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FOREWORD

Calcretes have always been a problem - sometimes perform well and sometimes don't - not always predictable by standard tests. Roadbuilders' opinions vary from that they are above average materials to that they should not be used as base.

In large tracts of the drier parts of Southern Africa calcretes are the only conventional roadbuilding materials. They are, however, often very difficult to find.

Details of how the project came about, initial sponsorship by S.W.A.A.R.

Netterberg on secondment from Kantey and Templer for part of the time?

This report represents a condensation of parts of the detailed report submitted by Dr. Netterberg as a doctoral thesis to the University of the Witwatersrand.

While including sufficient of the more fundamental aspects to promote a better understanding of these materials, this research report is concerned mainly with the more practical aspects of the use of calcretes in road construction and it is hoped that it will play a part in the progress towards cheaper and better calcrete roads.

P.J. RIGDEN

DIRECTOR : NATIONAL INSTITUTE FOR ROAD RESEARCH.

C O N T E N T S

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SYNOPSIS

The composition, distribution, origin and age of South African calcretes are briefly reviewed and simple classification suitable for engineering use described. Methods of searching for economic calcrete deposits and of testing calcretes for roadmaking are discussed and it is concluded that special methods of location and precautions during testing are necessary if the best results are to be achieved. The engineering properties of calcretes and specifications for calcrete roads are also dealt with in some detail. Depending on stage of development, these materials range in properties from an almost useless powder to that 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 are 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.

INTRODUCTION

Calcretes are a common sight in arid and semiarid lands everywhere and in Southern Africa they constitute in addition an important source of borrow for road construction. According to the areal extent of its use in the Republic (45.1%), calcrete ranks third after dolerite (48.9%) and sandstone (46.1%) in importance[2] and is second only to dolerite with respect to the actual quantities of materials used (Dr. H.H. Weinert, 1969; personal comm.). It probably rates as the most widely used road construction material in Southern Africa as a whole.

A calcrete can be regarded simply as almost any terrestial originally unconsolidated material which has undergone cementation and/or replacement by dominantly calcium carbonate. The unconsolidated material could be alluvium, colluvium, weathered rock, aeolian sand, residual soil or a soil in the pedological sense, but for the sake of convention, cave deposits are excluded. The carbonate may be calcite or dolomite and a calcrete may be pedogenic or non-pedogenic in origin, or both.

The field work undertaken and samples tested in connection with this project were carried out and came from almost entirely within South West Africa and the Republic of South Africa. The extensive deposits of calcrete in Botswana were not visited. It is thought, however, that most of the results of this work can probably be applied to all calcrete deposits everywhere. 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.

COMPOSITION/ ....

COMPOSITION

By definition, a calcrete is composed of the cementing and/or replacing carbonate and the host material cemented and/or replaced by it. Its composition is thus extremely variable and the term 'calcrete' can justifiably be applied to materials varying from an almost pure limestone in which hardly any trace of the host material can be seen to a material consisting to a large extent of the host material. The carbonate is usually dominantly calcite, but dolomite is often present, sometimes in amounts which would justify calling the material a dolocrete. No other carbonates have been positively identified from 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 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. The matrix is often veined and apparently replaced by coarser (often five micron) probably recrystallized calcite. Replacement of quartz and felspars by the coarser calcite is common and probably provides the main source of the opaline silica cement which silicifies some calcretes, resulting in Mohs' hardnesses of up to 6 being attained. Calcretes tend to increase in carbonate content relative to the allogenic component with stage of development and the average hardpan calcrete compares reasonably with Clark's average 'limestone'.

Powder calcretes, nodular calcretes and to a lesser extent

honeycomb/ ....

honeycomb calcretes are composed of two consistency components: indurated nodules, cutans, pedotubules or coalesced nodules (honeycomb types) which are composed as described above and the looser matrix in which they occur. 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 on moving about under the microscope, but can be broken by pressure from a needle. On treatment with acid they can be seen to contain hundreds of finer particles. The strength of the particles and their proportion of cementing matrix to cemented grains increases with particle size and the larger nodules may possess aggregate crushing values of below 20%. The minus two millimetre and minus 0.42 mm fractions of nodular calcretes are often composed largely of carbonate-cemented particles. Fossil calcretes may, however, possess non- or only slightly calcareous fines. The mineralogy of the above 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 two micron fraction is largely composed of calcite, quartz, clay minerals and dolomite.

Attapulgite, montmorillonite and sepiolite are probably the most commonly encountered clay minerals in calcretes. 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 largely calcium or magnesium saturated.

Calcretes possess therefore a composition which is unusual among road materials. The high carbonate content, the unusual clay minerals present (attapulgite and sepiolite) and the presence of amorphous silica microfossil remains can be expected to cause them to exhibit some unusual properties.

CLASSIFICATION/ ......

CLASSIFICATION

Many calcrete profiles are much more complicated than those illustrated here, but all the different types of calcrete seen in the Republic and South West Africa can be resolved into six basic types: calcified soils, powder calcretes, nodular calcretes, honeycomb calcretes, hardpan calcretes, and boulder calcretes. They can easily be recognised in the field by untrained personnel and each group possess a significantly different range of engineering properties as well as representing particular stages in the development of a pedogenic calcrete. The classification thus possesses both engineering and geological significance.

Calcified Soil

Calcified soil represents an early stage of calcrete formation in which the host material has normally not yet become very strongly cemented. A calcified soil does not normally exhibit nodular structure. The total carbonate content (calcite plus dolomite) varies between about 10 and 50 percent and largely occurs only filling the voids between the host grains, in other words the carbonate has not yet succeeded in mechanically replacing much of the host material. The common types are calcified gravels (Plate 1) and calcified sands (Plate 2), but it is occasionally convenient to place other materials like calcified weathered rock and the occasional calcified clay in this category as well. 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 filling fissures and cementing the fragments in calcified weathered rocks.

The consistency[3] of most calcified sands is only firm or stiff, but although often also friable they do not normally slake in water. Calcified gravels such as those in Plate 2 may however be very stiff in consistency and are picked with difficulty.

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

<u>Powder calcrete</u>

Usually composed of a loose powder consisting of predominantly silt and sand-sized carbonate particles and little or no nodular development (Plate 3). Nodular calcretes may form in host powder calcretes and a grading modulus of 1.5 affords a useful separation between the two, though the really typical powder calcrete possesses a grading modulus nearer 0.6. (The grading modulus is defined as the cumulative percentage retained on the U.S. No. 10, 40 and 200 sieves divided by 100[4]). Powder calcretes can usually readily be differentiated from calcified sands by their very low content of host soil grains, while calcified sands invariably possess a high percentage of sand-sized quartz grains. Apart from normally also having formed in a different host material, a powder calcrete has also succeeded in mechanically replacing most of its host material with fine carbonate.

<u>Nodular calcrete</u>

Discrete, usually intact, soft to very hard concentrations of carbonate-cemented host material (nodules) in a usually calcareous

matrix/ ....

matrix - a gravel in fact (Plate 4). The nodules actually vary in size from silt size to about 6 cms ($2\frac{1}{2}$ inches) and their shape varies from more or less spherical in the early stages of development to highly irregular in the later stages when several nodules have coalesced together. 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 size.

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 an horizon of much greater overall strength than a nodular calcrete, but relatively uncemented material still fills the voids between the partly coalesced nodules. 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 therefore usually be ripped and crushed by grid rolling to yield a gravel composed of relatively hard and angular fragments.

Hardpan calcrete

Hardpans usually occur as a harder sheetlike crust seldom more than about eighteen inches thick overlying less well cemented material of a lower stage of development (Plate 6). If the hardpan was previously a honeycomb calcrete, it represents the final stage wherein the voids between the coalesced nodules have become completely cemented up. The original nodules are usually no longer readily visible. If the hardpan was previously a calcified soil, it represents the final stage wherein the carbonate content has 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. Little of the host material is sometimes visible to the naked eye, but when it is appreciable, the hardpan may be described as gravelly or sandy, depending on the size of the particles visible.

Hardpans are usually intact, but may be shattered (showing the onset of weathering), bedded (due to bedding inherited from the host material) platy (due to it having formed in thin plate-like layers), or tufaceous (the vlei limestone, diatomaceous pan limestone, Pfannenkalktuff and Schneckenkalk of previous authors). The tufaceous types usually posses very low densities (75 - 95 lbs/cu ft) in comparison with other hardpans, which range from about 135 to 165 lbs/cu ft, usually being more than 150 lbs/cu ft.

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 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 tend to shatter and disintegrate to more angular boulders which exhibit the usual limestone solution facetting on their upper surfaces exposed to rainfall.

Boulder calcrete is usually rippable, in spite of the interiors

of/ .....

of the boulders being usually still hard and sound (except for badly shattered outcropping types). The boulders are however difficult to reduce further, usually being too hard for grid rolling and often too large to crush.

DETAILED DESCRIPTION

When detailed description of a calcrete or calcrete-containing profile is necessary for engineering purposes, it is recommended that a combination of the general soil description method of Jennings and Brink[3] and this calcrete classification described be employed, the calcrete classification falling under the heading of 'origin' and 'inclusions' in the case of nodular and powder calcretes and under 'soil type' in the case of the indurated types. An example of such a procedure and the symbols suggested is shown in Figure 1.

DISTRIBUTION

Figure 2 shows the distribution of the different types of calcrete in Southern Africa. It can be seen that calcretes sufficiently well developed for use in road construction generally only occur in the drier areas, that is where the rainfall is less than about 550 mm (22 inches). Various climatic indices also afford good correlations with the limit of usable calcrete occurrence in Southern Africa. Usable calcrete generally only occurs where Weinert's[2] N = 5 line is more than 5, in areas possessing a Thornwaite[5] water deficiency of more than 20 cm (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 550-800 mm (22-32 inches) of rainfall have hardly altered the engineering properties of their host materials (except to calcium saturate the clay minerals present) and the nodular calcretes occurring in this region are normally too thin to be economically worked for road material.

Calcification is generally absent in the area receiving more than about 800 mm (22 inches) of rainfall. This area is also well defined by the zone enjoying an annual water surplus according to Thornthwaite[5] (moisture index of 0).

While it is apparent that climate, particularly rainfall, plays a strong part in determining the distribution of calcretes, it is only one of the 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 distribution of calcrete and climate is thus not possible. Probably almost as important as climate is the

availability/ ......

availability of carbonate, which in turn is controlled by parent material, topography (drainage) and climate.

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 owe their origin to one or both of two mechanisms: deposition of carbonate in the host material above a shallow water table, and downward leaching of carbonate from the upper soil horizons by infiltrating rainwater and its deposition lower down in the profile. These two basic origins can be simply referred to as 'non-pedogenic' and 'pedogenic' respectively, though this perhaps would not meet with universal approval.

Examples of calcretes of the first type are thought to be 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.

Pedogenic calcretes are very common and the mature profile typically consists 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 and non-pedogenic calcretes are almost invariably capped by a crust of pedogenic hardpan calcrete. Non-pedogenic calcretes can form in a completely non-calcareous profile.

Stages of development

The different calcretes of the suggested classification

represents/ ....

represent the possible stages in the development of a pedogenic calcrete and appear to be related as shown in Table 1.

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 (SOLUANS)	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 (rocklike sheet)	
5			BOULDER CALCRETE (discrete boulders formed by weathering)	

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 shown are possible. The stages shown are merely those which are easily recognised, easily named

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and which possess significantly different engineering properties.

Any of the developmental stages in Table 3 may be pedogenic in origin, but it is thought that non-pedogenic 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 <u>development</u> of all calcretes. Boulder calcrete represents the weathering stage.

Mechanisms of calcification

The usual mechanism involved by most authors to account for carbonate crystallisation from solution is simply evaporation. There is however evidence to show that simple evaporation can only be important in about the upper three feet of the profile and that the effect of soil suction (pF) changes on the solubility of carbonate is probably the most important mechanism of 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 only of importance in the formation of the parent materials of the tufaceous calcretes. 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}) of 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) in CO_2-free water ($P_{CO_2} = 0$) is about 12 parts per million (ppm), while in equilibrium with atmospheric air ($P_{CO_2} = 0.00032$, equivalent to 0.032% by volume) it rises to 53 ppm. An increase in temperature lowers both the

solubility/ ....

solubility of CO_2 and the solubility of carbonate in water. The net result is that solution of carbonate is favoured by an increase in water content, CO_2 content and pressure, and a decrease in temperatures. Precipitation of carbonate is favoured by an increase in temperature and a decrease in water content, CO_2 content and pressure.

