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286 Calcrete in road construction

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BULLETIN 10 — ERRATA

1. Page vii, Contents; p.47 : delete "using calcrete as base and subbase".

2. Page 2, left hand column, 2nd last paragraph:- "Plate 2" should read "Plate 1".

3. Page 6 : replace "Figure 1".

4. Page 6, 5. DISTRIBUTION, 1st paragraph, 11th line down:- "20mm" should read "200mm".

5. Page 9, right hand column, 10th line from top:- "grading" should read "gradient".

6. Page 10, left hand column, 2nd paragraph, 3rd line down:- "the potential of the dissolved" should read "the potential dissolved" (ie delete : of the)

7. Page 10, right hand column, 8. PROSPECTING METHODS, 1st paragraph, 4th line down:- "particularly those of fossil origin." should read "particularly those which are fossil."

8. Page 21, TABLE 6, Nodule size mm, last line:- "63,5" should be ">63,5"

9. Page 22, left hand column, Hardpan calcretes, 2nd paragraph, 7th line down:- "true in regard to" should read "true except in regard to".

10. Page 22, right hand column, 2nd last paragraph, 2nd line down:- delete reference no "28"

11. Page 22, right hand column, 2nd last paragraph 5th line down:- "10mm" should read"19,1mm"

12. Page 23, FIGURE 14, Cryptocrystalline Calcrete symbols:- "□ 0,245mm" should read "□ 0,425mm"

13. Page 27, right hand column, 5th line from top:- "abration" should read "abrasion"

14. Page 30, Surfacing chips, add to end of last paragraph:- "Excessively porous calcretes may require unusually large quantities of bitumen if long term absorption problems are to be avoided. It is believed that 10% bitumen is used in the successful asphaltic concrete surface courses in New Mexico."

15. Page 31, right hand column, 2nd paragraph, 7th line down:- reference "34" should be "23"

16. Page 31, right hand column, foot note:- "Design Factor is 0,4" should read "Design Factor is 0,6"

17. Page 32, right hand column, end of 5th paragraph:- "(4,4mm)" should read "(44mm)"

18. Page 34, right hand column, 2nd last line:- "the correct size" should read "the percentage passing 0,425mm"

19. Page 40, right hand column, no. 27, in 2nd line:- "five" should read "six".

20. Page 45, heading:-
    "APPENDIX A"
    should be under main heading.

21. Page 45, Reference column
    Flow Index (I_f) :- "2(w_{10} blows - w_{32} blows) should read
    2(w_{10} blows - w_{32} blows)

22. Page 69 : add two columns to Table 18 (continued)

23. Page 73 : complete last two columns of Table 18 2nd Group (continued)

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SYNOPSIS

The composition, distribution, origin, and age of South African calcretes are briefly reviewed and a simple classification suitable for engineering use described. Methods of searching for economic calcrete deposits and of testing calcretes for roadmaking are considered and it is concluded that special methods of locating these must be used and that precautions must be taken during testing if the best results are to be achieved. The engineering properties of calcretes and specifications for calcrete roads are dealt with in some detail. Depending on their stage of development, these materials range in properties from those of an almost useless powder to those of rock, and this and their unusual composition and mode of formation cause them to exhibit unusual and often beneficial properties. Depending on the actual stage of development reached, calcretes can be suitable and have been used for all classes of road material, including surfacing chips, base, subbase, gravel road wearing courses and concrete aggregate. Some of the usual soil constants and grading requirements can be considerably relaxed when these materials are employed and significant economic benefit can result.

SINOPSIS

'n Kort oorsig oor samestelling, verspreiding, ontstaan en ouderdom van Suid-Afrikaanse kalkrete word gegee en 'n eenvoudige klassifisering vir ingenieursdoeleindes word verstrek. Metodes om ekonomiese neerslae van kalkreet op te spoor en hulle vir padboudoeleindes te toets, word oorweeg. Daar word bevind dat spesiale metodes toegepas moet word om hulle voorkoms te bepaal en dat toetse daarop besondere voorsorg vereis om die beste resultate te kan verkry. Die ingenieurseienskappe van kalkreet en spesifikasies vir die gebruik daarvan by padbou word uitvoerig behandel. Na gelang van die ontwikkelingstoestand wissel die eienskappe van hierdie materiaal van dié van 'n feitlik nuttelose poeier tot dié van rots en dit, tesame met die ongewone samestelling en ontwikkelingswyse van kalkrete, laat hulle buitengewone en dikwels gunstige eienskappe vertoon. Na gelang van die ontwikkelingstoestand wat hulle bereik het, kan kalkrete bruikbaar wees en is hulle reeds as klipslag vir die oppervlaklaag, kroonlaag, stutlaag, die ryvlak van gruispaaie en as aggregaat in beton gebruik. Sommige van die gewone vereistes ten opsigte van grondkonstantes en gradering kan minder streng wees wanneer kalkreet gebruik word, en dit kan merkbare ekonomiese voordeel meebring.

ACKNOWLEDGEMENTS

This work was initially financed by a grant from the Roads Department of the South West Africa Administration. The project was carried out in the Soil Engineering Group of the NIRR under the general guidance of Mr A.A.B. Williams and Dr. H.H. Weinert. Much of the routine laboratory soil testing was carried out under the author's supervision by Mr T.E. O'Connor.

The author also wishes to express his appreciation of the generosity and cooperation of the many persons and organizations who freely made information, test results, hospitality, or assistance of some kind available; they are acknowledged in full in the detailed report[1].

FOREWORD

Calcretes have always posed a problem for roadbuilders in South Africa as sometimes these materials perform well and in accordance with the behaviour predicted by standard tests and on the occasions the standard tests fail to predict their behaviour on the road. Opinions as to their suitability for road construction vary from district to district, and from man to man, over large tracts of the drier parts of Southern Africa where calcretes are the only local roadbuilding materials available.

In December 1963 the Roads Department of the South West Africa Administration agreed to sponsor a research project on calcretes entitled: "A study of a manner of formation and engineering properties of the surface limestones (calcretes) especially in the north, but also elsewhere in South West Africa and in the Republic". In May 1964 Mr. Netterberg was appointed to lead this project, and although the contract with South West Africa was terminated in June 1967 the research was continued as part of the national programme of research of this Institute until its completion at the end of 1969.

This report is a condensation of parts of a very comprehensive report submitted by Mr. Netterberg to the University of the Witwatersrand as a doctoral thesis and readers requiring more details should refer to this work. Although there have been several very interesting findings of fundamental importance, such as the identification of palygorskite as a clay mineral of common occurrence; the composition of calcretes which often contain much magnesium; the importance of micro-fossils in these soils, and a new explanation for the mode of formation of calcretes, in this Bulletin the emphasis is placed on the practical findings which are of importance in road construction.

As a result of this research several other papers have appeared in proceedings of conferences or in technical journals. The drawing together here of all practical aspects for application in more efficient road foundation design and construction should result in more economic use of our natural resources.

S. H. KÜHN
Director

National Institute for Road Research.

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METRICATION

This Bulletin has been metricated in accordance with the rules laid down for the South African road industry. Much of the original research, however, has been done when the Imperial System of Weights and Measures was still used in South Africa and metrication has therefore not been applied to tables or parts of tables where metrication would give values which were not obtained in the actual tests, as well as where reference has been made to non-metric specifications.

1. INTRODUCTION

Calcretes are of common occurrence in arid and semi-arid lands everywhere and in Southern Africa they constitute an important source of borrow material for road construction. As regards the extent of its use in the Republic (45,1%), calcrete ranks third in importance[2] after dolerite (48,9%) first and sandstone (46,1%) second while in respect to the actual quantities of materials used calcrete is second only to dolerite*. It probably rates as the most widely used road construction material in Southern Africa as a whole.

A calcrete can be regarded as a material formed by the *in situ* cementation and/or replacement of almost any pre-existing soil by calcium carbonate deposited from the soil water – the term 'soil' being used here in its wide engineering sense. A calcrete is not a sedimentary rock, but a member of that most useful group of road materials known as pedogenic materials (of which ferricrete is another familiar member).

The field work undertaken in connection with this project was carried out within South West Africa and the Republic of South Africa and the samples tested all came from the same areas. The extensive deposits of calcrete in Botswana were not visited. It is thought, however, that many of the results of this work can be applied to calcrete deposits elsewhere. The terms 'Southern Africa' will be used to refer to that portion of Africa lying south of about latitude 15°S and 'South Africa' to the Republic and South West Africa.

2. COMPOSITION

By definition, a calcrete is composed of a cementing and/or replacing carbonate, usually calcite, and the host material which is cemented and/or replaced by it. As this definition implies, calcretes can be so variable in composition that the term 'calcrete' can justifiably be applied to materials varying from an almost pure limestone in which there is hardly any trace of the host material to a material consisting to a large extent of the host material. Although usually the carbonate present is dominantly calcite, dolomite is often present too, sometimes in amounts which might justify the designation of the material as *dolocrete*. Apart from possible rare traces of aragonite, no other carbonates have been positively identified in calcretes. Quartz, usually in the form of sand grains, is the chief allogenic mineral in calcretes, but variable and lesser amounts of felspars, clay minerals, opaline silica (often in the form of microfossil skeletons), accessory amounts of other minerals, and rock fragments are also usually present.

The typical indurated calcrete (mainly nodules, hardpans, boulders and strongly calcified sands and gravels) is chiefly made up of an authigenic cryptocrystalline (about two micron) calcite or dolomite matrix in which 'float' variable amounts of allogenic quartz and occasional felspar grains. In some instances the matrix, which is often veined, has apparently been replaced by a coarser (often five micron), probably recrystallized, calcite. Replacement of the allogenic quartz and felspars by this coarser calcite is common and probably provides the main source of the opaline silica cement which silicifies some calcretes, and which results in a Mohs hardness of up to 6 being attained. Calcretes tend to increase in carbonate content according to their stage of development and relative to the amount of the allogenic component present so that the average *hardpan* calcrete compares reasonably with the average 'limestone'.

Powder calcretes, nodular calcretes and to a lesser extent honeycomb calcretes are composed of two consistency components. These are, for instance, indurated nodules, cutans, pedotubules or coalesced nodules (honeycomb types), composed as described above, and embedded in a loose matrix. The indurated nodules vary in size from silt-sized aggregations of finer particles weakly cemented by carbonate to very hard gravel-sized nodules with Mohs hardnesses of up to 6. The silt-sized aggregations do not disintegrate in water or when they are moved about under the microscope, but they can be broken up by pressure from a needle. On treatment with acid they can be seen to be compound particles made of hundreds of finer particles cemented together by the carbonate. The strength of the particles and the proportion of cementing matrix to cemented grains increases with particle size, so that the larger nodules may possess aggregate crushing values of below 20 per cent. The porosity and bulk specific gravity of the particles also vary with size fraction. The minus 2mm and minus 0,425mm fractions of nodular calcretes are often composed largely of carbonate-cemented particles. Fossil calcretes may, however, possess non-calcareous or only slightly calcareous fines. The mineralogy of these two size-fractions differs little from that of the coarser particles except that the carbonate content is invariably lower and the quartz and clay content higher in the finer fractions. The minus 2mm fraction is large-composed of calcite, quartz, clay minerals and dolomite.

Probably the most commonly encountered clay minerals in calcretes are palygorskite, montmorillonite and sepiolite. Illite is also found, but kaolinite occurs only in fossil calcrete fines which are being leached of their carbonate. Except possibly in the latter case, all the clay minerals present are probably to a great extent, saturated with calcium or magnesium. Palygorskite and sepiolite possess a number of unusual properties, most of which are likely to be beneficial to a road material[1]. For example, palygorskite clay (and

*WEINERT, H.H. Personal communication, 1969.

probably also sepiolite) possesses a far greater shear strength at the same moisture content than other clays.

Calcretes therefore possess a composition which is unusual among road materials. Because of the high carbonate content, the unusual clay minerals present (palygorskite and sepiolite) and the presence of compound porous particles and amorphous silica microfossil remains, they can be expected to exhibit some unusual properties.

3. CLASSIFICATION

Although many calcrete profiles are much more complicated than those illustrated here, the different types of calcrete seen in the Republic and South West Africa can be resolved into six basic types which can easily be recognized in the field by untrained personnel. These are: calcified soils, powder calcretes, nodular calcretes, honeycomb calcretes, hardpan calcretes, and boulder calcretes. Each group not only possesses a significantly different range of engineering properties but also represents a particular stage in the development of a pedogenic calcrete. The classification is thus of both engineering and geological significance.

Calcified soil

Calcified soil represents an early stage in calcrete formation in which normally the host material has not yet become very strongly cemented. A calcified soil does not usually exhibit nodular structure. The total carbonate content (calcite plus dolomite) varies between about 10 and 50 per cent and largely only fills the voids between the host grains, in other words, the carbonate has not yet succeeded in mechanically replacing (pushing apart) much of the host material. Although the common types are: calcified gravels (Plate 1) and calcified sands (Plate 2), it is occasionally convenient to place other materials such as calcified weathered rock and the occasional calcified clay in this category. The members of this group only possess structures inherited from the host material, such as the bedding in Plate 2 or those caused by the calcrete that fills fissures and cements the fragments in calcified weathered rocks.

Most calcified sands are of a consistency that is only slightly cemented and although they are often friable they do not normally slake in water. Calcified gravels such as those in Plate 2 may, however, be cemented (very stiff) in consistency and can only be picked with difficulty.

It is important here to distinguish between a calcified soil and a calcareous soil — the former term should only be applied to materials which have become significantly cemented and the term 'calcareous soil' to those materials which are so weakly calcareous (probably about 1–10% carbonate) that no cementation of engineering significance has taken place.

NB

Powder calcrete

Ideally, this type of calcrete is composed of a loose powder consisting predominantly of silt and sand-sized carbonate particles with little or no nodular development (Plate 3). Such a material possesses a 'grading modulus' of around 0,6, i.e. the cumulative percentage retained on the U.S. No. 10 (2,00mm), 40 (0,425mm), and 200 (0,075mm) sieves divided by 100[4]. Nodular calcretes sometimes form in host powder calcretes, and a grading modulus of 1,5 affords a useful means of separating the two, though a few nodular calcretes do fall below this figure. Usually, powder calcretes are readily differentiated from calcified sands by their very low content of host soil grains (calcified sands invariably possess a high percentage of sand-sized quartz grains). Also, not only is powder calcrete usually formed in a host material different from calcified sand but most of the host material has been mechanically replaced by fine carbonate.

Nodular calcrete

Nodular calcrete is, in fact, a gravel. It consists of discrete, usually intact, soft to very hard concentrations (nodules) of carbonate-cemented host material in a (usually) calcareous matrix (Plate 4). The nodules vary in size from silt size to about 60mm and their shapes vary from more or less spherical in the early stages of development to highly irregular in the later stages when several nodules have coalesced. The overall consistency of a nodular horizon is usually loose, occasionally medium-dense, but the individual nodules vary widely in consistency, generally increasing in hardness with both nodule size and grading modulus.

Nodular calcrete is the most useful type of calcrete from the roadmaking point of view.

Honeycomb calcrete

Honeycomb calcrete represents a stage of development intermediate between that of a nodular calcrete and a hardpan calcrete. The nodules have coalesced to form a horizon of much greater overall strength than a nodular calcrete, but the voids between the partly coalesced nodules are still filled with relatively uncemented material. The example shown in Plate 5 is a honeycomb type at an advanced stage in which the host soil has been washed out of the exposed voids by rain.

The overall consistency of a honeycomb layer is usually intermediate between that of nodular calcrete and hardpan. While the nodules themselves are normally hard, the horizon can usually be ripped and crushed by grid rolling, to yield a gravel composed of relatively hard, angular fragments.

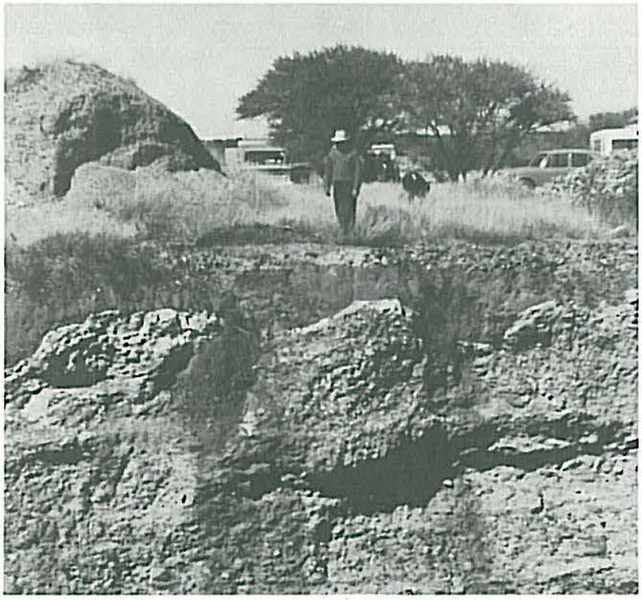

PLATE 1

CALCIFIED GRAVEL

Note also the makondos (solution hollows) extending downwards from the surface of the calcified gravel, indicating the onset of solutional weathering and boulder formation.

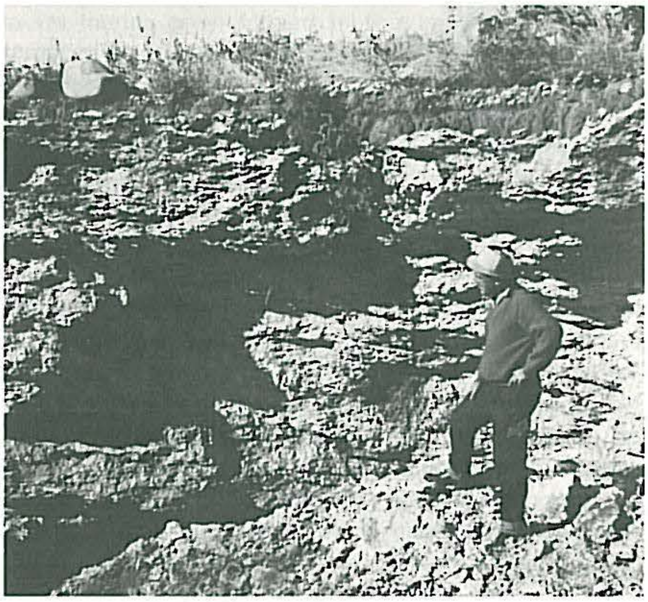

PLATE 2

CALCIFIED SAND

The bedding is inherited from the alluvial sand host material and is not an invariable feature of calcified sands.

POWDER CALCRETE

Note the deficiency of coarse aggregate.

Hardpan calcrete

Hardpans usually occur as a harder sheetlike crust seldom more than about half a metre thick which overlie less well-cemented material of a lower stage of development (Plate 6). Only very old calcrete hardpans such as those capping the calcified gravels of the Weissrand are likely to be thicker than half a metre. If the hardpan was previously a honeycomb calcrete, it represents the final stage, in that the voids between the coalesced nodules have become completely cemented up. The original nodules are usually no longer readily discernible. If the hardpan was previously a calcified soil, it represents the final stage of development, the carbonate content having increased to the point where it has replaced so much of the host material that the host grains have been forced apart and now 'float' in the carbonate matrix. Sometimes, little of the host material is visible to the naked eye, but when an appreciable amount is visible the hardpan may be described as gravelly or sandy, depending on the size of the visible particles.

Hardpans are usually intact, but may be shattered (showing the onset of weathering), bedded (due to bedding inherited from the host material), platy (due

PLATE 4

NODULAR CALCRETE
Essentially a loose to medium dense sandy gravel. Individual nodules and even compound nodules made up of several coalesced smaller nodules seldom exceed approximately 60 mm in diameter.

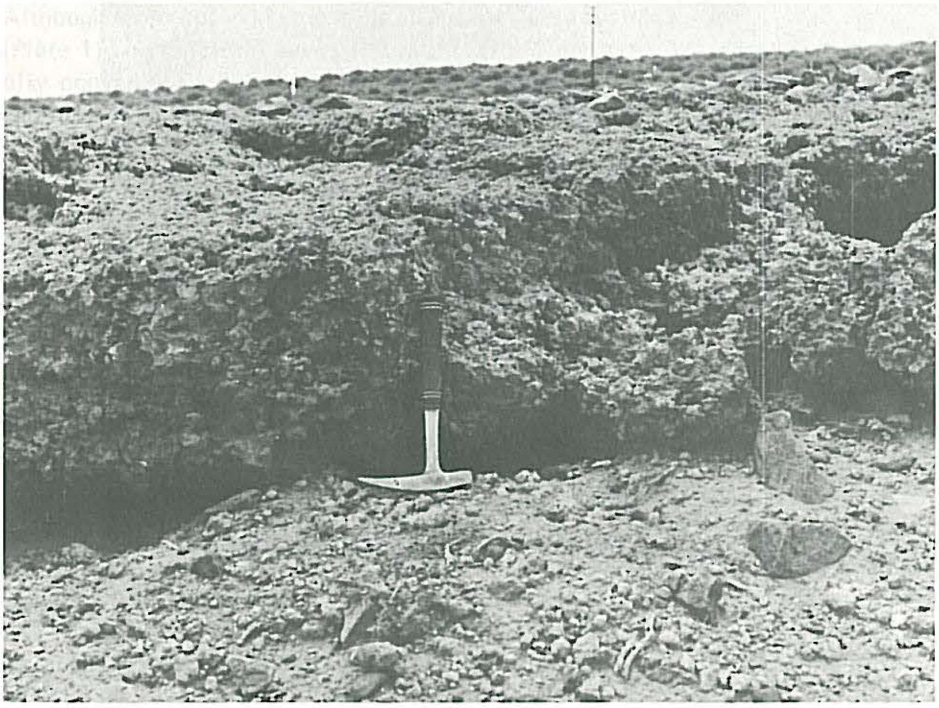

PLATE 5

HONEYCOMB CALCRETE
A layer made up of partially coalesced nodules.

to its having been formed in thin plate-like layers), or tufaceous (the vlei limestone, diatomaceous pan limestone, Pfannenkalktuff and Schneckenkalk of previous authors). The tufaceous types usually possess very low densities (1200–1520 kg/m^3) in comparison with other hardpans, which range from about 2060 to 2640 kg/m^3, the density usually being more than 2400 kg/m^3.

Boulder calcrete

On weathering, calcrete hardpans tend to form boulders. When covered by soil, the upper surfaces of the boulders assume a typically rounded appearance (Plate 7) due to solution and they exhibit solution holes or hollows filled with non-calcareous soil which can probably be regarded as representing the onset of boulder formation (Plate 1). Outcropping hardpans exhibit the usual limestone solution facetting on the upper surfaces exposed to rainfall.

PLATE 6

HARDPAN CALCRETE

An intact hardpan layer overlying a faintly bedded, weakly developed nodular powder calcrete.

PLATE 7

BOULDER CALCRETE

Note the rounded shape of the boulders and the holes filled with soil.

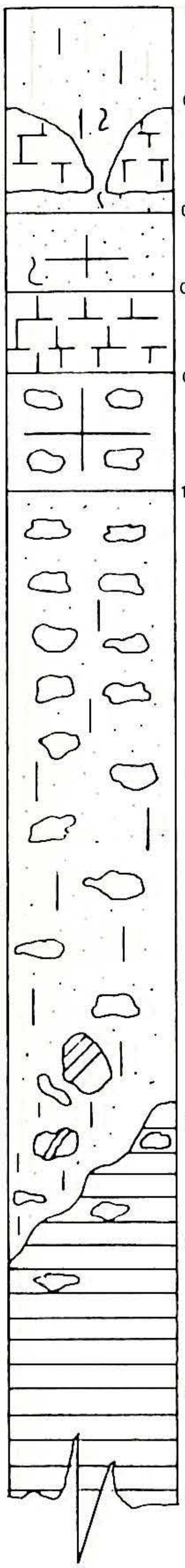

FIGURE 1

Suggested method of describing a hypothetical calcrete profile for engineering purposes.

Boulder calcrete is usually rippable, in spite of the interiors of the boulders being usually still hard and sound (except for badly-shattered outcropping types). The boulders are however difficult to reduce further, as they are usually too hard for grid rolling and often too large to crush.

4. DETAILED DESCRIPTION

When a detailed description of a calcrete or calcrete-containing profile is required for engineering purposes, it is recommended that the principles of the general soil description method of Jennings and Brink[3], and the calcrete classification given above, be employed. An example of the suggested procedure and the symbols used is shown in Figure 1. The usual additional information pertinent to the drilling or excavation method employed, samples taken, etc., will also be required.

5. DISTRIBUTION

Figure 2 shows the distribution of the different types of calcrete in Southern Africa. It can be seen that calcretes that are sufficiently well developed for use in road construction generally occur only in the drier areas, that is, where the rainfall is less than about 550mm. Various climatic indices also afford good correlation with the limit of occurrence of usable calcrete in Southern Africa. Generally, calcrete only occurs where Weinert's[2] N-value is more than 5; in areas possessing a Thornthwaite[5] water deficiency of more than 20mm (moisture index of less than -20), and in Thornthwaite's[6] Arid Warm (EB'd) and Semiarid Warm (DB'd) climates. The scattered calcrete nodules and soluans occurring in the drier half of Thornthwaite's subhumid zone (CB'd, Subhumid Warm, moisture deficient in all seasons) receiving 550–800mm of rainfall, i.e. where N is between 2 and 5, have hardly altered the engineering properties of their host materials (though their presence would normally mean that the clay minerals present are calcium saturated). Also the nodular calcretes occurring in this region are normally too thin to be economically worked for road material.

Calcification is generally absent in areas that receive more than 800mm of rainfall. According to Thornthwaite[5] these areas are defined by zones enjoying an annual water surplus (moisture index of 0), or where N is less than 2.

While it is apparent that climate, particularly rainfall, plays an important part in determining the distribution of calcretes, it is only one of several pedogenic factors: climate, parent material (or merely host material in the case of some calcretes), topography, and biotic influence, all of which may again vary independently with time. A perfect correlation between the

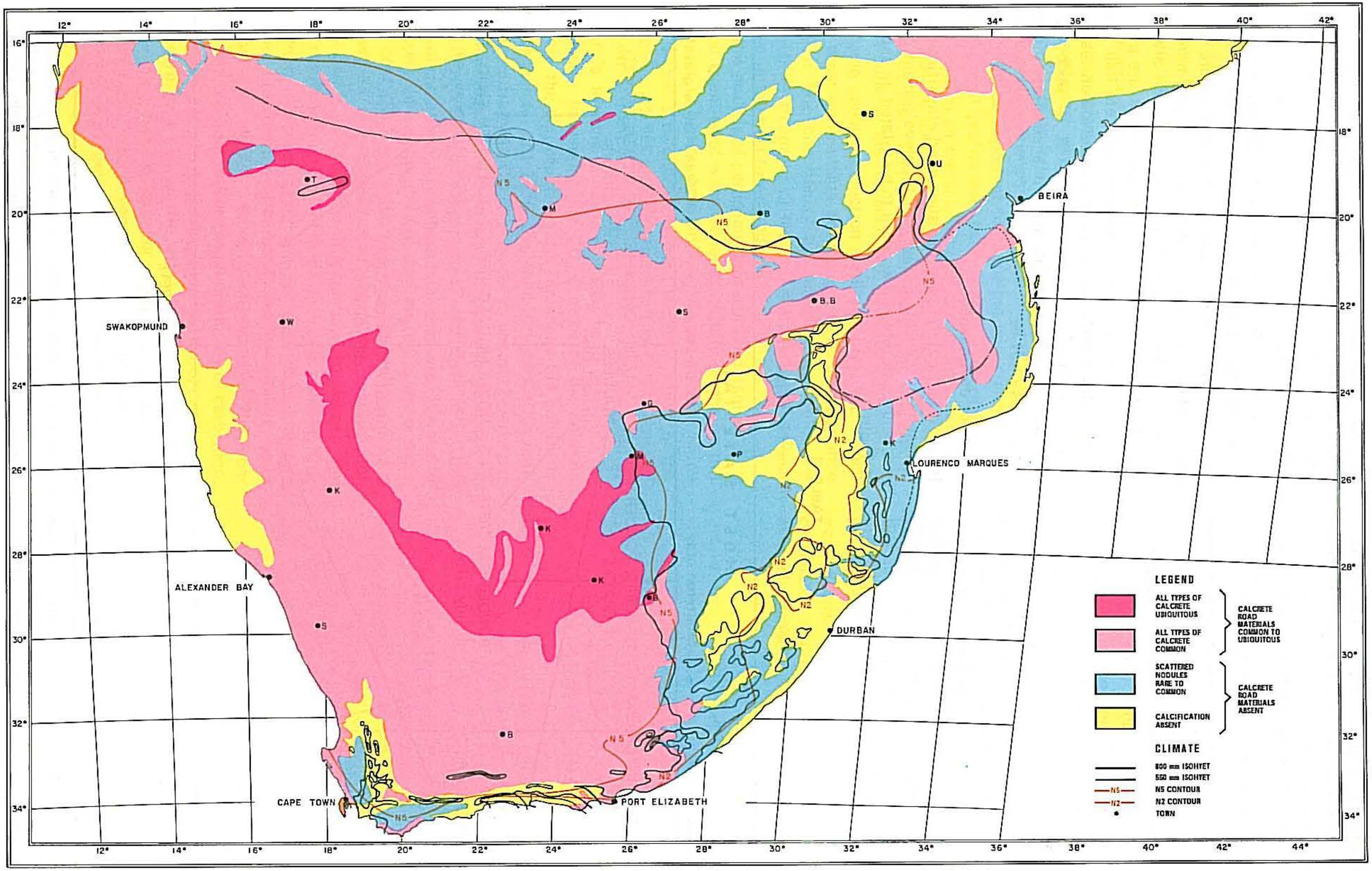

FIGURE 2

Distribution of calcretes in Southern Africa in relation to climate.

distribution of calcrete and climate is thus not possible. Probably almost as important as climate is the availability of carbonate, which in turn is controlled by parent material, topography (drainage) and climate.

6. ORIGIN

General

While many theories of origin have been proposed for calcrete-like materials in different parts of the world, it would seem that most of the Southern African deposits were formed in one or both of two ways: (1) by deposition of carbonate in the host material above a shallow perched or permanent water table, or (2) by downward leaching of carbonate from the upper soil horizons by infiltrating rainwater and its deposition lower down in the profile. Calcretes of these two basic origins can be simply referred to as ground water calcretes and pedogenic calcretes respectively.

The thick deposits of calcified alluvial sands and gravels of the Vaal and many other rivers (particularly in South West Africa), and the many relatively thick calcretes associated with present-day shallow water tables in the Republic are thought to be of the first type.

Pedogenic calcretes are very common, and the mature profile consists typically of a harder calcrete crust overlying weaker calcrete of a less mature type (lower stage of development), for example hardpan calcrete overlying nodular calcrete. It is not always easy to decide whether the immature types are pedogenic or non-pedogenic. Pedogenic calcrete can form in any calcareous profile. Ground water calcretes are almost invariably capped by a crust of pedogenic hardpan calcrete, and can form in a completely non-calcareous profile.

Stages of development

The different calcretes of the suggested classification represent the possible stages in the development of a pedogenic calcrete and appears to be related as shown in Table 1[7].