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. The presence of vegetation would complicate, but not invalidate the argument. Two practical examples are newly deposited alluvium and conditions too saline to support vegetation (for example the floor of the Etosha Pan, which is undergoing solonchak soil formation at its edges.) 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 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 the 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 gradient is increased and the line curves away to more negative pressures (higher suctions of pF) in the upper few feet (A). (For moisture to move upwards in the liquid phase against gravity, the suction gradient must 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 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 precipitate carbonate, regardless of whether evaporation takes place or not. If the soil air contains 0.032% CO_2, the solubility of calcite at 25°C at the water table would be 53 ppm. Below the water table the solubility would be greater than this and above it less. At a height of 100 cm (3.3 ft), equivalent to a pF of 2 (minus 1.42 psi) the solubility should theoretically be about 51 ppm if the pore water pressure diagram is still an extention of the hydrostatic line at this point. It can be similarly calculated that at a pF of 3.02 (equivalent to minus 14.7 psi, i.e. one atmosphere or 1036 cm (34 ft) of water) or more, the solubility should only be 12 ppm. As the equilibrium relative humidity of the soil air is almost 100% at suctions less than 3, it is apparent that liquid water moving upwards from the water table will have lost 53 minus 12 = 41 ppm or 77% of its dissolved $CaCO_3$ before any evaporation has even taken place. Only 23% of the $CaCo_3$ remains to be precipitated by evaporation higher up in the profile. Most (75%) crystallisation of $CaCO_3$ would take place at depths corresponding to where the pF changes from 2 to 3. The actual depth in practice that this pF change would take place depends on the suction profile, which in turn depends on the depth of the water table and the extent to which evapo-transpiration is desiccating the soil profile.

It can also be seen that the solubility of carbonate depends on the position of the water table. If the water table rises, the potential solubility at every point in the profile possessing a pF of less than 3 is increased. Similarly, if the water table drops, precipitation of carbonate will tend to occur, again mostly at depths where the pF changes from 2 to 3. It is thought that most non-pedogenic calcretes such as calcified alluviums are formed by the mechanism outlined 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 start 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 to the upper part of the pore water pressure diagram

during/ ....

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 will 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 (corresponding to the depth at which the pF changes from 2 to 3) not directly by evaporation, but as the pF changes, partly due to evaporation from the surface and partly due to an attempt to re-establish suction equilibrium. The effect of transpiring vegetation is merely 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 similar arguments to those preciously given that, once again, most deposition will 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 transpiration normally accounts for a much greater water loss from the soil than does evaporation.

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 - 13,000,000 years old), First Intermediate (35,000 - 45,000 years B.C.?), Second Intermediate (9 - 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 Terrance Gravels of the Ugab. First Intermediate calcretes are also mainly calcified alluviums. Second Intermediate calcretes are particularly widespread and most of the calcretes

used/ ....

used by the road builder were probably formed at this time. 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 and a few immature types like scattered 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 before present a hardpan for example could, however, be younger, having reached that stage more recently than for example the nodular calcrete underlying it. It is not true to state for example that all hardpans are older than all nodular calcretes, but it would be correct to state that all hardpans are more developed than all nodular calcretes.

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 which are fossil. Likely areas of calcrete occurrence are at the sides of pans and drainage lines, both present day and fossil, and these can often te detected on airphotos. The insides of bends are the best places to look (Plate 8). White calcareous anthills show up well on airphotos, but generally indicate fairly plastic calcrete below.

Various calciphilous plants have proved to be very useful indicators of the presence of calcrete or calcareous soil at shallow depths in Southern Africa. The gabbabos (Catophractes alexandri) is perhaps the most reliable and occurs almost all over that part of South West Africa receiving less than about 500 mm (20 inches) of rainfall. In Ovamboland where the rainfall is more than about 500 mm its place is taken by the saliebos (pechuel-Loeschea-leubnitziae). The vaalbos (Tarchonanthus camphoratus) is another useful

indicator/ ....

indicator in the South West African Okavango and in the northern Cape Province. A dense growth of these bushes usually indicates a calcrete rather than only a calcareous soil deposit, and they sometimes give rise to characteristic airphoto patterns. (PHOTOGRAPHS OF THESE BUSHES ARE AVAILABLE).

A most useful preliminary indication of the presence or absence of calcrete within a usable depth can be obtained in less than a minute in sandy soils by means of a rapid probing device (Figure 4) invented by the late Senior Road Inspector at Upington, Mr. van der Westhuizen (Figure 4). 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 affect 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 much faster (about ten times) than any hand auger.

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

It is considered that the combined use of airphoto interpretation, topographic setting, plant indicators, probing and pitting offers the best solution to the problem at present. Even using all these tools presently available, the certainty of detecting all calcrete deposits in a given area is probably less than 100%. Future work on prospecting methods for calcretes should include the use of infrared imagery, and colour, camouflage detection,

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and infrared films.

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 into larger, somewhat porous particles by the carbonate. Silt-sized calcite is non-plastic and does not shrink when dried from the liquid limit, but it does possess a measurable liquid limit of about 22. The engineering properties of a calcrete therefore depend largely on the nature (mainly texture) of the host material and the extent to which it has been cemented and replaced by the carbonate. If for example the host material was highly plastic, the calcrete will also be plastic, though less so than the host material, while if the host material was a non-plastic sand, the calcrete will also be non-plastic, though its liquid limit might be higher than that of the sand. Once the carbonate has firmly cemented and begun to separate the host grains (usually at about 50% $CaCO_3$ + $MgCO_3$ 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 3), 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 3 that soil texture plays a very important role in the early stages of calcrete development, given time and the right condition, all calcretes will develop a hardpan. In general, the crushing strength, bulk density, hardness and grading modulus increase with development, while 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 which are common to all calcretes, and then to deal with the properties of each type. The following remarks on soil constants, particles specific gravity, self-cementation, chemical stabilization and soluble salt problems apply particularly to calcified soils, powder calcretes and nodular calcretes, but could also be applied to a hardpan crusher run for example.

Soil Constants

As a host material becomes calcified, the exchange sites of the clay minerals present will become saturated with calcium, if this is not already the case. Only one or two percent $CaCO_3$ is necessary for this and the effect may be decidedly beneficial. If for example the clay mineral present is sodium montmorillonite, exchange of the Na^+ for Ca^{++} would reduce the liquid limit and plasticity index to as little as half of their original values. If the clay mineral present is attapulgite or illite, relatively little effect, or even a slight increase would be noticed, but if much magnesium is present (as in a dolocrete or very dolomitic calcrete) the plasticity and liquid limit of attapulgite may be lowered. Further addition of carbonate merely acts as a mechanical stabilizer. Experiments 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 22 at 100% silt sized $CaCO_3$. The plastic limit was found to be linearly decreased by additions of $CaCO_3$ of up to 50% down to a plastic limit of about 22, beyond which no further decrease took place. 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 relation 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/ .....

area of the Casagrande chart and that they do not obey these two published average liquid limit versus PI relationships.

The shrinkage limits of calcretes are often higher 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[15] results. Its cause is unknown and it is not even certain whether it is caused by a deficiency in the test method or an intrinsic property of the calcretes, such as the high porosity of their particles or the unusual clay minerals often present (attapulgite and sepiolite). The shrinkage limit is both the water content at which shrinkage ceases and the boundary between the semi-solid and solid states. A material can thus theoretically not be plastic below the shrinkage limit. These results cast doubt upon the validity of Atterberg limits and/or the test methods employed when applied to calcretes. The effect of $CaCO_3$ on the shrinkage limit is unknown. (CONSIDER EFFECT OF DIFFERENT SHRINKAGE LIMIT METHOD). A number of persons spoken to during the course of this investigation expressed doubt as to the reliability of Atterberg limits on calcretes and the term 'apparent PI' has been used (Dr. H.H. Weinert, personal comm.). As will be shown presently, these doubts are well founded, and this term will be used in this report whenever it is desired to emphasize the point. As used in this report it is however 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 relation between linear shrinkage and liquid limit for the calcretes tested compared with the published relations for the Transvaal[14] and Texas[15] materials. It can be seen that once again, neither of the two published curves provide a good fit and it can also be seen that calcretes tend to have higher liquid limits for the same shrinkage than do other materials. This may be 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/ ....

The high shrinkage limits of calcretes also indicate that shrinkage ceases during drying at a high moisture content (some moisture probably occurs in the probably porous particles) thus causing the linear shrinkage to be low in comparison with the liquid limit and plasticity index. The calcretes with the highest values of shrinkage limit minus plastic limit are those which contain the most fossil diatoms. Diatoms (tiny microfossil skeletons) can be expected to hold water in their shells, but are non-plastic.

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 was kindly made available by a number of sources.

It can be seen that there is generally a good correlation between the results of the test, indicating that the two tests are generally measuring the same property[14]. 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 many calcretes tend to yield a somewhat higher PI for the same linear shrinkages than do many other materials, especially dolerites. It is considered to be at least partly due to the porous nature of the calcrete particles, which would raise all the Atterberg limits but not affect the shrinkage. In addition, the most common clay mineral in calcretes is probably attapulgite, which possesses a PI equal to calcium montmorillonite[17] (and an even greater liquid limit), but is non-expansive.

Particle specific gravity

The mean particle specific gravity of the minus 40 U.S. mesh (0.42 mm) 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 particle specific gravity

TABLE 2 /...

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					
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 - 2.33	0.84	43
Weathered dolerites	Bloemfontein-Winburg					
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	-	-	-	2005
'Soils'	Texas[15]	0.55	-	0.43 - 0.60	-	-
'Soils'	Transvaal[14]	0.46	-	-	0.90	-

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 other minerals. Opaline silica, montmorillonite, attapulgite, sepiolite, kaolinite and allophane can be expected to lower the particle specific gravity and

minerals /...

minerals like carbonates, some illites and very heavy minerals like magnetite, ilmenite, rutile, zicon, garnet, etc. to raise it. The average density of the minus 40 mesh particles in calcretes is unknown, but in many cases will be considerably less than the figures quoted above for particle specific gravity.

Self-cementation

An important difference between all pedogenic materials and other roadbuilding materials is that they may actually be forming on excavation, 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 even expressed the opinion that at least some calcrete roads cement themselves to some extent. This phenomenon can in fact be observed on many unsurfaced calcrete roads, but evidence of its significance 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 or dolomite is slightly soluble in water, so that solution, migration and recrystallization of carbonate during suction, moisture content and temperature changes in the base is perfectly possible. It seems likely[18] that the diurnal temperature cycle in the base will act as a "salt pump" transporting 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 considerable increases in CBR are possible, during laboratory wetting and drying cycles at least.

It also shows that in some case a decrease in CBR is possible. The cause of this decrease is unknown. The sample concerned (2223) was the only powder calcrete tested (all the others being nodular calcretes), so that the weak aggregate characteristic of this type of calcrete may possibly have softened under this treatment. An increase in CBR from around 80 to around 140 merely on curing as for stabilized materials has also been reported to

TABLE 3 EFFECT OF WETTING AND DRYING CYCLES ON THE CBR

	FOUR DAY SOAKED MOD. AASHO CBR* (%)						
NO. OF CYCLES	0	2**	5**	10**	0	10+	20+
SAMPLE NO.							
2114	56	83	95		64	133	122
2116	55	78	64		45	-	-32
2223	47	41	20		48	24	29
2555	-	-	-		41	86	-
2689	21	24	41		-	-	-

* Minus three quarter inch 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.

occur in some calcretes from the Karasburg area (of which 2555 is an example) by Mr. J. Caiger (Personal comm.). A decrease in CBR on curing is also known (Table 9). 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 may also result in the formation of a relatively impermeable crust at the top of the layer - of possible value in the control of moisture and salt migration.

Another possible cause for self-cementation in some calcretes is 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) and fossil oxidized pedogenic materials will not therefore 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 self-cementation due

to /...

to this mechanism. Calcretes containing several percent ferrous and/or ferric iron are known. Nascimento et al's[20,21] petrifaction degree test or a CBR after one day drying cycle at 105°C is probably suitable for detecting self-cementation by this mechanism. The petrifaction degrees of the calcretes tested ranged from 0.40 to 0.74 (1.0 would indicate complete petrifaction or cementation), suggesting that this mechanism is possible in calcretes. In all cases, however, the absorption limits exceeded the liquid limits, while 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 defined as the shrinkage limit divided by the absorption limit, it is obvious that the high shrinkage limits of calcretes affect the the results, and in all probability nothing less that a petrifaction degree of 0.9 is significant. None of the calcretes tested were freshly taken; all had been in dry storage for months or years before testing. Any available ferrous iron would probably thus 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. Petrifaction degrees of about 1.0 were obtained.

A third possible mechanism whereby some improvement in quality can possibly be affected is the slow breakdown of clay minerals under the high temperature (up to 75°C) and pH (up to 10) conditions reached by many calcrete bases in the semiarid and arid environment. At high temperatures or a pF greater than 3.02, the pH of the liquid water in a calcrete base will be raised to about 10, due to the elimination of carbon dioxide from the solution. Given sufficient time, reaction between calcium carbonate and amorphous silica to form cementitious calcium silicate hydrates would also be possible.

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 /...

strength of portland cement - soil mixes and concrete. This means that the usual rule that clayey materials stabilize best with lime and sandy materials with cement does not hold with calcretes. Each calcrete must be treated on its own merits and some calcretes have been known to yield a greater and more rapid strength increase 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. (More details are available on these aspects if required).

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+ - psi
1601	Nodular calcrete	4	42,000	65
1569	Nodular calcrete	SP	21,000	52
1567	Calcified sand	NP	15,000	48
1624	Nodular calcrete	20	2,000	32
2114	Nodular calcrete	14	1,200	18
2116	Nodular calcrete	11	900	19

* Before stabilization

** In the minus 40 U.S. mesh (0.42 mm) fraction)

+ Seven day soaked unconfined compressive strength of 4" x 2" cylindrical specimens compacted from soil mortar (-10 mesh, 2 mm) to 115 lbs/cu ft dry density with an admixture of 4% high calcium slaked lime.

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 with strongly calcareous fines. A strength two thirds of that attained with an equal quantity of cement was yielded by the one calcrete tested.

Soluble salts/...

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 like NaCl, Na_2SO_4, $MgSO_4$ and Na_2CO_3 and relatively insoluble salts like 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 like blistering of the surfacing and loss of density and cohesion of the upper base. Complete loss of bond between the loose base and the surfacing results, with consequent cracking, scabbing and potholing. All the sulphates including gypsum can react with the reaction products of lime or cement stabilization or directly with the clay minerals present to form ettringite, leading 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 shows the range of the chief roadmaking properties of the most important calcrete types. Some actual test results upon which Table 5 was partly based, are shown in Tables 15-18 in the Appendix.*

Most calcified soils are either calcified sands or calcified gravels, and are readily told apart. The calcified gravels are generally of a higher quality than the sands and will nearly always be of low plasticity, good aggregate strength and 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 weak (10% F.A.C.T. value usually about 3 tons, ACV about 45%) and the grading of the excavated and placed material depends largely on this. Mechanical analysis of all the weaker

calcified /...

* Appendix 1 if Tables 8 and 9 form Appendix 2.

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. No 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 to various road authorities varies greatly, an indication of the differing opinions held on them and the extent to which the particular authority is forced to make use of them. The best calcified sands fall within the specification of the Texas Highwas Department[23] for caliche (calcrete) base, 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 be acceptable to the South African Department of Transport only as fill or selected subgrade. A few calcified sands fulfil the requirements of Fossberg[24] and the Orange Free State for gravel road wearing courses.

The main uses of calcified sands have been as gravel road materials. 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.

<u>Powder calcretes</u>

Powder calcretes are usually loose, but may be up to stiff in situ. On excavation and placing, however, the latter types normally degrade to a powder with a grading modulus of less than 1.0. Calcretes which have degraded to a powder on the road, which 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 for convenience 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 /...

mechanical analyses must therefore be regarded as unreliable. They are however shown in Figure 9. Samples 1515 and 2223 are regarded as the type specimens of natural powder calcretes and 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.

The aggregate strength of the coarse aggregate is usually poor. Only A.P.T. figures are available, the AFV ranging from 25 to 94% and the APV from 8 to 63%. There is some 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, such powder calcretes have progressed further towards becoming nodular calcretes and could also be older than their more powdery relatives.

The general rule that CBR decreases with PI does not always hold in the case of calcretes, many of them possessing apparent PIs fitting them only for lower fill, but CBRs befitting subbase. Most powder calcretes will in fact yield Department of Transport 7000 lb wheel load subbase CBRs (25-80), but would fail the subbase specification on grading and sometimes also soil constants. Little compaction data is available, but it is known that calcretes in general and powder calcretes in particular are prone to have exceptionally high optimum moisture contents (up to 20%) or more - probably largely due to diatoms and other porous particles. Modified AASHO densities probably range from 100 to 130 lbs/cu ft.

The type specimen natural powder calcretes 1515 and 2223 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 dozing or excavating. Ripping is unnecessary and scraper loaders can probably load without predozing. They sometimes, however, underlie hard intact hardpan which may require blasting,

and /...

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. Repeated applications of water for several days while being compacted by traffic or rolling is said to assist greatly in "blad" formation. It is probable that this is caused by self-cementation due to solution and recrystallization of carbonate, so that the above procedure is theoretically sound, provided the wetting periods are interspersed with drying cycles. Once the crust is broken, disintegration and 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 partly 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 of course no direct transition point from powder to nodular calcretes, but a separation at a grading modulus of 1.5 has been found to be both useful and practicable, being easily estimated in the field. A minimum grading modulus of 1.5 is also often specified for subbase for secondary roads.

The soil constants of nodular calcretes are particularly variable, and even higher PIs than those shown in Table 7 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, but the soil fines 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.

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 trather 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 the degraded particles become recemented with time 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 and the sand hunch is once again apparent. The kink between 200 mesh and 0.060 mm is probably due to a deficiency in the method rather than a characteristic of the calcretes.

The aggregate strength of nodular calcretes is very variable. 10% F.A.C.T. values range from 5 to 19 tons on the $\frac{1}{2}$" - $\frac{3}{4}$" fraction, but aggregate strengths increase with size, as shown by Table 6.

TABLE 6

VARIATION OF AGGREGATE STRENGTH WITH SIZE FRACTION TESTED

SAMPLE NO.	2114	2116	
NODULE SIZE*	10% FACT tons	10% FACT tons	LAA+ %
$\frac{3}{8}$" - $\frac{1}{2}$"	3.6	7.4	74.2
$\frac{1}{2}$" - $\frac{3}{4}$"	4.9	9.7	55.5
$\frac{3}{2}$" - 1"	7.7	13.0	56.5
1" - 1$\frac{1}{2}$"	7.5	13.9	54.8
1$\frac{1}{2}$" - 2"	-	15.0	52.4
2" - 2$\frac{1}{2}$"	-	13.9	-
+2$\frac{1}{2}$"	-	16.8	46.7

* All crushed to $\frac{3}{8}$" - $\frac{1}{2}$" before testing

\+ 2500 gm samples, using 11 spheres and 500 revolutions

The dry /...

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 under a surfacing the aggregate will be in a moist environment the latter order is probably the correct one. Unfortunately, little information is available on the sulphate soundness, LAA and soaked ACV and 10% F.A.C.T. values of nodular calcretes. Nodular calcretes with soaked 10% F.A.C.T. values of as little as 2 tons have made 4% lime and cement stabilized bases which have given no problems in the up to four years of service to date which can be attributed to aggregate degradation. It is doubtful if such low strengths would however 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 114 to 135 lbs/cu ft. CBR swell is usually low, even in the case of calcretes with high apparent PIs and low CBRs.

Most nodular calcretes are A-2 materials with group indices seldom rising 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 do 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. Most would be semi sand clays according to the South African Department of TRansport classification 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 aggregate may degrade during a stockpiling and compaction. 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 /...

deficiency of fines, but an improvement in grading may result.

Few nodular calcretes meet the natural base specifications of the Transvaal and South West Africa and even fewer (none of those tested) those of the South African Department of Transport, but about a third of those tested would probably be acceptable as caliche base to the Texas Highway Department[23] and perhaps a tenth to the Federal Aviation Agency[25]. About a third of those tested would be acceptable to the Transvaal and South West Africa for 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 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 Transvaal, South West Africa, Orange Free State, Department of Transport or Fossberg[24]. Most of the samples tested were either used as a gravel road wearing course or as lime or cement stabilized base and, apart from some salt problems in one case, all have yielded satisfactory performance to date.

Honeycomb calcretes

Honeycomb calcretes are relatively rare, and little is known of their properties. The actual 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 firm to hard, sheetlike, more or less massive calcrete layer usually underlain by softer or looser material. No tight restriction on hardness or strength is therefore intended, but

description /...

description should always of course include mention of the consistency, i.e. stiff, hard 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 as to the strength and other properties of hardpan and boulder calcretes can also be applied to the aggregate fraction of other calcretes about which less is known. This is thought to be true as far as the shape of the aggregate (i.e. more or less spherical nodules as against sharp, angular, crushed hardpan and boulders) would affect the test results concerned. Some nodules also possess a somewhat powdery outer surface which is normally only present at the bottom of some hardpans.

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 liquid limit of 25 and PI of 6, and it is therefore apparent that care must be taken when using these materials as crushed bases if this specification (which is considered too severe for calcretes) is not to be exceeded.

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

Tufaceous hardpans at about firm to stiff are the weakest form of hardpans and their 10% F.A.C.T. values range from about 2.5 to 5 tons. 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 7-10 tons and in their most advanced stages all hardpans may become slightly silicified, yielding Mohs' hardnesses of up to 6, dry 10% F.A.C.T. values of up to 22 tons (13 tons 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 1000 to 3000 psi are known from the Lichtenburg area and air dry values of up to 8000 psi (6000 psi moist) are

known /...

known from the United States of America[27]. Judging by their ACVs, the strongest calcretes should yield dry strengths up to 26,000 psi.

The strength of indurated calcretes depends on the density (Figure 11) up to a Mohs' hardness of about 4½, 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 0.01 mm never seem to become harder than about 3½ or to possess 10% F.A.C.T. values of more than about 10 tons. All the hardest and strongest calcretes are invariably crypto-crystalline (carbonate crystal diameter less than 0.01 mm). 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. While the effect of water varies, on the average the dry strength is reduced by one third on soaking.

Figure 14 shows the performance of some hardpan and boulder calcretes in the 20 cycle Department of Transport[28] sodium sulphate soundness test compared with one Nosib (?) quartzite and one silcrete. According to the percentage passing the ¾" 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 1.21 g/cc (75 lbs/cu ft) in the case of tufaceous types to a maximum of about 2.64 g/cc (165 bls/cu ft) in the case of the most indurated types. A fair degree of correlation exists between the density of calcretes and the crushing strength (not shown).

The water absorption of hardpan calcretes crushed to ⅜" - ½" and soaked for 24 hours varied between 2.8 and 24%. Only tufaceous types take up as much water as the latter figure, however, and most hardpans fall within the

range /...

range 3-6%. A fair correlation exists between water absorption and crushing strength, and it would seem that merely setting an upper limit of 10% on the water absorption of calcrete aggregate would eliminate the use of all aggregate possessing a dry 10% F.A.C.T. value of less than 11 tons.

Calcretes are not prone to yield excessively flaky aggregate on crushing and the values on laboratory crushed $\frac{1}{4}$" - $\frac{5}{16}$" aggregate determined according to SABS 647:1961[29] were found to vary from 2.0 to 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 (1965)[30] values on laboratory crushed $\frac{1}{4}$" - $\frac{5}{16}$" 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 popular stones like dolomite. Areas in which calcretes occur are not heavily populated at present, so that polishing is not likely to be a problem, and any calcrete which meets the other requirements of a surfacing stone could be used.

Calcretes vary widely in their adhesion to bitumen, according to the detachment test[31], 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 best ones in premix and penetration macadam. 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 /...

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. The pocket hardness tester, water absorption and Treton testing would be useful here. Four calcretes (two hardpans, 1724 and 1562, and two boulder calcretes, 1028X and 1627) have been tested by the National Building Research Institute[32] for their suitability as concrete coarse aggregate. All four were found reasonably satisfactory, yielding 7 day prism compressive strengths of 3900 - 5000 psi, except for 1724 which only yielded 2000 psi, with satisfactory durability and shrinkage. One was found to be possibly potentially alkali-reactive if a high alkali cement were to be used. 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 7.1 tons) not used for 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 so that, while rippable, the large soil-filled solution hollows between the hard boulders make ripping very hard on the machines and once 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.

Owing to the fact that non-outcropping boulder calcretes are essentially chemically weathered hardpans with the weakest parts dissolved out, they usually yield excellent aggregate crushing values, values of more than 30% being seldom obtained. Similarly, soil constants and conductivities of the fines produced in the Los Angeles machine were found to be very low. Hardnesses and bulk densities are always high, the latter varying from 2.2 to 2.64 g/cc (137-165 lbs/cu ft) and the former normally never falling below $3\frac{1}{2}$-4. Water absorption

varied /...

varied from 3 to 8% and the polished stone values (1965) of the three samples tested ranged from 0.49 to 0.58. 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. This is perhaps one of the few cases where disintegration produces a product technically inferior to chemical weathering, though in practice the disintegrated boulder calcrete may prove to be more usable than the large hard rounded boulders produced by solution (Plate 7).

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 0.420 mm fraction) for soil constants and sedimentation analysis. Methods differ in three main features : whether the fines are obtained simply by dry sieving or by a process involving slaking in water to release fines adhering to the aggregate, whether any prohibition is placed on heating, and 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 roads authorities adhere to this procedure however. At the National Institute for Road Research[33] simple dry screening is used, heating above room temperature is prohibited and presoaking overnight at about 1½

times /...

times the liquid limit is employed.