TABLE 1

Stages in the development of calcretes

STAGE	HOST MATERIAL			
0	Weathered rock	Shattered clay	Mixed texture	Clean sand or gravel
1	CALCRETE IN CRACKS	CALCRETE POWDER IN CRACKS	SCATTERED CALCRETE NODULES in host soil	CALCRETE-COATED GRAINS
2	CALCIFIED WEATHERED ROCK	POWDER CALCRETE (sandy silt)	NODULAR CALCRETE (sandy gravel)	CALCIFIED SAND OR GRAVEL (massive)
3			HONEYCOMB CALCRETE (coalesced nodules)	
4			HARDPAN CALCRETE (rock-like sheet)	
5			BOULDER CALCRETE (discrete boulders formed by weathering)	

Any of the developmental stages in Table 1 may be pedogenic in origin, but it is thought that ground water types do not often develop beyond stage 2. The mature calcrete profile is regarded as one which contains a capping of reasonably unbroken hardpan (stage 4). Hardpan thus represents the final stage of development of all calcretes. Boulder calcrete represents the weathered stage.

Calcrete formation is thus a process of deposition and crystal growth of carbonate in the host soil, in which the host particles are pushed apart and the relative carbonate content increases with development. Under ideal conditions this is probably a continuous process in that all gradations between the stages are possible. The stages shown are merely those which are easily recognized, easily named and which possess significantly different engineering properties.

Mechanisms of calcification

The mechanism postulated by most authors to account for carbonate crystallization from the soil solution during calcrete formation is simply evaporation. There is however evidence to show that simple evaporation can only be important in about the upper metre or so of the soil profile and that the effect of changes in soil suction on the solubility of carbonate is possibly the most important mechanism in calcrete formation at all depths[1]. Transpiration by plants and evaporation (particularly the former) are however important in bringing about the suction gradients involved. Algae and other water plants are thought to be of importance only in the formation of the tufaceous calcretes. A very brief treatment of the soil suction hypothesis is given in the next few paragraphs.

The amount of carbonate that can be dissolved in a given quantity of water depends chiefly on the partial pressure of carbon dioxide (P_{CO_2}) in the air with which the water is in equilibrium and the temperature of the system. As the total pressure or the CO_2 content of the air is increased, P_{CO_2} is also increased, and by Henry's law the solubility of CO_2 is in turn also increased. Increasing the CO_2 content of the solution lowers the pH, which in turn increases the solubility of carbonate. At 25°C the solubility of $CaCO_3$ (calcite) initially in CO_2-free water is about 12 parts per million (ppm), while in equilibrium with atmospheric air (P_{CO_2} = 0,00032 atmospheres, equivalent to 0,032% by volume at one atmosphere total pressure) it rises to 53 ppm. In equilibrium with soil air (up to one or more per cent CO_2) the solubility is even higher. An increase in temperature lowers both the solubility of CO_2 and the solubility of carbonate in water. The net result is that an increase in water content, CO_2 content, and pressure, and a decrease in temperature favour the solution of carbonate while an increase in temperature and a decrease in water content, CO_2 content and pressure favour the precipitation of carbonate.

Figure 3 shows the likely distribution of pore water pressure in a soil profile with a shallow water table undergoing evaporation. For the sake of simplicity it is assumed that no vegetation is present since the presence of vegetation would complicate, but not invalidate the argument. Two practical examples where no vegetation would be present are newly deposited alluvium and conditions too saline to support vegetation. Below the water table the pore spaces are essentially saturated with water under hydrostatic pressure. Above the water table the water is held under a pressure of less than one atmosphere by surface tension forces and there the pressure as well as the degree of pore saturation decreases upwards. Under an infinite impermeable cover, the pore water pressure diagram above the water table would, under shallow water table conditions, be an extension of the hydrostatic line, but in the case considered in Figure 3, evaporation is taking place, the suction grading is increasing and the line curves away to more negative pressures (higher suctions or pF) in the upper few feet (A). (For moisture to move upwards in the liquid phase against gravity, the suction gradient would have to be increased above that represented by an extension of the hydrostatic line).

Now, since the solubility of carbonate in water at constant temperature depends largely on the pore pressure of the CO_2 with which it is in equilibrium, it follows that a pore water pressure diagram is also a diagram of relative carbonate solubility. This means for example that carbonate-charged ground water moving upwards in the profile will tend to deposit carbonate, merely because of the increased suction to which it becomes subjected. It is thought that the

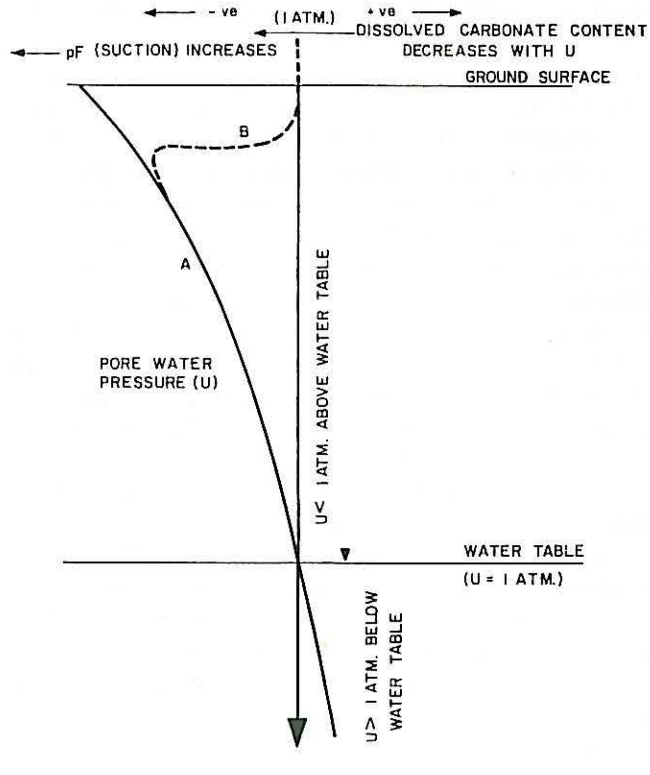

FIGURE 3

Probable pore water pressure distribution in a non-vegetated, homogeneous soil profile with a shallow water table undergoing evaporation (curve A) and during rainwater infiltration (curve B).

reason why most of the dissolved carbonate will be deposited is the CO_2-loss as the pF changes from about 2 to 3 before much evaporation can take place. Although at still higher pFs the solubility of Ca^{++} should rise again (owing to the formation of hydroxide ions), the moisture contents are too low to permit the retention of much dissolved $CaCO_3$.

It will also be evident that the solubility of carbonate will depend on the position of the water table. If the water table rises, the potential of the dissolved carbonate content at every point in the profile is increased. Similarly, if the water table drops, precipitation of carbonate will tend to occur, again mostly at depths were the pF changes from 2 to 3. It is thought that most ground-water calcretes such as calcified alluviums are formed by the mechanism outlined, i.e. by deposition from ground water rising under the suction gradient and as the water table fluctuates.

To consider the origin of pedogenic calcretes, let us commence with any calcareous parent material. It may even be a calcrete formed by the mechanism outlined above. Consider curve B in Figure 3, which shows what probably happens in the upper part of the pore water pressure diagram during rainwater infiltration. The curve swings sharply to the right, the horizontal portion representing the infiltration front. Solution of carbonate will now tend to occur in the uppermost portion of the profile and deposition at the infiltration front where the pressure drop is greatest. Again, most deposition should occur where the pF changes from 2 to 3. As the front moves downwards, this carbonate will tend to be redissolved and carried still deeper, until rainfall ceases and the pressure diagram slowly reverts to its original position (A). The net result is that carbonate is leached from the upper horizons and deposited in the lower horizon not directly by evaporation, but as the pF changes, partly owing to evaporation from the surface and partly because the suction equilibrium tends to re-establish itself. The effect of transpiring vegetation would be to modify the shape of the pressure diagrams and in this way to influence the depth and amount of deposition. It can be shown by arguments similar to those previously given that, once again, most deposition is likely to occur as the water moves along the suction gradient towards the plant roots in the zone where the pF changes from 2 to 3. It can also be shown that plant transpiration normally accounts for a much greater loss of water from the soil than does evaporation.

The relative importance of evaporation, transpiration, and CO_2 loss, due to pore pressure changes, to calcrete formation probably varies from place to place, depending on the depth of the water table, soil texture, climate, and other factors, and cannot be dealt with further here. A detailed discussion has been given elsewhere[1].

7. AGE

Calcretes are geologically very young, and it would appear that most of the calcretes of Southern Africa can be classified into four important age groups[9]: Pliocene (about 2 000 000 – 13 000 000 years old), First Intermediate (at least 35 000 – 45 000 years B.C.), Second Intermediate 9 000 – 15 000 years B.C.) and Recent (present – 8 000 years B.C.). Examples of Pliocene calcretes are the Kalahari Limestone and probably some calcified river gravels such as the Basal Older Gravels of the Vaal and the Main Terrace Gravels of the Ugab. First Intermediate calcretes are mainly calcified alluviums. Second Intermediate calcretes are particularly widespread and most of the calcretes used by the road builder were probably formed during this period. Many are pedogenic in origin. Widespread calcrete hardpan formation does not appear to have taken place in Southern Africa in recent times and only a few boulder calcretes (weathering products of hardpans) and a few immature types such as calcrete nodules seem to be forming at present.

The stage of development (Table 1) usually increases with age thus, for example, boulder and hardpan calcretes are normally older than nodular calcretes in the sense that they have taken longer to reach that stage of development. In terms of age, i.e. period of formation before present, a hardpan, for example, could be younger than, for example, the nodular calcrete underlying it because it has reached that stage more recently. While it would not be true to state that all hardpans are older than all nodular calcretes, it would be correct to state that all hardpans are more *developed* than all nodular calcretes.

8. PROSPECTING METHODS

Airphoto interpretation[1,10,11,12] is a very useful preliminary tool, especially in the Kalahari, but cannot be completely relied upon to detect all calcrete deposits, particularly those of fossil origin. The areas on airphotos in which to look for calcrete deposits, both present day and fossil, are at the sides of pans and drainage lines and also on the insides of bends (Plate 8). White, calcareous anthills show up well on airphotos, although generally these indicate fairly plastic calcrete below.

In Southern Africa, various calciphilous plants have proved to be very useful indicators of the presence of calcrete or calcareous soil at shallow depths. The gabbabos *(Catophractes alexandri)*, which occurs over almost all that part of South West Africa receiving less than about 500mm of rainfall and where N is more than 5 is perhaps the most reliable of these. In Ovamboland where the rainfall is more than about 500mm but where the N-value is still more than 5, its place is taken by the saliebos *(Pechuel-Loeschea-leubnitziae)*. The vaalbos *(Tarchonanthus camphoratus)* is

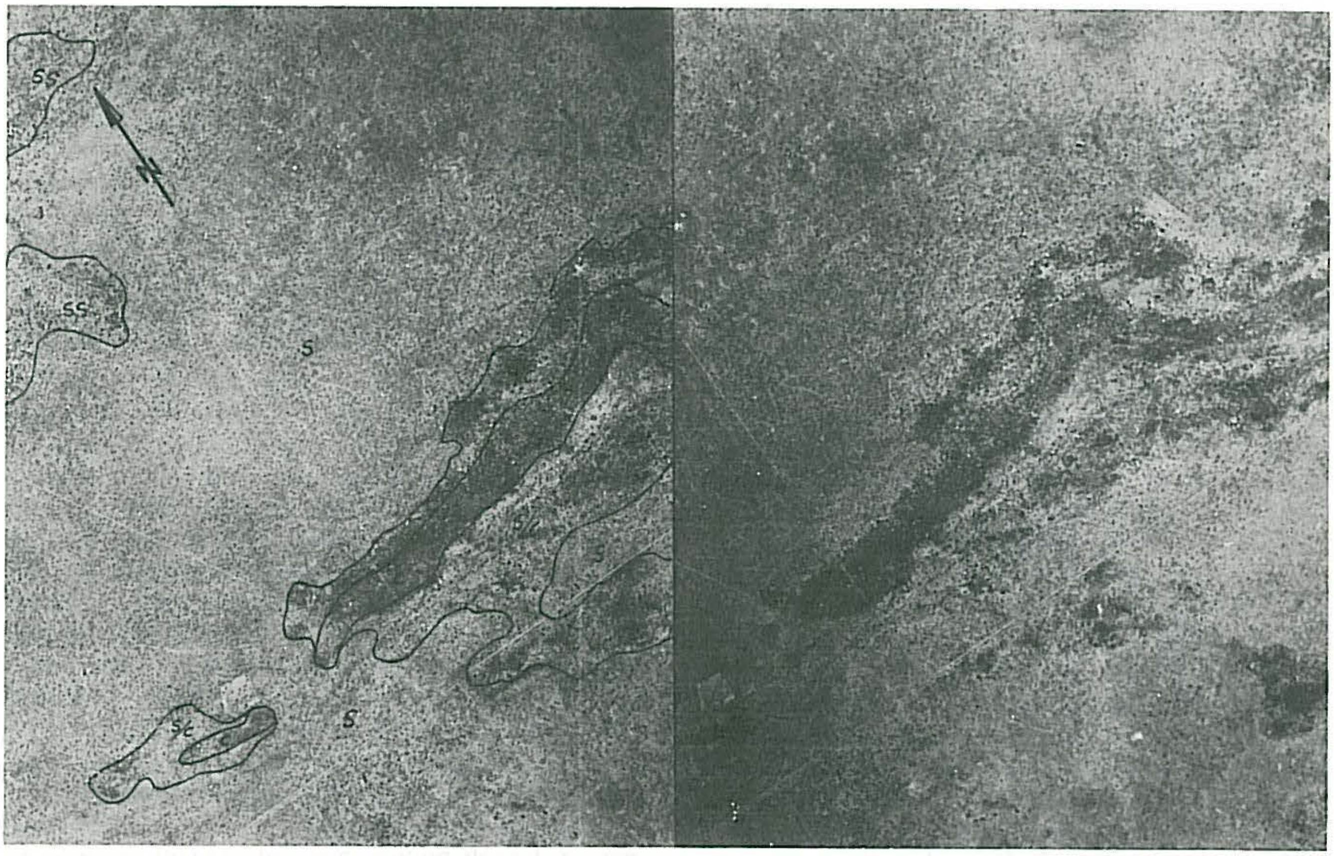

PLATE 8

Calcrete (C) and sand overlying calcrete (S/C) in a fossil drainage line on the farm Operet 312 northwest of Tsumeb, South West Africa. Airphoto interpretation after Mountain (1964).

another useful indicator in the Okavango region of South West Africa and in the northern Cape Province. The dense growth of these bushes which sometimes give rise to characteristic airphoto patterns, usually indicates a calcrete rather than a calcareous soil deposit.

A most useful preliminary indication of the presence or absence of calcrete within a usable depth can be obtained in sandy soils in less than a minute by means of a rapid probing device (Figure 4) invented by the late Senior Road Inspector at Upington, Mr. T.H. van der Westhuizen. The device consists essentially of a double-acting pipe hammer fitted to a length of high tensile steel rod with a suitable piston to take the hammer blows welded to the upper end of the rod[13]. The rod is simply hammered into the ground until refusal or until its maximum range is reached, upon which it is hammered upwards to effect a rapid withdrawal. Any calcrete encountered will have stuck to the point, and with a little practice the type of calcrete can also be deduced. This probe is about ten times faster than any hand auger.

Figure 5 shows an idealized section across a typical riverside or pan calcrete deposit. It is important to prospect for calcretes more thoroughly than is perhaps usual in materials surveys since the indications of the presence of these materials are often so subtle that an important deposit can easily be missed. Use of the calcrete probe ensures that a given area can be propsected about ten times as thoroughly as it can with a hand auger in the same time.

It is considered that at present the combined use of airphoto interpretation, topographic setting, plant indicators, probing and pitting offers the best solution to the problem. Even using all these tools presently available, the certainty of detecting every calcrete deposit in a given area is probably not 100 per cent. Future work on prospecting methods for calcretes should include the use of infrared imagery and colour, camouflage detection, and infrared films.

9. ENGINEERING PROPERTIES

Except for some calcretes in the most advanced stages of development, calcretes cannot be regarded as being made up of hard, discrete particles. They must rather be regarded as being composed of a large number of small host soil-particles cemented together by the carbonate into larger, somewhat porous particles.

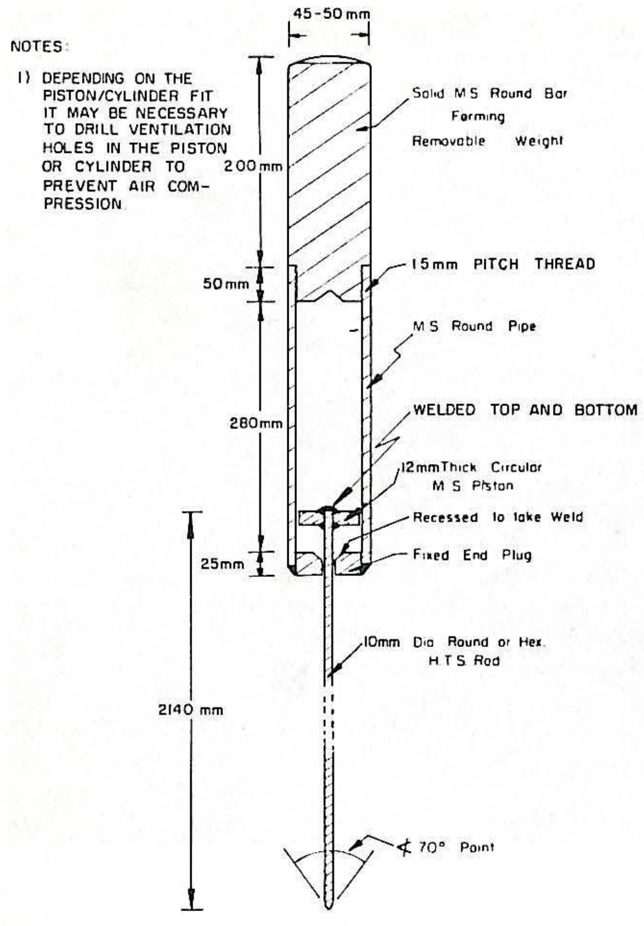

FIGURE 4

Rapid calcrete probing device.

Silt-sized calcite which possesses a measurable Liquid Limit of about 27 is non-plastic and does not shrink when dried from the Liquid Limit. The engineering properties of a calcrete therefore depend largely on the nature (mainly texture) of the host material and extent to which it has been cemented and replaced by the carbonate. If, for example, the host material is highly plastic, the calcrete will also be plastic, though less so than the host material, while if the host material is non-plastic sand, the calcrete will also be non-plastic, even though its Liquid Limit may be higher than that of the sand. Once the carbonate has firmly cemented and separated the host grains (at 30–50% $CaCO_3$ + $MgCO_3$ depending on the host soil type), the properties of the host material become less important and the properties of the calcrete come to depend more on the stage of development (Table 1), i.e. the extent of cementation and replacement. This in turn is controlled to a large extent by the age of the calcrete. While it is plain from Table 1 that soil texture plays a very important role in the early stages of calcrete development, given time and the right condition, all calcretes will eventually develop a hardpan. In general, the crushing strength, bulk density, hardness and grading modulus increase with development, while the Plasticity Index, Linear Shrinkage, water absorption and soluble salt content decrease with development.

In discussing the engineering properties of calcretes it is convenient to deal first with those properties which are common to all calcretes, and then to deal with the properties of each type. Although the

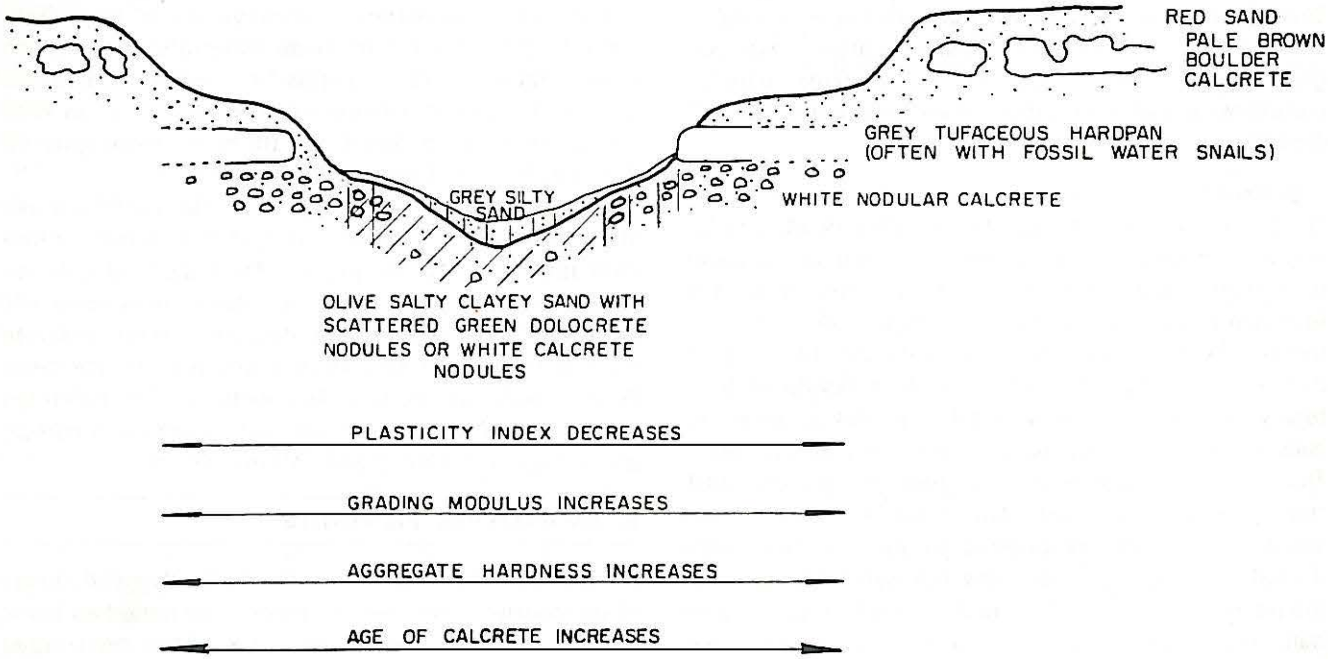

FIGURE 5

Idealized section across pan or omuramba.

following remarks on soil constants, particle specific gravity, self-cementation, chemical stabilization and soluble salt problems apply particularly to calcified soils, powder calcretes and nodular calcretes, they could, for example, also be applied to a hardpan crusher run.

Soil constants

As a host material becomes calcified, the exchange sites on the clay minerals present will become saturated with calcium, if this has not already happened. Only one or two percent $CaCO_3$ is necessary to saturate the clay minerals and the effect may be decidedly beneficial. If for example the clay mineral originally present was sodium montmorillonite, exchange of the Na^+ for Ca^{++} would reduce the Liquid Limit and Plasticity Index to as little as half their original values. On the other hand, if the clay mineral present were palygorskite or illite, the exchange of Na^+ for Ca^{++} would have relatively little effect, or could cause even a slight increase in the Liquid Limit and Plasticity Index. If, however, much magnesium is present (as in a dolocrete or very dolomitic calcrete) the Plasticity and Liquid Limit of palygorskite might be lowered and any further carbonate added would merely act as a mechanical stabilizer. Experiments[1] with a calcium-saturated montmorillonitic black clay indicated that the Liquid Limit is reduced linearly in proportion to the carbonate content to a minimum of about 27 at 100% silt-sized $CaCO_3$. The Plasticity Index was also reduced by the $CaCO_3$, but owing to the combined effect of the $CaCO_3$ on the Liquid and Plastic Limits, the PI versus $CaCO_3$-content relationship only became linear after the Plastic Limit became constant (50% $CaCO_3$). The Bar Linear Shrinkage curve was found to be exactly the same shape as the PI curve.

Figure 6 shows the position of some calcretes on the Casagrande plasticity chart compared with the average Liquid Limit versus PI relationship of some Transvaal[14] and Texas[15] materials, a small percentage of which were calcretes. It can be seen that calcretes do not fall in any particular area of the Casagrande chart and they do not obey these two published average Liquid Limit versus PI relationships.

The Shrinkage Limits of calcretes are often unusually high, higher even than their Plastic Limits (by up to 9 percentage units), an extremely rare feature which may be confined to these materials. This feature is also shown by Gillette's[16] results. Its cause appears to be the high porosity of the calcrete particles and the diatoms (tiny hollow microfossil shells) which are often present. It should also be noted that the Shrinkage Limit method involves a drying cycle, so that some cementation and development of structure may be possible since the addition of cement or lime to a soil normally increases the Shrinkage Limit. Although the effect of $CaCO_3$ on the

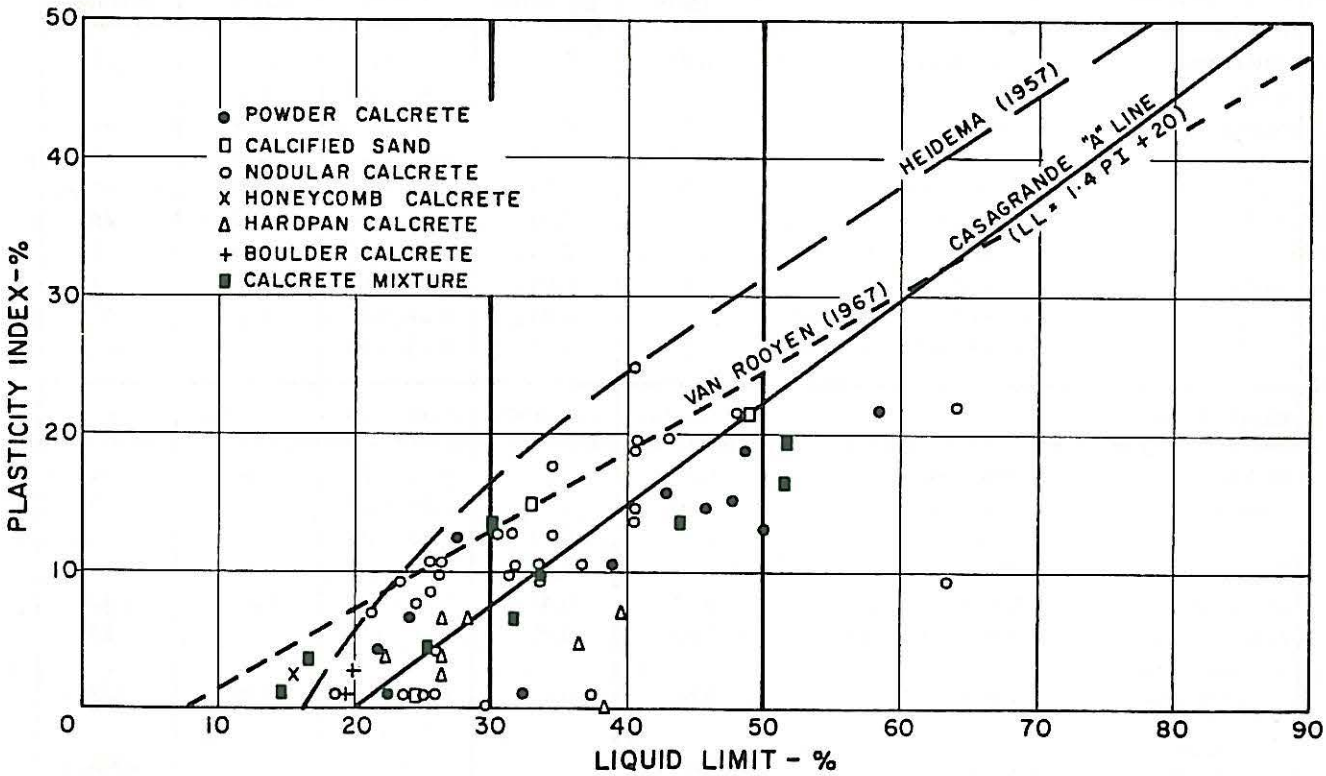

FIGURE 6

Position of calcretes on the Casagrance plasticity chart.

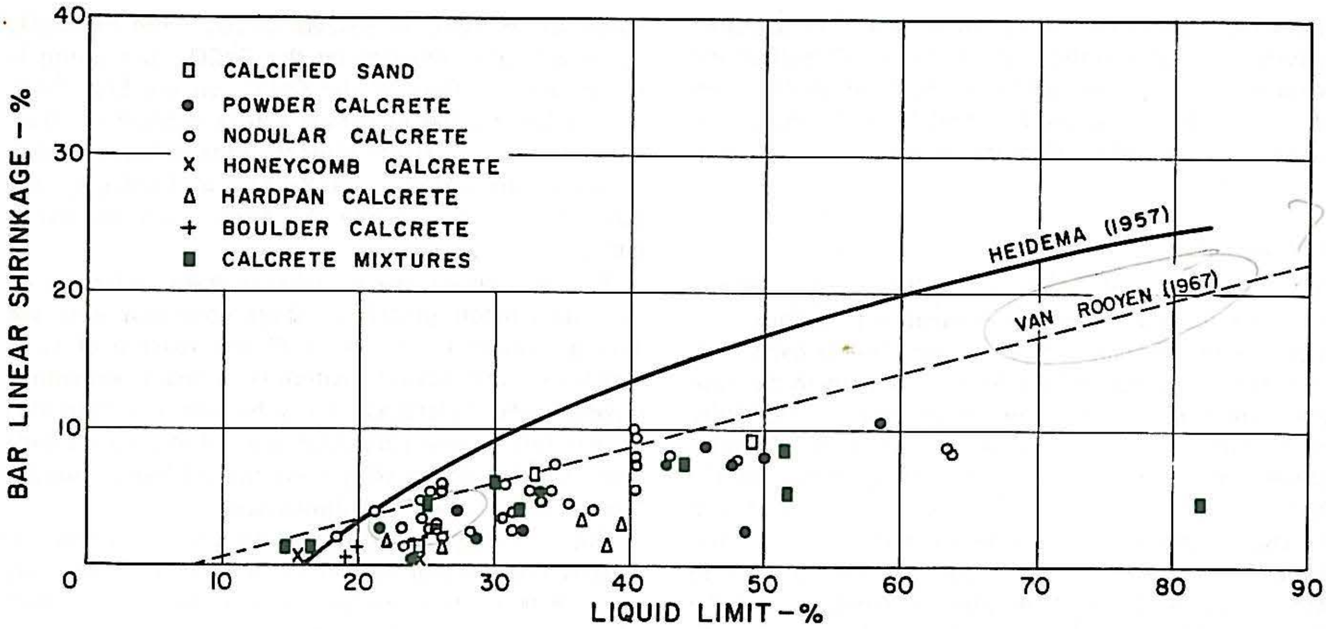

FIGURE 7

Relation between Bar Linear Shrinkage and Liquid Limit.

TABLE 2

Relations between Bar Linear Shrinkage and Plasticity Index for calcretes and other materials

Material	Locality	Mean LS/PI	Std. Deviation	Range	Correlation Coeff.	No. of Samples
Calcretes	Ondangua	0,38	0,06	0,25–0,54	0,97	67
"	Runtu	0,44	0,09	0,25–0,54	0,76	14
"	Various	0,43	0,17	0,09–1,08	0,76	54
"	Port Elizabeth	0,42	0,06	0,30–0,56	0,93	26
"	Wasser-Asab	0,50	0,10	0,16–0,70	0,86	38
"	Cape Flats	0,43	0,05	0,36–0,50	0,83	29
"	Tsumeb-Operet	0,35	0,08	0,23–0,50	0,73	8
"	Operet-Oshikati	0,46	0,06	0,23–0,62	0,96	192
"	Bloemfontein–Winburg	0,40	0,08	0,21–0,57	0,92	43
"	Lichtenburg–Deelpan	0,51	0,03	0,40–0,63	0,98	307
Weighted mean		0,4654	0,0592	0,09–1,08	0,9360	–
Mixtures	Wasser-Asab	0,48	0,07	0,30–0,60	0,84	68
Topsoils	Ondangua	0,27	0,09	0,10–0,48	0,94	39
"	Runtu	0,38	0,07	0,25–0,48	0,99	8
Mostly shales	Wasser-Asab	0,48	0,08	0,25–0,69	0,85	116
Weathered dolerites	Various	0,81	0,47	0,34–2,23	0,84	43
Weathered dolerites	Bloemfontein–Winburg	0,43	0,05	0,30–0,58	0,77	26
Various, except calcretes and dolerites	Various	0,60	0,31	0,18–4,00	0,86	233
1664 'soils' + 41 caliches	Texas[15]	0,53	–	–	–	2 005
'Soils'	Texas[15]	0,55	–	0,43–0,60	–	–
'Soils'	Transvaal[14]	0,46	–	–	0,90	–

Shrinkage Limit is unknown, pure silt-sized $CaCO_3$ has been found to possess a Shrinkage Limit of 36, and diatoms Shrinkage Limits in excess of 100.