It would appear from the methods used in Texas[34] in general (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 calcretes certainly do possess clay-coated aggregate this factor may well be significant. Some work has however been carried out on the effect of dry and moist heat on the soil constants of South African calcretes. Six calcretes (five nodular and one powder, samples 2114, 2116, 2223, 2367, 2493 and) possessing all the clay mineral combinations known to occur in calcretes, apparent liquid limits between 24 and 36 and apparent plasticity indices between 10 and 15 were chosen for the study. All samples had been previously 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 1½ times the liquid limit and then drying out at 50°C, (d) heating for 24 hours at 105°C, and (e) soaking for 24 hours at about 1½ tines the liquid limit and then heating for 24 hours at 105°C. The soil fines were in each case then soaked for 24 hours at about 1½ times the liquid limit before actual testing began. It was found that heating the air dried or previously soaked soil fines to 105°C resulted in every case in a lowering of the liquid limits by up to 2 percentage units, the plasticity indices by 1 to 4, and, generally, in a raising of the plastic and shrinkage limits by up to 3 and 1 respectively. Linear shrinkage was not affected in three cases, in one case (sample 2114) went up by 1.8 percentage units and in another down by 0.7. The behaviour of the shrinkage ratios, flow indices, toughness indices, absorption limits and petrifaction degrees were erratic, in some cases increasing, in some decreasing and in other evincing no change. The behaviour of all samples at 50°C was much the same as at 105°C, except that the results were

somewhat /...

somewhat more erratic and the effects less pronounced, though the PI of the powder calcrete (2223) was decreased by 5 (almost entirely through an increase in the plastic limit) as against 4 after heating at 105°C. It is apparent therefore, that if all other factors were equal, the Department of Transport method would yield for example 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

Liquid limit methods actually vary quite widely in detail and it is not surprising that they do not always yield comparable results. The methods in use can firstly be differentiated on the basis of whether they employ a prior period of wet equilibration. Methods routinely employing presoaking are the Department of Transport since October 1963 (12 hours near LL), though how widely this is followed is uncertain and National Institute for Road Research methods (overnight at about 1.5 LL). Others, such as the AASHO, ASTM, Texas and BS methods normally merely employ the dry soil fines. The effect of presoaking on calcretes is unknown. If it has 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 road authorities. The liquid limit device itself also varies, the ASTM devices usually possessing harder rubber bases, soft rubber feet, and differing in other apparently minor detail. It would seem from one published paper[35] and some unpublished experiments (Mr. J. Caiger, personal comm.), that the BS device yields liquid limits higher by 1-5 units than the ASTM devices used. An experiment with seven calcretes (samples 1569, 1602, 2114, 2116, 2223, 2367 and 2555) with liquid limits between 21 and 34 by the BS device showed that the BS device used (base hardness) yielded consistently higher liquid limits by 1-2 units than the ASTM device (base hardness).

The /...

The flow indices determined by the ASTM device were generally higher than those yielded by the BS device. Grooving tools in use also vary, most methods specifying the ASTM type, while the BS method employs the Casagrande type. The effect of grooving tool is probably small, though the ASTM type has been criticised.[36]

Besides the liquid limit device itself and the unknown effect of presoaking on calcretes, another important factor affecting the liquid limit of calcretes appears to be the direction of approach, i.e. whether the 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 Department of Transport 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 proceeded only from wet to dry, the liquid limit could be estimated to within ± 0.5 units in about 93% of cases. The maximum probably interpretation error varied from ± 0.1 to $^{7.1}_{1.5}$. If the best straight line was merely drawn through all the points obtained, a liquid limit was obtained which was within ± 0.5 of the most probable value in 80% of cases. The difference between the liquid limit obtained by this latter procedure and the most probable value was found to vary from ± 0.0 to $^{+1.3}_{-2.5}$. In about 17%

of /...

of cases the flow curves were not linear on the semilog plot but were curved, concave upwards. It was also found that where the material was rewetted and remixed to obtain another point to fill a gap on the flow curve, that point seldom fell on the line drawn through the other points (3 out of 22 cases) but nearly always fell above the line (17 cases out of 22, or 77%). Only in 2 cases out of 22 did it fall below the line. This may be due to insufficient mixing after rewetting or to cementation by crystallization of dissolved carbonate during the drying inherent in the National Institute for Road Research method (cementation might also increase the porosity), the remixing being insufficient to completely break up the cementation bonding. Cementation 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, so that a method such as that of the Department of Transport will generally (77% of the time) yield a spuriously high liquid limit if it is not obtained at the first approximation and rewetting after drying has to be employed. The magnitude of the possible error is unfortunately 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 (AASHO,ASTM, Texas and BS alternatives) must be of dubious accuracy and reliability. The one point method was apparently developed by the Washington State Highway Department. Calcretes have not been used in that State in any quantity, if at all.

Plastic limit

Most plastic limit methods 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 /...

the plastic limits of calcretes. This difficulty was usually 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 facts 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 other materials.

The factors affecting liquid limit and PI determinations on calcretes (and possibly on all materials) can probably be summarised as follows : soil fines preparation method (drying temperature and whether wet or dry screened), presoaking and its duration, mixing time and effort, liquid limit device, grooving tool, direction of moisture change in liquid limit procedure and probably the number of wetting and drying cycles, addition of dry soil to previously seasoned soil, 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), silt content, and accuracy of moisture content determinations (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 other factors beyond the soil fines preparation method, possibly presoaking and the possibility of ion exchange 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.

In view of the difficulties with 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. The fact that a good correlation exists between the PI and linear shrinkage means taht both tests essentially measure the same property. It also 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 on calcretes. Its advantages can be vriefly summarised as follows[15,37]: The test is extremely rapid and of course 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, it costs less than 20% of 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 and 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 will however be far less than on the PI.

Mechanical analysis

Mechanical (sieve plus sedimentation analysis) analysis is of course only applicable to powder calcretes, nodular calcretes and some calcified soils in their natural state. The following remarks can however be taken to apply to all calcretes as actually used on the road, with the only 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 /...

clay to yield reliable analyses.

Calcretes may give rise to problems during mechanical analysis in three ways : by weak aggregate breakdown during sieving or dispersion for sedimentation analysis, by clay-coated aggregate, and by flocculation during sedimentation analysis. The first difficulty may be partly overcome by 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 abrasion of coarse particles than do mechanical stirring devices.[38] 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 calcareous clayey sands showed that wet sieving produced a different grading curve from dry sieving, together with up to 15% more passing 0.420 mm (40 mesh) and 20% more passing 0.074 (200 mesh).. 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 percent, but removal of carbonate from a calcrete with acid should not be resorted to since the carbonate may be the major component and 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 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 calcrete particle specific gravity and shape on readings obtained by the hydrometer method can theoretically be regarded as negligible for all practical purposes. Calcretes with very porous

particles /...

TABLE 5 SOME PROPERTIES OF SOME IMPORTANT CALCRETE TYPES

CALCRETE TYPE	$CaCO_3$ + $MgCO_3$ CONTENT %	GRADING MODULUS	HRB CLASS.	GROUP INDEX	CASA-GRANDE CLASS.	M.AASHO CBR %	APPARENT PI %	L S %	−0.42 mm CONDUCTIVITY mmhos/cm	A C V %	10% FACT Tons	A.P.T. A F V %	A.P.T. A P V %	MOHS' HARD-NESS	CONSIS-TENCY	SEIS-MIC VELO-CITY ft/sec.	WORKABILITY
CALCI-FIED SAND	10-50	~~Varia-ble~~ 1.5 – 1.8 ?	A-1-b to A-2-7	0-2	GP-GF to SF-GF	25-50 ?	NP-21	1-9	0.25 – 2.3	35-50 ?	3-9 ?	75-95 ?	20-40 ?	2-3	Med. dense-dense or firm-stiff	2000 - 6000 ?	Variable, shovel-rip
POWDER CALCRETE	50-95	0.5 - 1.5	A-2-4 to A-7-5	0-13	ML-GF	25-70	SP-22	1-11	0.14 – 2.1	35-50 ?	3-9 ?	25-95 ?	5-65 ?	2-3	Loose-stiff	1300 - 3500	Doze or shovel
NODULAR CALCRETE	50-75	1.5 - 2.5	A-1-a to A-6	0-3	SF-GF to GP	50-120+	NP-25	1-10	0.17 – 7.4	21-40 ?	5-19 ?	0-100	0-87	2-5	Loose (nod-ules soft to very hard)	2000 - 3000	Doze or shovel
HONEY-COMB CALCRETE	70-85	-	Rock?	-	Rock?	-	SP-7+ ?	1-3?		16-35 ?	9-23 ?	-	-	3-6	Stiff - very hard	3000 - 4000	Rip (usually)
HARDPAN CALCRETE	70-95	-	Rock	-	Rock	-	NP-7	1-3	0.09 – 0.16	19-53	3-22 ?	-	-	2-6	Stiff - very hard	3000 - 15000	Rip or blast
BOULDER CALCRETE	80-95	-	Boul-ders	-	Boul-ders	-	NP-3	1-2	0.06 – 0.16	20-33	11-21	-	-	3.5-6	Boul-ders very hard	Erra-tic	Rip

1) Consistency convention as per Jennings and Brink (1961) with addition of 'hard' (Mohs' hardness 2-3.5) and 'very hard' (3.5-6) as suggested in NIRR RS/5/65.

2) PIs ~~and~~ LSs and conductivities of honeycomb, hardpan and boulder calcretes on the fines produced in the Los Angeles Abrasion test. ~~Soil constants of~~ Tests on adhering soil may ~~be higher~~ yield higher results.

TABLE 8 SOME DETAILS OF FIVE ROADS EMPLOYING CALCRETE AS BASE AND SUBBASE

ROAD	YEAR COMPLETED	SURFACING	TOTAL THICKNESS OF IMPORTED (CALCRETE) LAYERS	NATURAL SUBGRADE	12 HOUR TRAFFIC COUNT, vpd		RAINFALL mm	N VALUE	THORNTHWAITE'S MOISTURE INDEX	REGIONAL DESIGN FACTOR
					CARS	HEAVIES				
PORT ELIZABETH MARINE DRIVE (MAIN ROAD 4)	Before 1936	$\frac{3}{4}$" surface treatment	?	?	$220^{\times}$ $800^{\times}$	$5^{\times}$ $18^{\times}$	~540 ~770?	~2.6 ?	~-18 ?	0.50 0.50?
PORT ELIZABETH - ADDO (MAIN ROAD 1)	Before 1936 to 1940	$\frac{3}{4}$" surface treatment to 2" penetration	?	?	$290^{\times}$ $350^{\times}$	$70^{\times}$ $40^{\times}$	~500 ~400	~3 ~5	? ~-30	0.50 0.50
LANSDOWNE ROAD, CAPE FLATS	1926, surfaced 1952	2 coat surface treatment	11" - [illegible]"	Sand or grey soil	$3000^{\times\times}$	$760^{\times\times}$	~500	~3.7	~-7	0.50
DUINEFONTEIN ROAD (NO. 122), CAPE FLATS	1950	?	8" - [illegible]"	Sand	$960^{\ddagger}$ $1100^{\ddagger}$	$520^{\ddagger}$ $630^{\ddagger}$	~500 ~500	~3.7 ~3.7	~-7	0.50
WELTEVREDE ROAD (NO. 156), CAPE FLATS	1960	?	12" - 6"	Sand	?	?	~500	~3.7	~-7	0.50

$^{\times}$ Average of 6 day period (Monday to Saturday) in July 19[illegible], from 6 am to 6 pm near each end of road. Vehicles, excluding cars, with payloads in excess of $\frac{1}{2}$ ton counted as heavies.

$^{\times\times}$ On 12.1.67 from 7 am to 7 pm. All vehicles over 3 tons counted as heavies.

‡ At the same point on 15.1.67 and 24.1.66 respectively, from 7 am to 7 pm. All vehicles over 3 tons counted as heavies.

particles will however possess a lower effective specific gravity and this would give rise to erroneously low results.

It is considered that sedimentation analysis is unnecessary for routine work and that wet sieving to 200 or 270 mesh is quite adequate.

Aggregate strength and durability tests

The aggregate tests in common use are the aggregate crushing tests (ACT)[28,39], the 10 percent fines aggregate crushing test (10% F.A.C.T.)[29,40], the Treton test[41], the sulphate soundness tests[28,29,42] and the Los Angeles wear[42] or abrasion(LAA). Of these, only the first four are normally used in South Africa, while the Los Angeles test and the sulphate tests are those most commonly used in the United States and Australia. These tests are applicable to all calcretes from which aggregate of suitable size can be obtained by sieving or by crushing and the ACT (ACV), 10% F.A.C.T. and LAA are considerably more reproducible than the older tests employing cylindrical specimens.[8]

Both the aggregate crushing test and the 10% F.A.C.T. appear to yield meaningful results on calcretes, but no correlation with the service of these materials is known. Figure 15 shows that the relation between the results of the two tests on crushed calcretes is essentially that of Shergold and Hosking's[40] average curve, calcretes apparently falling near the curve for porous materials. Very few results are available for other than crushed hardpans and boulders.