These results throw doubt upon the validity of Atterberg Limits and/or the other test methods employed when applied to calcretes; in fact, a number of persons spoken to during the course of this investigation expressed doubt as to the proper interpretation of Atterberg Limits on calcretes. One of them* used 'apparent PI'. These doubts are well founded (see discussion below) but this term 'apparent PI' will only be used in this report when it is desired to emphasize this point – in this report 'PI' is synonymous with the PI determined in the usual way.

Bar Linear Shrinkage from the Liquid Limit is considered to be the most reliable soil constant for application to calcretes. Figure 7 shows the relationship between Linear Shrinkage and Liquid Limit for the calcretes tested compared with the published relationships for the Transvaal[14] and Texas[15] materials. It will be seen that, once again, neither of the two published curves provides a good fit and it will also be seen that calcretes tend to have higher Liquid Limits than other materials for the same shrinkage. This is probably partly due to the relatively high content of silt and fine sand-sized carbonate, which presumably contributes to the Liquid Limit, but not to the PI and shrinkage. The high Shrinkage Limits of calcretes also indicate that shrinkage ceases during drying at unusually high moisture contents (some moisture no doubt occurs in the probably porous particles) thus causing the Linear Shrinkage to be low in comparison with the Liquid Limit and Plasticity Index.

A commonly-used rule of thumb is that the PI is equal to about twice the Linear Shrinkage. Table 2 shows some LS/PI ratios for calcretes and other materials. All the materials except the 2005 Texas soils possessed PIs of less than 20. The raw data were kindly made available by a number of sources.

It can be seen that generally there is good correlation between the results of the two tests, indicating that the two tests are measuring the same property[14]. This is, in fact, merely due to the shrinkage between the different limits[1] and, as will be shown presently, the scatter is more likely to be caused by inaccuracies in the PI than in the Linear Shrinkage. It is also apparent that for the same Linear Shrinkage many calcretes tend to yield a somewhat higher PI than many other materials, especially dolerites. This is considered to be due, at least partly, to the porous nature of the calcrete particles (and diatoms), which would raise all the Atterberg Limits but which would not affect the shrinkage. In addition, the most common clay mineral in calcretes is probably palygorskite, which possesses a PI equal to calcium montmorillonite (and an even greater Liquid Limit), but is non-expansive.

Particle specific gravity

Calcretes are not made of discrete, solid particles, but rather, are composed of somewhat porous compound particles made up of smaller discrete, solid host-material particles cemented together. The bulk specific gravity of calcrete particles may therefore be much less than the specific gravity of solids, and as the bulk specific gravity also appears prone to vary with particle size, the reliability of a number of common tests is reduced. A study of this phenomenon has been attempted[1]. Although the results on the finer fractions are questionable, it was found that the coarse and medium sand-fractions of most of the seven calcretes tested did tend to possess lower bulk-particle specific gravities than the other fractions. It also seems likely that below a certain size (that of the larger host material particles) the bulk-particle specific gravity should increase, and approach that of the solid particle specific gravity, since the number of porous particles must decrease.

The mean solid-particle specific gravity of the minus 40 U.S. mesh (0,425mm) fraction of 44 samples of calcified sands, powder calcretes, nodular calcretes, and calcrete mixtures was found to be 2,66, with a standard deviation of 0,06 and a range of 2,47 to 2,80. The solid-particle specific gravity reflects the mineralogical composition of the calcrete. If quartz and calcite were the only minerals present it would vary between 2,65 and 2,71. Variations outside this range indicate the presence of additional minerals. Minerals such as opaline silica, montmorillonite, palygorskite, sepiolite, kaolinite and allophane can be expected to lower the solid-particle specific gravity while carbonates, some illites and very heavy minerals such as magnetite, ilmenite, rutile, zircon, garnet, etc. can be expected to raise it. Owing to the uneven distribution of carbonate with size fraction the solid-particle specific gravity of the minus 0,425mm fraction is not necessarily the same as that for the whole calcrete crushed and ground minus 40 mesh. The average bulk specific gravity of the minus 0,425mm particles in calcrete is unknown, but in many cases it will be considerably less than the figures quoted for particle specific gravity.

Self-cementation

An important difference between all pedogenic materials and other road building materials is that on excavation they may actually reinstate the process of developing, i.e. improving in quality – unlike a weathered basic igneous rock for example, which deteriorates with time. A number of persons both in this country and elsewhere have expressed the opinion that because of this some calcrete roads cement themselves to some extent, though conclusive evidence

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* WEINERT, H.H. personal communication

that this happens, particularly under a bituminous surfacing, is still lacking. The idea of a calcrete base cementing itself with time under a blacktop surfacing is perfectly compatible with the known origin of the material. Calcite and dolomite are slightly soluble in water, so that solution, migration and recrystallization of carbonate during suction, moisture-content and temperature changes in the base are possible. It seems likely[18] that the diurnal temperature cycle in the base will act as a 'pump' which transports soluble matter upwards. The net result should be the formation of a hard crust at the surface of the base. On all roads the compaction water will also play a role and on gravel roads rainwater will be important. Table 3 shows that during laboratory wetting and drying cycles considerable increases in CBR are possible.

It also shows that in some cases a decrease in CBR is possible, although the reason for this is unknown. The sample concerned (2223) was the only powder calcrete tested, all the others being nodular calcretes. Table 3 only shows the behaviour of the 0,1in CBRs, but 0,2in values always showed a similar trend, the percentage increases often being even greater. It may well be possible to increase the *in situ* CBR of a calcrete road layer by subjecting it to a number of wetting and drying cycles before placing the next layer or the surfacing. This also could result in the formation of a relatively impermeable crust at the top of the layer which would be of value in controlling moisture and salt migration.

TABLE 3

Effect of wetting and drying cycles on the CBR

	Four day soaked 0,1in. Mod. AASHO CBR* (%)						
No. of cycles	0	2**	5**	10**	0	10†	20†
Sample No.							
2114	56	83	95	121	64	133	122
2116	55	78	64	107	45	–	132
2223	47	41	20	38	48	24	29
2555	–	–	–	–	41	86	–
2689	21	24	41	–	–	–	–

* Minus 19mm material only, no compensation
** One cycle = soaking for one day and drying at 105°C for one day
† One cycle = soaking for two days and drying at 105°C for two days

leads us to another possible cause of self-cementation in some calcretes, i.e. the oxidation of soluble ferrous iron to ferric oxide as suggested by Aitchison and Grant[19] for ferricretes. Iron is only appreciably soluble under reducing conditions (such as in the presence of organic matter), therefore oxidized fossil pedogenic materials will not be able to undergo cementation by this mechanism. In all probability only an actively forming pedogenic material in a damp, reduced state, excavated, placed and compacted so that it is not allowed to dry out before compaction, can exhibit

TABLE 4

Effect of soil fines diatom content on the strength of lime-stabilized calcretes

Sample number	Calcrete Type	Apparent PI* %	Microfossils per gram**	Unconfined compressive strength † kPa With 4% lime
1601	Nodular calcrete	4	42 000	450
1569	Nodular calcrete	SP	21 000	360
1567	Calcified sand	NP	15 000	320
1624	Nodular calcrete	20	2 000	220
2114	Nodular calcrete	14	1 200	110
2116	Nodular calcrete	11	900	130

* Before stabilization
** In the minus 40 U.S. mesh (0,425mm) fraction
† Seven-day soaked unconfined compressive strength of 4in x 2in cylindrical specimens compacted from soil mortar (–10mesh, 2mm) to 1840 kg/m^3 dry density with an admixture of 4% high calcium slaked lime.

An increase in CBR from around 80 to around 140 merely on curing as for stabilized materials has also been reported* in some calcretes from the Karasburg area (of which 2555 is an example). A decrease in CBR on curing is also known (Table 9, p.33). This self-cementation due to this mechanism. These calcretes probably possess negative redox potentials and contain more than the usual amount of organic matter. Calcretes containing several percent ferrous and/or ferric iron are known. Nascimento *et al's*[20,21] Petrifaction Degree Test or a CBR after a one-day drying cycle at 105°C is probably suitable for detect-

* CAIGER, J.H. personal communication

ing self-cementation by this mechanism. Petrifaction Degrees of the calcretes tested ranged from 0,40 to 0,79 (1,0 would indicate complete petrifaction or cementation), which suggests that this mechanism is possible in some calcretes. In all cases, however, the Absorption Limits exceeded the Liquid Limits, whereas the test theory suggests that the Absorption Limit of a self-cementing material should be between the Liquid Limit and the Shrinkage Limit. Since the Petrifaction Degree is obtained by dividing the Shrinkage Limit by the Absorption Limit, it is obvious that the high Shrinkage Limits of calcretes affect the results, and in all probability nothing less than a Petrifaction Degree of about 0,9 is significant. None of the calcretes tested were freshly taken; all had been in dry storage for months or years before testing, so that any available ferrous iron would probably have been oxidized by the storage conditions. That the Petrifaction Degree Test is valid in practice as well as in theory was shown by adding 4% cement to two calcretes, upon which Petrifaction Degrees of about 1,0 were obtained. The Petrifaction Degree was also found to increase after a number of wetting and drying cycles[1].

Several other mechanisms also exist which could possibly in time cause an improvement in the quality of a calcrete base, particularly under the high pHs and temperatures which must be encountered in bases in the warm semi-arid environment[1].

Chemical stabilization

Some calcretes contain amorphous silica in the form of tiny microfossil skeletons, mainly diatoms, which reacts rapidly with lime to form cementitious tobermorite gel – the chief compound responsible for the strength of portland cement-soil mixes and concrete. This means that the usual rule that clayey materials stabilize best with lime, and sandy materials best with cement does not hold for calcretes. Each calcrete must be treated on its own merits since some calcretes have been known to yield a greater and more rapid increase in strength when stabilized with lime than with an equal quantity of cement. Table 4 shows how the early strength attainable with lime is controlled by the microfossil content. This was confirmed by adding diatoms to the non-reactive samples, upon which a considerable increase in strength was obtained.

The strengths attained by the most reactive samples in Table 4 are comparable with those obtained by Gregg[22] on a sand stabilized with 4% cement.

Sodium silicate is another stabilizer which will greatly improve the strength of any calcrete containing strongly calcareous fines (a strength two-thirds of that attained with an equal quantity of cement was yielded by the one calcrete tested).

Apart from the comments made in the last few paragraphs, calcretes appear to behave in a similar fashion to other materials as far as the effect of lime and cement on the soil constants and compaction characteristics is concerned.

why if Ca^{++} saturated

Soluble salts

An important property of calcretes at an early stage of development (up to, but probably excluding the hardpan stage) is that they may contain highly soluble salts such as NaCl, Na_2SO_4, $MgSO_4$ and Na_2CO_3 and relatively insoluble salts such as gypsum ($CaSO_4 . 2H_2O$). The very soluble salts migrate to the top of the base, crystallize there and, if present in excess, give rise to problems such as blistering of the surfacing and loss of density and cohesion of the upper base. The result is loss of bond between the loose base and the surfacing, with consequent cracking, scabbing and potholing. All the sulphates (except barite), but including gypsum, can react with the reaction products of lime or cement stabilization to form ettringite, which leads to still further disintegration.

Table 5 shows the range of conductivities encountered in calcretes, indicating that soluble salt problems are possible when calcified soils, powder calcretes, nodular calcretes and possibly honeycomb calcretes are used, but will be unlikely in the case of hardpans and boulder calcretes.

Calcified soils

Table 5 also shows the range of the chief roadmaking properties of the most important calcrete types. Some of the actual test results upon which Table 5 was partly based, are shown in Tables 14–17 pp. 48 to 63 in Appendix B.

Most calcified soils are either calcified sands or calcified gravels, and are easily distinguishable. The calcified gravels are generally of a higher quality than the sands and are nearly always of low plasticity, with good aggregate strength and will yield base course CBRs on ripping and crushing or grid rolling. Some will require presplitting before ripping and may not be crushable with a grid roller. Calcified sands are generally rather weak (10% F.A.C.T. value usually about 25 kN, ACV about 45%) and the grading of the excavated and placed material will be affected by this. Mechanical analysis of all the weaker calcified sands must be regarded as unreliable owing to their friable nature. Some are shown in Figure 8. The term 'apparent grading' is probably a meaningful one as far as many calcretes are concerned. Little information is available on the CBR or compaction characteristics of calcified sands, but the better ones should yield at least subbase CBRs and are A-1-b(0) materials.

The acceptability of all calcretes by the different road authorities varies greatly, an indication of the diverse opinions held and the extent to which a particular authority is forced to make use of them. The best calcified sands fall within the specification of

TABLE 5

Summary of some properties of calcretes in comparison with calcareous soils

Material type	Total Carbonate[1] as $CaCO_3$ %	Grading Modulus	Revised PRA class.	Group Index	Extended Casagrande class.	Mod. AASHO CBR %	Apparent PI[1,2] %	LS[1,2]	−0,425mm Conductivity[1,2] $m\Omega^{-1} cm^{-1}$	ACV %	10% Fact kN	A.P.T.		Mohs hardness[4]	Overall consistency	Seismic velocity m/sec.	Workability
												A.F.V. %	A.P.V. %				
Calcareous soil	1–10?[5]	Variable	Variable	Varible	Variable	Variable	Variable	Varible	Variable	Varible	Varible	Varible	Varible	Varible	Variable	300–900	Variable
Calcified sand	10? – 50	1,5? – 1,8?	A–1–b to A–2–7	0–2	GP–GF? to SF–GF?	25? – 60?	NP – 20	1–9	0,2–2,3	35? – 55?	18? – 70?	70? – 95?	20? – 50?	2–3	Med. dense or firm – dense or stiff	600? – 1200	Doze-rip
Calcified gravel	10? – 50	>1,8?	A–1–a to A–1–b	0–1?	GW to GF?	>80?	<8?	<3	<1?	25? – 35?	70? – 135?	90? – 100?	50? – 90?	3+?	Med. dense or firm – very dense or very stiff	1200? – 2450?	Rip?
Powder Calcrete	70 – 99	0,4 – 1,5	A–2–4 to A–7–5	0–13	ML to GF	25? – 70?	SP – 22	1–11	1–21	33? – 55	18 – 90?	25 – 95	5 – 65	2–3	Loose – stiff	400 – 1070	Doze-shovel
Nodular calcrete	50 – 75	1,5 – 2,3	A–1–a to A–6	0–3	SF–GF to GP	40 – 120+	NP–25	1–12	0,2–7,4	20 – 57	9 – 178	0 – 100	0 – 90	1–5	Loose – med. dense	600 – 900	Doze-shovel
Honeycomb calcrete	70 – 90	–	Rock?	–	Rock?	>80?[3]	SP+	1–3	0,1–1?	16 – 35	80? – 205	90? – 100	60? – 100	3–6	Stiff – Very stiff	900 – 1200	Rip and grid
Hardpan calcrete	50 – 99	–	Rock	–	Rock	10? – 100+[3]	NP – 7	1–3	0,1–0,6	19 – 53	27 – 196	75? – 100	30? – 100	2–6	Stiff – hard	900 – 4500	Rip – blast
Boulder calcrete	50 – 99	–	Boulders	–	Boulders	>100[3]	NP – 3	1–2	0,1–0,2	20 – 33	98 – 205	95? – 100	70? – 100	3,5 – 5	Very stiff – hard	Erratic	Rip and crush

(1) Without the loose soil in the large voids in honeycomb and boulder calcretes.
(2) On the 0,425mm fines produced after 500 revolutions in the Los Angeles test in the case of honeycombs, hardpans and boulders.
(3) Crusher run
(4) Essentially that of the carbonate cement.
(5) Up to 50% when many nodules present.

the Texas Highway Department[23] for caliche (calcrete) base, and would be acceptable in the Transvaal and South West Africa as natural subbase, and in South West Africa as lime or cement-stabilized base, but would only be acceptable to the South African Department of Transport as fill or selected subgrade. A few calcified sands fulfil the requirements of Fossberg[24] and the Orange Free State authority for gravel road wearing courses.

The main use of calcified sands has been as gravel road material. The few used so far as lime-stabilized base (Ondangua area) have performed satisfactorily for the four years that they have seen service. Calcified gravels have been little used and would be worth exploiting for base.

Powder calcretes

Powder calcretes are usually loose, but may be up to stiff *in situ*. On excavation and placing, however, the stiffer types normally degrade to a powder with a grading modulus of less than 1,0. For convenience, calcretes which have degraded to a powder on the road, but fulfil the requirement of a grading modulus of less than 1,5 and are low in host soil grains, are also placed in this category if the original type of calcrete is unknown. It is sometimes impossible to distinguish in the field between a powder calcrete and a degraded nodular calcrete.

The strength of all the particle sizes in powder calcretes is low and mechanical analyses must therefore be regarded as unreliable. They are, however, shown in Figure 9. Samples 1515 and 2223 which are regarded as type specimens of natural powder calcretes, are composed of about 75% silt and fine sand-sized carbonate. A marked sand hunch is present. This excess of fine sand coupled with a deficiency of coarse sand is a characteristic of most calcified sands, powder calcretes and nodular calcretes, which may be due to variation in bulk-particle specific gravity with size fraction (see p. 27).

The aggregate strength of the coarse aggregate is usually poor, the AFV* (p. 28) ranging from 25 to 95% and the APV**(p. 28) from 5 to 65%; the 10% F.A.C.T. values are always less than 90 kN (usually much less). There is a tendency for the aggregate strength to improve with increasing grading modulus. This is not surprising, since a higher grading modulus indicates a higher stage of development. In other words, powder calcretes of the stiffer type have progressed further

* Aggregate Finger Value
** Aggregate Pliers Value

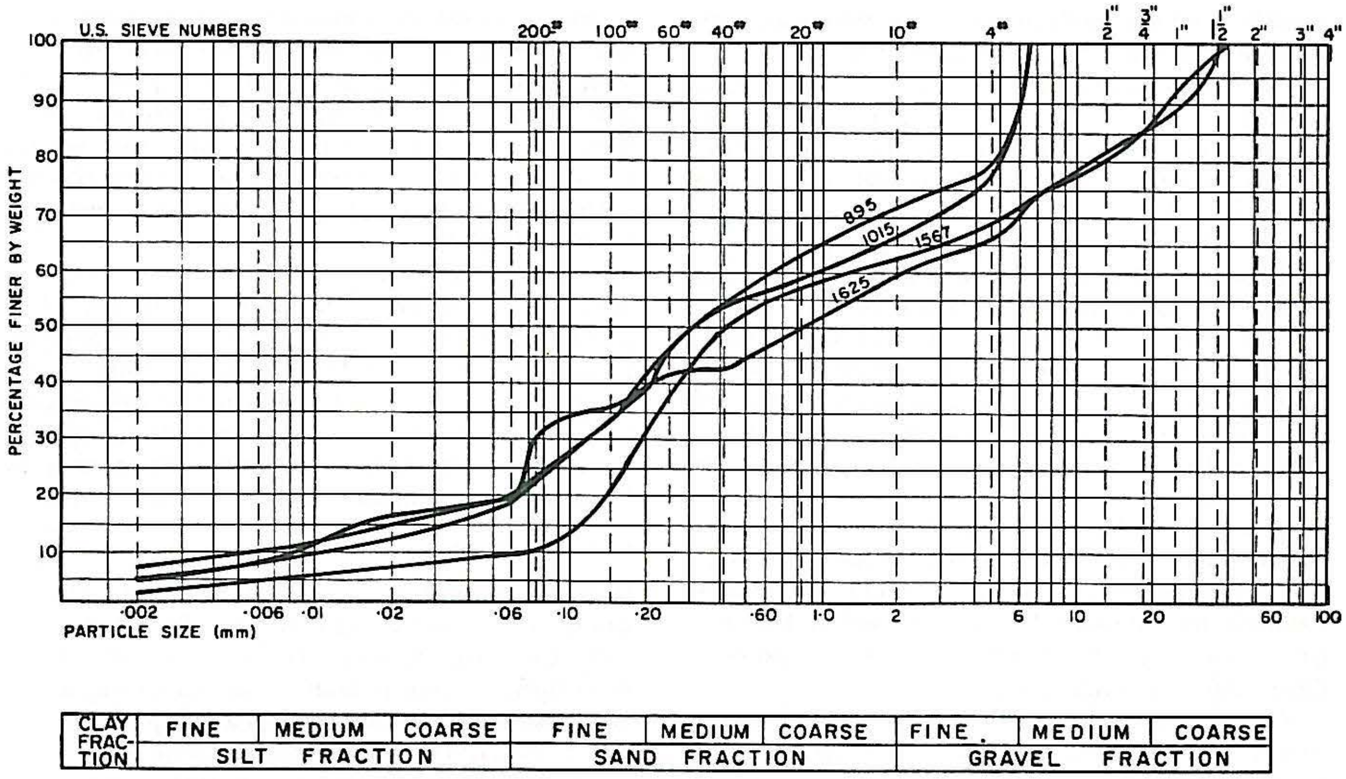


FIGURE 8

Particle size distribution of some calcified sands.

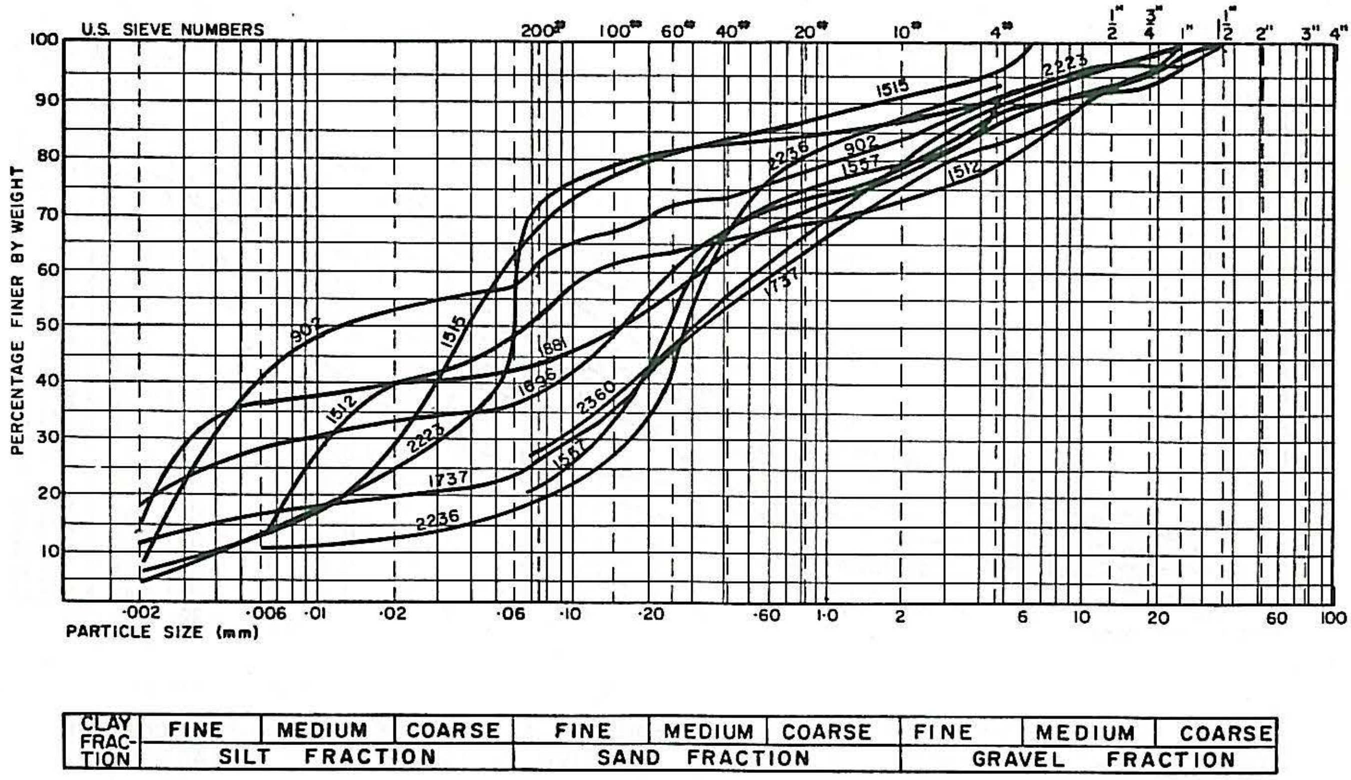


FIGURE 9

Particle size distribution of powder calcretes.

towards becoming nodular calcretes than their more powdery relatives and could also be older.

The general rule that CBR decreases with increasing PI does not always hold in the case of calcretes, as many of them possess PIs which apparently make them only fit for lower fill, although their CBRs suggest they are suitable for subbase. Although most powder calcretes will yield Department of Transport 31 kN wheel load subbase CBRs (25–80), they would fail the subbase specification on grading and sometimes also on soil constants. Little compaction data is available, but it is known that calcretes in general and powder calcretes in particular tend to have exceptionally high optimum moisture contents (up to 20% or more), probably due largely to the diatoms and other porous particles present. Modified AASHO densities probably range between 1600 to 2100 kg/m^3.

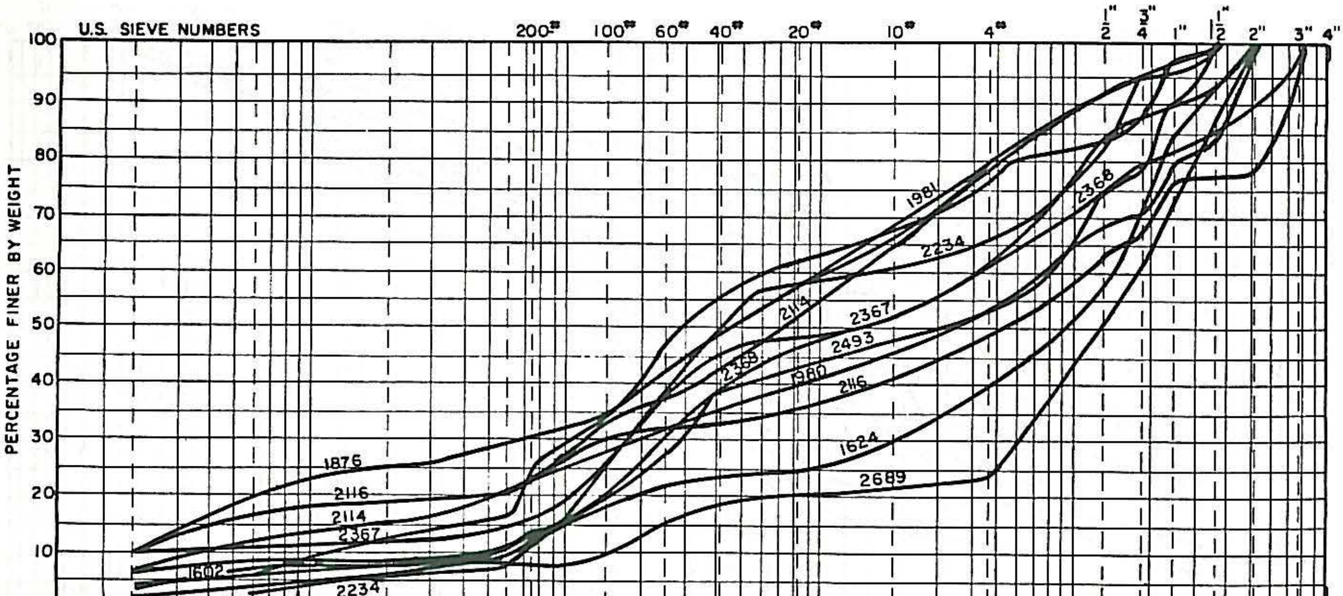
The natural powder calcretes 1515 and 2223 which are type specimens classify as A-4-(7) and A-4-(8) materials on the revised PRA scale and as ML (inorganic silts with slight plasticity) in the extended Casagrande classification.

All powder calcretes are easily worked by bulldozing or excavating. Ripping is unnecessary and scraper loaders can probably load without first bulldozing. Sometimes, however, powder calcretes underlie hard intact hardpan which may require blasting, and tend to dissipate the blast if the shot holes are drilled right through the hardpan.

Powder calcretes seldom meet the requirements of any authority for any material other than fill or, at most, selected subgrade. They are generally also unsuitable for gravel-road wearing courses (though sample 1512 is said to perform well), but may last for a while if a hard crust or 'blad' can be obtained on the surface by careful compaction with water and rolling. 'Blad' formation is said to be greatly assisted by repeated applications of water for several days while being compacted by traffic or rolling. It is probable that this is because of self-cementation due to solution and recrystallization of carbonate, so that the above procedure is theoretically sound, provided the wetting periods alternate with drying periods. Once the crust is broken, disintegration with the formation of large powder potholes is rapid.

Nodular calcretes

Nodular calcretes are by far the most useful group of calcretes for roadmaking purposes. By defining them with a minimum grading modulus of 1,5 to separate them from the more nodular powder calcretes, a minimum standard is already set for the group. There is no direct transition point between powder and nodular calcretes, but a separation at a grading modulus of 1,5 has been found to be both useful and practicable, especially as this can easily be estimated in the field. A minimum grading modulus of 1,5 is often specified for subbase for secondary roads.



CLAY FRACTION	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE
	SILT FRACTION			SAND FRACTION			GRAVEL FRACTION		

FIGURE 10

Particle size distribution of nodular calcretes.

The soil constants of nodular calcretes are particularly variable, and even higher PIs than those shown in Table 7 (p. 31) are occasionally found. Nodular calcretes formed in topographic vertisols (black clays) for example may be very well developed, with reasonable gradings and good, hard aggregate. The soil fines, however, may largely consist of black clay, which is difficult to screen out and difficult to reduce in plasticity with sand. Lime stabilization is usually the only suitable treatment for such materials. Since a host soil of mixed texture seems to be necessary for their formation, genuinely non-plastic nodular calcretes probably do not exist. Even those reported as non-plastic may have Linear Shrinkages of up to 4, while those reported as slightly plastic may have Linear Shrinkages of up to 7, probably due to the uncertainty of Atterberg Limit determinations on calcretes.

Mechanical analyses of the better nodular calcretes are probably reasonably reliable but, as will be shown presently, the strength of calcrete nodules decreases with size, so that the finer particles are more likely to degrade during sample preparation and analysis than the coarser particles. As has already been mentioned, calcrete particles are not made up of discrete particles but rather of many smaller host-material particles cemented together. This cement may be relatively weak, but the degradation process is reversible and it is quite possible that with time the degraded particles will become recemented in the road. Mechanical analyses of some nodular calcretes are shown in Figure 10. It can be seen that nodular calcretes are not particularly well graded by weights and the sand hunch is once again apparent. The kink between 0,075 mm and 0,060mm is probably due to a deficiency in the method rather than to a characteristic of the calcretes.