No information is available on the application of the Treton test to calcretes, though it is used on calcretes by some laboratories. In view of its simplicity it is surprising that this test is not more common. Walker and Stewart[41] 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 /...

the advantage that little apparatus is required. They have however been criticised by several authors[1,2,44,45] 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 on calcretes that the test does not simply detect calcrete with a poor wet strength, but rather that which loses a relatively large proportion of its 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 seems to apply to all types of aggregate. A survey in the United States by Erickson[46] 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 apparent that critical re-appraisal of the ACT, the 10% F.A.C.T. and the Treton is therefore also called for. Tests reported as helpful include a modified Los Angeles using no steel charge[47,48], 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[49], 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.

A new aggregate strength test of great simplicity has recently been described[1,13] and can be carried out by any roadworker using his own fingers, a pair of standard pliers and simple arithmetic. This test was designed

originally /...

originally for nodular calcrete gravel road wearing courses, but with modification (for example the use of larger pliers) may prove suitable for field control of all wearing course and base course gravels, as well as in laboratory testing when too little material is available for the standard crushing tests.

Attention is drawn to the good correlation found between Mohs' hardness and the aggregate crushing strength (ACV and 10% F.A.C.T.) of calcretes shown in Figure 12. 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 has been developed to make this test a most convenient one. Even the mere use of the average brass key (hardness about $3\frac{1}{2}$) would enable one to distinguish calcretes that 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. 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 exceeds about 50%.

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 calcified soils vary, normally increasing with size and development.

<u>Self-cementation</u>

This has already been dealt with in a previous section. The petrifaction degree or a strength test such as a soaked CBR 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

calcretes which cement themselves due to wetting and drying cycles. In all cases the experiments should be carried out in duplicate or triplicate and, in the case of those involving 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 still remains unproven. Methods probably capable of detecting self-cementation in the road itself include 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 been the subject of a recent paper[50] 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. These could however be detected and measured conductometrically by a dilution technique. 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 unconfined compressive strength testing before and after wetting and drying cycles. In both tests, however, some leaching of very soluble salts will take place, so that the results may be optimistic in such cases. Reliable results should be obtained only when gypsum or anhydrite is present.

Miscellaneous /...

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 in calcretes. A method which determines OH^- in the presence of CO_3^{--} such as the California 338-D (Parts II and IV) would, however, be suitable.

Apart from their inherent variability, calcretes are not known to present any problems in the obtaining of moisture-density and CBR data. Drying should however 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 can take up moisture if reproducible results are to be obtained.

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 and it has already been shown here that 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 $\frac{5}{8}$ - $\frac{3}{8}$" aggregate and a maximum loss of 20% in the Australian Standard

No. /...

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 successfully used (Mr. A.J. Scala, personal comm.).

Nodular calcrete yielding dry 10% F.A.C.T. values of 2.4-6.3 tons not surprisingly yielded "not very satisfactory" results in Nigeria as surface dressing aggregate.[51]

In view of their 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 it 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 general recent survey of soft limestones is that of Beaven[52]. Gillette discussed a number of caliche-based roads in the United States of varying performance. Some of his conclusions were that caliches with PIs less than about 10, flocculation factors up to 1.7 and without gel-producing colloids should prove satisfactory as bases under thin surface treatments under average moisture conditions, that caliches with PIs of 10 to 15, flocculation factors up to 2.5 and without colloidal gel should prove satisfactory under semiarid 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 40 mesh fraction and that the minimum thickness of base course should be eight inches, preferably compacted in four inch layers.

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

Table 7. /...

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[53]	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	$1\frac{3}{4}$	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.

It can be seen that only a liquid limit, PI and three grading requirements are normally listed. Only in one case is aggregate strength specified. The PIs are comparable to those suggested by Gillette and it is apparent that these authorities also consider the quality and quantity passing the 40 mesh sieve to be important. It should be noted that the above specifications are mucy less severe than normally required by other authorities and by the FAA and Texas for other materials and also that no strength requirement (e.g. CBR or triaxial classification) is called for, although this might still be specified in the contract documents themselves. Personal communication has also revealed that most of the FAA airports utilising the above specifications are of the utility category serving aircraft of less than 12,500 lbs gross weight, and the FAA would 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 used unstabilized 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 /...

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.

With regard to the plasticity limits in Table 5 it should also be noted that calcretes only occur in relatively dry climates anyway (in Australia[54] the PI is increased from the usual figure of 6 to 9 in areas receiving less than 15 inches of rainfall and in South Africa the regional design factor would be 0.5 or less in all calcrete areas) and that both authorities (the CAA is now the FAA) specify slaking methods for the preparation of the soil fines and that heating above 60°C is forbidden. The extent to which the soil constants of calcretes prepared by the American methods differ from those prepared by the South African method is yet unknown. The Australian authorities[54] also recognised that higher soil constants are allowable for some "limestone rubbles" (nodular calcretes?).

Evidence from South Africa

Tables 8 and 9 show details of five roads in South Africa employing calcrete as base and subbase kindly made available by the Cape Provincial Roads Department and the Cape and Port Elizabeth Divisional Engineers. On the basis of their present specifications the Cape Provincial Roads Department had quite rightly rejected all similar materials for future 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 unsuitable) for base and the soil constants too high even for subbase. In the case of the three Cape Flats roads, some relaxation of grading is now allowed for the calcretes there, though not of soil constants and CBR requirements. All five 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 (and probably also those of all other Provinces). Of the 30 base course samples, DE.7415-7417 in the case of the 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. DE.7419, DE.7384 and DE.7389 would fail the usual CBR requirement of 80. All samples listed would even fail the Department of Transport and all except DE.7385 the Transvaal grading requirements for stabilized base, though DE.7416, DE.7418, D.9559, V.2330 and V.2331 would be acceptable on a grading basis to the Transvaal if screened minus 1½ inch. 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 30, 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 to these materials. On a grading and soil constant basis only one base course sample out of 30 (3%), i.e. sample DE.7418, would pass the FAA specification and only 3 out of 30 (10%), i.e. DE.7382, V.2329 and V.2333, would pass the Texas specification. Many, however, fail only on the maximum size criteria, but if these are waived the figures are still only raised to 2 (7%) and 15 (50%) passing the FAA and Texas specifications respectively.

Table 10 /...

Table 10 summarises the results on the four roads for which some idea of the traffic count is available. The worst results are shown at the foot of the table.

In the case of the <500 vpd traffic category, it can be seen that the worst PI 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 is probably applied to roads in the medium (500-1000 vpd) category in Table 10. Here their limit of 50% passing the 40 mesh sieve agrees well with a grading modulus of 1.5 (in the case of calcretes a grading modulus of 1.6 is very closely equivalent to 50% passing the 40 mesh sieve - Figure 16) and their maximum PI of 12 is not exceeded by the worst in Table 10 (9).

Correlation is less good in the case of the heaviest traffic category, which is presumably roughly equivalent to the Texas medium volume primary highways. Their grading requirement of a maximum of 35% passing the 40 mesh sieve (equivalent to a grading modulus of 2.0) is more severe than the usual grading modulus of 1.5 in Table 10 (one sample even possessed a grading modulus of 1.2 and 74% passing 40 mesh). A grading modulus of more than about 2.0 is usually regarded as satisfactory for natural base[4]. The worst liquid limits and PIs in Table 10 of 20 and 8 are much less than the Texan limits of 35 and 12.

It is apparent from the foregoing that soil constants and grading requirements, and possibly even CBR in the lower traffic categories can be relaxed somewhat for calcrete bases and subbases and the following interim specifications are suggested in Table 11.

TABLE 10 /...

TABLE 10 SUMMARY OF TEST RESULTS ON 26 SAMPLES OF CALCRETE BASES

ROAD	TRAFFIC CATEGORY (vehicles per 12 hour day) 500 (<20% >½ ton payload)	No. of samples	500-1000 (2% >½ ton payload)	No. of samples	1000-4000 (<35% >3 tons)	No. of samples
PORT ELIZABETH MARINE DRIVE			%-40# : 28 - 56	8		
			GM : 1.5 -2.2	8		
			LL : 20 - 24	8		
			PI : 4 - 9	8		
			LS : 2.5-4.0	8		
			CBR : 62-90	2		
			Swell : 0.0-0.1	2		
			GI : 0.0	8		
PORT ELIZABETH - ADDO	% -40# : 39-56	8				
	GM : 1.4-1.8	8				
	LL : 28-33	8				
	PI : 11-15	8				
	LS : 5.5-6.5	8				
	CBR : 47-66	2				
	Swell : 0.1-0.2	2				
	GI : 0.2-0.5	8				
LANDS-DOWNE ROAD					% -40#: 43-56	5
					GM : 1.5-1.9	5
					LL : 17-20	5
					PI : 6-8	5
					LS : 2.5-3.0	5
					CBR : 112-134	3
					Swell : 0.0	3
					GI : 0.0	5
DUINE-FONTEIN ROAD					% -40#: 48-74	5
					GM : 1.2-1.7	5
					LL : 18-20	5
					PI : 4-7	5
					LS : 1.5-3.0	5
					CBR : 87-107	2
					Swell : 0.0	2
					GI : 0.0	5
Max.% -40#	56		56		74	
Min. GM	1.4		1.5		1.2	
Max. LL	33		24		20	
Max. PI	15		9		8	
Max. LS	6.5		4.0		3.0	
Min. CBR	47		62		87	
Max. Swell	0.2		0.1		0.0	
Max. GI	0.5		0.0		0.0	

Table 11 /...

TABLE 11 SUGGESTED INTERIM SPECIFICATIONS FOR CALCRETE NATURAL OR CRUSHED NATURAL GRAVEL BASES

TRAFFIC CATEGORY (vpd)*, with <35% over 3 tons	<500	500-1000	1000-4000
Max. particle size	2"	2"	2"
Max. apparent % passing 0.42 mm (40 mesh)	56	56	56
Min. apparent grading modulus	1.4	1.5	1.5
Max. apparent liquid limit (%)	45	40	35
Max. apparent PI (%)	15	12	10
Max. bar linear shrinkage (%)	6.5	4.0	3.0
Max. -40 mesh conductivity, Cl^- (m mhos/cm)	1.8	1.8	1.8
Max. -40 mesh conductivity, SO_4^{--} (m mhos/cm)	0.7?	0.7?	0.7?
Min. dry 10 F.A.C.T. value** (tons)	-	10?	12?
Min. soaked/dry 10% F.A.C.T. value+ (%)	-	60?	60?
Min CBR (Modified AASHO) (%)	80	80	80
Max. CBR Swell (Mod. AASHO & OMC) (%)	0.5	0.5	0.5?
Min. relative compaction (%)	As required		
Max. 24 hour water absorption+ (%)	15?	15?	10
Max. group index	0.5	0.0	0.0

* 12 hour (daylight) day

** Or equivalent ACV (Figure 15)

\+ On $\frac{1}{2}$"-$\frac{3}{4}$" aggregate

The grading and soil constant specifications are a blend of Tables 7 and 10. Apart from the group index, grading and soil constants have been considered separately, but they should really be considered together. A limit on the maximum value of the linear shrinkage divided by the grading modulus or the linear shrinkage multiplied by the percentage passing the 40 mesh sieve might be a good way of doing this. Conductivity limits for calcrete bases have been tiven by Netterberg[50] and assume non-saline compaction water. If saline compaction water is used, the limits should either be adjusted according to the calculated amount of salt to be added by the compaction water or applied to material taken from the full depth of base course after compaction. The limit for SO_4^{--} of 0.7 m mhos/cm may possibly be too high. The dry 10% F.A.C.T. value of 12 tons for the 1000-4000 vpd traffic category is the usual Department of Transport requirement and may be too severe. No results are

available /...

available for calcrete bases. The 10 ton value is likewise werely a guess. The percentage ratio of soaked to dry strength is about 6% less than the average for crushed calcrete hardpans. Very few results are available for nodular calcretes and calcified soils. Water absorption after 24 hours or the APT is suggested as a rapid field control test as a substitute for an aggregate crushing test. The figures of 10 and 15% are roughly equivalent to dry 10% F.A.C.T. values on crushed calcrete hardpans of 12 and 10 tons respectively. APT values are not yet available for bases. CBR and swell are normal South African requirements. 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 it is considered that if in practice a c lcrete road layer meets the CBR requirements and difficulty is obtained in meeting the density requirement, the latter could be relaxed by one or two percent. The percent passing 40 mesh (or the grading modulus), linear shrinkage, conductivity, CBR, swell and relative compaction are regarded as essential criteria. All others are optional.

As far as the crusher run calcrete base is concerned, in the absence of any information to the contrary it would be wise to specify the usual grading required by the authority concerned. Provided no non-calcareous binder is used, the soil constant limits for the heavy traffic category in Table 11 should be adequate. The conductivity limits should also be applied, though hardpans and boulders are not normally saline. The aggregate strength requirements for the heavy traffic category in Table 11 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. <u>It is likely that the suggested specifications (Table 11) only apply to calcretes with strongly calcareous soil fines.</u>

Line /...