The aggregate strength of nodular calcretes is extremely variable. 10% F.A.C.T. values usually range from 45 to 180 kN on the 13,2mm – 19,0mm fraction, but aggregate strengths increase with size, as shown by Table 6.

The dry 10% F.A.C.T., ACV and APV tests generally seem to place nodular calcretes in the same order of merit, but the soaked 10% F.A.C.T. and the LAA at 500 revs put them in a different order. Since in practice, the aggregate will be in a moist environment under a surfacing, the latter order is probably the correct one. Unfortunately, little information is available on the Sulphate Soundness, LAA and soaked 10% F.A.C.T. values of nodular calcretes. Figure 13, (p. 23) suggests that calcrete nodules lose a higher percentage of their dry strength on soaking than do crushed hardpans and boulders. Nodular calcretes with soaked 10% F.A.C.T. values of as little as 18 kN have been used with 4% lime and 4% cement in stabilized bases which have presented no problems. This can be attributed to aggregate degradation in the up to four years of service to date. It is doubtful, however, if such low strengths would prove satisfactory in ordinary flexible bases.

Nodular calcretes will generally yield modified AASHO CBRs of at least 40 and a number of values of over 100 have been recorded. Optimum moisture contents range from about 7 to 14% and maximum dry densities from about 1825 to 2165 kg/m^3. CBR swell is usually low.

Most nodular calcretes are A-2 materials with group indices that seldom rise above 1,0. In the Casagrande classification most nodular calcretes would class as GF (gravel-sand mixtures with an excess of fines) materials, while the better ones would not quite make the GC (well graded gravel-sands with small clay contents) or GP (poorly graded gravel-sand mixtures with little or no fines) categories. According to the South African Department of Transport classification most would be semi-sand clays and only a few would qualify as gravels.

Nodular calcretes never present any workability problems unless they are overlain by a hardpan. Their normally loose consistency ensures that they can readily be worked by excavator, bulldozer or scraper loader. Those which possess particularly weak aggregates may degrade during stockpiling and compaction. Although most compaction degradation takes place during the first few passes and will be most severe in the case of gap graded materials with a deficiency of fines, an improvement in grading may result.

Few nodular calcretes meet the natural base specifications of the Transvaal and South West Africa authorities and even fewer (none of these tested) those of the South African Department of Transport, but about a third of those tested would probably be acceptable to the Texas Highway Department[23] as caliche base and perhaps a tenth to the Federal Aviation Agency[25]. About a third of those tested would be

TABLE 6

Variations in aggregate strength with size fraction tested

Sample No.	2114	2116	
Nodule size* mm	10% F.A.C.T. kN	10% F.A.C.T. kN	LAA † %
9,5 – 12,7	31	66	74,2
12,7 – 19,1	44	86	55,5
12,7 – 25,4	68	116	56,5
25,4 – 38,1	67	124	54,8
38,1 – 50,8	–	133	52,4
50,8 – 63,5	–	124	–
63,5	–	149	46,7

* All crushed to 9,5 – 12,7 mm before testing

† 2500 g samples, using 11 spheres and 500 revolutions

acceptable to the Transvaal and South West Africa authorities as natural subbase and perhaps a tenth to the Department of Transport. About two-thirds of the samples tested would probably meet at least one of the Transvaal, South West Africa, Department of Transport, or Fossberg and Gregg's[26] specifications for a lime or cement-stabilized base and about one third should be acceptable to two or more of these authorities. About half the materials tested should be acceptable as gravel road wearing course to at least one of the following: Transvaal, South West Africa, Orange Free State authorities, the Department of Transport or Fossberg[24]. Most of the samples tested were either used as a gravel road wearing course or as a lime or cement-stabilized base.

Honeycomb calcrete

Honeycomb calcretes are relatively rare, and little is known of their properties. The partially coalesced nodules are generally hard and yield good crushing values, but the overall consistency of the horizon is often such that it can be ripped and grid-rolled. With the addition of a suitable binder or lime to reduce any excessive plasticity of the host soil still filling the voids, most honeycomb types should be suitable as base or subbase.

Hardpan calcretes

Hardpan calcrete as defined is merely a hard, sheet-like, more or less massive calcrete layer usually underlain by softer or looser material. No tight restriction on hardness or strength is therefore intended, but any description should mention the consistency, e.g. "hard", "stiff", etc.

Since hardpan calcretes are merely the final stages of *development* of all other calcretes it is reasonable to suppose that much of the following information on the strength and other properties of hardpan and boulder calcretes is applicable to the aggregate fraction of other calcretes about which less is known – this is thought to be true in regard to the degree to which the shape of the aggregate (i.e. more or less spherical nodules as against sharp, angular, crushed hardpan and boulders) might affect the test results concerned. There are also some nodules only found at the bottom of some hardpans which possess a rather powdery outer surface.

The soil constants of the fines produced in the Los Angeles test of some hardpans were found to just exceed the common base specification of a Liquid Limit of 25 and a PI of 6; it is therefore apparent that care must be taken when using these materials as crushed bases, if this specification is not to be exceeded.

Conductivity testing indicates that the soluble salt content is generally low. However, the method used does not detect gypsum, and gypseous hardpans are known along the coast of South West Africa in the Swakopmund-Walvis Bay area.

The weakest form of hardpans are tufaceous hardpans at about firm to stiff. Their 10% F.A.C.T. values range from about 20 to 45 kN. Soaked values are about half this and ACVs are over 50%. With continued development, the tufaceous types become better cemented to materials yielding 10% F.A.C.T. values of 50–90 kN and in their most advanced stages all hardpans tend to become slightly silicified, yielding Mohs' hardnesses of up to 6; dry 10% F.A.C.T. values of up to 195 kN (115 kN soaked), and ACVs of 19 or 20%. Very little information is available on the unconfined compressive strength of calcrete hardpans. A few figures varying from 7 to 21 MPa are known from the Lichtenburg area and air-dry values of up to 55 MPa (41 MPa moist) are known from the United States of America[27]. Judging by their ACVs, the strongest calcretes should yield dry strengths up to 180 MPa.

The strength of indurated calcretes depends on the density up to a Mohs' hardness of about 4,5 (Figure 11); the hardness (Figure 12), the degree of crystallinity, and the amount of host soil grains still present. Very sandy calcretes are usually relatively weak. Strength seems to decrease with increasing carbonate-crystal size and calcretes in which the cementing carbonate has a crystal size greater than about 0,01mm never seem to become harder than about 3,5 (Mohs) or to possess 10% F.A.C.T. values of more than about 90 kN. All the hardest and strongest calcretes are invariably cryptocrystalline (carbonate-crystal diameter less than 0,01mm). The correlation between Mohs' hardness and strength should be particularly useful, since Mohs' hardness is a property easily determined in the field.

Figure 13 shows the effect of water on the aggregate crushing strength. In the case of crushed calcretes the dry strength is reduced by about one third on soaking.

Figure 14 shows the performance of some hardpan and boulder calcretes in the Department of Transport[28] 20-cycle sodium-sulphate soundness test compared with one Nosib (?) quartzite and one silcrete. According to the percentage passing the 10mm sieve, all the calcretes disintegrated severely in this test. The percentage passing the smaller sieves is clearly related to the 10% F.A.C.T. value.

The density of calcrete hardpans varies from about 1210 kg/m^3 in the case of tufaceous types to a maximum of about 2640 kg/m^3 in the case of the most indurated types. These density determinations were carried out on waxed hand specimens. Crushed 9,5mm –13,2mm aggregate tested according to ASTM C127 yielded apparent specific gravities from 2,49 to 2,74; dry bulk specific gravities from 1,67 to 2,68, and saturated surface dry bulk specific gravities from

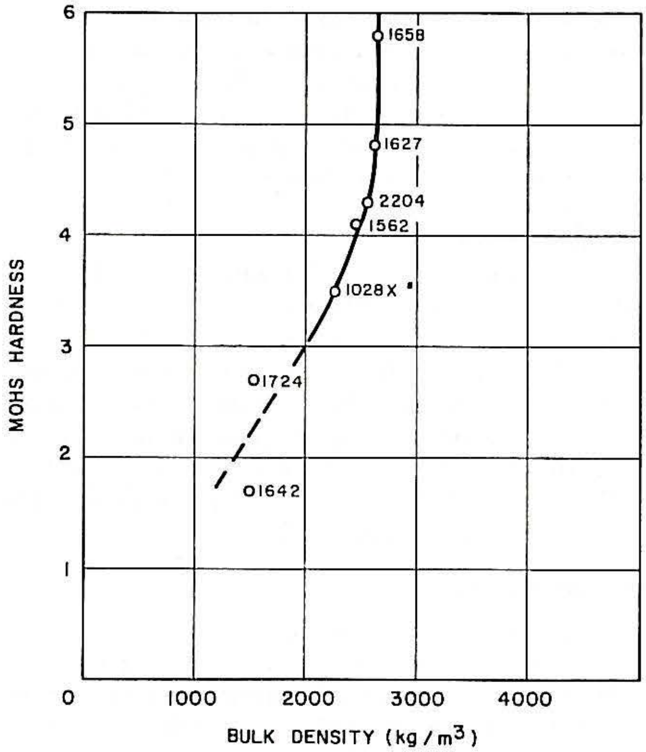


FIGURE 11

Effect of bulk density on hardness of cryptocrystalline calcretes.

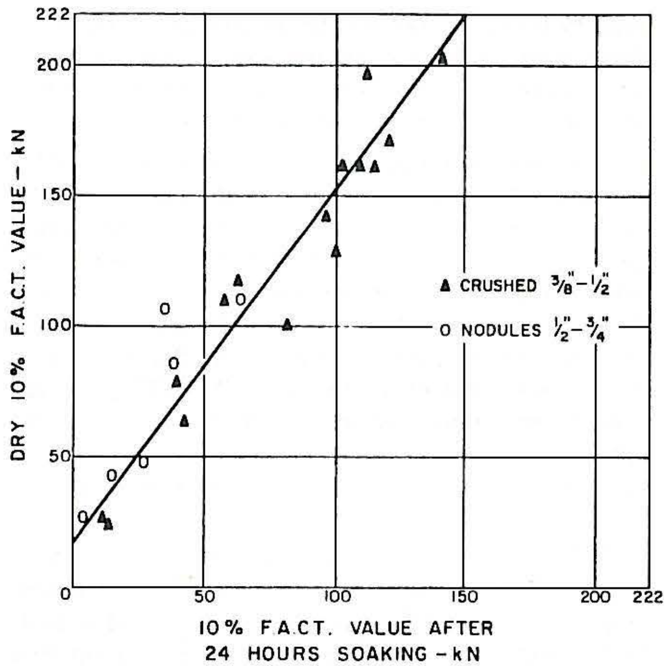


FIGURE 13

Effect of soaking on aggregate crushing strength of calcretes.

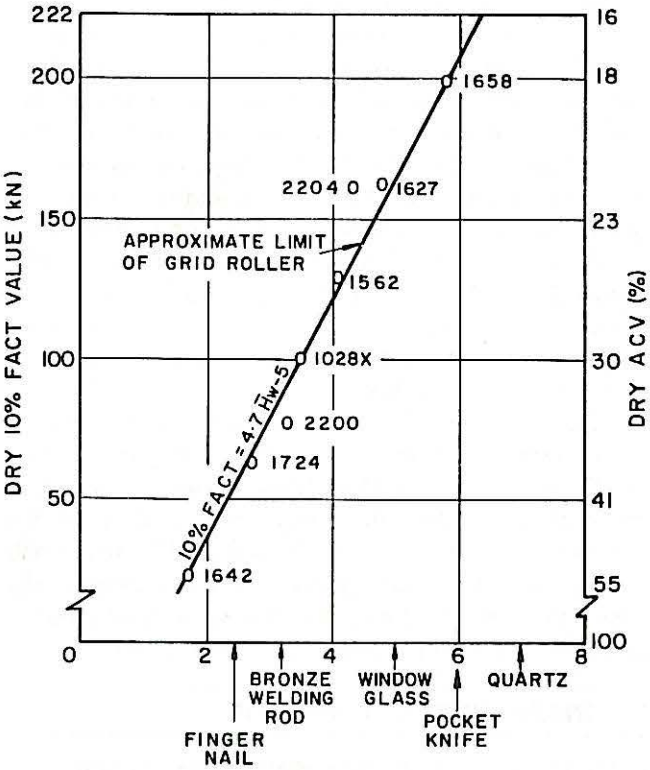


FIGURE 12

Relation between aggregate crushing strength and Mohs hardness by minerals on hand specimens before crushing

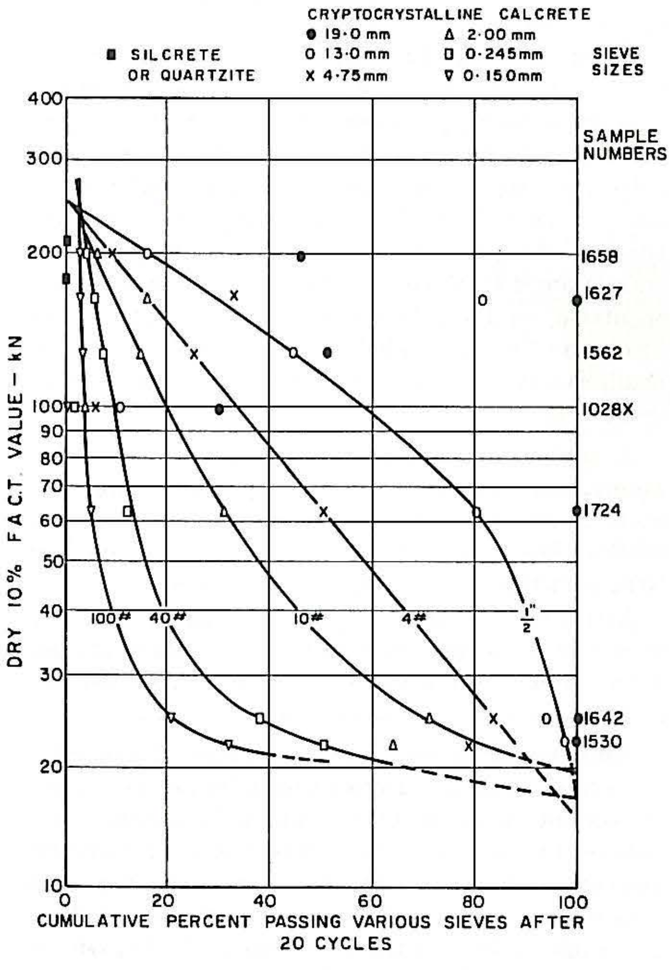


FIGURE 14

Relation between 10% F.A.C.T. and the amount of disintegration occurring in the 20 cycle sodium sulphate soundness test.

2,00 to 2,69. Tufaceous types can be expected to yield even lower values of bulk specific gravity. A fair degree of correlation exists between the density of calcretes and the crushing strength.

The water absorption of hardpan calcretes crushed to 9,5mm – 13,2mm and tested according to ASTM C127 varied between 0,4 and 20%. Only tufaceous types take up as much water as the latter figure, however, and most hardpans fall within the range 1–4%. A fair correlation exists between water absorption and crushing strength, and it would seem that by setting up an upper limit of 5% to the ASTM C127 water absorption of calcrete aggregate, would eliminate the use of all aggregate possessing a dry 10% F.A.C.T. value of less than 100 kN and a soaked value of less than 55 kN.

Calcretes do not usually yield excessively flaky aggregate on crushing and the values on *laboratory crushed* 6,7mm – 8,0mm aggregate determined according to SABS 647 : 1961[29] were found to vary between 2,0 and 8,5.

Polishing of surfacing chips can be a problem under heavy traffic conditions and carbonates are often particularly susceptible to this. Nine polished stone values (1965)[30] on *laboratory crushed* 6,7mm – 8,0mm aggregate were found to vary from 0,47 to 0,65, though the latter figure is probably unreliable owing to the rapid wear during polishing. The next highest figure was 0,60. As far as laboratory values are concerned, calcretes compare quite well with other materials in common use in South Africa and some rate better than some of the popular stones such as dolomite. At present, areas in which calcretes occur are not heavily populated, so that polishing is not likely to be a problem, therefore any calcrete which meets the other requirements of a surfacing stone could be used in these areas.

According to the detachment test[31] calcretes vary widely in their adhesion to bitumen, varying from the worst to the best yet tested at the National Institute for Road Research. Values varying between zero and 100% detachment at 10 days were obtained.

Virtually all hardpans except some of the very weak tufaceous types require ripping or blasting prior to excavation. Hardpan layers are often thin and may therefore be difficult and expensive to work.

On the basis of their aggregate crushing strengths, many calcrete hardpans are suitable for use as soil and waterbound-macadam, crusher run and concrete aggregate, and the next best ones in premix and penetration macadam, while the very best ones would just be acceptable as surface dressing chips to some authorities such as the Transvaal, but not to the Department of Transport. The variability of hardpan deposits probably makes it unwise to use them as surface dressing chips on all but the lightest trafficked roads unless very good laboratory control is available during construction. This is a case where the pocket hardness test, water absorption and Treton test would be useful. Four calcretes (two hardpans, 1724 and 1562, and two boulder calcretes, 1028 and 1627) were tested by the National Building Research Institute[32] for their suitability as concrete aggregate. All four were found reasonably satisfactory, yielding 7-day prism compressive strengths of 27–34 MPa (except for 1724 which only yielded 14 MPa) with satisfactory durability and shrinkage. It was found that there was one which, possibly, would be alkali-reactive if it were used with a high alkali cement. But in general, the materials were considered satisfactory provided the mix was properly designed and the weaker calcretes such as 1724 (10% F.A.C.T. value only 63 kN) were not used in work calling for high-strength concrete.

Boulder calcrete

Boulder calcretes are possibly the least desirable of all calcretes as far as their roadmaking properties are concerned. Until quite a late stage of hardpan weathering the boulders are still partly connected to each other so that, while they are rippable, the large soil-filled solution hollows between the hard boulders make ripping hard on the machines and even after being ripped the boulders may prove to be too large to be fed into the average road crusher and uneconomic to reduce further by hand or blasting.

Because non-outcropping boulder calcretes are essentially weathered hardpans with the weakest parts dissolved out, they usually yield excellent crushing values – values of more than 30% being seldom obtained. Similarly, soil constants and conductivities of the fines produced in the Los Angeles machine are found to be very low. Hardnesses and bulk densities are always high, the latter usually varying from 2200 to 2640 kg/m^3, the hardness normally never falling below 3,5 – 4. ASTM C127 water absorption varies between 1 and 4. Generally speaking, chemically-weathered boulder calcretes have properties slightly superior to most hardpans.

Disintegration of hardpan (usually confined mainly to outcropping hardpans in the more arid areas) produces smaller, cracked boulders which have lost much of their strength, though the interiors of the smaller fragments may still be reasonably sound and in practice the disintegrated boulder calcrete may prove to be more usable than the large hard rounded boulders produced by solution (Plate 7, p. 5).

10. ENGINEERING TEST METHODS

Calcretes differ from most roadbuilding materials in that they have an unusual composition and may exhibit unusual properties. For these reasons certain precautions are necessary when carrying out the usual standard tests, and even then the results may in some cases be of dubious reliability. In addition, non-standard tests are necessary to measure some of the unusual properties.

Preparation of the soil fines

Several methods are in use in various parts of the world for the preparation of the soil fines (the minus 0,425mm fraction) for soil constants and sedimentation analysis. Methods differ in three main features depending upon a) whether the fines are obtained simply by dry sieving or by a process involving slaking in water to release any fines adhering to the aggregate; b) whether any prohibition is placed on heating, and c) whether the prepared fines are presoaked before actual testing commences. The method probably most commonly used in South Africa[28] involves a period of slaking by boiling, and drying at about 105°C. Not all the South African road authorities, however, adhere to this procedure. At the National Institute for Road Research[33] simple dry screening is used, heating above room temperature is prohibited and presoaking overnight at about 1,5 times the liquid limit is employed.

It would appear from the methods generally used in Texas[34] where much calcrete is used, and the method specified by the United States Federal Aviation Agency[25] for caliche (calcrete) base, that the use of a wet preparation method (i.e. slaking) and prohibition of heating at temperatures greater than 60°C are necessary in the preparation of calcrete soil fines. The importance of slaking and presoaking has not been evaluated for South African calcretes, but as some S.A. calcretes possess clay-coated aggregate this factor may well be significant. The work of Gillette[35] and McDowell[36], in which some of the materials used were caliches, showed that the wet AASHO T146 and TEX-101 soil preparation methods tend to yield higher Liquid Limits, Plasticity Indices and percentages passing five micron and lower percentages passing 425 micron than the AASHO T87 dry method. Apart from a tendency for the AASHO wet method to produce more minus 425 micron material than the TEX wet method (probably due to the fact that pulverization before slaking is not allowed in the TEX method) the soil constants yielded by the two wet methods were reasonably comparable. The caliches were generally among the materials most strongly affected by the method of preparation as far as the percentage passing 425 microns was concerned (probably due to their friable nature), but did not behave much differently from the other materials as far as the soil constants were concerned. Although experimental evidence is lacking, it can be assumed that Southern African calcretes will behave in a similar manner to the United States caliches. Some work has however been carried out on the effect of dry and moist heat on the soil constants for South African calcretes. Five calcretes (four nodular and one powder) possessing nearly all the clay mineral combinations known to occur in calcretes and having apparent Liquid Limits between 24 and 36 and apparent Plasticity Indices between 10 and 15, were chosen for the study. All samples had previously been only air dried. In each case soil constants were determined after a) air drying only, b) heating for 24–36 hours at 50°C until dry, c) soaking for 24 hours at about 1,5 times the Liquid Limit and then drying at 50°C, d) heating for 24 hours at 105°C, and e) soaking for 24 hours at about 1,5 times the Liquid Limit and heating for 24 hours at 105°C. In each case the soil fines were then soaked for 24 hours at about 1,5 times the Liquid Limit before actual testing began. In every case it was found that heating the air dried or previously soaked soil fines to 105°C resulted in a lowering of the Liquid Limits by up to two percentage units, a lowering of the Plasticity Indices by 1 to 4, and, generally, in raising the Plastic and Shrinkage Limits by up to 3 and 1 respectively. Linear Shrinkage was relatively little affected, the maximum variation from the air dry value being 0,7. The behaviour of the shrinkage ratios, flow indices, toughness indices, absorption limits and degrees of petrifaction were erratic, as in some cases they increased, in some decreased and in others evinced no change. The behaviour of all samples at 50°C was much the same as at 105°C, except that the results were even more erratic and the effects less pronounced, though the PI of the powder calcrete was decreased by 5 (almost entirely because of an increase in the Plastic Limit) as against 4 after heating at 105°C. All these effects can be expected to be even more pronounced where the soil is oven dried but not pre-soaked before testing. It is apparent that if all other factors were equal, the Department of Transport method, for example, would yield lower Liquid Limits and PIs than the Texas and FAA methods while the National Institute for Road Research method would yield the highest values of all.

Liquid Limit

Methods of determining Liquid Limits vary quite widely in detail so that it is not surprising that they do not always yield comparable results. The methods in use can be differentiated on the basis of whether or not they employ a prior period of wet equilibration. Methods which routinely employ presoaking are those of the Department of Transport – since October 1963 – (12 hours near LL), and the National Institute for Road Research (overnight at about 1,5 LL). Others, such as the AASHO, ASTM, Texas and BS methods normally use the dry soil fines. The effect of presoaking on calcretes is unknown. If it does have any effect, it can be expected to raise the Liquid Limit, PI and Linear Shrinkage. The effect of mixing time is also unknown, but in view of the friable nature of many calcrete fines the Liquid Limit, PI and Linear Shrinkage probably increase with mixing time. It is believed that a 10 minute mixing time[28] is employed by all South African roads authorities. Further, the design of the Liquid Limit device and the grooving tools[37, 38] affect the Liquid Limit.

Apart from the effect of the Liquid Limit device itself and the unknown effect of presoaking on calcretes, another important factor that affects the Liquid Limit of calcretes appears to be the direction of approach, i.e. whether Liquid Limit is approached from the wetter or the drier state. The ASTM, AASHO and BS flow curve methods all proceed from the drier to the wetter state, while the National Institute for Road Research method proceeds from the wetter to the drier state. In the Texas methods the moisture content is adjusted by adding dry soil or water until the Liquid Limit is reached (hand method) or until the groove closes at between 20 and 30 blows, after which the Liquid Limit is estimated from a chart of flow curves (mechanical method). In the Department of Transport method the moisture content is adjusted by adding water or mixing until the Liquid Limit is reached. In the latter method a sample could thus undergo several cycles of wetting and drying before reaching the Liquid Limit, though it is stated that it is better to work from a slightly too wet condition to the Liquid Limit. Examination of the flow curves of about 60 calcretes determined by the National Institute for Road Research method showed that calcretes tend to exhibit an unusually large scatter of points about the best straight line. Provided, however, obviously erratic points were rejected and the test proceeds only from wet to dry, it should be possible to estimate the Liquid Limit to within ±0,5 units in about 93 per cent of cases. The maximum probable interpretation error varied between ±0,1 and $^{+7,1}_{-1,5}$. When the best straight line was merely drawn through all the points obtained, the Liquid Limit was obtained, which was within ±0,5 of the most probable value in 80 per cent of cases. The difference between the Liquid Limit obtained by this latter procedure and the most probable value was found to vary between ±0,0 and $^{+1,3}_{-2,5}$. In about 17 per cent of cases the flow curves were not linear on the semilog plot but were curved upwards, i.e. concave. It was also found that where the material was rewetted and remixed to obtain another point to fill a gap on the flow curve in eighteen out of 22 cases, that point did not fall on the line drawn through the other points and seventeen out of 22 cases fell above the line – only in two cases out of 22 did it fall below the line. This could have been due to insufficient mixing after rewetting or to what is inherent in the National Institute for Road Research method – cementation by crystallization of dissolved carbonate during the drying, the remixing being insufficient to completely break up the cementation bonding. Cementation might also increase the porosity and could also possibly be the cause of the concave flow curves. Whatever the reason, this phenomenon means that rewetting a partly dried sample effectively increases the Liquid Limit. Unfortunately the magnitude of any possible error is unknown, but inspection of the flow curves suggests that it could be up to several units. The scatter of the points about many calcrete flow curves is such that a one-point method for Liquid Limits (AASHO, ASTM, Texas and BS alternatives) must be of dubious accuracy and reliability for calcretes.

Plastic Limit

Most methods of determining the Plastic Limits are closely comparable, though small differences do exist. In the Texas method for example, dry or wet soil is added to adjust the moisture content.

Several persons engaged in soil testing in the Republic and South West Africa have mentioned that difficulty is often experienced in determining the Plastic Limits of calcretes. This difficulty has usually been attributed to the high silt content of many calcretes. The actual operator variability and reproducibility of Plastic Limit determinations on calcretes is unknown.

Plasticity Index

In view of what has been said regarding Liquid and Plastic Limit determinations on calcretes; their relatively high Atterberg Limits and Plasticity Indices in comparison with their linear shrinkage; the fact that CBR and swell do not necessarily decrease and increase respectively as the PI increases, and the fact that calcretes with substandard PIs have nevertheless performed satisfactorily as bases (see later) leads to the conclusion that the Plasticity Index of a calcrete must be regarded as relatively unreliable when compared with the PIs of other materials.

The factors which affect Liquid Limit and PI determinations on calcretes (and possibly on all materials) can probably be summarized as follows: a) methods of preparing soil fines (drying temperature and whether wet or dry screened); b) presoaking and its duration; c) mixing time and effort; d) Liquid Limit device; e) grooving tool; f) direction of grooving; g) direction of moisture change in Liquid Limit procedure and probably the number of wetting and drying cycles; h) addition of dry soil to previously seasoned soil; i) soluble salts in material and/or water (cementation on drying out, surface tension effects, solution, ion exchange, flocculation, and loss of water of hydration on heating); j) silt content, and k) accuracy of moisture content determination (often a problem in field laboratories).

Bar Linear Shrinkage

The Bar Linear Shrinkage from the Liquid Limit test does not seem to be affected by any factors other than a) method of preparing the soil fines, and b) possibly presoaking and c) the possibility of ion exchange and flocculation if distilled water is not used. Even the effect of heat was found to be relatively much less on the Linear Shrinkage than on the PI.

Because of the difficulties encountered in determining the Liquid Limit and PI on calcretes *it is suggested that more weight should be attached to the Linear Shrinkage than to the PI and Liquid Limit.* It seems very likely that as suggested by Heidema[15], the Bar Linear Shrinkage test is actually more accurate (±2% of the value obtained) than the PI. This is probably especially so in the case of calcretes. Its advantages can be briefly summarized as follows[15, 39]: the test is extremely rapid and the calcrete need only be dried to a little below the Shrinkage Limit; it does not depend on the results of two other tests; no moisture content determination is required; its cost is less than 20 per cent that of the PI; hardly any apparatus is required; the personal factor plays a very small role, and if a specially calibrated rule is used, all calculation can be eliminated. Multiple troughs are available to speed up the test even further, also bowing and cracking of highly plastic materials can be eliminated or reduced by an initial air drying or by the use of a special trough open on two sides instead of one.

While the initial Moisture Content is not particularly critical, since the Linear Shrinkage test starts from the Liquid Limit, in cases where a Liquid Limit, which is incorrectly high by several units, is obtained, this may begin to affect the Linear Shrinkage, probably increasing it slightly. The effect on the Linear Shrinkage, however, will be far less than on the PI.

Mechanical analysis

Mechanical analysis, i.e. sieve plus sedimentation analysis, is only applicable to powder calcretes, nodular calcretes and some calcified soils in their natural states. The following remarks can however be taken to apply to all calcretes actually used on the road, with the possible exception of crushed hard hardpan and boulder calcrete. The particles in these two latter cases are probably sufficiently hard and free of salts and clay and probably possess a sufficiently constant specific gravity to yield reliable and analytical results.

Problems are likely to arise during mechanical analysis of calcretes owing to: (a) breakdown of the aggregate during sieving or dispersion for sedimentation analysis; (b) clay-coated aggregate; (c) flocculation during sedimentation analysis, and (d) variation in the dry bulk particle specific gravity with size fraction.

The first difficulty may be partly overcome by the use of a standard procedure, though much must inevitably be left to the operator's discretion. It is believed that air dispersion yields better dispersion of fine particles and less abration of coarse particles than mechanical stirring devices[40].

The second difficulty can be readily overcome by use of a wet preparation technique – slaking is an additional requirement of the FAA[25] caliche method and is standard practice in Texas[34]. Experiments with four calcretes and two calcarous clayey sands showed that wet sieving produced a different grading curve from dry sieving, together with up to 15% more passing 0,425mm and 20% more passing 0,075mm. No slaking experiments have been carried out on South African calcretes.

Flocculation problems with all calcareous or saline materials during sedimentation analysis seem to be common. Methods which have been used to combat flocculation include increasing the amount of dispersant; trying a different dispersant, and removal of the carbonate with acid, or the more soluble salts by washing. Removal of very soluble salts by washing is regarded as acceptable, since they seldom make up more than one per cent, but removal of carbonate from a calcrete with acid should not be resorted to since the carbonate may be the major component and a completely false analysis will be obtained. Increasing the amount of dispersant by a factor of two to four usually results in approximate results still being obtainable, while the National Institute for Road Research method using air dispersion and sodium pyrophosphate as dispersant has been found to yield satisfactory results on all the calcretes tested during the course of this project.