Lime and cement stabilized base

In view of the findings for natural basis it is also reasonable to relax some of the requirements for lime and cement stabilized calcrete. The following seems reasonable, but no information is actually available on the long term performance of calcrete bases to such a specification. (Table 12).

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. As with other materials, this form of stabilization is probably only suitable for calcretes of relatively low plasticity, fines and porosity.

Subbase and lower layers

In practice the Texas caliche base specification in Table 7 is applied to subbase for "high type" highways and probably to both base and subbase for their farm to market and secondary roads. 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 light traffic category base specification in Table 11 be used as subbase for secondary and minor roads and that the medium traffic category calcrete specification should be adequate as subbase to the heaviest flexible base design. CBR, swell (if relaxed) and group index requirements can of course 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 limies in Table 11 to subbase as well. If the completed base will be completely non-saline (-40 mesh conductivity less than about 0.1 m mhos/cm), the limits could probably be relaxed somewhat (perhaps doubled).

No information is available on specifications for calcrete selected subgrade, but it seems reasonable to relax the apparent grading modulus requirement to 0.8-1.0, (or 72-80% passing 40 mesh), apparent PI to about 17,

Table 12 /...

TABLE 12. SUGGESTED INTERIM SPECIFICATIONS FOR LIME AND CEMENT STABILIZED CALCRETE BASES

PROPERTY	REQUIREMENT
Max. particle size Max. apparent % passing 0.42 mm Min. apparent grading modulus	As required > 56 < 1.5
Max. soil constants	Not to exceed the appropriate natural base requirement (Table 11) after stabilization. Materials with apparent PIs much above 25 before stabilization may be difficult to mix. Liquid limit requirements can probably be waived for pavements not subject to frost heave
Max. soluble salts	Conductivity limits as in Table 11 before stabilization. Gypsum and anhydrite limited to 0.25% SO_3 if the apparent PI or the two micron clay 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 11.
Strength	As required, bearing in mind that the in service strength is likely to be about 40% of the laboratory strength. For example, a minimum laboratory CBR after 7 days curing and 4 days soaking of about 200 is thus called for when strength and not reduction of plasticity is the primary requirement.
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 granual and silty soils and 7% for clay soils[55] 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.

PROPERTY	REQUIREMENT
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 not yielding high early strengths.

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 until more is known of salt migration in roads. In all cases the usual CBR, swell and compaction requirements should be maintained.

Gravel road wearing course

A good calcrete gravel road will possess a hard surface crust or "blad", (perhaps formed in a similar manner to hardpan in nature) which enables the road to last for some time without grading. The crust does not corrugate or raise excessive dust, but 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 (one inch) layer of Kalahari-type sand. A crust can normally be obtained by compaction with water.

It is important to distinguish between a gravel and a 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 limits for the simple pliers test are shown below (Table 13).

The limits are based on the results of tests on 21 samples from calcrete gravel road wearing courses of known performance varying from excellent to very poor. Grading and plasticity were entirely disregarded,

in /...

TABLE 13. SUGGESTED LIMITS FOR PLIERS TEST

	AFV total % passing fingers test)	% passing fingers test but failing pliers	APV (% passing pliers)	Test as a whole (AFV TO APV)
Suggested lower limit	60	30	20	All of foregoing
% samples obeying above limit	90	60	75	100
Operator variability	±10	±5	±5	-
Approx. 10% F.A.C.T. value (tons)	>5	5	10	-

in other words, all samples which obeyed the limits in Table 13 performed satisfactorily, regardless of grading and soil constants. Although loose surface gravel which had been on the road for at least twelve months has always so far been found to possess an APV of at least 77%, it would appear that the most important criterion for unused gravel is rather the AFV (assuming a sufficient quantity of aggregate is present). Merely specifying that at least 60% of the ½"-¾" aggregate shall be breakable by hand (AFV>60%) would apparently ensure a satisfactory performance in 90% of cases. It may be advisable to add the 10% operator variability to this and actually specify 70%.

In the case of sand-clays, the aggregate floats in a matrix of fines and its strength becomes unimportant. For the best performance of such a material, the percentage passing 40 mesh times the linear shrinkage should lie between 100 and 200. It should never exceed 350. Linear shrinkage should lie between 3 and 8. These same figures can also usefully be applied to gravels (MODIFY AFTER ANALYSIS OF W.TVL. SAMPLES).

A number of authorities relax their plasticity and grading requirements for calcrete gravel road wearing courses. The Orange Free State Roads Department[56] state that the percentage passing the 40 U.S. mesh sieve

(0.42 mm /...

(0.42 mm) should normally be 32-37%, but that in the case of calcretes this can be 40% or more. In dry areas the PI should normally be 14 when the percent passing 0.42 mm is 45% and 18 when it is only 20%, but when calcretes are used, PIs of 16 to 20 can be used. In Rhodesia[57], a PI of up to 20 is tolerated in calcrete wearing courses, while in the Transvaal[58] unspecified latitude is allowed on both plasticity and grading and in Australia[54] the PI relaxed somewhat from 2-9 to 12.

About 30 samples of calcrete wearing courses of known performance were obtained and tested. All the samples came from sites in the Republic and South West Africa receiving an annual rainfall of less than about 550 mm (22 inches). The apparent PIs of satisfactory wearing courses were found to be between 10 and 20, with the very best ones tending towards the upper limit - though they apparently tended to become somewhat slippery in wet weather. Calcretes which corrugated were prone to have PIs below about 10, while those which tended to form powder holes either also had PIs below 10 or else above 20. Calcretes which formed powder holes also usually possessed an AFV below 60%. A suggested grading envelope is shown in Figure 17 which is essentially an extension of that suggested by Fossberg[24] for relatively dry climates. The very best calcrete wearing courses tended towards the fine side (upper limit) of this envelope. There is some evidence that still finer materials may be satisfactory in some cases. Both grading and soil constants should really be considered together, and a similar procedure to that suggested for base course might prove satisfactory. The grading of wearing courses is probably more important than the grading of calcrete bases. Wet sieving to 200 or 270 mesh should be adequate.

Dust palliatives

When a hard crust or "blad" has not been obtained, calcrete gravel roads can be among the most dusty of roads. While a number of dust palliatives are available[59] and a locally available waste product, sulphite lye, has been

found /...

found to be particularly effective on calcrete roads[60], it would appear 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 make by use of the suggested relaxed calcrete specifications it will be economic to surface roads carrying less traffic than normal.

CONCLUSIONS AND SUMMARY

1. Calcretes are one of the most widely used classes of road materials in Southern Africa. They are not 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. The classification represents particular stages of calcrete development and each stage differs significantly in its engineering properties from any other stage.

4. Calcretes of roadmaking quality only occur in the drier areas of Southern Africa, in general where the annual rainfall is less than 550 mm (22 in.)

5. Calcretes in South Africa can be divided into two basic types, those formed by deposition of carbonate in the host material above a shallow water table (non-pedogenic, either permanent or perched), and those formed by downward leaching of carbonate from the upper soil horizons and its deposition in the lower (pedogenic). Both these types proceed through definite developmental stages, used as a basis for the calcrete classification referred to above.

6. The mechanism of calcification is not thought to be direct evaporation as is commonly supposed, or even transpiration, but a decrease in solubility brought about by carbon dioxide loss due to a decrease in the pressure of the pore water (increase in suction or pF) caused indirectly mainly by evaporation and transpiration. Most calcification probably takes place as the pF changes from 2.8 to 3.02.

7. Calcretes are geologically very young. The oldest calcrete important in roadbuilding is probably late Pliocene (about 2-5 million years old) while the youngest is probably Second Intermediate (9-15,000 years B.C.) in age.

8. The best solution to the problem of prospecting for calcretes lies at present in the combined use of airphoto interpretation, topographic setting, plant indicators and a certain rapid probing device. One hundred percent success in detecting all calcrete deposits within a given area in a reasonable time is, however, probably not to be had 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, acting always as a non-plastic mechanical stabilizer, often as a cementing agent, and ensuring that the clay minerals are always calcium-saturated.

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 calcrete fines.

11. /...

11. The solid particle specific gravity (-0.42 mm fraction) at 2.66, with a standard deviation of 0.06, does not differ significantly from that of other soils. The bulk particle specific gravity may however be much less than this.

12. Calcretes are prone to yield poor apparent gradings, with an excess of fine sand and a deficiency of coarse, probably due to degradation of the coarse sand particles in the test.

13. Calcretes tend to yield good CBRs and low swells in spite of their apparently poor gradings and Atterberg limits.

14. While the practical importance of self-cementation of calcrete roads remain 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. Four self-stabilization mechanisms appear to be possible in the case of calcretes : oxidation of soluble ferrous iron to insoluble ferric oxide, solution and recrystallization of carbonate, and reaction between calcium carbonate, amorphous silica and clay minerals under the high pH and high temperature conditions encountered under a blacktop surfacing in the arid and semiarid environment. Tests are available for detecting self-stabilization by all three mechanisms.

16. The properties of lime stabilized calcretes are controlled largely by their content of amorphous silica microfossil skeletons. Some calcretes yield higher and more rapid strengths 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. One to one and a half inch nodules for example may yield twice the 10% F.A.C.T. value of $\frac{3}{8}$"-$\frac{1}{2}$" nodules.

19. The dry strength of indurated calcrete particles depends largely on their Mohs' hardness, density and degree of crystallinity, increasing with density and hardness and decreasing with carbonate crystal size. A particularly good correlation exists between Mohs' hardness (H) and the dry 10% F.A.C.T. value = 4.7 H - 5). The soaked strength is usually about two thirds of 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 calcrete) to material with the character of rock, possessing 10% F.A.C.T. values of up to 22 tons (ACV of 19%) Mohs' hardnesses of up to 6 and densities up to 165 lbs/cu ft, which requires blasting for excavation.

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

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^{o}C for 24 hours resulted in every case in a lowering of the liquid limit by up to 2 percentage units and the plasticity index by 1 to 4. Linear shrinkage was not affected in three cases, in one case went up and in the other case went down. The effect of heating at 50^{o}C was somewhat similar, though generally less pronounced and more erratic.

23. Examination of a number of calcrete flow curves suggests that reasonably reliable ($\pm$0.5 units in 93% of cases) liquid limits can only be obtained from a flow curve method employing at least four points progressing

smoothly /...

smoothly from a wetter to a drier state (or possibly from a drier to a wetter state). One point methods will 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 about 77% of cases where at least one cycle of drying and rewetting is employed. Calcretes will thus tend to yield erratic, but generally high liquid limits in the standard South African method. 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 lead to the conclusion that the liquid limits and plasticity indices of calcretes are generally unreliable, but tend to yield results which are generally unrealistically high.

25. Linear shrinkage is the only soil constant which is considered to yield reliable results on calcretes and this is suggested as the primary control measure in place of the liquid limit and plasticity index. The bar linear shrinkage from the liquid limit is rapid and reliable.

26. Calcretes may possess a relatively weak aggregate fraction and testing of this is probably advisable in the case of medium and heavily trafficked bases. It is particularly important in the case of gravel road wearing courses. Several simple tests (Treton, APT, water absorption and Mohs' hardness) 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 leads to the conclusion that the normal South African specifications currently applied to these materials are too severe, and that considerable relaxation of apparent grading,

liquid /...

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APPENDIX : 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 course of the calcrete project are shown in Tables 15 to 18. Some details of the methods used are shown in Table 14 below :

TABLE 14 TEST METHODS USED AND ABBREVIATIONS

TEST	REFERENCE
Absorption limit (w_A)	Nascimento et al[20]
Liquid limit (w_L or LL)	NIRR[33]
Plastic limit (w_p)	"
Shrinkage limit (w_S)	Lambe[61]
Shrinkage ratio (SR)	"
Plasticity index (I_P or PI)	$w_L - w_p$
Flow index (I_f)	$2\ (w_{10}\text{blows} - w_{32}\text{blows})$
Toughness index (I_t)	$\frac{I_p}{I_f}$
Bar linear shrinkage (LS)	NIRR[33]
Petrifaction degree (PD)	$\frac{w_S}{wA}$ Nascimento et al[20]
Particle specific gravity (G_s)	NIRR[33]
Conductivity ($G_{15.8}$)	Netterberg[50]
pH	"
Mechanical analysis	NIRR[33]
Dry 10% Fines aggregate crushing test (10% F.A.C.T.)	Shergold and Hosking[10], with sieves for different aggregate size as for ACV[28]. Results are in 2000 lb tons.
Soaked 10% F.A.C.T.	10% F.A.C.T on 24 hour soaked, surface dry aggregate
Aggregate crushing value (ACV)	Department of Transport[28]

Los /...

TEST	REFERENCE
Los Angeles abrasion (LAA)	California 211.-B, using preferred abrasive charge, only one size fraction and 2500 g sample.
Aggregate pliers test (APT)	Netterberg[13]
Water absorption	On crushed hardpans, from the results obtained during the soaked 10% F.A.C.T. On nodules, ASTM C127-59[42]
Bulk density (γ_D)	On hardpan lumps before crushing, by coating with wax and weighing in air and in water : $\gamma_D = \dfrac{a}{b-c - \dfrac{(b-a)}{(G_{wax})}}$ 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[42]
Optimum moisture content (OMC)	Department of Transport[28]
Maximum dry density (MDD)	" " "
California Bearing ratio (CBR)	" " "
Grading modulus (GM)	Kleyn[4]
Effective size (D_{10})	Maximum size of the smallest 10% by weight[62]
Uniformity coefficient (C_u)	$\dfrac{\text{Maximum size of the smallest 60\% by weight}^{62}}{D_{10}}$
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)	" " "
Sulphate soundness	Department of Transport[28]
Sound cycles	Number of completed cycles before onset of disintegration

TEST	REFERENCE
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[63] or hardness tester[1], weighting 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]

TABLE ~~11~~ 9 TE

ROAD	SAMPLING POINT	ROAD PROFILE	SAMPLE DETAILS			PERCENTAG		
			CUPWAU No.	DEPTH	MATERIAL	2.0	1.5	1.
PORT ELIZABETH MARINE DRIVE	Opposite Noordhoek quarry	?	DE.7400	Base	Calcrete	100/98 98/100	81	72
	"	"	DE.7401	"	"	100/60	46	46
	"	"	DE.7402	"	"	100	67	62
	"	"	DE.7403	"	"		100	90
	"	"	DE.7404	"	Mixture of 7400-7404	-	-	-
	Mile 5	?	DE.7415	Base	Calcrete	100	78	78
	" "	"	DE.7416	"	"	100/53	53	53
	" "	"	DE.7417	"	"	100	73	61
	" "	"	DE.7418	"	"	100	79	75
	" "	"	DE.7419	"	Mixture of 7415-7419	-	-	-
PORT ELIZABETH - ADDO	Mile 29.5	?	DE.7380	Base	Calcrete		100	87
	" "	"	DE.7381	"	"		100	93
	" "	"	DE.7382	"	"		100	85
	" "	"	DE.7383	"	"		100	96
	" "	"	DE.7484	"	Mixture of 7380-7384	-	-	-
	Mile 26	?	DE.7385	Base	Calcrete		100	93
	" "	"	DE.7386	"	"			100
	" "	"	DE.7387	"	"			
	" "	"	DE.7388	"	"		100	94
	" "	"	DE.7389	"	Mixture of 7385-7388	-	-	-
LANDSDOWNE ROAD, CAPE FLATS	Mile 0.1	0 - 24" Calcrete	D.9558	0 - $4\frac{1}{2}$"	Calcrete		100	96
	" "	24"+ Sand	D.9559	$4\frac{1}{2}$" - 14"	"	97	85	73
	Mile 0.2	0 - 12" Calcrete	D.9561	0 - 4"	"	100	98	93
	" "	12 - 17" Calcrete + sand 17"+ Sand + calcrete	D.9562	4"-12"	"	92	86	80
	-	-	-	-	Mixture of 9558 & 9561	-	-	-
	Mile 0.4	0-17" Calcrete	D.9564	0-6"	Calcrete	95	88	81
	" "	17"+ Sand	D9565	6"-12"	"	94	81	70
	Mile 0.5	0-$10\frac{1}{2}$" Calcrete	D9566	0-6"	"	100	96	88
	" "	$10\frac{1}{2}$" - 18" Greyish soil	D9567	6"-$10\frac{1}{2}$"	"	95	94	87
	Mile 0.6	0-13" Calcrete	D9569	0-3"	"	99	97	87
	" "	13"+ Greyish soil	D9570	3"-8"	"	96	92	86

TABLE ~~11~~ 9 TEST RESULTS N FIVE ROADS EMPLOYING CALCRETE AS BASE AND SUBBASE

	PERCENTAGE PASSING (INCHES, MILLIMETRES, U.S. SIEVES)									HYDROMETER ANALYSIS (mm)						CONSTANTS		
										SAND				SILT	CLAY			
										COARSE	FINE							
ERIAL	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	<2.0 >0.42	<0.42 >0.246	<0.246 >0.147	<0.147 >0.05	<0.05 >0.005	<0.005	LL %	PI %	LS %
rete	100 / 98 / 98	81	72	72	72	72	71	56	17	21	12	20	23	8	16	22	4	2.5
"	100 / 60	46	46	46	45	44	44	42	15	5	12	30	19	14	20	23	5	2.5
"	100	67	62	62	59	55	50	45	16	10	17	25	15	13	20	23	5	2.5
"		100	90	90	86	80	75	55	19	27	10	24	14	9	16	21	5	2.5
ure of -7404	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
rete	100	78	78	71	71	65	61	48	15	21	15	22	17	9	16	23	8	3.5
"	100 / 53	53	53	51	49	44	37	28	9	24	14	21	17	9	15	24	9	4.0
"	100	73	61	59	57	54	50	41	14	18	15	20	20	11	17	23	8	3.5
"	100	79	75	74	74	65	58	33	49	43	10	18	13	6	10	20	6	2.5
ure of -7419	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
rete		100	87	85	83	79	70	56	23	20	8	11	29	20	12	28	12	5.5
"		100	93	93	86	71	67	55	18	18	8	13	34	18	9	31	14	6.0
"		100	85	79	77	68	61	49	16	20	10	13	31	16	11	31	11	5.5
		100	96	96	93	82	79	56	21	29	8	11	25	17	9	28	12	5.5
ure of -7384	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
ete		100	93	93	88	76	66	39	13	41	10	11	19	10	9	32	14	6.5
			100	97	95	84	73	42	13	43	10	10	19	10	8	33	15	6.5
				100	99	90	80	52	17	35	9	12	23	11	10	31	13	6.0
		100	94	94	94	83	72	53	17	26	12	12	26	12	12	31	13	6.0
ure of 7388	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
ete		100	96	90	88	75	68	65	21	19	16	15	19	11	20	19	7	3.0
	97	85	73	66	62	51	46	37	11	20	18	18	20	12	13	19	6	2.5
	100	98	93	90	88	78	71	56	20	21	16	17	19	11	17	19	6	3.0
	92	86	80	72	[illegible]	57	51	41	14	20	18	17	19	12	15	19	6	2.5
re of 79561	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-
ete	95	88	81	76	75	67	61	48	16	21	17	16	19	11	15	20	8	3.0
	94	81	70	68	62	56	52	42	10	19	19	21	20	10	10	18	6	2.5
	100	96	88	77	71	60	54	43	12	20	17	19	21	14	9	17	6	2.5
	95	94	87	87	84	75	69	56	16	19	19	20	19	12	11	18	7	2.5
	99	97	87	85	78	63	57	45	15	21	16	18	20	13	12	16	5	2.5
	96	92	86	80	73	59	53	43	13	19	17	18	22	16	8	18	5	2.5

	DENSITY			MOD. AASHO OMC, CBR AND SWELL								
					4 DAYS SOAKING		CURED 7 DAYS SOAKED 4 DAYS		CURED 28 DAYS SOAKED 4 DAYS			
ERIAL	IN-SITU lbs/cu ft	M. AASHO lbs/cu ft	RELATIVE %	OMC %	CBR %	SWELL %	CBR %	SWELL %	CBR %	SWELL %	APPROX. GRADING MODULUS	APPROX. GROUP INDEX
rete	-	-	-	-	-	-	-	-	-	-	1.5	0.0
"	-	-	-	-	-	-	-	-	-	-	1.9	0.0
"	-	-	-	-	-	-	-	-	-	-	1.6	0.0
"	-	-	-	-	-	-	-	-	-	-	1.5	0.0
ure of -7404	-	110	-	14.4	90	0.1	36	0.0	38	0.0.0	-	—
rete	[illegible]	-	-	-	-	-	-	-	-	-	1.7	0.0
"	-	-	-	-	-	-	-	-	-	-	2.2	0.0
"	-	-	-	-	-	-	-	-	-	-	1.8	0.0
"	-	-	-	-	-	-	-	-	-	-	2.0	0.0
ure of -7419	-	116	-	12.4	62	0.0	85	0.0	63	0.0	-	—
rete	-	-	-	-	-	-	-	-	-	-	1.4	0.4
"	-	-	-	-	-	-	-	-	-	-	1.4	0.5
"	-	-	-	-	-	-	-	-	-	-	1.7	0.2
	-	-	-	-	-	-	-	-	-	-	1.4	0.3
ure of -7384	-	116	-	12.2	66	0.2	75	0.0	68	0.1	-	—
ete	-	-	-	-	-	-	-	-	-	-	1.8	0.2
	-	-	-	-	-	-	-	-	-	-	1.7	0.3
	-	-	-	-	-	-	-	-	-	-	1.5	0.3
	-	-	-	-	-	-	-	-	-	-	1.5	0.3
ure of 7388	[illegible]	117	-	13.1	47	0.1	29	0.0	19	<0.05	-	—
ete	121	121	100	-	-	-	-	-	-	-	1.5	0.0
	-	-	-	-	-	-	-	-	-	-	2.0	0.0
	123	121	102	-	-	-	-	-	-	-	1.5	0.0
	-	-	-	-	-	-	-	-	-	-	1.9	0.0
re of 79561	-	121	-	10.0	112	0.0	-	-	-	-	-	—
ete	118 113	121	98	10.6	125	0.0	-	-	-	-	1.7	0.0
	-	-	-	-	-	-	-	-	-	-	1.9	0.0
	121	125	97	9.5	134	0.0	-	-	-	-	1.9	0.0
	-	-	-	-	-	-	-	-	-	-	1.5	0.0
	121	±122	±99	-	-	-	-	-	-	-	1.8	0.0
	-	-	-	-	-	-	-	-	-	-	1.9	0.0

TABLE ~~21~~ 9 (Continued)

ROAD	SAMPLING POINT	ROAD PROFILE	SAMPLE DETAILS: C.P.A. No.	SAMPLE DETAILS: DEPTH inches	SAMPLE DETAILS: MATERIAL	PERCENTAGE PASSING (INCHES, MILLIMETRES, 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	HYDROMETER ANALYSIS (mm): SAND: COARSE < 2.0 > 0.42	SAND: FINE <0.42 >0.246	SAND: FINE <0.246 >0.147	SAND: FINE <0.147 >0.05	SILT <0.05 >0.005	CLAY <0.005	CONSTANTS: LL %	PI %	LS %	DENSITY: IN-SITU lbs/cu ft	M. AASHO lbs/cu ft	RELATIVE %	OMC %
DUINE-FONTEIN ROAD, CAPE FLATS	Mile 0.2	0-13" Calcrete	V396	0-5	Calcrete	100	99	96	92	88	77	69	53	18	23	15	17	19	13	13	[illegible]	7	3.0	130	127	102	9.
	" "	13"+ Sand	V397	5-10	"	100	95	91	87	81	74	68	54	18	21	15	17	21	14	13	21	8	3.0	-	-	-	-
	Mile 0.6	0-14" Calcrete	V400	0-6	Calcrete		100	93	90	87	74	66	52	17	21	16	17	20	14	12	19	6	2.5	129	127	102	-
	" "	14"+ Sand	V401	6-14	"	81	79	75	71	69	62	58	46	16	21	15	16	21	14	13	20	7	3.5	--	-	-	-
	Mile 1.0	0-8" Calcrete	V402	0-8	Calcrete	100	99	99	97	96	91	88	74	16	16	23	25	18	18 10	8	18	4	1.5	118	123	96	9.
	" "	8"+ Sand	V403	8-18	Sand							100	74	2	26	34	30	7	1	1	-	NP	0.0	-	-	-	-
	Mile 1.4	0-13" Calcrete	V404	0-4	Calcrete	97	91	88	83	[illegible]	67	61	48	16	21	16	16	20	13	13	18	5	2.5	132	127	104	-
	" "	13-18" Sand	V405	4-13	"		100	97	96	95	87	81	61	19	25	15	18	19	12	12	19	6	2.5	-	-	-	-
	Mile 1.8	0-14" Calcrete	V406	0-4	Calcrete	100	95	95	86	82	72	65	52	18	20	15	18	21	14	13	20	6	2.5	128	127	101	-
	" "	14-18" Sand	V407	4-14	"	100	98	91	81	81	71	64	49	14	23	16	15	23	13	9	20	6	2.5	-	-	-	-
WELTE-VREDE ROAD, CAPE FLATS	Mile 0.1	0-14" Calcrete	V2327	0-4	Calcrete	100	98	90	88	82	66	59	44	15	25	17	17	16	11	14	21	6	2.5	123	124	99	10.
	" "		V2328	4-14	"		100	94	89	85	76	69	52	17	25	17	17	16	11	14	18	6	2.5	-	-	-	-
	Mile 0.2	0-16" Calcrete	V2329	0-4	Calcrete		100	94	90	84	68	59	44	15	25	16	16	16	11	15	18	6	2.5	118	±121	±98	-
	" "		V2330	4-16	"	90	87	78	75	70	57	51	37	13	27	18	17	14	10	15	[illegible]	7	2.5	-	-	-	-
	Mile 0.3	0-13" Calcrete	V2331	0-5	Calcrete	98	90	84	76	72	61	55	40	14	27	18	17	14	10	15	19	7	2.5	112	118	95	12.
	" "	13"+ Sand	V2332	5-13	"	100	98	98	91	89	78	71	54	18	24	18	18	15	11	14	18	6	2.5	-	-	-	-
	Mile 0.4	0-12" Calcrete	V2333	0-3	Calcrete		100	95	88	82	70	63	46	15	27	16	17	16	10	14	19	5	2.5	118	±121	±98	-
	" "	12"+ Sand	V2334	3-12	"	100	94	94	88	76	66	59	44	15	25	17	17	15	12	13	18	5	2.5	-	-	-	-