The effect of particle specific gravity on particle size distribution is probably considerable in the case of calcretes. Where the coarse and medium sand fraction possesses a relatively low, and the finer fraction a relatively high, bulk particle specific gravity, the usual method of expressing the particle size distribution (by weight) underestimates the amount of coarse and medium sand and overestimates the fines present by volume. It is the particle size distribution by *volume* which determined the maximum density to which a soil can be compacted; which is only the same as the particle size distribution by weight if all particles possess the same bulk specific gravity.

The sedimentation analysis is probably the source of a similar error: porous particles settle more slowly than solid particles of the same diameter. Their effective specific gravity during sedimentation will probably approximate that of the saturated surface dry bulk specific gravity. In addition, the needle-like shape of palygorskite and sepiolite particles is rather different from the flaky shape of the other common clay minerals. Although the sedimentation analysis can probably be improved by applying it only to minus 200 mesh material (in order to exclude as many of the porous particles as possible) instead of to the minus 10 or 40 mesh, there is probably no easy way of correcting mechanical analyses by weight, since determination of the necessary bulk particle specific gravity data would be both time consuming and unreliable on the finer fractions. There is therefore probably no practical alternative to that of accepting that mechanical analyses on calcretes may be unreliable by normal criteria. It will be shown presently that calcretes with inferior particle size distributions by weight may nevertheless prove satisfactory in practice.

Sedimentation analysis is considered unnecessary for routine road work and wet sieving to 0,075mm or 0,053mm is quite adequate.

Aggregate strength and durability tests

Since the aggregate strength of calcretes is so variable, it is considered that the use of some form of test of aggregate strength is advisable when calcretes are to be used as base course, concrete aggregate, or surfacing chips. A test is also necessary in the case of gravel road wearing courses possessing less than about 50 per cent passing 0,425mm.

The aggregate tests in common use are the Aggregate Crushing Test[28,41] (ACT); the 10 per cent Fines Aggregate Crushing Test (10% F.A.C.T.)[29,42], the Treton Test[42]; the Sulphate Soundness Tests[28,29,44], and the Los Angeles Wear or Abrasion Test (LAA)[44]. Of these, only the first four are normally used in South Africa, whereas in the United States and Australia the Los Angeles Test and the Sulphate Tests are those most commonly used. These tests are applicable to all calcretes from which aggregate of suitable size can be obtained by sieving or by crushing.

Both the Aggregate Crushing Test and the 10% F.A.C.T. appear to yield meaningful results on calcretes, but no correlation between these results and the service of these materials is known. Figure 15 shows that the relation between the results of the two tests on the stronger calcretes is essentially that of Shergold and Hosking's[42] average curve, while the weaker calcretes fall between the curves for porous and dense materials.

No information is available on the application of the Treton Test to calcretes, though it is used on calcretes by some laboratories, and in view of its simplicity, it is surprising that this test is not more common. Walker and Stewart[43] concluded that 'in practice, the Treton (brittleness) and Absorption of Water tests together, will be sufficient' for the testing of road aggregates 'in nearly every case'.

The Sulphate Soundness Tests are favoured by many authorities and have the advantage that little apparatus is required. They have however been criticized by several authors[1,2,46,47] and suffer from being excessively lengthy, probably relatively poorly reproducible, and even the five-cycle test may be too severe for South African conditions. It would appear from a few experiments[1] that the test can be used to detect those calcretes which lose a relatively large proportion of their strength on soaking. A much more rapid substitute for it may therefore be the dry and soaked 10% F.A.C.T. with a limit set on the percentage strength loss on soaking, as advocated by Weinert[2].

Calcretes do not give rise to any problems as far as Los Angeles testing is concerned, but it is believed that United States experience is that both LAA and Sulphate Tests yield unreliable results when applied to calcretes. This does not appear to be confined to calcretes, for a survey in the United States by Erickson[48] showed that in fact out of 36 States which had experienced aggregate degradation, '30 reported that the Los Angeles Test, 21, the Sulfate Soundness Test and 13, the Freeze-and-Thaw Test, did not differentiate degrading aggregates from good aggregates'. Considerable dissatisfaction was expressed with these present tests and it is therefore apparent that a critical re-appraisal of the A.C.T., the 10% F.A.C.T. and the Treton is called for. Tests reported as helpful include a modified Los Angeles using no steel charge[49,50], soil constants on the fines thus produced, before and after gradings on the Los Angeles material, wet tumbling and calculation of a degradation factor based on breakdown and sand equivalent of fines[51], wet ball mill tests (TEX-116-E and Calif. 210) and others. In general, particle-to-particle abrasion (i.e. without steel shot) under wet conditions seems to yield the most reliable results as far as the prediction of the amount and nature of in-service degradation is concerned. The necessity for a wet test is supported by the findings[35,36] already mentioned in connection with dry versus wet soil fines preparation methods.

N.B

A new aggregate strength test of great simplicity has recently been described[1,13] which can be carried out in the field by any roadworker using his own fingers, a pair of standard pliers and simple arithmetic. This test, known as the Aggregate Pliers Test (A.P.T.), was designed originally for nodular calcrete gravel-road wearing courses, but may prove suitable for the field control of all wearing course and base course gravels, as well as for laboratory testing when too little material is available for the standard crushing tests. Figure 16 shows that it correlates reasonably with the dry 10% F.A.C.T.

Attention is drawn to the good correlation found between Mohs hardness and the aggregate crushing strength (A.C.V. and 10% F.A.C.T.) of calcretes shown in Figure 12 (p. 23). The hardness of a calcrete is a property easily determined in the field and in the laboratory and a special pocket hardness tester (Plate 9) the size of a small pocket knife is being developed[1]

175 kN ≈ 20 tons Sherg + Hosking Crushed rocks

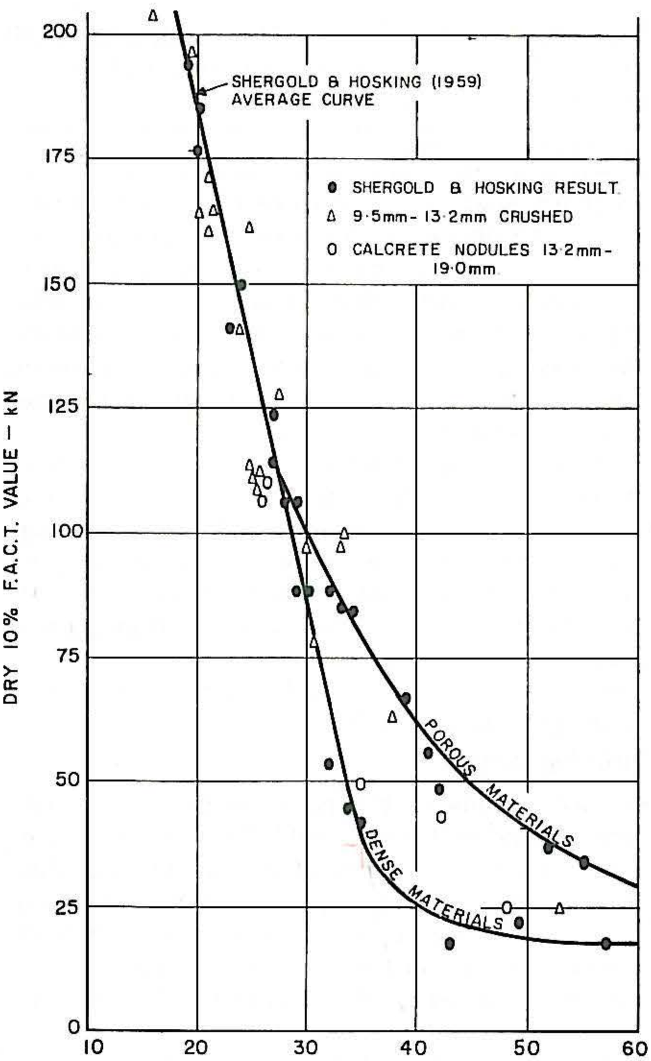


FIGURE 15

Relation between the 10% F.A.C.T. and the ACV for calcretes and other materials.

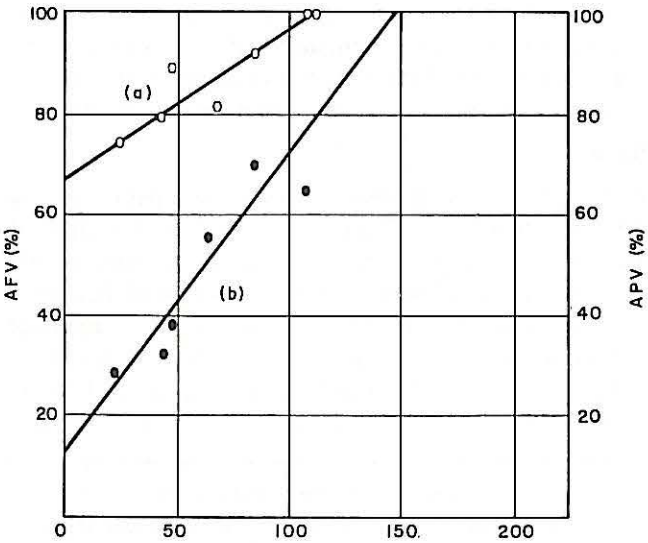


FIGURE 16

Relation between the AFV (a) and APV (b) and 10% F.A.C.T. value for calcrete nodules.

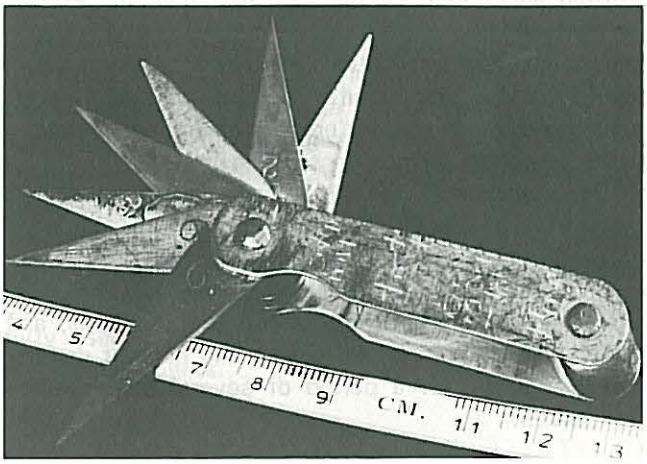


PLATE 9

Prototype pocket Mohs hardness tester. The blades are marked 1 to 6,5 (6.) and the numbers scratched on the container are Mohs numbers and their equivalent Vickers numbers, 10% F.A.C.T. values and ACVs. The device is suitable for the determination of Mohs and Vickers hardnesses of minerals and metals and the aggregate crushing strength of calcretes and possibly very fine-grained rocks in general. Length when closed 6 cms (2,5 in.), weight 40 g (1,5 oz). Photo: Graphic Arts, CSIR.

to make this test a most convenient one. Even the mere use of the average brass key (hardness about 3,5) would enable one to distinguish calcretes that would pass or fail the common base course and concrete aggregate requirement of an ACV of not more than 30%. In cases where the calcrete is variable in hardness, the use of a weighted arithmetic-mean hardness calculated from the estimated percentage by volume of the various hardness components has been found to yield good results, and was in fact used in the construction of Figure 12 (p. 23). Estimation of the crushing strength of calcretes from Mohs hardness is probably not too reliable in cases where the percentage by volume of sand grains exceed 50 per cent.

In all work on the aggregate strength of calcrete it should be borne in mind that the strength of all particles in nodular and powder calcretes and possibly also in calcified soils varies and normally increases with size of particle and development of the calcrete.

Self-cementation

This has already been effectively dealt with in a previous section. The petrifaction degree or a strength test such as a soaked CBR test after one drying cycle at 105°C for one day should be sufficient to show up any self-cementation caused by simple drying, while the usual soaked CBR test preceded by several (two to five) cycles of wetting and drying, consisting of one day soaking and one day drying at 105°C in the CBR moulds, appears to be adequate to distinguish calcretes which cement themselves as a result of the wetting and drying cycles. In all cases the experiments should be carried out in duplicate or triplicate and, in those that involve CBR testing, with the perforated plates and annular weights in place at all times, in order to prevent erosion of the exposed compacted calcrete surface on soaking.

The significance of self-cementation of pedogenic materials in practice is still not clear. Methods probably capable of detecting self-cementation in the road itself include combined undisturbed and disturbed Shrinkage Limits, petrifaction degrees and CBRs, and combined curvature or deflection and nuclear density measurements over a period of several years at the same sites.

Soluble salts

The types of soluble salts found in road construction materials in general and calcretes in particular and methods for their detection have recently been considered[1,52] and it was concluded that the soil paste conductivity method together with a qualitative sulphate test affords a very useful and rapid indicator test for the presence of all the very soluble (and most deleterious) salts. The method does not, however, in its standard form indicate the presence of the very much less soluble salts gypsum and anhydrite. Conductivity methods for gypsum and anhydrite are available, however[52]. Calcretes which are greenish in colour and those which flocculate during sedimentation analysis should always be suspected of containing very soluble salts such as NaCl.

The presence of all deleterious substances in lime and cement stabilized materials will probably be shown up by the wet-dry-brushing tests such as the AASHO T135 or possibly by testing the unconfined compressive strength before and after the wetting and drying cycles. In both tests some leaching of very soluble salts will take place, so that the results may be optimistic. Reliable results will only be obtained when gypsum or anhydrite is present.

Miscellaneous methods

The EDTA method or any method which determines Ca and/or Mg as a measure of the cement or lime content of stabilized soils is unsuitable for use with calcretes. A method which determines OH^- in the presence of CO_3^{--} such as California 338-D (Parts II and IV) should be suitable.

Apart from their inherent variability, calcretes are not known to present any problems regarding moisture-density and CBR data. Natural calcretes normally yield higher CBRs at 0,2in penetration than at 0,1in. (particularly at the higher densities), while the reverse is usually the case with stabilized specimens. Drying should be adequate to ensure the removal of moisture held in porous particles and, in all tests involving the addition of water, sufficient equilibration time should be allowed to ensure that the porous particles take up the moisture necessary for obtaining reproducible results.

11. SPECIFICATIONS

Calcretes have been extensively and successfully used for all classes of road material from gravel road material to subbase and base in the Republic, South West Africa, Texas, New Mexico, South Australia, Victoria, New South Wales, Arizona, and elsewhere. They have also been successfully used in asphaltic concrete surface courses in New Mexico and in surface treatments in South Australia.

It has long been felt by many workers in the field of road materials, both in this country and elsewhere, the higher Plasticity Indices and poorer gradings could be allowed in the case of calcretes than for most other materials, especially since, as has already been shown in this report, the Atterberg Limits, Plasticity Indices and mechanical analyses of calcretes can be unreliable, and usually seem to yield unduly pessimistic results.

Surfacing chips

Crushed calcrete hardpan possessing a maximum Los Angeles wear of 35% on the 0,625in – 0,375in aggregate and a maximum loss of 20% in the Australian Standard No. 77–1957 Sulphate Soundness Test has made satisfactory surface treatment chips in South Australia. Under light traffic conditions even crushed nodules with a Los Angeles wear of 40–50% have been used successfully*.

Nodular calcrete yielding dry 10% F.A.C.T. values of 20–55kN not surprisingly yielded 'not very satisfactory' results in Nigeria as surface-dressing aggregate[53].

In view of its generally satisfactory adhesion and resistance to polishing, there does not appear to be any objection to using crushed calcrete as surfacing aggregate provided it meets the usual strength requirements for the traffic expected and is not too porous.

Base

Relatively little has been published on specifications for calcrete bases. The only detailed work is that of Gillette[16], published in 1934, while a more recent general survey of soft limestones is that of Beaven[54]. Gillette discussed a number of caliche-based roads of varying performance in the United States. Some of his conclusions were that caliches with PIs less than 10, flocculation factors** up to 1,7 and without gel-

* SCALA, A.J., Personal communication

** The flocculation factor was defined as 'the ratio of pores to solids in a sediment formed in 24 hours from a mixture of 5000mm³ absolute volume of powdered soil solids thoroughly dispersed in 39 ml of distilled water and 1 ml of chemical deflocculant', and the colloidal gel referred to is that formed above the sediment.

producing colloids should prove satisfactory as bases under thin surface treatments and under average moisture conditions; that caliches with PIs of 10 to 15, flocculation factors of up to 2,5 and without colloidal gel should prove satisfactory under semi-arid or well drained conditions, and that the better materials even with much gel may make satisfactory bases for thicker surfacings. Gillette also concluded that the stability of caliche bases depends on the quality and quantity of the minus 0,425mm fraction and that the minimum thickness of base course should be 200mm, preferably compacted in 100mm-layers.

layers such as subbase for 'high type' highways or as base for farm to market and secondary roads, but it would appear from private communications that crushed natural gravel caliches with PIs of up to 12, Liquid Limits of up to 35 and percentages passing the 40-mesh sieve of up to 35% have made satisfactory bases for up to Texas medium-volume, primary highways. In the case of their very low volume facilities (anticipated traffic 250–600 vpd), natural gravel caliches with Liquid Limits of 45 and PIs of 15 have been successfully used under one or two course penetration-type surface treatments.

TABLE 7

Some United States specifications for calcrete bases

Authority	Specification	Max. LL %	Max. PI %	Max. Size in.	% Passing		Other
					U.S. 40	U.S. 200	
CAA[55]	Caliche Base	35	10	2	15–35	0–20	LAA < 80%
FAA[25]	Caliche Base (Item P.210)	35	10	2	15–35	0–15	–
TEXAS[23]	Caliche Base (Grade 1, Item 232)	40	12	1,75	15–50	–	–

Note: Soil constants are all determined after slaking and by preparation methods involving heating at less than 140°F (60°C) – Tex. – 101–E[34] or FAA P.210 – 2,2 modification[25] of AASHO T87.

Table 7 shows some United States specifications for caliche (calcrete) bases.

It will be noted that only a Liquid Limit, PI and three grading requirements are listed. Only in one case is aggregate strength specified. The PIs are comparable to those suggested by Gillette and it is apparent that the authorities quoted in Table 7 also consider the quality and quantity passing the 40-mesh sieve to be important. It should be noted that the above specifications are much less severe than those normally specified by other authorities and by the FAA and Texas for most other materials. Also no strength requirement (e.g. CBR or triaxial classification) is called for, although this presumably might still be specified in the contract documents. Regarding severity of traffic conditions, personal communication has revealed that most of the FAA airports which utilize the above specifications are of the utility category serving aircraft of less than 5 500kg gross mass, and that the FAA only use non-plastic caliches of superior quality for their 'high type' bases. No aggregate strength tests are specified as both the LAA and the Sulphate Soundness Tests are considered unreliable on caliches. The Texas Highway Department specification is also only applied to unstabilized

It should be borne in mind when considering Plasticity Limits in Table 7 (above) that calcretes only occur in relatively dry climates* and that both authorities quoted (the CAA is now the FAA) specify slaking methods for the preparation of the soil fines and forbid heating above 60°C. All the Texas specifications[34] allow unusually high Liquid Limits and Plasticity Indices for base and McDowell[36] has recently suggested that AASHO should consider revising M-147 to permit the use of materials with Liquid Limits of up to 35 and PIs of up to 12 in the top 150–200mm of base when the wet method of preparation is used. McDowell's report casts considerable doubt upon the reliability of the Liquid Limit and PI as indicators of the performance of bases under Texan conditions. The extent to which the soil constants of calcretes prepared by the American methods differ from those prepared by the South African methods is as yet unknown. The Australian authorities[56] also recognize that higher soil constants are allowable for some 'limestone rubbles' (nodular calcretes?).

*In Australia[56] the PI can be increased from the usual figure of 6 to 10 in areas that receive less than 375mm of rainfall while in South Africa the Regional Design Factor is 0,4 or less in all calcrete areas.

Evidence from South Africa

Tables 8 and 9 (kindly made available by the Cape Provincial Roads Department and the Cape and Port Elizabeth Divisional Engineers) show details of six roads in South Africa in which calcrete has been used as base and subbase. Actual test results on these roads are given in Table 18 in Appendix C (p. 66). On the basis of their present specifications the Cape Provincial Roads Department had, quite rightly, rejected all materials similar to those used in the first five roads for use as base. In the case of the Marine Drive, the gradings and CBRs were regarded as unsatisfactory and the soil constants borderline to high. In the case of the Port Elizabeth-Addo road, the CBRs were too low, and probably also the gradings were unsuitable for base, and the soil constants too high even for subbase. The gradings were generally regarded as unsuitable in the case of the first three Cape flats roads, even though some relaxation of grading is now allowed for the calcretes there, but not of soil constants and CBR requirements. All six roads have performed entirely satisfactorily to date and no failures have been observed.

Examination of the test results shows that every single sample listed would fail the base course grading requirements of the Department of Transport and the Transvaal Provincial authority, and probably also those of all other Provinces, or would be regarded as borderline. Of the 32 base course samples detailed in Table 18, p. 66 DE.7415–74–7 in the case of Marine Drive, all samples from the Port Elizabeth-Addo road, D.9558 and D.9564 from Landsdowne road, V.396 from Duinefontein road and V.2331 from Weltevrede road would also fail the usual maximum PI requirement of 6, and DE.7419, DE.7394 and DE.7389 would fail the usual CBR requirement of 80. All the samples listed would even fail the Department of Transport and all except DE.7385 the Transvaal grading requirements for stabilized base, although DE.7416, DE.7418, D.9559, V.2330 and V.2331 would be acceptable on a grading basis to the Transvaal if screened minus 38,1mm. Not one of the base course samples would be acceptable on a grading basis to the Department of Transport even as subbase, and all the samples from the Port Elizabeth-Addo road except DE.7380 would also fail on PI. Only DE.7401–7403, DE.7418, D.9566, D.9569, V.404, V.2331 and V.2333 (i.e. 9 base samples out of 32, or 30%) would probably qualify in the Transvaal for use as subbase. It is apparent that no authority in South Africa would use any of these materials for natural base course, while Provincial authorities would accept some of them for use as stabilized base or as subbase, presumably also only on other than national roads and other roads not designed for heavy traffic.

It is also most interesting to apply the FAA and Texas caliche base specifications in Table 7 (p. 31) to these materials. On a grading and soil constant basis only one base-course sample out of 32 (3%), i.e. DE.7418, would pass the FAA specification and only 3 out of 32 (10%), i.e. DE.7382, V.2329 and V.2333, would pass the Texas specification. Many fail only on the maximum size criteria, but if these are waived the figures are still only raised to 4 (13%) and 17 (53%) which would pass the FAA and Texas specifications respectively.

Table 9 (p. 33) summarizes the results on the six roads. The information is grouped simply according to total traffic counts. On an equivalent 80kN single-axle load basis the Port Elizabeth-Addo road and the Port Elizabeth Marine Drive probably carry about the same traffic, while the Weltevrede, Olieboom and Landsdowne roads probably carry about 1,5 times the traffic of Duinefontein road.

In the case of the less than 500 vpd traffic category, it can be seen that the worst PI (15) is exactly the same as the Texas limit of 15 for roughly the same traffic category. The worst Liquid Limit, 33, is much lower than the Texas limit of 45.

The Texas specification in Table 7 (p. 31) is probably applied to roads in the 500 – 1 000 vpd category. Here their limit of 50% passing the 40-mesh sieve agrees well with the limit of 56% possessed by most of the road materials in Table 9 (only one sample out of the 32 in Table 18, p. 66, exceeded this) and their maximum PI of 12 is not exceeded by the worst in Table 9 (9). The Texas soil preparation method probably yields less coarse minus 0,425mm material and possibly more clay (and hence higher soil constants) than the Department of Transport[28] methods used by the Cape Roads Department.

The last four groups in Table 9 are comparable with a similar Texan category – the medium-volume primary highway specification except that the 40–56% passing 0,425mm in Table 9 would be somewhat higher than the 35% allowed by Texas, and the maximum Liquid Limit and PI of 21 and 8 less than the Texan limit of 35 and 12, but once again the difference in preparation and test procedures could account for at least some of these discrepancies which are still in the correct direction. The maximum sizes in the last three roads in Table 9 vary on either side of the Texan limit of 1,75 inches (4,4mm).

Suggested specifications

It is apparent from both overseas and local experience that calcretes, which would fail the usual AASHO, British and South African base course requirements, have nevertheless made perfectly satisfactory flexible bases for light and medium trafficked roads under relatively thin surfacings. In the light of these findings it seems reasonable to relax certain requirements and the specifications suggested are shown in Table 10.

TABLE 8

Some details of six roads employing calcrete as base and subbase

Road	Year completed	Surfacing	Total thickness of imported layers	Natural subgrade	12 Hour traffic count, vpd		Approx. rainfall mm	Approx. N value	Approx. Thorn-Thwaite's moisture index	Regional design factor
					Cars	Heavies				
Port Elizabeth Marine Drive (Main Road 4)	Before 1936	20mm surface treatment to 50mm penetration	150–250mm calcrete	Sand	220^{+} 800^{+}	5^{+} 18^{+}	540 770	2,6 ?	–18 ?	0,60 0,60?
Port Elizabeth–Addo (Main Road 1)	Before 1936 to 1940	20mm surface treatment to 50mm penetration	150–200mm calcrete	Brown Soil	290^{+} 350^{+}	70^{+} 40^{+}	500 400	3 5	? –30	0,60 0,40 or 0,60
Landsdowne Road, Cape Flats	1926, surfaced 1952	2 coat surface treatment to 25mm premix	280–610mm calcrete	Sandy or grey soil	3000^{++}	760^{++}	500	3,7	–7	0,60
Duinefontein Road (No. 122), Cape Flats	1950	25mm penetration & 13mm chip wearing course	150–360mm calcrete	Sand	960† 1100†	520† 630†	500 500	3,7 3,7	–7	0,60
Weltevrede Road (No. 156), Cape Flats	1960	25mm premix	300–400mm calcrete	Sand	882$^{\bullet}$	820$^{\bullet}$	500	3,7	–7	0,60
Olieboom Road (No. 222), Cape Flats	1963	13mm chip wearing course	150–300mm calcrete 180mm lateritic(?) sandstone	Sand	564$^{\bullet}$	829$^{\bullet}$	500	3,7	–7	0,60

\+ Average of 6 day period (Monday to Saturday) in July 1967; from 0600 to 1800 hrs near each end of road. Vehicles, excluding cars, with payloads in excess of 0,5 ton counted as heavies.

++ On 12.1.67 from 0700 to 1900 hrs. All vehicles over 3 tons counted as heavies.

† At the same point on 15.1.67 and 24.1.66 respectively, from 0700 to 1900 hrs. All vehicles over 3 tons counted as heavies.

• Average over 3 days from 0700 to 1900 hrs during November 1969. All vehicles over 3 tons counted as heavies.

TABLE 9

Summary of test results on 32 samples of calcrete bases

Property	Traffic category (vehicles per 12 hour day)											
	350–400 (10, 20%>0,5 ton payload)		220–820 (2%>0,5 ton payload)		1500–1700 (35%>3 tons payload)		1400 (60%>3 tons payload)		1700 (48%>3 tons payload)		3800 (20%>3 tons payload)	
	P.E.-Addo Road	No. of Samples	P.E. Marine Road	No. of Samples	Duinefontein Road	No. of Samples	Olieboom Road	No. of Samples	Weltevrede Road	No. of Samples	Landsdowne Road	No. of Samples
Max. size (mm)	19 – 37,5	8	37,5 – 53	8	37,5 – >53	5	75,0	2	37,5 – >53	4	37,5 – >53	5
GM	1,4 – 1,8	8	1,5 – 2,2	8	1,2 – 1,7	5	2,1, 2,4	2	1,7 – 1,9	4	1,7 – 1,9	5
% – 0,425mm	39 – 56	8	28 – 56	8	48 – 74	5	22, 32	2	40 – 46	4	43 – 56	5
LL (%)	28 – 33	8	20 – 24	8	18 – 20	5	20, 21	2	18 – 21	4	17 – 20	5
PI (%)	11 – 15	8	4 – 9	8	4 – 7	5	4, 5	2	5 – 7	4	6 – 8	5
LS (%)	5,5 – 6,5	8	2,5 – 4,0	8	1,5 – 3,0	5	1,5, 2,0	2	2,5	4	2,5 – 3,0	5
LS x % – 0,425mm	254 – 330	8	83 – 168	8	111 – 159	5	44, 64	2	100 – 115	4	107 – 168	5
GI	0,2 – 0,5	8	0,0	8	0,0	5	0,0	2	0,0	4	0,0	5
Revised PRA class.	A–2–6(0) to A–2–6(1)	8	A–1–b(0) to A–2–4(0)	8	A–1–b(0) to A–2–4(0)	5	A–1–a(0) to A–1–b(0)	2	A–1–b(0) to A–2–4(0)	4	A–1–b(0) to A–2–4(0)	5
0,1" LAB. CBR (%)	47, 66	2	69, 90	2	87, 107	2	110	1	95, 98	2	112 – 134	3
0,2" LAB. CBR (%) @	61, 83	2	81, 92	2	108, 133	2	146	1	120, 125	2	133, 142	2
Rel. LAB. Compaction %	100, 100	2	100, 99	2	100, 100	2	100	1	?	2	102 – 98	3
CBR Swell (%)	0,1, 0,2	2	0,0, 0,1	2	0,0, 0,0	2	0,0	1	0,0, 0,0	2	0,0	3
Rel. Field Compaction(%)	?	0	?	0	96 – 104	5	103, 105	2	95 – 99	4	97 – 102	5
Estim. 0,1" Field CBR(%)	–	0	–	0	70, 92	2	>100?	1	90, 91	2	60 – 118	3
Estim. 0,2" Field CBR(%)	–	0	–	0	74, 120	2	>120?	1	114, 103	2	77 – 142	3

E.M.C.

D.C.P ??

NOTE: Basic tests carried out by the Cape Provincial Roads Dept. according to the methods of the Dept. of Transport (1958). All compaction data Mod. AASHO.

If the specifications in Table 10 are only applicable to calcretes and not to all materials in the warm semi-arid and arid environment, there must presumably be some minimum value of fine carbonate content below which the material will not behave as a calcrete. This limiting carbonate content is not known with any certainty (and it may even be less important than the presence of palygorskite or sepiolite), but limiting the total carbonate content (calcite plus dolomite) of the minus 0,425mm fraction to a minimum of 5–8% CO_2 should be safe. Until more is known it would be advisable to apply the authority's normal specifications when the CO_2 content drops below 5–8%. A qualitative test on the soil fines with hot dilute hydrochloric acid (both calcite and dolomite effervesce strongly) or some simple semi-quantitative test should suffice. An alkaline pH (more than about 7,5) might even suffice. Calcrete soil fines of an overall white, light grey, cream or pale green colour are normally strongly calcareous; those brown or red in colour are normally low in carbonate content.

Table 10 is a blend of Table 7, p. 31, Table 9, p. 33 and existing South African requirements and the derivation of most of it should be obvious. Detailed grading requirements do not appear to be warranted. Merely specifying a maximum size and a Grading Modulus or the percentage passing 0,425mm is probably sufficient. The Grading Modulus is a quite well accepted method of providing a simple control on the grading in the Transvaal and South West Africa and Figure 17 suggests that the correct size and Grading Modulus are sufficiently closely related to be inter-

TABLE 10

Suggested interim specifications for calcrete natural gravel or crushed natural gravel bases

PROPERTY	Expected Traffic Category (vpd) <20% >3 tons			
	<500	500 – 1000	1000 – 2000	2000 – 4000
Max. size (mm)	19 – 37,5	37,5 – 53	37,5 – 53	37,5 – 53
Min. GM	1,5	1,5	1,5	1,5
Percentage passing 0,425mm by weight	15 – 55	15 – 55	15 – 55	15 – 55
Max. LL (%)	40	35	30	25
Max. PI (%)	15	12	10	8
Max. BAR LS (%)	6,0	4,0	3,0	3,0
Max.–0,425mm cond., Cl^- ($m\Omega^{-1}/cm$)	1,8	1,8	1,8	1,8
Max.–0,425mm cond., SO_4^{--} ($m\Omega^{-1}/cm$)	0,7?	0,7?	0,7?	0,7?
Max. Group Index	0,5	0,0	0,0	0,0
Worst revised PRA class.	A-2-6	A-2-4	A-2-4	A-2-4
Max. BAR LS x % –0,425mm	320	170	170	170
Min. Dry 10% F.A.C.T. Value (kN)	–	80?	110?	110?
Max. Dry ACV (%)	–	35?	30?	30?
Min. Soaked/dry 10% FACT value (%)	–	50?	50?	50?
Max. Water absorption (%)	–	–	5?	5?
Min. Dry APV	50?	60?	70?	70?
Min. 98% M.AASHO 0,1'' CBR (%)	60	80	80	80
Min. 98% M.AASHO 0,2'' CBR (%)	80	80	100	100
Max. M.AASHO CBR Swell (%)	0,5	0,5	0,5	0,5
Min. Rel. Field Compaction (%)	98	98	98	98

Notes:

1. Department of Transport (1971) routine test methods to be used.
2. Traffic counts are per 12 hours (daylight) day.
3. Conductivity is measured on the saturated paste at 16°C at a consistency between about 1 and 30 blows of the Liquid Limit device.
4. All aggregate strength tests are carried out on 13,2mm–19,0mm aggregate and soaked tests after 24 hours soaking

changeable in the case of calcretes. The Linear Shrinkage is regarded as the most reliable calcrete soil constant and the values in Table 10 are those suggested in Table 9; except for the figure of 6 in the lightest traffic group, the figures of 4 and 3 suggested are in line with what is usually required in South West Africa (4) and the Republic (3) for all classes of surfaced roads. The Liquid Limits and PIs required by these authorities are always lower than those suggested in Table 10 and were decided on after consideration of: Table 9, the values allowed by Texas; the probable reliability of the two tests; and the range of Liquid Limit and PI possessed by calcretes with the Linear Shrinkages shown (Table 2, p. 14). The soil-paste conductivities[1,52] assume that the construction water is non-saline and apply to the full depth of base course. If the water is saline (indicated by a conductivity of more than about 0,5–1,0 mΩ^{-1}/cm for example) the limits should either be adjusted to allow for the amount of salt to be added by the compaction plus any curing water. The limits for Linear Shrinkage times the percentage passing 0,425mm give an indication of the CBR[1] and the potential volume change of the soil fines.

No information is available on the aggregate strengths required of calcrete bases and the figures given are intended as rough guides only. The dry 10% F.A.C.T. value of 110 kN in the 1 000 vpd traffic category and its equivalent ACV are the usual Department of Transport requirements for national roads. The 80 kN value and the percentage ratio of soaked to dry strength are merely guesses. Water absorption after 24 hours or the A.P.T. may prove useful as a rapid field control test as a substitute for an aggregate crushing test. In view of the few CBR results available and in the absence of information concerning the practical importance and rate of self-cementation, it is not as yet feasible to relax CBR or density requirements, though the application of a 0,2" CBR of 80 in the lightest traffic category would be a relaxation in South Africa (though not in Britain or the U.S.A.). The maximum size, percentage passing 40 mesh (or the Grading Modulus), Linear Shrinkage, conductivity, CBR, swell and relative compaction are regarded as essential criteria. All other are regarded as optional at this stage of our knowledge of these materials.

The question of surfacing thickness is not too clear. Table 8, p. 33 suggests that a surface treatment may often be adequate but the surfacings do vary by up to 50mm of penetration macadam in some cases. Until more information is available it would probably be safe to use the authority's normal surfacing for the traffic expected. The information available does not permit minimum thickness of base course to be suggested as in most cases the entire imported cover used was of a similar quality to the base. Until further information is available it seems reasonable to suggest that a 150mm base be used in the three lower traffic categories and an 200mm base in the highest traffic category underlain by a minimum subbase thickness of 75mm in the case of the lowest traffic category and 150mm in the case of the other three traffic categories, and that the actual thickness of subbase and lower layers required be calculated by the usual CBR design procedure.

As far as crusher run and macadam calcrete is concerned, in the absence of any information to the contrary it would be wise to specify the usual grading and aggregate strength required by the authority concerned. The soil constant limits for the heaviest traffic category in Table 10 should be adequate provided the soil fines are calcareous. The conductivity limits required by the authority should also be applied, for, although hardpans and boulders are not normally saline, the binder could be. The aggregate strength requirements for the heaviest traffic category in Table 10 should be adequate for calcrete crusher runs.

If a calcrete is to be mechanically stabilized with non-calcareous fines the authority's usual grading and soil-constant requirements should be specified.

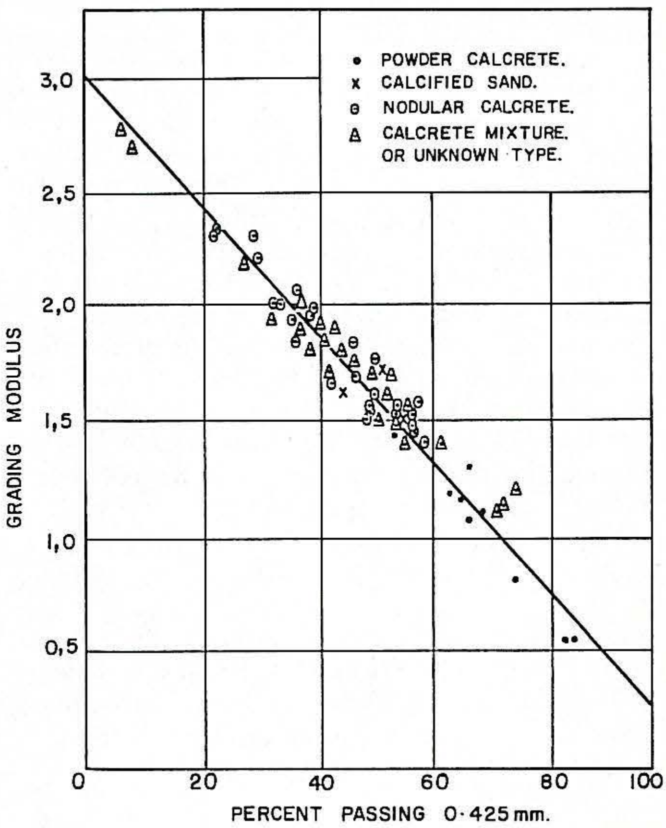


FIGURE 17

Relation between Grading Modulus and percentage passing 0,425mm.

Lime and cement stabilized base

In view of the findings for natural bases it is also reasonable to relax some of the requirements for lime and cement stabilized calcrete. Those set out in Table 11 seem reasonable, but no information is actually available on the long term performance of calcrete bases to such a specification.

As far as durability of lime and cement stabilized calcretes is concerned, the high pH environment in a calcrete base and the generally dry climate in which calcretes occur is favourable towards the preservation of the stabilization reaction products. Troubles due to salt action will be avoided if the specifications suggested are adhered to. Aggregate strength requirements are unknown, although they will be less severe than for an unstabilized base, and will also depend on the nature of the fines produced by aggregate degradation and on the grading and density of the stabilized layer.

No detailed information is available on the bituminous stabilization of calcretes, though it is believed that bituminous stabilization of these materials has been successfully employed in New Mexico.

Subbase and lower layers

Less information is available on specifications for calcrete subbases than for bases. In practice, the Texas caliche base specification in Table 7, p. 31 is applied to subbase for 'high type' highways and probably to both base and subbase for their farm to market and secondary roads. Tables 8, p. 33 and 18, p. 66 are of little help, as the subbase is in most cases of comparable quality to the base. It is apparent therefore that each base specification in Table 10, p. 34 can be used for both base and subbase. Applying the same specification to both base and subbase seems to be common United States practice (although McDowell[36] has suggested maximum Liquid Limits and PIs of 35 and 12 and 40 and 20 for the top 150–200mm of base and for subbase respectively in Texas).

Further information comes from the Western Transvaal, where the Schweizer Reneke-Bloemhof road (built in 1962), has successfully employed a calcrete subbase under a cement-stabilized alluvial gravel base between milestones 15,1 and 17,7. Seven test results carried out during the construction of this length of road by the Transvaal Roads Department

TABLE 11

Suggested interim specifications for lime and cement stabilized calcrete bases

Property	Requirement
Max. particle size Max. apparent % passing 0,425mm Min. apparent Grading Modulus	53mm or less 55? 1,5?
Max. Soil Constants	Not to exceed the appropriate natural base requirement (Table 10) after stabilization. Liquid Limit requirements can probably be waived for pavements not subject to frost heave.
Max. soluble salts	Conductivity limits as in Table 10 before stabilization. Gypsum and anhydrite limited to 0,24% SO_3 if the apparent PI or the minus two micron content of the material before stabilization is greater than 13 and 15 respectively, otherwise 1,0% SO_3.
Min. aggregate strength	As for one traffic category below that which would be specified for natural base in Table 10
Stabilized strength CBR swell and relative compaction	As required (normal requirement of the authority concerned.)
Durability	In order to assess general durability and effect of salts, particularly gypsum and anhydrite, a reasonable requirement is probably that the compacted specimen of cement stabilized calcrete or lime stabilized calcrete showing high early strength should not suffer a weight loss of more than 14% for granular soils, 10% for the more plastic granular and silty soils and 7% for clay soils after having been subjected to 12 cycles of the AASHO T135 wet-dry test. A maximum limit of 2% on the volume change after 12 cycles may also be useful.
Delay between mixing and compaction	As for cement stabilized base in the case of cement stabilized calcretes and lime stabilized calcretes showing high early strength. As for lime stabilized base for lime stabilized calcretes that do not yield high early strengths.

were as follows: LL: very low through 23 to 56, PI: 0–19, LS: 0–10, per cent passing 0,425mm: 20–45%, LS x % minus 0,425mm: 0–320 (mostly 200–300), Modified AASHO densities: 1745–1790 kg/m^3, and OMC*: 14–16,5%. Relative field compaction was 95–103% and field CBRs all 25 plus. Traffic on this road is light, of the order of approximately 120 vpd.

In view of the above and the findings in South Africa with regard to what is satisfactory as base course, it is suggested that the base specification of the lightest traffic category in Table 10 should be adequate as subbase under any flexible base. Compaction, CBR, swell (if required) and Group Index requirements can be lowered to the usual subbase requirements.

In the absence of information regarding the distance which soluble salts can migrate in the road structure, it would be advisable to apply the conductivity limits in Table 10 to the subbase as well. If the completed base is to be completely non-saline (–0,425mm conductivity less than about 0,1 mΩ^{-1}/cm), these limits could probably be relaxed somewhat (perhaps doubled). Aggregate strength requirements are probably not important, and for light and medium trafficked roads a specification is probably unnecessary. For example, in West Australia 'coastal limestone' (aeolianite or dunerock – most of them probably equivalent to a non-plastic calcified sand) with dry 10% F.A.C.T. values down to 10kN is successfully employed as subbase under thin crushed-stone flexible bases and thin bituminous surfacings.

No information is available on specifications for calcretes for the layer of selected subgrade, but it seems reasonable to relax the apparent Grading Modulus requirement to 0,8–1,0 (or 72–80% passing 0,425mm); apparent PI to about 17; Bar Linear Shrinkage to 7,0; and Group Index by one unit. Group Index requirements for the rest of the fill could presumably be similarly relaxed by one unit. It would probably be advisable to apply subbase conductivity limits to the selected subgrade and perhaps even to the upper subgrade until more is known of salt migration in roads. *In every case the usual CBR, swell and compaction requirements should be maintained.*

*Optimum moisture content

Gravel road wearing course

A good calcrete gravel road will possess a hard surface crust or 'blad', probably formed in a similar manner to natural hardpan which can normally be obtained by compaction with water. This crust enables the road to last for some time without grading, since it does not corrugate or raise excessive dust, although it may tend to pothole, after which it breaks up rapidly if not maintained; excessive grading also damages the crust. A useful form of protection is a thin (25mm) layer of Kalahari-type sand.

It is important to distinguish between a gravel and sand-clay when choosing a calcrete for a gravel-road wearing course. In the case of gravels, the aggregate strength has been found to be the greatest single factor determining the performance of calcrete roads and specifications based solely on the results of a simple pliers test have been suggested[13]. In the case of sand-clays, the aggregate floats in a matrix of fines and its quality becomes of less importance, while that of the fines becomes all important. In this respect, information from a number of sources[56,58,59,60] suggests that unsurfaced calcrete roads can tolerate poorer gradings and higher Plasticity Indices than many other materials. The results of a recent study[1] confirm this. Existing specifications are not, however, entirely suitable for calcretes, in that, while they do succeed in eliminating bad materials, they do this only at the cost of rejecting many good calcretes.

Table 12 and Figure 17, p. 35 show a suggested specification for calcrete wearing courses which gives a good fit to the data available on 29 satisfactory, seven slippery and/or rutting, five powder-holing

TABLE 12

Suggested soil constants and aggregate strength requirements for calcrete wearing courses

Property		Per cent passing 0,425mm by weight			
		15–45	45–55	55–65	65–75
LL	(%)	65–25	45–20	40–20	30–18
PI	(%)	23–8	22–5	13–8	13–5
LS	(%)	9–3^2	8–2	5–3^2	5–2
LS x % – 0,425mm		380–100	420–100^3	330–100^3	330–100^3
AFV	(%)	>65	>60^1	–	–
APV	(%)	>25	>20^1	–	–

(1) An aggregate strength requirement is probably unnecessary for materials possessing more than 50% passing 0,425mm.

(2) Linear Shrinkages down to 2 are permissible if some looseness and dust is tolerable.

(3) Values down to 70 are permissible if some looseness and dust is tolerable.

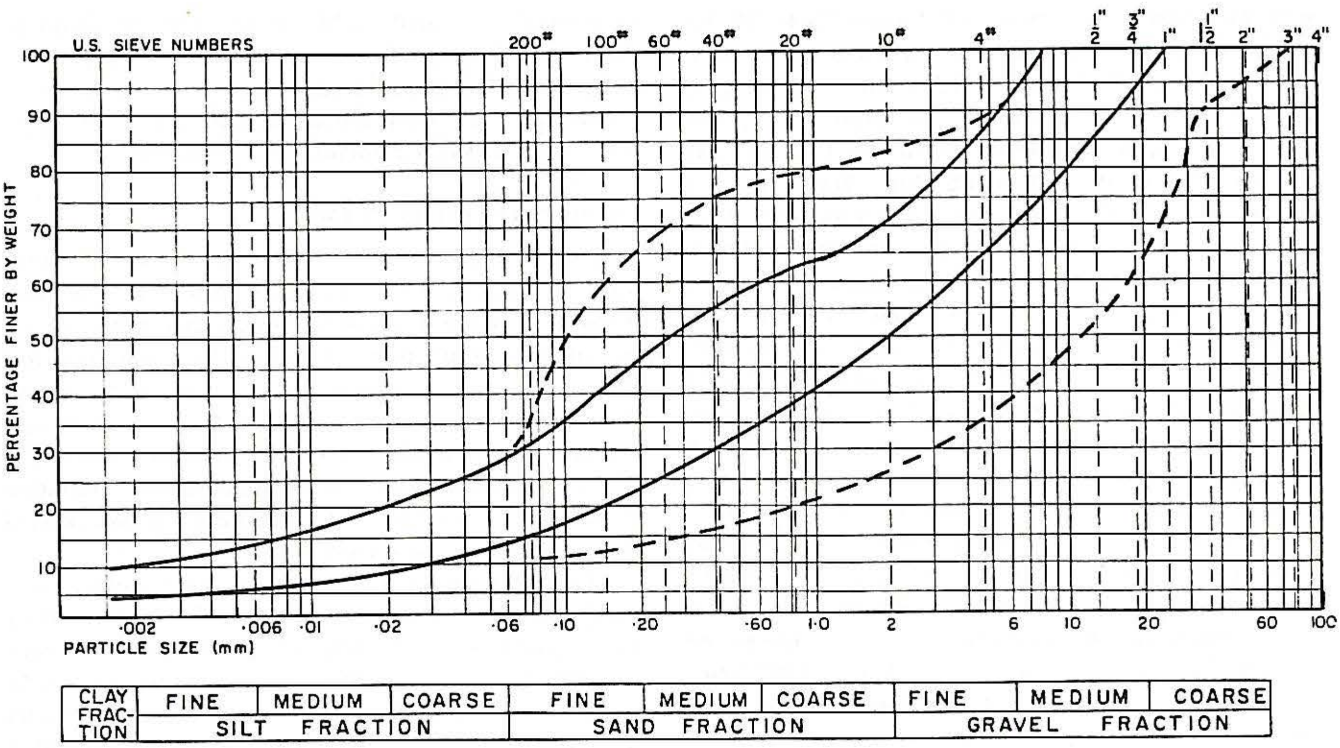


FIGURE 18

Probably desirable particle size distribution of calcrete wearing courses (solid) and the known satisfactory range (dashed).

and two potholing calcrete wearing courses from the Western Transvaal and South West Africa[1]. The specification is applied as follows:–

1. The material should possess a particle size distribution falling within the outer limits of the envelope on Figure 18.
2. The specifications in Table 12 are then applied according to the percentage passing 0,425mm.

It will be observed that a wide tolerance in the grading is permitted, but that the soil constants allowable depend on the amount passing 0,425mm and that an aggregate strength test is required on the coarser calcretes. The latter is necessary to ensure that the coarser materials do not degrade under traffic to a material with unsuitable soil constants for a sand-clay wearing course. If the soil constants in Table 12 are exceeded, the road will be slippery when wet, while if the minimum requirements are not met, the data suggest that the road will be dusty. The maximum value of Linear Shrinkage multiplied by the percentage passing 0,425mm ensures a reasonable CBR which will prevent rutting when wet, while the minimum value probably ensures that the material possesses sufficient cohesion when dry to prevent looseness and corrugation and to minimize dust. The very best all-weather calcrete wearing courses will probably fall within the inner (solid) envelope on Figure 18; possess a maximum size of 19mm; a Linear Shrinkage between 3 and 6; an AFV in excess of 65%; an APV in excess of 25%; and a value of LS x % minus 0,425mm of between 100 and 200. As there is a high probability that a calcrete with a value of Bar Linear Shrinkage multiplied by the percentage passing 0,425mm of less than 200 will possess a modified AASHO CBR of at least 60, this upper limit of 200 should be specified when heavy traffic is expected.

The specifications should be generally applicable to all calcretes in Southern Africa, though as in the case of base course, it may be advisable as an interim measure to restrict their application to those calcretes possessing strongly calcareous fines, which inspection, or a qualitative test with hydrochloric acid should suffice to indicate. In areas receiving more than about 500mm of rainfall it may also prove desirable to lower the maximum allowable PI by one or two units and to lower the maximum allowable Linear Shrinkage by 0,5 if slipperiness is to be avoided.

When minimal testing is to be undertaken the testing can probably be simplified by omitting the Liquid Limit and PI determination. As nearly all calcretes with less than about 50% passing 0,425mm possess a Linear Shrinkage between 2 and 9, it is even possible to omit the Linear Shrinkage Test and merely apply the Aggregate Strength Test (see[13]) in such cases.

In all cases, the importance of proper compaction is emphasized.

Dust palliatives

When a hard crust or 'blad' has not been obtained, calcrete gravel roads can be the dustiest of roads. A number of dust palliatives are available[61] and a locally available waste product, sulphite lye, has been found to be particularly effective for laying dust on calcrete roads[62]. It would appear, however, that, except in rare cases, the use of dust palliatives is not an economic proposition in the calcrete areas of Southern Africa. Attention should rather be paid to the proper selection of gravel and its compaction to attain a hard surface crust. A good crust can be just as effective as a dust palliative. In view of the savings it is possible to effect by using the suggested relaxed calcrete specification, it will be economic to surface roads carrying less traffic than normal.

12. CONCLUSIONS AND SUMMARY

1. Calcretes contribute one of the most widely used classes of road materials in Southern Africa. They are not sedimentary rocks, but form by growing in the soil itself and are members of the pedogenic group of road materials.
2. They possess a chemical and mineralogical composition which differs greatly from that of most other road materials and can therefore be expected to exhibit some unusual properties.
3. For engineering and most geological purposes the calcretes of South Africa can be classified into six basic types: calcified soils, powder calcretes, nodular calcretes, honeycomb calcretes, hardpan calcretes and boulder calcretes. The classification represents particular stages of calcrete development and each stage differs significantly in its engineering properties from each of the other stages.
4. Calcretes of roadmaking quality only occur in the drier areas of Southern Africa, in general where the annual rainfall is less than about 550mm or the Weinert N-value is more than 5.
5. Calcretes in South Africa can be divided into two basic types: (1) ground water calcretes formed by deposition of carbonate in the host material above a shallow water table, and (2) pedogenic calcretes which are formed by downward leaching of carbonate from the upper soil horizons and its deposition in the lower horizons. Both these types proceed through definite developmental stages. These stages are used as a basis for the calcrete classification referred to above.
6. The mechanism of calcification is not as is commonly supposed direct evaporation, or even transpiration, but by a decrease in solubility brought about by the loss of carbon dioxide due to a decrease in the pressure of the pore water (increase in suction or pF) which is mainly, although indirectly, caused by transpiration and evaporation.
7. Geologically calcretes are very young. The oldest calcrete important in roadbuilding is probably late Pliocene (about 2–5 million years old), a younger calcrete used is probably Second Intermediate (9 000–15 000 years B.C.?) in age while the youngest may still be forming at the present time.
8. At present the best solution to the problem of prospecting for calcretes lies in the combined use of airphoto interpretation, topographic setting, plant indicators, and a certain rapid probing device. One-hundred per cent success in detecting all calcrete deposits within a given area and within a reasonable time, is probably not possible at the present time.
9. The engineering properties of a calcrete depend on the nature of the host material and the extent to which the host material has been cemented and replaced by the carbonate (i.e. the stage of development of the calcrete). The carbonate fraction is decidedly beneficial, as it always acts as a non-plastic mechanical stabilizer, often as a cementing agent, and ensures that the clay minerals are always calcium-saturated. The clay minerals probably most commonly present in calcretes (palygorskite and sepiolite) possess a number of unusual engineering properties not shared by the other common clays.
10. Calcretes are prone to have higher apparent Liquid Limits, Shrinkage Limits and Plasticity Indices than most other materials of the same Linear Shrinkage. This is probably largely due to the porosity of the fine calcrete particles.
11. The solid-particle specific gravity does not differ greatly from that of other soils. Owing to particle porosity, the bulk-particle specific gravity may, however, be appreciably less, and also may vary with size fraction.
12. Calcretes are prone to yield apparently poor gradings by weight, with an excess of fine sand and a deficiency of coarse sand, generally coupled with a high fines content. The reason appears to be due partly to the variation in bulk-particle specific gravity and partly the variation in particle strength with grain size.

13. Calcretes with apparently poor gradings and Atterberg Limits often tend to yield good CBRs and low CBR swell.

14. While the practical importance of self-cementation of calcrete roads remains to be proven, up to a trebling of the soaked CBR has been obtained in the laboratory after a number of wetting and drying cycles.

15. A number of possible mechanisms exist whereby calcretes could, in theory, undergo a measure of self stabilization. Tests are available for detecting whether self stabilization has occurred.

16. The properties of lime stabilized calcretes are controlled largely by their content of amorphous silica microfossil skeletons. Calcretes with high amorphous silica contents yield higher strengths more rapidly when stabilized with lime than with cement.

17. Some calcretes may contain deleterious amounts of soluble salts which can cause disintegration of the upper base and blistering of a blacktop surfacing. Conductivity and durability testing of proposed base materials is suggested as a preventive measure.

18. The aggregate strength of nodular calcretes increases with size. 25mm–37,5mm nodules for example may yield twice the 10% F.A.C.T. value of 9,5mm–13,2mm nodules.

19. The dry strength of indurated calcrete particles depends largely on their Mohs hardness, density and degree of crystallinity, and the dry strength increases with density and hardness and decreases with carbonate crystal size. A particularly good correlation exists between Mohs hardness (H) and the dry 10% F.A.C.T. (10% F.A.C.T. value = 4,7 H – 5). The soaked strength is usually about two-thirds the dry strength.

20. Calcretes, as a group, vary from sands (powder calcretes and calcified sands) through gravels (nodular calcretes and calcified gravels) and boulders (boulder calcretes) to material with the character of rock, possessing 10% F.A.C.T. values of up to 195 kN (ACV of 19%), Mohs hardnesses of up to 6 and densities of up to 2640 kg/m^3, which requires drilling and blasting for excavation.

21. Depending largely on the stage of development, calcretes have uses which vary from surfacing chips through base and subbase to gravel-road wearing courses and fill.

22. The soil constants of some calcretes are affected by heating at temperatures much above room temperature. In the five calcretes studied, heating the soil fines to 105°C for 24 hours resulted in every case in a lowering of the Liquid Limit by as much as 2 percentage units and the Plasticity Index by 1 to 4. Linear Shrinkage was relatively little affected, the maximum variation from the air dry value being 0,7. The effect of heating at 50°C was somewhat similar, though generally less pronounced and more erratic than when heated to 105°C.

23. Examination of a number of calcrete flow curves suggests that reasonably reliable (±0,5 units in 93% of cases) Liquid Limits can only be obtained by using a flow curve method which employs at least four points which progress smoothly from a wetter to a drier state (or possibly from a drier to a wetter state). One-point methods tend to yield erratic results, and methods which permit the moisture content to be adjusted by wetting and drying until the Liquid Limit is reached, will yield Liquid Limits which are too high by up to several units in many cases. Calcretes will thus tend to yield erratic, but generally high Liquid Limits if the standard South African method is used. In addition, difficulties with the Plastic Limit have been reported.

24. The combined difficulties in determining the Liquid and Plastic Limits of calcretes and their statistically high Liquid Limits, Shrinkage Limits and Plasticity Indices in comparison with their Linear Shrinkages suggest that the Liquid Limit and Plasticity Index tests on calcretes tend to yield results which are unrealistically high and generally unreliable.

25. Linear Shrinkage is the only soil constant which is considered to be reliable in regard to calcretes and this is suggested as the primary means of control in place of the Liquid Limit and Plasticity Index. The Bar Linear Shrinkage from the Liquid Limit is a rapid and reliable test, the results of which seem to correlate well with known performance.

26. Calcretes may possess a relatively weak aggregate fraction and testing of this is probably advisable in the case of the base of medium and heavily-trafficked roads. It is particularly important in the case of the coarser gradings for gravel-road wearing courses. Several simple tests (Treton, APT, Water Absorption and Mohs Hardnesses) are available which are suitable for field and central laboratory use.

27. Examination of some overseas specifications and the performance of five South African calcrete-based roads has led to the conclusion that parts of the normal South African specifications currently applied to these materials are too severe, and that considerable relaxation of apparent

grading, apparent Liquid Limit, apparent Plasticity Index and even Linear Shrinkage, in some cases, is possible.

28. Other than as crusher run or stabilized crusher run, calcretes are not suitable as base for the heaviest flexible pavement designs.

29. Revised specifications are suggested for all classes of calcrete road material.

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APPENDIX A: TEST METHODS USED, LIST OF ABBREVIATIONS AND SUMMARY OF TEST RESULTS

The results of most of the tests carried out at the National Institute for Road Research during the coarse of the calcrete project are shown in Tables 14 to 17. A reference to each test method used, is given in Table 13 below:

TABLE 13

Test methods, definitions and abbreviations employed

Test	Reference
Absorption limit (w_A)	Nascimento et al[20]
Liquid Limit (w_L or LL)	NIRR[33]
Plastic Limit (w_p)	NIRR[33]
Shrinkage Limit (w_S)	Lambe[63]
Shrinkage ratio (SR)	Lambe[63]
Plasticity Index (I_p) or (PI)	$w_L - w_p$
Flow Index (I_f)	2 (w_{10} blows − w_{32} blows)
Toughness Index (I_t)	I_p/I_f
Bar Linear Shrinkage (LS)	NIRR[33]
Petrifaction degree (PD)	$\frac{w_S}{w_A}$ Nascimento et al[20]
Particle specific Gravity (G_S)	NIRR[33]
Conductivity	Netterberg[52]
pH	Netterberg[52]
Mechanical Analysis	NIRR[33]
Dry 10% Fines Aggregate Crushing Test (10% F.A.C.T.)	Shergold and Hoskings[42], with sieves for different aggregate size as for ACV[28]. Results are in kN.
Soaked 10% F.A.C.T.	10% F.A.C.T. on 24 hour soaked, but surface dry aggregate.
Aggregate crushing value ACV	Department of Transport[28].
Los Angeles Abrasion LAA	California 211–B,using preferred abrasive charge, only one size fraction and 2500 g sample.
Aggregate pliers test (A.P.T.)	Netterberg[13]
Water absorption	On crushed hardpans, from the results obtained during the soaked 10% F.A.C.T. On nodules, ASTM C127 – 59[44].
Bulk density (γ)	On hardpan lumps before crushing, by coating with wax and weighing in air and in water: $\gamma = \dfrac{a}{b - c - \dfrac{(b-a)}{(G_{wax})}}$ where a = specimen weight in air; b = wax coated specimen weight in air; c = wax coated specimen weight in water; G_{wax} = specific gravity of wax (about 0,90). On nodules, ASTM C127–59[44]
Optimum moisture content (OMC)	Department of Transport[28]
Maximum dry density (MDD)	Department of Transport[28]
California Bearing Ratio (CBR)	Department of Transport[28]

Test	Reference
Grading Modulus (GM)	Kleyn[4]
Effective size (D_{10})	Maximum size of the smallest 10% by weight[64]
Uniformity coefficient (C_u)	$\frac{\text{Maximum size of the smallest 60\% by weight}^{64}}{D_{10}}$
D.O.T. classification	Department of Transport[28], G = gravel, S = sand, SSC = semi sand clay, SC = sand clay.
Extended Casagrande classification	Road Research Laboratory[8]
Revised PRA (HRB, AASHO)	Road Research Laboratory[8]
Sulphate soundness	Department of Transport[28]
Sound cycles	Number of completed cycles before onset of disintegration.
Polished stone value (1965)	Maclean and Shergold[30]
Detachment	Karius and Dalton[31]
Flakiness	SABS 647:1961[29]
Bulk density (rapid method)	Normal specific gravity method by weighing in air and water, but reading in water taken as soon as possible (within 5 seconds at most) before much water absorption takes place.
Weighted mean Mohs hardness ($\bar{H}_w$)	Normal method, using minerals[65] or hardness tester[1], according to estimated percentage by volume of each hardness component.
$\bar{H}_w + H_{MAX} - H_{MIN}$	Mean hardness plus range of hardness (correlates with detachment)[1].

APPENDIX B

Tables of results on different types of calcretes.

TABLE 14 SUMMARY OF TEST RESULTS

MATERIAL TYPE	SAMPLE NUMBER	MINUS 40 MESH FRACTION (SOIL FINES) ABSORPTION LIMIT %	LIQUID LIMIT %	PLASTIC LIMIT %	SHRINKAGE LIMIT (1) %	SHRINKAGE RATIO	PLASTICITY INDEX %	SHRINKAGE INDEX %	FLOW INDEX %	TOUGHNESS INDEX	BAR LINEAR SHRINKAGE %	PETRIFACTION DEGREE (1)	SOLID PARTICLE SPECIFIC GRAVITY	CONDUCTIVITY $m\Omega^{-1}/cm$	pH
CALCIFIED SANDS	895	50,1	33,0	18,1	22,3[(2)]	1,68	14,9	-4,2	13,0	1,1	6,7	0,44	2,667	2,3	9,2
	1015	-	24,7	-	-	-	S.P.		~8,0	~0 1	1,3	-	2,691	-	-
	1567	45,6	-	-	35,9[(2)]	1,36	N.P.	-	-	-	-	0,79	2,650	-	-
	1625	59,7	49,0	27,7	29,2	1,52	21,3	-1,5	0,8	26,6	9,1	0,49	2,729	0,25	8,6
	2506	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	2531	-	-	-	-	-	-	-	-	-	-	-	-	1,10	7,7
POWDER CALCRETES	897	-	48,7	29,9	-	-	18,8	-	4,6	4,1	2,6	-	2,676	-	-
	902	51,2	58,4	36,6	26,0[(2)]	1,61	21,8	10,6	~6 ?	~3,6 ?	10,7	0,51	2,782	21,3	8,5
	1017	-	~50	36,7	-	-	~13	-	~14 ?	~0,9 ?	8,0	-	2,652	-	-
	1020	-	42,7	27,1	-	-	15,6	-	14,6	1,1	7,3	-	2,649	0,19	8,1
	1512	38,9	28,7	18,5	21,9	1,67	10,2	-3,4	3,4	3,0	2,0	0,54	2,634	-	-
	1515	41,6	22,1	-	23,1	1,66	S.P.	-	10,8	~0,9	-	0,56	2,676	-	-
	1557	43,4	27,4	15,0	18,9[(2)]	1,70	12,4	-3,9	0,4	31,0	4,0	0,44	2,489	0,56	8,2
	1696	39,5	45,8	31,3	24,8[(2)]	1,55	14,5	6,5	~2,2 ?	~0,7 ?	8,7	0,63	2,498	0,48	8,2
	1737	-	21,8	17,7	-	-	4,1	-	2,8	1,5	2,7	-	-	-	-
	1881	34,9	33,3	23,7	25,8	1,61	9,6	-2,1	3,6	2,7	5,3	0,74	2,746	-	-
	2223	42,7	24,0	17,5	20,6	1,74	6,5	-3,1	4,8	1,4	0,6	0,48	2,709	0,14	8,2
	2236	-	32,1	-	-	-	S.P.	-	6,0	~0,2	2,7	-	2,607	-	-
	2360	43,7	47,7	32,7	28,7[(2)]	1,50	15,0	4,0	1,2	12,5	7,3	0,66	2,638	1,6	8,7

NOTE: (1) USING G_s SHRINKAGE LIMIT METHOD
(2) SOAKED BEFOREHAND AS FOR LL
(3) USING 2500g AGGREGATE AND 11 SPHERES

CALCIFIED SANDS AND POWDER CALCRETES

PARTICLE SIZE DISTRIBUTION - CUMULATIVE PERCENTAGE FINER BY WEIGHT (INCHES, MILLIMETRES, U.S. SIEVES)																	
3·00 76·1	2·00 50·8	1·50 38·1	1·00 25·4	0·75 19·0	0·50 12·7	0·25 6·35	0·185 4·76 4	0·0787 2·00 10	0·0331 0·841 20	0·0164 0·420 40	0·0098 0·250 60	0·0058 0·149 100	0·0029 0·074 200	0·0024 0·060	0·0008 0·020	0·0002 0·006	0·00008 0·002
-	-	-	-	-	-	100	80	72	64	56	45	33	24	20	17	9	5
-	-	-	-	-	-	100	77	68	60	55	47	34	24	19	13	8	6
-	-	100	89	86	82	73	70	63	58	50	38	21	11	10	7	3	2
-	-	100	93	86	81	73	67	60	50	44	42	37	31	20	15	10	7
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	-	100	97		97	93	91	83	78	74	72	67	61	57	54	41	9
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	-	100	96	88	87	82	79	73	69	66	63	61	52	48	40	14	5
-	-	-	-	-	-	100	96	91	87	84	82	79	68	63	30	13	8
-	-	100	98	96	92	85	83	78	75	68	64	34	22	-	-	-	-
-	-	100	96	94	93	-	88	79	73	68	61	49	38	36	33	29	18
-	-	-	-	-	-	-	86	75	64	53	43	32	(17)	23	20	17	12
-	-	-	-	-	-	-	87	79	71	63	57	49	(38)	42	40	36	15
-	-	-	100	95	93	90	90	87	84	83	81	79	73	51	26	9	6
-	-	-	-	-	-	-	93	87	80	65	43	27	19	18	14	11	-
-	-	-	100	98	97	91	88	75	65	56	48	38	29	28	28	21	16

T A B L E 14 (continued)

MATERIAL TYPE	HALF INCH TO THREE QUARTER INCH AGGREGATE										
	10% F.A.C.T.		A C V	L A A (3)		A.P.T.		WATER	SPECIFIC GRAVITY		
	DRY kN	SOAKED kN	%	100 REVS. %	500 REVS. %	A F V %	A P V %	ABSORPTION %	Apparent	Bulk, dry	Bulk, saturated surface dry
CALCIFIED SANDS	-	-	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	72	19	-	-	-	-
	-	-	-	-	-	94	39	-	-	-	-
	25	3	48,4	47,4	96,0	-	-	21,7	-	-	-
	-	-	-	-	-	-	-	-	-	-	-
POWDER CALCRETES	-	-	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	94	53	-	-	-	-
	-	-	-	-	-	75	63	7,9	2,72	2,24	2,42
	-	-	-	-	-	75	33	-	-	-	-
	-	-	-	-	-	25	8	-	-	-	-
	34	-	-	-	-	-	-	7,0	2,60	2,21	2,36
	-	-	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	-	-	-	-	-	-
	25	-	-	-	-	75	28	-	-	-	-
	-	-	-	-	-	94	53	-	-	-	-
	-	-	-	-	-	83	44	16,1	2,58	1,83	2,12

MODIFIED AASHO						CLASSIFICATION			REMARKS
OMC %	MDD kg/m³	CBR %	GRADING MODULUS	EFFECTIVE SIZE mm	UNI-FORMITY COEFF.	D.O.T.	EXTEND-ED CASA-GRANDE	REVISED PRA	
-	-	-	1,48	0,007	86	SSC	SF-GF	A-2-6(0)	
-	-	-	1,53	0,009	94	SSC	GF	A-2-4(0)	
-	-	-	1,76	0,06	17	SSC	GP-GF	A-1-b(0)	
-	-	-	1,65	0,006	333	SSC	GF-SF	A-2-7(2)	
-	-	-	-	-	-	-	-	-	
-	-	-	-	-	-	-	-	-	
20,0	1618	54	-	-	-	-	-	-	NATURAL
-	-	-	0,82	0,002	35	SC	CH ?	A-7-5(13)	NATURAL
-	-	-	-	-	-	-	-	-	POWDERED BY TRAFFIC ?
-	-	-	-	-	-	-	-	-	POWDERED BY TRAFFIC ?
—	-	-	1,09	0,004	35	SSC	GF	A-4(3)	NATURAL
-	-	-	0,57	0,003	17	SC	ML	A-4(7)	NATURAL
-	-	-	1,32	-	-	SSC	SF	A-2-6(0)	NATURAL
-	-	-	1,15	0,001	240	SSC	SF	A-7-5(2)	POWDERED BY TRAFFIC
-	-	-	1,45	0,0015	400	SSC	SF	A-2-4(0)	NATURAL
-	-	-	1,20	0,0015	200	SSC	SF	A-4(1)	FROM GRAVEL ROAD SURFACE
8,7	2035	37	0,57	0,007	9·3	SC	ML	A-4(8)	NATURAL
-	-	-	1,29	0,005	76	SC	SF-SP	A-2-4(0)	NATURAL
-	-	-	1,40	0,0006	1000	SSC	SF	A-2-7(1)	UNKNOWN SOURCE

TABLE 15 SUMMARY OF TEST

MATERIAL TYPE	SAMPLE NUMBER	MINUS 40 MESH FRACTION (SOIL FINES)													
		ABSORBTION LIMIT %	LIQUID LIMIT %	PLASTIC LIMIT %	SHRINKAGE LIMIT % [1]	SHRINKAGE RATIO	PLASTICITY INDEX %	SHRINKAGE INDEX %	FLOW INDEX %	TOUGHNESS INDEX %	BAR LINEAR SHRINKAGE %	PETRIFACTION DEGREE [1]	SOLID PARTICLE SPECIFIC GRAVITY	CONDUCTIVITY $m\Omega^{-1}/cm$	pH
NODULAR CALCRETES	913	49,4	34,5	16,8	19,7	1,77	17,7	-2,9	5,4	3,3	7,3	0,40	2,687	-	-
	976	-	40,5	21,6	-	-	18,9	-	30,0	0,6	7,3	-	2,534	-	-
	978	-	63,5	54,4	-	-	9,1	-	-	-	8,7	-	2,523	1,71	8,1
	1016	-	~64	42,0	-	-	~22	-	~24?	~0,9?	8,0	-	2,646	0,17	8,1
	1021M	52,9	40,4	26,8	30,5	1,45	13,4	-3,7	1,6	8,4	5,3	0,58	2,610	-	-
	1024	-	24,6	17,0	-	-	7,6	-	3,0	2,5	4,7	-	2,654	0,53	8,0
	1024M	36,6	26,1	16,2	19,0	1,77	9,9	-2,8	4,2	2,4	5,3	0,52	-	-	-
	1026	-	23,2	14,1	-	-	9,1	-	3,2	2,8	2,7	-	2,693	-	-
	1026M	36,4	25,6	15,0	17,8	1,83	10,6	-2,8	4,2	2,5	5,3	0,49	-	-	-
	1513	45	34,2	21,5	21,8 [2]	1,65	12,7	-0,3	1,4	9,1	5,3	0,48	2,566	0,33	8,0
	1514	43,2	43,0	23,1	21,5 [2]	1,64	19,9	1,6	3,0?	6,6?	8,0	0,50	2,534	0,36	7,9
	1516	45,7	31,3	21,5	23,5 [2]	1,62	9,8	-2,0	6,2	1,6	2,7	0,51	2,595	0,25	8,0
	1533	42,3	33,4	22,8	24,3	1,67	10,6	-1,5	1,2	8,8	4,7	0,57	2,804	2,0	-
	1561	~42	25,7	17,5	23,2 [2]	1,60	8,2	-5,7	6,0	1,4	3,0	~0,55	2,540	2,4	8,2
	1565	53,1	40,3	15,4	13,9 [2]	1,83	24,9	1,5	4,0	6,2	10,0	0,26	2,471	0,60	7,8
	1569	39,6	24,9	-	19,8	1,72	S.P.	-	4,0	~0,3	3,3	0,50	2,596	0,29	8,1
	1601	42,0	26,0	21,9	30,9	1,46	4,1	-9,0	2,4	1,7	2,7	0,74	2,656	1,0	8,5
	1601R	44,9	25,8	-	27,5	1,54	S.P.	-	2,4	0,4	3,7	0,62	-	-	-
	1602	43,0	31,5	18,7	24,1	1,63	12,8	5,4	3,6	3,6	4,0	0,61	2,662	1,9	8,7
	1618	-	21,2	14,2	-	-	7,0	-	3,0	2,3	4,0	-	-	-	-
	1620	-	36,8	26,5	-	-	10,3	-	1,6	6,4	4,7	-	-	-	-
	1624	47,1	40,8	21,2	19,6	1,75	19,6	1,6	~14?	~1,4?	9,3	0,42	2,677	0,29	8,5
	1876	-	31,9	21,6	-	-	10,3	-	4,0	2,6	5,0	-	-	-	-
	1877	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	1879	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	1880	-	-	-	-	-	-	-	-	-	-	-	-	-	-
	1979	-	29,7	-	-	-	N.P.	-	~2 6	0	4 0	-	-	-	-
	1980	39,8	30,5	17,6	19,3 [2]	1,75	12,9	-1,7	1,0	12,9	3,3	0,48	2,636	0,42	8,3
	1981	~45	48,0	26,5	20,1 [2]	~1,66	21,5	6,4	17?	~13?	7,7	~0,45	2,594	2,5	8,8
	2114	40,2	40,4	26,1	22,3	1,68	14,3	3,8	17,0	0,8	8,0	0,56	2,680	0,34	7,9
	2116	36,1	26,1	15,4	14,7	1,95	10,7	0,7	~4,8	~2,2	6,0	0,41	2,694	0,28	8,4
	2234	-	23,6	-	-	-	S.P.	-	6,2	~0,2	1,3	-	2,650	-	-
	2367	~52	31,8	-	21,7 [2]	1,70	S.P.	-	6,0	~0,2	6,3	0,42	2,689	2,4	9,0
	2368	34,5	-	-	21,7 [2]	1,67	S.P.	-	-	-	2,0	0,63	2,616	1,3	8,0
	2493	41,0	32,9	23,6	23,2 [2]	1,65	9,3	0,4	~10,6?	~0,9?	5,3	0,57	2,662	0,33	8,0
	2555	35,6	20,8	-	22,2	1,68	S.P.	-	3,2	~0,3	1,3	0,63	2,678	0,46	8,0
	2566	33,9	-	-	20,3	1,73	N.P.	-	-	-	0,0	0,60	2,656	0,27	8,0
	2588	~65	37,2	-	44,0 [2]	1,20	S.P.	-	-	-	4,0	~0,68	2,541	0,35	7,8
	2689	48,9	41,7	21,9	16,1	1,78	19,8	5,8	4,2	4,7	12,3	0,33	2,504	0,64	7,7

RESULTS – NODULAR CALCRETES

PARTICLE SIZE DISTRIBUTION – CUMULATIVE PERCENTAGE FINER BY WEIGHT (INCHES, MILLIMETERS, U.S. SIEVES)																	
3·00 76·1	2·00 50·8	1·50 38·1	1·00 25·4	0·75 19·0	0·50 12·7	0·25 6·35	0·185 4·76 4	0·0787 2·00 10	0·0331 0·841 20	0·0164 0·420 40	0·0098 0·250 60	0·0058 0·149 100	0·0029 0·074 200	0·0024 0·060	0·0008 0·020	0·0002 0·006	0·00008 0·002
-	-	-	-	-	100	-	70	58	53	47	40	31	25	22	19	11	7
-	-	-	-	-	-	-	-	-	-	-	-	-	19	-	-	-	-
-	-	-	-	-	-	-	-	54	-	38	-	-	17	-	-	-	-
-	-	-	-	-	-	-	-	54	-	38	-	-	13	-	-	-	-
-	-	-	100	97	96	86	83	74	66	58	47	29	16	15	14	11	10
-	-	-	-	-	-	-	72	61	57	54	50	41	28	23	13	10	5
-	100	92	90	89	85	76	72	64	59	54	48	39	28	24	20	15	9
-	-	-	-	-	-	-	78	63	59	58	55	48	37	26	23	19	17
-	-	-	-	100	98	86	79	63	59	57	53	46	34	29	25	18	5
100	97	-	93	90	-	82	79	67	56	49	44	41	33	27	24	15	5
-	-	100	94	84	80	71	67	58	52	49	46	43	39	-	-	-	-
-	-	100	86	80	78	64	59	47	41	37	35	34	29	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	100	96	95	91	87	81	79	72	64	56	45	29	19	-	-	-	-
-	100	86	72	60	48	39	37	35	33	30	26	19	14	-	-	-	-
-	100	85	-	82	75	64	59	51	40	33	27	22	14	9	7	4	3
-	100	88	-	-	84	74	70	62	58	51	41	25	16	12	9	6	-
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	-	100	98	88	84	81	77	68	63	56	48	34	27	17	13	7	5
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	100	93	85	72	56	45	40	30	25	23	22	19	13	11	9	7	5
-	-	-	-	-	-	-	83	67	57	50	44	36	(21)	30	26	21	11
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	100	83	80	71	68	58	54	46	40	36	33	29	24	-	-	-	-
-	-	100	96	96	92	85	81	69	58	49	42	35	26	-	-	-	-
-	-	100	97	95	93	84	80	65	51	43	38	34	24	22	16	13	8
100	79	-	77	67	63	52	48	41	37	34	33	31	23	21	19	18	10
-	-	-	-	100	82	-	66	61	59	50	38	25	11	10	7	3	-
-	100	93	-	89	84	68	62	52	49	46	39	26	17	15	13	12	11
100	90	-	82	80	75	65	61	53	47	39	31	20	12	8	6	4	2
-	-	100	90	80	75	56	-	48	44	39	28	20	14	11	9	9	7
-	100	94	92	87	79	63	57	44	40	37	31	20	12	11	8	6	4
100	83	-	69	63	55	43	39	33	32	29	21	12	7	6	4	4	3
100	89	83	77	75	64	52	48	35	26	22	19	15	10	8	5	4	2
-	100	89	75	62	51	33	25	22	20	18	15	9	8	8	8	8	3

NOTE: (1) USING 6_s SHRINKAGE LIMIT METHOD
(2) SOAKED BEFOREHAND AS FOR LL
(3) USING 2500g AGGREGATE AND 11 SPHERES

TABLE 15 (continued)

MATERIAL TYPE	HALF INCH TO THREE QUARTER INCH AGGREGATE											
	10% F.A.C.T.			ACV	LAA(3)		A.P.T.		Water Absorption %	SPECIFIC GRAVITY		
	DRY kN	SOAKED kN	Soaked moisture content %	%	100 Revs %	500 Revs %	AFV %	APV %		Apparent	BULK, DRY	BULK Saturated, Surface dry
NODULAR CALCRETES	-	-	-	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	-	0	0	-	-	-	-
	-	-	-	-	-	-	87	54	15,0	2,66	1,90	2,19
	-	-	-	-	-	-	96	73	-	-	-	-
	-	-	-	-	-	-	70	34	-	-	-	-
	-	-	-	-	-	-	39	19	-	-	-	-
	-	-	-	-	-	-	95	52	6,3	2,60	2,24	2,38
	-	-	-	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	-	92	14	-	-	-	-
	-	-	-	-	-	-	77	25	-	-	-	-
	-	-	-	-	-	-	78	37	-	-	-	-
	-	-	-	-	-	-	98	87	-	-	-	-
	11	-	-	57	-	-	-	-	20,6	2,49	1,65	1,99
	-	-	-	-	-	-	93	77	2,5	2,66	2,50	2,57
	-	-	-	-	-	-	91	48	5,0	2,64	2,33	2,46
	-	-	-	-	-	-	78	25	-	-	-	-
	-	-	-	-	-	-	-	-	-	-	-	-
	152?	-	-	-	-	-	69	21	2,6	2,69	2,51	2,58
	-	-	-	-	-	-	93	51	-	-	-	-
	-	-	-	-	-	-	78	24	-	-	-	-
	-	-	-	-	-	-	93	73	-	-	-	-
	-	-	-	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	-	91	44	-	-	-	-
	-	-	-	-	-	-	100	77	-	-	-	-
	-	-	-	-	-	-	99	77	-	-	-	-
	-	-	-	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	-	-	-	5,3	2,68	2,35	2,47
	-	-	-	-	-	-	-	-	-	-	-	-
	44	15	9,3	42,4	21,8	64,1	80	33	6,4	2,63	2,26	2,40
	86	39	5,7	33,1	-	-	92	69	3,7	2 66	2,42	2,51
	-	-	-	-	-	-	93	66	-	-	-	-
	110	63	5,4	26,7	-	37,2	100	69	-	-	-	-
	49	27	8,4	35,1	-	~ 50	89	39	-	-	-	-
	108	35	10,2	26,3	14,0	46,3	100	65	-	-	-	-
	-	-	-	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	-	-	-	12,7	2,69	2,01	2,26
	172	-	-	-	-	-	-	-	3,3	2,67	2,46	2,54

– 40 # LAA FINES			MODIFIED AASHO			GRADING MODULUS	EFFECTIVE SIZE mm	UNIFORMITY COEFF.	CLASSIFICATION		
LIQUID LIMIT %	PLASTICITY INDEX %	BAR LINEAR SHRINKAGE %	OMC %	MDD kg/m³	CBR %				D.O.T.	EXTENDED CASAGRANDE	REVISED P.R.A.
-	-	-	-	-	-	1,70	0,005	440	SSC	GF	A-2-6(1)
-	-	-	-	-	-	-	<0,074	-	SSC	-	A-2-7(1)
-	-	-	-	-	-	1,91	<0,074	-	SSC	GF	A-2-5(0)
-	-	-	-	-	-	1 95	<0,074	-	SSC	GF	A-2-7(1)
-	-	-	-	-	-	1 52	0,002	250	SSC	SF-GF	A-2-6(1)
-	-	-	-	-	-	1,57	0,006	300	SSC	GF	A-2-4(0)
-	-	-	-	-	-	1,54	0,003	400	SSC	GF	A-2-4(0)
-	-	-	-	-	-	1,42	<0,001	-	SSC	GF	A-4(1)
-	-	-	-	-	-	1,46	0,003	333	SSC	GF	A-2-6(0)
-	-	-	-	-	-	1,51	0,003	333	SSC	GF	A-2-6(1)
-	-	-	-	-	-	1,54	<0,074	-	SSC	GF	A-6(3)
-	-	-	-	-	-	1,87	<0,074	-	G	GF	A-2-4(0)
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	1,53	<0,074	-	SSC	GF-SF	A-2-4(0)
-	-	-	-	-	-	2,21	<0,074	-	G	GF	A-2-6(0)
-	-	-	-	-	-	2,02	0,065	77	SSC	GC-GF	A-1-b(0)
-	-	-	-	-	-	1,71	0,030	33	SSC	GF	A-2-4(0)
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	1,49	0,011	55	SSC	GF	A-2-6(0)
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	2,34	0,040	325	G	GC-GF	A-2-7(0)
-	-	-	-	-	-	1,62	0,002	500	SSC	GF	A-2-4(0)
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	1,94	<0,074	-	G	GF	A-2-6(0)
-	-	-	-	-	-	1,56	<0,074	-	SSC	GF	A-2-7(1)
21,1	S.P.	2 0	10,6	1986	80	1,68	0,003	433	SSC	GF	A-2-6(0)
20,0	S.P.	0 7	7,0	2163	59	2,02	0,002	5500	G	GF	A-2-6(0)
-	-	-	-	-	-	1,78	0,060	22	SSC	GP-GF	A-1-b(0)
18,5	S.P.	2 0	-	-	-	1,85	0,0015	2670	SSC	GP-GF	A-1-b(0)
-	-	-	13,6	1826	80	1,96	0,065	62	SSC	GP-GF	A-1-b(0)
30,3	9,7	4,0	~11	~1900	~110	1,99	0,050	160	G	GP-GF	A-2-4(0)
-	-	-	-	-	-	2,07	0,055	100	G	GP-GF	A-1-b(0)
-	-	-	-	-	-	2,31	0,11	136	G	GP	A-1-a(0)
33,3	S.P.	3,3	-	-	-	2,31	0,074	149	G	GC	A-1-a(0)
-	-	-	10,5	2035	38	2,52	0,180	100	G	GU-GC	A-2-7(0)

TABLE 16 SUMMARY OF TEST RESULTS – HONEYCOMB, HARDPAN

MATERIAL TYPE	SAMPLE NUMBER	— 40 # LAA FINES: LIQUID LIMIT %	PLASTIC LIMIT %	PLASTICITY INDEX %	FLOW INDEX %	TOUGHNESS INDEX %	BAR LINEAR SHRINKAGE %	CONDUCTIVITY $m\Omega^{-1}/cm$	pH
HONEYCOMB CALCRETES	1931	–	–	–	–	–	–	–	–
	1573	24,5		S.P.	1,4	~0,7	3,0	0,10	8,6
	2494	15,8	13,5	2,3	–	–	0,7	0,11	8,5
HARDPAN CALCRETES	1530	–	–	N.P.	–	–	–	0,19	8,4
	1562	–	–	S.P.	–	–	–	0,09	9,5
	1623	–	–	N.P.	–	–	–	0,62	8,7
	1642	36,5	31,7	4,8	6,8	0,7	3,3	0,18	10,0
	1658	26,2	23,8	2,4	7,0	0,3	2,0	0,10	9,4
	1658 x	39,5	32,5	7,0	–	–	3,0	0,12	8,6
	1724	38,3	39,0	0	7,4	0	1,3	0,61	9,1
	1828	–	–	–	–	–	–	0,18	–
	2200	–	–	S.P.	–	–	–	0,10	8,7
	2204	22,1	18,5	3,6	5,6	0,6	2,0	0,15	8,8
	2225	26,3	22,5	3,8	5,6	0,7	1,3	0,13	8,9
	2226	26,3	20,0	6,3	0,0	∞	1,3	0,08	8,6
	2227	28,1	21,9	6,2	3,4	1,8	2,6	0,15	8,1
	2505	23,3	21,7	1,6	3,4	0,5	2,7	0,35	8,0
	2507	21,3	18,8	2,5	5,0	0,5	2,7	0,30	8,1
BOULDER CALCRETES	1028	–	–	S.P.	–	–	–	–	–
	1028 x	–	–	–	–	–	–	0,16	9,9
	1621	15,5	–	6,5	5,2	1,3	1,3	0,08	8,3
	1622	21,4	–	2,1	3,4	0,6	2,0	0,09	8,2
	1627	20,0	17,3	2,7	4,8	0,6	1,3	0,06	9,2
	1627 x	19,2	–	S.P.	4,6	~0,2	0,6	0,11	8,3
	2125	22,6	–	1,6	6,8	0,2	1,3	0,07	8,8
	2228	–	–	N.P.	–	–	–	0,12	8,3
	2302	–	–	N.P.	–	–	1,3	0,18	7,9
MIXTURE OF A, B AND C	890	–	–	–	–	–	–	0,41	9,2
VITREOUS SILCRETE	890 A	–	–	–	–	–	–	–	–
VITREOUS SILCRETE	890 B	–	–	–	–	–	–	–	–
DOLOMITIC SANDSTONE	890 C	–	–	–	–	–	–	0,33	9,3
NOSIB (?) QUARTZITE	1833 x	16,2	14,9	1,3	7,2	0,2	0,0	0,054	7,9

NOTES: (1) Using 2500 g aggregate and 11 spheres. (2) By pocket hardness

AND BOULDER CALCRETES AND MISCELLANEOUS SAMPLES

3/8" — 1/2" CRUSHED AGGREGATE										
10 % F.A.C.T.			A C V	L A A (1)			SPECIFIC GRAVITY			
DRY kN	SOAKED kN	SOAKED MOISTURE CONTENT %	%	100 REVS %	500 REVS %	WATER ABSORPTION %	APPARENT	BULK DRY	BULK, SATURATED SURFACE DRY	MOHS (2) HARDNESS $\bar{H}$
–	–	–	–	–	–	1,2	2,68	2,59	2,63	4,0
142	98	5,7	23,6	–	–	2,4	2,68	2,52	2,58	4,8
205	142	4,9	16,0	5,5	24,5	2,0	2,74	2,59	2,64	4,5
				–						
22	13	20,0			–	–	–	–	–	2,3
129	101	4,8	27,1	(20,3)	(85,8)	2,5	2,61	2,45	2,51	4,2
–	–	2,8	20,9	–	–	0,4	2,71	2,68	2,69	3,4
25	12	–	52,8	–	–	–	–	–	–	<2
198	114	4,5	19,3	–	–	1,6	2,74	2,63	2,67	5,0
162	116	4,4	21,0	–	–	–	–	–	–	5,0
63	44	24,0	37,5	13,0	44,2	19,8	2,49	1,67	2,00	2,8
–	–	–	–	–	–	–	–	–	–	–
78	40	4,3	30,6	–	–	1,6	2,71	2,60	2,64	3,1
164	109	4,9	20,2	7,6	34,1	2,0	2,72	2,58	2,63	4,2
166	–	–	21,1	9,3	41,6	2,3	2,69	2,54	2,60	–
113	–	–	24,9	13,7	57,6	1,8	2,69	2,57	2,62	3,0
99	–	–	30,0	16,2	56,5	2,7	2,70	2,52	2,59	3,2
110	60	5,8	24,9	11,6	48,9	3,8	2,69	2,44	2,53	–
80-133	44-53	5,9	–	12,2	49,0	3,2	2,70	2,49	2,56	4,0
118	64	–	–	–	–	–	–	–	–	–
101	82	8,4	33,1	–	–	4,4	2,62	2,35	2,45	3,8
–	–	3,8	20,4	–	–	1,0	2,68	2,61	2,64	3,9
–	–		23,1	–	–	1,4	2,70	2,60	2,63	4,0
162	103	2,8	24,6			–	–	–	–	4,0
173	122	3,4	20,8	7,8	36,1	1,3	2,71	2,62	2,66	4,3
–	–	5,7	25,3	–	–	2,6	2,68	2,50	2,57	4,0
112	–	–	25,2	13,8	59,0	2,2	2,69	2,54	2,60	–
80-142	56	7,3	26,8	–	–	–	–	–	–	4,0
212	–	3,4	17,7	–	–	–	–	–	–	–
–	–	–	–	–	–	–	–	–	–	–
–	–	–	–	–	–	–	–	–	–	–
177	–	5,1	–	–	–	1,3	2,64	2,55	2,58	–
210		2,9	18,0	7,2	28,0	0,8	2,57	2,52	2,54	–

tester. (3). By minerals.

TABLE 16 (continued)

MATERIAL TYPE	SAMPLE NUMBER	3/4" — 1" CRUSHED AGGREGATE: 20 CYCLE SODIUM SULPHATE SOUNDNESS: CUMULATIVE PERCENT PASSING							1/4"–5/16" CRUSHED AGGREGATE		
		3/4"	1/2"	4#	10#	40#	100#	SOUND CYCLES	POLISHED STONE VALUE (1965)	DETACHMENT %	FLAKINESS %
HONEYCOMB CALCRETES	1931	-	-	-	-	-	-	-	-	-	-
	1573	-	-	-	-	-	-	-	-	-	-
	2494	-	-	-	-	-	-	-	-	-	-
HARDPAN CALCRETES	1530	100	98	79	64	51	32	2	-	-	-
	1562	51	44	25	15	8	3	7	0,60	0,5	5,7
	1623	-	-	-	-	-	-	-	-	-	-
	1642	100	94	84	71	38	21	1	-	-	-
	1658	48	16	10	6	4	3	7	0,47	15	5,0
	1658 x	-	-	-	-	-	-	-	-	-	-
	1724	100	80	50	31	12	5	4	(0,65)	100	4,7
	1828	-	-	-	-	-	-	-	-	-	-
	2200	-	-	-	-	-	-	-	-	-	-
	2204	-	-	-	-	-	-	-	-	-	-
	2225	-	-	-	-	-	-	-	0,53	-	8,5
	2226	-	-	-	-	-	-	-	0,48	-	2,0
	2227	-	-	-	-	-	-	-	0,54	-	2,7
	2505	-	-	-	-	-	-	-	-	-	-
	2507	-	-	-	-	-	-	-	-	-	-
BOULDER CALCRETES	1028	-	-	-	-	-	-	-	-	-	-
	1028 x	31	12	6	4	1	1	4	0,49	53	8,1
	1621	-	-	-	-	-	-	-	-	-	-
	1622	-	-	-	-	-	-	-	-	-	-
	1627	100	81	33	16	6	3	8	0,51	26	9,2
	1627 x	-	-	-	-	-	-	-	-	-	-
	2125	-	-	-	-	-	-	-	-	-	-
	2228	-	-	-	-	-	-	-	0,58	-	2,7
	2302	-	-	-	-	-	-	-	-	-	-
MIXTURE OF A, B AND C	890	0	0	0	0	0	0	20	0,47	47	2,2
VITREOUS SILCRETE	890 A	-	-	-	-	-	-	-	0,43	55	5,7
VITREOUS SILCRETE	890 B	-	-	-	-	-	-	-	-	-	-
DOLOMITIC SANDSTONE	890 C	-	-	-	-	-	-	-	0,59	30	7,1
NOSIB (?) QUARTZITE	1833 x	0	0	0	0	0	0	20	0,56	13	5,8

HAND SPECIMENS					
BULK DENSITY (RAPID METHOD)	CALCULATED BULK DENSITY (WAXED METHOD)	MOHS[3] HARDNESS $\bar{H}_w$	$H_w + H_{max} - H_{min}$	EXTENDED CASAGRANDE CLASSIFICATION	REMARKS
–	–	–	–	ROCK ?	
–	–	–	–	" ?	*RIPPABLE*
–	–	–	–	" ?	*RIPPABLE*
–	–	–	–	–	*RIPPABLE BY ALLIS – CHALMERS H.D 16. SURFACE LAYER OF STIFF CALCIFIED SAND*
2,45	2,45	4,1	7,1	ROCK	*TYPE 3, VERY STIFF — HARD, INTACT.*
–	–	–	–	"	*HARD, INTACT HARDPAN*
1,59	1,50	1,7	(3,2)	—	*FRIABLE TUFACEOUS TYPE*
2,64	2,64	5,8	5,8	ROCK	*HARD, INTACT HARDPAN AND BOULDERS. REQUIRED BLASTING. SEISMIC VELOCITY 10,000 ft./sec.*
–	–	–	–	"	*SAID TO COME FROM THE SAME SITE AS 1658*
1,64	1,55	2,7	4,5	"	*SOMEWHAT INDURATED TUFACEOUS TYPE, STIFF*
1,39	1,21	–	–	—	*FRIABLE TUFACEOUS TYPE*
2,63	2,63	3,2	(3,7)	ROCK	*STIFF, INTACT, TYPE 6*
2,57	2,57	4,3	4,7	"	*HARD, INTACT, MAINLY TYPE 1*
2,51	2,51	–	–	"	*MIXTURE OF HARD TYPE 3, AND VERY STIFF TYPE 6. INTACT.*
2,46	2,46	–	–	"	*SOME TYPE 6 PRESENT, BUT MAINLY TYPE 3*
2,24	2,23	–	–	"	*MAINLY HARD, INTACT TYPE 3*
–	–	–	–	"	*VERY STIFF – HARD*
–	–	–	–	"	*VERY STIFF GRAVELLY HARDPAN*
–	–	–	–	BOULDERS	
2,29	2,28	3,5	(5,5)	"	
–	–	–	–	"	
–	–	–	–	"	
2,62	2,62	4,8	5,3	"	
–	–	–	–	"	
–	–	–	–	"	
2,48	2,48	–	–	"	
–	–	–	–	"	
2,59	2,59	7,7	8,2	ROCK	
2,53	2,53	7,7	7,7	"	
–	–	–	–	"	
2,63	2,63	7,8	8,6	"	
2,55	2,55	7,7	9,2	"	

TABLE 17 SUMMARY OF TEST RESULTS – CALCRETE

MATERIAL TYPE	SAMPLE NUMBER	MINUS 40 MESH FRACTION (SOIL FINES)												
		ABSORPTION LIMIT %	LIQUID LIMIT %	PLASTIC LIMIT %	SHRINKAGE LIMIT (1) %	SHRINKAGE RATIO	PLASTICITY INDEX %	FLOW INDEX %	TOUGHNESS INDEX	BAR LINEAR SHRINKAGE %	PETRIFACTION DEGREE (1)	SOLID PARTICLE SPECIFIC GRAVITY	CONDUCTIVITY $m\Omega^{-1}/cm$	pH
	896	-	25,5	21,4	-	-	4,1	7,2	0,6	1,3	-	2,654	-	-
	900	-	82,0	64,4	-	-	17,6	-	-	4,7	-	2,468	-	-
	910	-	110,3	63,7	-	-	46,6	-	-	13,3	-	2,681	-	-
	911	-	14,9	13,7	-	-	1,2	4,0	0,3	1,3	-	2,717	-	-
	912	-	16,5	12,6	-	-	3,9	4,2	0,9	1,3	-	2,724	-	-
	918	-	51,9	32,7	-	-	19,2	5,4	3,6	5,3	-	2,629	-	-
	977	-	51,6	35,3	-	-	16,3	2,4	6,8	8,7	-	2,538	0,50	8,1
	1018	-	31,8	23,4	-	-	8,4	3,2	2,6	4,0	-	2,636	-	-
	1019	-	44,0	30,6	-	-	13,4	11,6	1,2	7,3	-	2,647	0,24	8,2
	1022	-	30,0	16,5	-	-	13,5	0,0	∞	6,0	-	2,659	0,55	8,0
	2588													
CALCAREOUS SOILS	891	-	55,7	19,4	-	-	36,3	-	-	12,0	-	2,700	-	-
	893	-	43,0	20,1	-	-	22,9	12,0	1,9	5,7	-	2,683	-	-
	894	-	164	20,7	-	-	143,3	-	-	20,0	-	2,720	-	-
	894 R	-	152	21,3	-	-	130,7	-	-	19,3	-	2,669	-	-
	916	-	49,6	21,0	-	-	28,6	14,6	2,0	6,7	-	2,758	-	-
MISCELLANEOUS	892	-	86,6	31,7	-	-	54,9	-	-	19,3	-	2,577	-	-
	908	-	-	-	-	-	N.P.	-	-	-	-	2,639	-	-
	909	-	-	-	-	-	N.P.	-	-	-	-	2,681	-	-
	1965	-	7	-	-	-	N.P.	-	-	-	-	2,661	-	-
	2235	-	-	-	-	-	N.P.	-	-	-	-	2,654	-	-
	2363	30,6	-	-	17,7	1,83	~ N.P.	-	-	-	0,58	2,687	-	-

NOTE: (1) USING G_s SHRINKAGE LIMIT METHOD

MIXTURES, CALCAREOUS SOILS AND MISCELLANEOUS SAMPLES

PARTICLE SIZE DISTRIBUTION – CUMULATIVE PERCENTAGE FINER BY WEIGHT (INCHES, MILLIMETRES, U.S. SIEVES)

3·00 76·1	2·00 50·8	1·50 38·1	1·00 25·4	0·75 19·0	0·50 12·7	0·25 6·35	0·185 4·76 4	0·0787 2·00 10	0·0331 0·841 20	0·0164 0·420 40	0·0098 0·250 60	0·0058 0·149 100	0·0029 0·074 200	0·0024 0·060	0·0008 0·020	0·0002 0·006	0·00008 0·002
-	-	-	-	-	100	92	87	82	76	55	28	14	10	-	-	-	-
-	-	100	93	92	86	73	68	56	46	40	32	18	12	9	6	4	3
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	30	18	11	8	7	6	5	3	3	2	0,4
-	-	-	-	-	-	-	21	13	8	6	6	5	4	3	2	1	0,3
-	-	-	-	-	-	-	100	87	77	72	61	39	27	23	20	19	12
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	-	100	93	92	92	89	88	84	79	71	60	43	32	25	23	19	16
-	-	-	-	100	96	87	83	68	46	32	25	20	15	14	10	7	4
-	-	-	-	-	-	-	100	99	93	77	58	45	40	23	22	22	19
-	-	-	-	-	-	-	-	100	96	84	62	41	27	17	14	14	14
-	-	-	-	-	-	100	94	93	90	83	70	59	53	36	34	33	30
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
-	-	-	-	-	-	100	94	94	87	71	53	38	30	22	18	18	17
-	-	-	-	-	-	-	-	100	99	96	92	88	84	68	65	62	48
-	-	-	-	-	-	-	-	-	-	100	94	56	51	2	1	1	1
-	-	-	-	-	-	-	-	-	100	97	66	26	12	5	4	3	3
-	-	-	-	-	-	-	-	-	100	95	60	23	6	5	5	5	5
-	-	-	-	-	-	-	-	-	100	88	51	23	7	4	2	2	-
-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-

TABLE 17 (continued)

MATERIAL TYPE	SAMPLE NUMBER	½"–¾" aggregate A.P.T. AFV %	APV %	MODIFIED AASHO OMC %	MDD kg/m³	CBR %	Grading modulus	Effective size mm	Uniformity coeff.	CLASSIFICATION D.O.T.	Extended Casagrande	Revised PRA
	896	-	-	-	-	-	1,53	0,074	6,1	SC	SF	A-2-4(0)
	900	-	-	-	-	-	1,92	0,065	47	SSC	GF	A-2-7(0)
	910	-	-	23,0	1538	-	-	-	-	-	-	-
	911	-	-	-	-	-	2,69	0,800	-	G	GC ?	A-1-a(0)
	912	-	-	-	-	-	2,77	1,00	-	G	GC ?	A-1-a(0)
	918	-	-	21,0	1538	60	1,14	0,0015	153	SC	SF	A-2-7(1)
	977	-	-	-	-	-	-	-	-	-	-	-
	1018	93	61	-	-	-	-	-	-	-	-	-
	1019	24	21	-	-	-	1,13	<0,001	-	SC	SF	A-2-7(1)
	1022	51	12	-	-	-	1,85	0,020	75	SSC	GF	A-2-6(1)
	2588											
CALCAREOUS SOILS	891	-	-	11,0	1954	5	0,84	<0,001	> 270	SC	SF	A-7-6(7)
	893	-	-	-	-	-	0,89	<0,001	>220	SC	SF	A-2-7(2)
	894	-	-	-	-	-	0,71	<0,001	>160	SC	CH	A-7-6(13)
	894 R	-	-	-	-	-	-	-	-	-	-	-
	916	-	-	12,0	1938	7	1,05	<0,001	>300	SC	SF	A-2-7(3)
MISCELLANEOUS	892	-	-	-	-	-	0,20	<0,001	>5,0	SC	CH	A-7-5(20)
	908	-	-	-	-	-	0,49	0,065	2,5	S	ML	A-4(0)
	909	-	-	-	-	-	0,91	0,070	3,3	S	SU	A-2-4(0)
	1965	-	-	-	-	-	0,99	0,090	2,8	S	SU	A-3(0)
	2235	-	-	-	-	-	1,05	0,090	2,9	S	SU	A-3(0)
	2363	-	-	~ 7	~2100	~96	-	-	-	-	-	-

REMARKS
MIXTURE OF CALCIFIED SAND AND NODULES, USED FOR GRAVEL ROAD
MIXTURE OF HARDPAN FRAGMENTS, NODULES AND CALCAREOUS SANDY CLAY, USED FOR GRAVEL ROAD
COMPACTED LUMPS (PROBABLY HARDPAN FRAGMENTS, NODULES AND CALCIFIED SAND) FROM GRAVEL ROAD
COMPACTED LUMPS (PROBABLY HARDPAN FRAGMENTS, NODULES AND CALCIFIED SAND) FROM GRAVEL ROAD
MAINLY CALCAREOUS SANDY CLAY, SOME HARDPAN FRAGMENTS AND NODULES, USED FOR GRAVEL ROAD
MIXTURE OF HARDPAN FRAGMENTS, NODULES AND TOPSOIL FROM UNFAILED GRAVEL ROAD SURFACE
MAINLY CALCIFIED WEATHERED FELSPATHIC SANDSTONE GRAVEL, SAID TO BE GOOD GRAVEL ROAD MATERIAL
CHIEFLY NODULAR - SEE TABLE H.2
CLAYEY SAND USED FOR GRAVEL ROAD
CLAYEY SAND USED FOR GRAVEL ROAD
SANDY CLAY USED FOR GRAVEL ROAD
CLAYEY SAND USED FOR GRAVEL ROAD
BLACK SANDY CLAY
PALE BROWN SAND OF KALAHARI TYPE
REDDISH BROWN SAND OF KALAHARI TYPE
REDDISH BROWN SAND OF KALAHARI TYPE
PALE BROWN SAND OF KALAHARI TYPE
NODULAR FERRICRETE BASE COURSE, PROBABLY SELF-STABILIZING

APPENDIX C

TABLE 18 Test results on six roads employing calcrete as base and subbase.

TABLE 18 TEST RESULTS ON SIX ROADS

ROAD	SAMPLING POINT	ROAD PROFILE	SAMPLE DETAILS	
			C.P.A. NUMBER	DEPTH
PORT ELIZABETH MARINE DRIVE	OPPOSITE NOORDHOEK QUARRY	?	DE. 7400	BASE
	"	"	DE. 7401	"
	"	"	DE. 7402	"
	"	"	DE. 7403	"
	"	"	DE. 7404	"
	MILE 5	?	DE. 7415	BASE
	" "	"	DE. 7416	"
	" "	"	DE. 7417	"
	" "	"	DE. 7418	"
	" "	"	DE. 7419	"
PORT ELIZABETH – ADDO	MILE 29,5	?	DE. 7380	BASE
	" "	"	DE. 7381	"
	" "	"	DE. 7382	"
	" "	"	DE. 7383	"
	" "	"	DE. 7384	"
	MILE 26	?	DE. 7385	BASE
	" "	"	DE. 7386	"
	" "	"	DE. 7387	"
	" "	"	DE. 7388	"
	" "	"	DE. 7389	"
LANDSDOWNE ROAD, CAPE FLATS	MILE 0,1	0 - 600mm CALCRETE	D. 9558	0 - 115mm
	" "	600mm + SAND	D. 9559	115 - 360mm
	MILE 0,2	0-300mm CALCRETE	D. 9561	0 - 100mm
	" "	300 - 430mm CALCRETE + SAND	D. 9562	100 - 300mm
		430mm + SAND + CALCRETE		
	–	–	–	–
	MILE 0,4	0 - 430mm CALCRETE	D. 9564	0 - 150mm
	" "	430mm + SAND	D. 9565	150 - 300mm
	MILE 0,5	0 - 270mm CALCRETE	D. 9566	0 - 150mm
	" "	270 - 460mm GREYISH SOIL	D. 9567	150 - 270mm
	MILE 0,6	0 - 330mm CALCRETE	D. 9569	0 - 75mm
	" "	330mm + GREYISH SOIL	D. 9570	75 - 200mm

EMPLOYING CALCRETE AS BASE AND SUBBASE

MATERIAL	PERCENTAGE PASSING (INCHES, MILLIMETERS, U.S. SIEVES)								
	2,0	1,5	1,05	0,742	0,525	0,185 4	0,078 2 10	0,0164 0,42 40	0,0021 0,05 270
CALCRETE	100	81	72	72	72	72	71	56	17
	98								
	98								
	100								
"	60	46	46	46	45	44	44	42	15
"	100	67	62	62	59	55	50	45	16
"		100	90	90	86	80	75	55	19
MIXTURE OF 7400 - 7403	-	-	-	-	-	-	-	-	-
CALCRETE	100	78	78	71	71	65	61	48	15
	100								
"	53	53	53	51	49	44	37	28	9
"	100	73	61	59	57	54	50	41	14
"	100	79	75	74	74	65	58	33	9
MIXTURE OF 7415 - 7418	-	-	-	-	-	-	-	-	-
CALCRETE		100	87	85	83	79	70	56	23
"		100	93	93	86	71	67	55	18
"		100	85	79	77	68	61	49	16
"		100	96	96	93	82	79	56	21
MIXTURE OF 7380 - 7383	-	-	-	-	-	-	-	-	-
CALCRETE		100	93	93	88	76	66	39	13
"			100	97	95	84	73	42	13
"				100	99	90	80	52	17
"		100	94	94	94	83	72	53	17
MIXTURE OF 7385 - 7388	-	-	-	-	-	-	-	-	-
CALCRETE		100	96	90	88	75	68	55	21
"	97	85	73	66	62	51	46	37	11
"	100	98	93	90	88	78	71	56	20
"	92	86	80	72	68	57	51	41	14
MIXTURE OF 9558 & 9561	-	-	-	-	-	-	-	-	-
CALCRETE	95	88	81	76	75	67	61	48	16
"	94	84	70	68	62	56	52	42	10
"	100	96	88	77	71	60	54	43	12
"	95	94	87	87	84	75	69	56	16
"	99	97	87	85	78	63	57	45	15
"	96	92	86	80	73	59	53	43	13

TABLE 18 (continued)

ROAD	C.P.A. NUMBER	HYDROMETER ANALYSIS (mm) SAND COARSE <2,0 >0,42	<0,42 >0,246	FINE <0,246 >0,147	<0,147 >0,05	SILT <0,05 >0,005	CLAY <0,005	CONSTANTS LL %	PI %	LS %
PORT ELIZABETH MARINE DRIVE	DE. 7400	21	12	20	23	8	16	22	4	2,5
	DE. 7401	5	12	30	19	14	20	23	5	2,5
	DE. 7402	10	17	25	15	13	20	23	5	2,5
	DE. 7403	27	10	24	14	9	16	21	5	2,5
	DE. 7404	-	-	-	-	-	-	-	-	-
	DE. 7415	21	15	22	17	9	16	23	8	3,5
	DE. 7416	24	14	21	17	9	15	24	9	4,0
	DE. 7417	18	15	20	20	11	17	23	8	3,5
	DE. 7418	43	10	18	13	6	10	20	6	2,5
	DE. 7419	-	-	-	-	-	-	-	-	-
PORT ELIZABETH - ADDO	DE. 7380	20	8	11	29	20	12	28	12	5,5
	DE. 7381	18	8	13	34	18	9	31	14	6,0
	DE. 7382	20	10	13	31	16	11	31	11	5,5
	DE. 7383	29	8	11	25	17	9	28	12	5,5
	DE. 7384	-	-	-	-	-	-	-	-	-
	DE. 7385	41	10	11	19	10	9	32	14	6,5
	DE. 7386	43	10	10	19	10	8	33	15	6,5
	DE. 7387	35	9	12	23	11	10	31	13	6,0
	DE. 7388	26	12	12	26	12	12	31	13	6,0
	DE. 7389	-	-	-	-	-	-	-	-	-
LANDSDOWNE ROAD, CAPE FLATS	D. 9558	19	16	15	19	11	20	19	7	3,0
	D. 9559	20	18	18	20	12	13	19	6	2,5
	D. 9561	21	16	17	19	11	17	19	6	3,0
	D. 9562	20	18	17	19	12	15	19	6	2,5
	-	-	-	-	-	-	-	-	-	-
	D. 9564	21	17	16	19	11	15	20	8	3,0
	D. 9565	19	19	21	20	10	10	18	6	2,5
	D. 9566	20	17	19	21	14	9	17	6	2,5
	D. 9567	19	19	20	19	12	11	18	7	2,5
	D. 9569	21	16	18	20	13	12	18	5	2,5
	D. 9570	19	17	18	22	16	8	18	5	2,5

DENSITY			MOD. AASHO OMC, CBR AND SWELL										
INSITU Kg/m³	M. AASHO Kg/m³	RELATIVE %	OMC %	4 DAYS SOAKING		CURED 7 DAYS SOAKED 4 DAYS		CURED 28 DAYS SOAKED 4 DAYS		APPROX. GRADING MODULUS	APPROX. GROUP INDEX	LS X % - 0,420mm	REVISED PRA CLASSIF.
				CBR %	SWELL %	CBR %	SWELL %	CBR %	SWELL %				
-	-	-	-	-	-	-	-	-	-	1,5	0,0	140	A-2-4(o)
-	-	-	-	-	-	-	-	-	-	1,9	0,0	105	A-1- (o)
-	-	-	-	-	-	-	-	-	-	1,8	0,0	112	A-1- (o)
-	-	-	-	-	-	-	-	-	-	1,5	0,0	137	A-2-4(o)
-	1762	-	14,4	90	0,1	38	0,0	38	0,0	-	-	124	—
-	-	-	-	-	-	-	-	-	-	1,7	0,0	168	A-1- (o)
-	-	-	-	-	-	-	-	-	-	2,2	0,0	112	A-2-4(o)
-	-	-	-	-	-	-	-	-	-	1,8	0,0	143	A-2-4(o)
-	-	-	-	-	-	-	-	-	-	2,0	0,0	83	A-1- (o)
-	1858	-	12,4	62	0,0	85	0,0	63	0,0	-	-	127	—
-	-	-	-	-	-	-	-	-	-	1,4	0,4	308	A-2-6(o)
-	-	-	-	-	-	-	-	-	-	1,4	0,5	330	A-2-6(1)
-	-	-	-	-	-	-	-	-	-	1,7	0,2	270	A-2-6(o)
-	-	-	-	-	-	-	-	-	-	1,4	0,3	308	A-2-6(o)
-	1858	-	12,2	66	0,2	75	0,0	68	0,1	1,5	0,4	304	—
-	-	-	-	-	-	-	-	-	-	1,8	0,2	254	A-2-6(o)
-	-	-	-	-	-	-	-	-	-	1,7	0,3	273	A-2-6(o)
-	-	-	-	-	-	-	-	-	-	1,5	0,3	312	A-2-6(o)
-	-	-	-	-	-	-	-	-	-	1,5	0,3	318	A-2-6(o)
-	1874	-	13,1	47	0,1	29	0,0	19	<0,05	1,6	0,3	289	—
1938	1938	100	-	-	-	-	-	-	-	1,5	0,0	165	A-2-4(o)
-	-	-	-	-	-	-	-	-	-	2,0	0,0	93	A-1-b(o)
1970	1938	102	-	-	-	-	-	-	-	1,5	0,0	168	A-2-4(o)
-		-	-	-	-	-	-	-	-	1,9	0,0	102	A-1-b(o)
-	1938	-	10,0	112	0,0	-	-	-	-	1,7	0,0	132	—
1890	1938	98	10,6	125	0,0	-	-	-	-	1,7	0,0	144	A-2-4(o)
-	-	-	-	-	-	-	-	-	-	1,9	0,0	105	A-1-b(o)
1938	2003	97	9,5	134	0,0	-	-	-	-	1,9	0,0	107	A-1-b(o)
-	-	-	-	-	-	-	-	-	-	1,5	0,0	140	A-2-4(o)
1938	±1954	±99	-	-	-	-	-	-	-	1,8	0,0	112	A-1-b(o)
-	-	-	-	-	-	-	-	-	-	1,9	0,0	107	A-1-b(o)

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TABLE 18

ROAD	SAMPLING POINT	ROAD PROFILE	SAMPLE DETAILS	
			C.P.A. NUMBER	DEPTH mm
DUINE FONTEIN ROAD, CAPE FLATS	MILE 0,2	0 - 330mm CALCRETE	V 396	0 - 130
	" "	330mm + SAND	V 397	130 - 250
	MILE 0,6	0 - 360mm CALCRETE	V 400	0 - 150
	" "	360mm + SAND	V 401	150 - 360
	MILE 1,0	0 - 200mm CALCRETE	V 402	0 - 200
	" "	200mm + SAND	V 403	200 - 460
	MILE 1,4	0 - 330mm CALCRETE	V 404	0 - 100
	" "	330 - 460mm SAND	V 405	100 - 330
	MILE 1,8	0 - 360mm CALCRETE	V 406	0 - 100
	" "	360 - 460mm SAND	V 407	100 - 360
WELTEVREDE ROAD, CAPE FLATS	MILE 0,1	0 - 360mm CALCRETE	V 2327	0 - 100mm
	" "		V 2328	100 - 360mm
	MILE 0,2	0 - 400mm CALCRETE	V 2329	0 - 100mm
	" "		V 2330	100 - 400mm
	MILE 0,3	0 - 330mm CALCRETE	V 2331	0 - 130mm
	" "	330mm + SAND	V 2332	130 - 330mm
	MILE 0,4	0 - 300mm CALCRETE	V 2333	0 - 75mm
	" "	300mm + SAND	V 2334	75 - 300mm
OLIEBOOM ROAD, CAPE FLATS	A	0 - 280mm CALCRETE	B 3230	0 - 140mm
	"	280 - 460mm LATERITIC SANDSTONE	B 3231	140 - 280mm
	"	460mm + SAND	B 3232	280 - 460mm
	B	0 - 150mm CALCRETE	B 3233	0 - 150mm
	"	150 - 330mm LATERITIC SANDSTONE	B 3234	150 - 330mm
	"	330mm + SAND	B 3235	330 - 460mm
	A & B	–	MIXTURE OF B 3230 & B 3233	–
	"	–	MIXTURE OF B 3232 & B 3234	–

2nd GROUP

MATERIAL	PERCENTAGE PASSING (INCHES, MILLIMETERS, U.S. SIEVES)								
	2,0	1,5	1,05	0,742	0,525	0,185 4	0,078 2 10	0,0164 0,42 40	0,0021 0,05 270
CALCRETE	100	99	96	92	80	77	69	53	18
"	100	95	91	87	81	74	68	54	18
CALCRETE		100	93	90	87	74	66	52	17
"	81	79	75	71	69	62	58	46	16
CALCRETE	100	99	99	97	96	91	88	74	16
SAND							100	74	2
CALCRETE	97	91	88	83	77	67	61	48	16
"		100	97	96	95	87	81	61	19
CALCRETE	100	95	95	86	82	72	65	52	18
"	100	98	91	81	81	71	64	49	14
CALCRETE	100	98	90	88	82	66	59	44	15
"		100	94	89	85	76	69	52	17
CALCRETE		100	94	90	84	68	59	44	15
"	90	87	78	75	70	57	51	37	13
CALCRETE	98	90	84	76	72	61	55	40	14
"	100	98	98	91	89	78	71	54	18
CALCRETE		100	95	88	82	70	63	46	15
"	100	94	94	88	76	66	59	44	15
CALCRETE	100-85	72	60	55	49	37	32	22	6
"	100-88	73	64	52	47	37	32	22	6
LAT. SANDSTONE	100-95	95	95	88	86	77	73	54	4
CALCRETE	100-96	83	70	64	58	48	43	32	9
LAT. SANDSTONE	100	98	98	95	90	80	77	69	3
GREY SAND	-	-	-	-	-	-	100	67	1
CALCRETE	-	-	-	-	-	-	-	-	-
LAT. SANDSTONE	-	-	-	-	-	-	-	-	-
	-	-	-	-	-	-	-	-	-

TABLE 18 (continued)

ROAD	C.P.A. NUMBER	HYDROMETER ANALYSIS (mm)						CONSTANTS		
		SAND				SILT	CLAY			
		COARSE		FINE		< 0,05 > 0,005	< 0,005	LL %	PI %	LS %
		< 2,0 > 0,42	< 0,42 > 0,246	< 0,246 > 0,147	< 0,147 > 0,05					
DUINE FONTEIN ROAD, CAPE FLATS	V 396	23	15	17	19	13	13	18	7	3,0
	V 397	21	15	17	21	14	13	21	8	3,0
	V 400	21	16	17	20	14	12	19	6	2,5
	V 401	21	15	16	21	14	13	20	7	3,5
	V 402	16	23	25	18	10	8	18	4	1,5
	V 403	26	34	30	7	1	1	-	NP	0,0
	V 404	21	16	16	20	13	13	18	5	2,5
	V 405	25	15	18	19	12	12	19	6	2,5
	V 406	20	15	18	21	14	13	20	6	2,5
	V 407	23	16	15	23	13	9	20	6	2,5
WELTEVREDE ROAD, CAPE FLATS	V 2327	25	17	17	16	11	14	21	6	2,5
	V 2328	25	17	17	16	11	14	18	6	2,5
	V 2329	25	16	16	16	11	15	18	6	2,5
	V 2330	27	18	17	14	10	15	18	7	2,5
	V 2331	27	18	17	14	10	15	19	7	2,5
	V 2332	24	18	18	15	11	14	18	6	2,5
	V 2333	27	16	17	16	10	14	19	5	2,5
	V 2334	25	17	17	15	12	13	18	5	2,5
OLIEBOOM ROAD, CAPE FLATS	B 3230	31	16	17	17	10	9	21	4	2,0
	B 3231	31	16	17	17	12	8	21	4	1,5
	B 3232	26	17	24	27	3	2	-	NP	0,0
	B 3233	26	16	17	20	9	12	20	5	2,0
	B 3234	10	13	33	39	3	2	-	NP	0,0
	B 3235	33	32	24	10	1	0	-	NP	0,0
	MIXTURE OF B 3230 & B 3233	-	-	-	-	-	-	-	-	-
	MIXTURE OF B 3232 & B 3234	-	-	-	-	-	-	-	-	-
		-	-	-	-	-	-	-	-	-

DENSITY			MOD. AASHO OMC, CBR AND SWELL							APPROX. GRADING MODULUS	APPROX. GROUP INDEX	LS X %- 0,420mm	REVISED PRA CLASSIF.
INSITU kg/m³	M. AASHO kg/m³	RELATIVE %	OMC %	4 DAYS SOAKING		CURED 7 DAYS SOAKED 4 DAYS		CURED 28 DAYS SOAKED 4 DAYS					
				CBR 0,1/0,2	SWELL %	CBR %	SWELL %	CBR %	SWELL %				
2083	2035	102	9,7	87	0,0	-	-	-	-	1,6	0,0	159	A-2-4(o)
-	-	-	-	-	-	-	-	-	-	1,6	0,0	162	A-2-4(o)
2067	2035	102	-	-	-	-	-	-	-	1,6	0,0	130	A-2-4(o)
-	-	-	-	-	-	-	-	-	-	1,7	0,0	161	A-2-4(o)
1890	1970	96	9,9	107	0,0	-	-	-	-	1,2	0,0	111	A-2-4(o)
-	-	-	-	-	-	-	-	-	-	1,2	0,0	0	A-3 (o)
2115	2035	104	-	-	-	-	-	-	-	1,7	0,0	120	A-1-b(o)
-	-	-	-	-	-	-	-	-	-	1,4	0,0	152	A-2-4(o)
2051	2035	101	-	-	-	-	-	-	-	1,7	0,0	130	A-2-4(o)
-	-	-	-	-	-	-	-	-	-	1,7	0,0	122	A-1-b(o)
1970	1986	99	10,9	95	0,0	-	-	-	-	1,8	0,0	110	A-1-b(o)
-	-	-	-	-	-	-	-	-	-	1,6	0,0	130	A-2-4(o)
1890	±1938	±98	-	-	-	-	-	-	-	1,8	0,0	110	A-1-b(o)
-	-	-	-	-	-	-	-	-	-	1,9	0,0	93	A-2-4(o)
1794	1890	95	12,5	98	0,0	-	-	-	-	1,9	0,0	100	A-2-4(o)
-	-	-	-	-	-	-	-	-	-	1,5	0,0	135	A-2-4(o)
1890	±1938	±98	-	-	-	-	-	-	-	1,7	0,0	115	A-1-b(o)
-	-	-	-	-	-	-	-	-	-	1,8	0,0	110	A-1-b(o)
2019	1954	103	11,1	-	-	-	-	-	-	2,4	0,0	44	A-1-a(o)
1858	1954	94	-	-	-	-	-	-	-	2,4	0,0	33	A-1-a(o)
1666	1746	95	14,6	-	-	-	-	-	-	1,6	0,0	0	A-3 (o)
2051	1954	105	-	-	-	-	-	-	-	2,1	0,0	64	I-I-b(o)
1650	1746	94	-	-	-	-	-	-	-	1,5	0,0	0	A-3 (o)
1730	1746	99	11,0	-	-	-	-	-	-	1,3	0,0	0	A-3 (o)
-	1954	100	11,1	110/146	0,0	-	-	-	-	2,2	0,0	0	A-3 (o)
-	1746	100	14,6	53/69	0,0	-	-	-	-	1,6	0,0	54	-
-	-	-	-	-	-	-	-	-	-	-	-	0	-