	SIS (mm)		CONSTANTS			DENSITY			MOD. AASHO OMC, CBR AND SWELL							
	SILT	CLAY								4 DAYS SOAKING		CURED 7 DAYS SOAKED 4 DAYS		CURED 28 DAYS SOAKED 4 DAYS		APPR GRAI MODU
47 5	<0.05 >0.005	<0.005	LL %	PI %	LS %	IN-SITU lbs/ cu ft	M. AASHO lbs/ cu ft	RELATIVE %	OMC %	CBR %	SWELL %	CBR %	SWELL %	CBR %	SWELL %	
	13	13	1[illegible]	7	3.0	130	127	102	9.7	67	0.0	-	-	- - -	-	1.6
	14	13	21	8	3.0	-	-	-	-	-	-	-	-	-	-	1.6
	14	12	19	6	2.5	129	127	102	-	-	-	-	-	-	-	1.6
	14	13	20	7	3.5	--	-	-	-	-	-	-	-	-	-	1.7
	18 10	8	18	4	1.5	118 118	123	96	9.9	107	0.0	-	-	-	-	1.2
	1	1	-	NP	0.0	-	-	-	-	-	-	-	-	-	-	1.2
	13	13	18	5	2.5	132	127	104	-	-	-	-	-	-	-	1.7
	12	12	19	6	2.5	-	-	-	-	-	-	-	-	-	-	1.4
	14	13	20	6	2.5	128	127	101	-	-	-	-	-	-	-	1.7
	13	9	20	6	2.5	-	-	-	-	-	-	-	-	-	-	1.7
	11	14	21	6	2.5	123	124	99	10.9	95	0.0	-	-	-	-	1.8
	11	14	18	6	2.5	-	-	-	-	-	-	-	-	-	-	1.6
	11	15	18	6	2.5	118	±121	±98	-	-	-	-	-	-	-	1.8
	10	15	1[illegible]	7	2.5	-	-	-	-	-	-	-	-	-	-	1.9
	10	15	19	7	2.5	112	118	95	12.5	98	0.0	-	-	-	-	1.9
	11	14	18	6	2.5	-	-	-	-	-	-	-	-	-	-	1.5
	10	14	19	5	2.5	116	±121	±96	-	-	-	-	-	-	-	1.7
	12	13	18	5	2.5	-	-	-	-	-	-	-	-	-	-	1.8

TABLE 15 SUMMAR

MATERIAL TYPE	SAMPLE NUMBER	MINUS 40 MESH FRACTION (SOIL FINES) ABSORPTION LIMIT %	LIQUID LIMIT %	PLASTIC LIMIT %	SHRINKAGE LIMIT %	SHRINKAGE RATIO	PLASTICITY INDEX %	FLOW INDEX %	TOUGHNESS INDEX %	LINEAR SHRINKAGE %	PETRIFACTION DEGREE	PARTICLE SPECIFIC GRAVITY	CONDUCTIVITY m mhos/cm	pH	PARTICLE SIZE DISTRIBUTION — CUMUL. 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
CALCIFIED SANDS	895		33·0	18·1			14·9			6·7		2·667									100
	1015		24·7				S.P.			1·3		2·691									100
	1567						N.P.					2·650					100	89	86	82	73
	1625	59·7	49·0	27·7	29·2		21·3			9·1	0·49	2·729	0·25	8·6			100	93	86	81	73
	1636																				
	1637																				
	1971																				
	2506																				
POWDER CALCRETES	897		48·7	29·9			18·8			2·6		2·676									
	902		58·4	36·6			21·8			10·7		2·782					100	97		97	93
	1017		~50	36·7			~ 13			8·0		2·652									
	1020		42·7	27·1			15·6			7·3		2·649									
	1512	38·9	28·7	18·5	21·9		10·2			2·0	0·54	2·634					100	96	88	87	82
	1515	41·6	22·1		23·1		S.P.				0·56	2·676									100
	1557		27·4	15·0			12·4			4·0							100	98	96	92	85
	1696		45·8	31·3			14·5			8·7							100	96	94	93	
	1737		21·8	17·7			4·1			2·7											
	1881	34·9	33·3	23·7	25·8		9·6			5·3	0·74	2·746									
	2223	42·7	24·0	17·5	20·6		6·5			0·6	0·48	2·709	0·14	8·2				100	95	93	90
	2236		32·1				S.P.			2·7		2·607									
	2360		47·7	32·7			15·0			7·3		2·638	1·6	8·7				100	98	97	91

S AND POWDER CALCRETES

.S. SIEVES)			HALF INCH TO THREE QUARTER INCH AGGREGATE									−40 # LAA FINES			MODIFIED AASHO						CLASSIFICATI	
			10% FACT.		A.C.V.	L.A.A.		A.P.T.		WATER ABSORPTION	BULK DENSITY	LIQUID LIMIT	PLASTICITY INDEX	LINEAR SHRINKAGE	O.M.C.	M.D.D.	C.B.R.	GRADING MODULUS	EFFECTIVE SIZE	UNIFORMITY COEFF.	D.O.T.	EXTENDED CASAGRAND
-0008 0.020	0.0002 0.006	0.00008 0.002	DRY tons	SOAKED tons	%	100 REVS %	500 REVS %	A.F.V. %	A.P.V. %	%	gms/cc	%	%.	%	%	lbs./cu.ft	%		mm			
17	9	5																1.48			SSC	SF-G
13	8	6																1.53			SSC	GF
7	3	2						72	19									1.76			SSC	GP-G
15	10	7						94	39									1.65			SSC	GF-S
			2.8	0.3	48.4	47.4	96.0			21.7												
															20.0	101	54					
54	41	9																0.82			SC	CH [illegible]
								94	53													
								75	63													
40	14	5						75	33									1.09			SSC	GF
30	13	8						25	8									0.57			SC	ML
																		1.32			SSC	SF
33	29	18																1.15			SSC	SF
20	17	12																1.45			SSC	SF
40	36	15																1.20			SSC	SF
26	9	6													8.7	127	37	0.57			SC	ML
14	11							94	53									1.29			SC	SF-S
																		1.40			SSC	

CLASSIFICATION			REMARKS
D.O.T.	EXTENDED CASAGRANDE	REVISED P.R.A	
SSC	SF-GF	A-2-6(0)	
SSC	GF	A-2-4(0)	
SSC	GP-GF	A-1-b(0)	
SSC	GF-SF	A-2-7(2)	
			NATURAL
SC	CH ?	A-7-5(13)	NATURAL
			POWDERED BY TRAFFIC ?
			POWDERED BY TRAFFIC ?
SSC	GF	A-4(3)	NATURAL
SC	ML	A-4(7)	NATURAL
SSC	SF	A-2-6(0)	NATURAL
SSC	SF	A-7-5 (2)	POWDERED BY TRAFFIC
SSC	SF	A-2-4 (0)	NATURAL
SSC	SF	A-4(1)	FROM GRAVEL ROAD SURFACE
SC	ML	A-4(8)	NATURAL
SC	SF-SP	A-2-4 (0)	NATURAL
SSC		A-2-7 (1)	NATURAL

TABLE 16 SUMM

MATERIAL TYPE	SAMPLE NUMBER	MINUS 40 MESH FRACTION (SOIL FINES) ABSORPTION LIMIT %	LIQUID LIMIT %	PLASTIC LIMIT %	SHRINKAGE LIMIT %	SHRINKAGE RATIO	PLASTICITY INDEX %	FLOW INDEX %	TOUGHNESS INDEX %	LINEAR SHRINKAGE %	PETRIFACTION DEGREE	PARTICLE SPECIFIC GRAVITY	CONDUCTIVITY m mhos/cm	pH	PARTICLE SIZE DISTRIBUTION — CU 3·00 76·1	2·00 50·8	1·50 38·1	1·00 25·4	0·75 19·0	0·50 12·7	
NODULAR CALCRETES	913	49·4	34·5	16·8	19·7		17·7			7·3	0·40	2·687								100	
	976		40·5	21·6			18·9			7·3		2·534									
	978		63·5	54·4			9·1			8·7		2·523									
	1016		~64	42·0			~22			8·0		2·646									
	1021 M	52·9	40·4	26·8	30·5		13·4			5·3	0·58	2·610						100	97	96	8
	1024		24·6	17·0			7·6			4·7		2·654									
	1024 M	36·6	26·1	16·2	19·0		9·9			5·3	0·52					100	92	90	89	85	7
	1026		23·2	14·1			9·1			2·7		2·693									
	1026 M	36·4	25·6	15·0	17·8		10·6			5·3	0·49								100	98	8
	1513		34·2	21·5			12·7			5·3					100	97		93	90		8
	1514		43·0	23·1			19·9			8·0							100	94	84	80	7
	1516		31·3	21·5			9·8			2·7							100	86	80	78	6
	1533	42·3	33·4	22·8	24·3		10·6			4·7	0·57	2·804	2·0								
	1561		25·7	17·5			8·2			3·0						100	96	95	91	87	81
	1565		40·3	15·4			24·9			10·0			0·60			100	86	72	60	48	3
	1569		24·9				S.P.			3·3		2·596	0·29	8·1		100	85		82	75	6
	1601	42·0	26·0	21·9	30·9		4·1			2·7	0·74	2·656	1·0	8·5		100	88			84	7
	1601 R	44·9	25·8		27·5		S.P.			3·7	0·62										
	1602	43·0	31·5	18·7	24·1		12·8			4·0	0·61	2·662	1·9	8·7			100	98	88	84	81
	1618		21·2	14·2			7·0			4·0											
	1620		36·8	26·5			10·3			4·7											
	1624	47·1	40·8	21·2	19·6		19·6			9·3	0·42	2·677	0·29	8·5		100	93	85	72	56	45
	1876		31·9	21·6			10·3			5·0											
	1877																				
	1879																				
	1880																				
	1979		29·7				N.P.			4·0											
	1980		30·5	17·6			12·9			3·3			0·42	8·3		100	83	80	71	68	58
	1981		48·0	26·5			21·5			7·7			2·5	8·8			100	96	96	92	85
	2114	40·2	40·4	26·1	22·3		14·3			8·0	0·56	2·680					100	97	95	93	8
	2116	36·1	26·1	15·4	14·7		10·7			6·0	0·41	2·694	0·28	8·4	100	79		77	67	63	5
	2234		23·6				S.P.			1·3		2·650							100	82	
	2367		18·5				S.P.			2·0		2·689	2·4	9·0		100	93		89	84	6
	2368						S.P.					2·616	1·3	8·0	100	90		82	80	75	6
	2493		32·9	23·6			9·3			5·3		2·662	0·33	8·0			100	90	80	75	5
	2555	35·6			22·2						0·63	2·678				100	94	92	87	79	6
	2566											2·656			100	83		69	63	55	4
	2588		37·2				S.P.			4·0			0·35	7·8	100	89	83	77	75	64	

SUMMARY OF TEST RESULTS – NODULAR CALCRETES

ION — CUMULATIVE PERCENTAGE FINER BY WEIGHT (INCHES, MILLIMETRES, U.S. SIEVES) 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	HALF INCH TO THREE QUARTER INCH AGGREGATE: 10% FACT. DRY tons	10% FACT. SOAKED tons	A.C.V. %	L.A.A. 100 REVS %	L.A.A. 500 REVS %	A.P.T. A.F.V. %	A.P.T. A.P.V. %	WATER ABSORPTION %	BULK DENSITY gms/cc	−40 # LAA FINES: LIQUID LIMIT %	PLASTICITY INDEX %	LINE. SHRIN. %
100		70	58	53	47	40	31	25	22	19	11	7												
																		0	0					
																		87	54					
																		96	73					
96	86	83	74	66	58	47	29	16	15	14	11	10						70	34					
		72	61	57	54	50	41	28	23	13	10	5						39	19					
85	76	72	64	59	54	48	39	28	24	20	15	9												
		78	63	59	58	55	48	37	26	23	19	17												
98	86	79	63	59	57	53	46	34	29	25	18	5												
	82	79	67	56	49	44	41	33	27	24	15	5						92	14					
80	71	67	58	52	49	46	43	39										77	25					
78	64	59	47	41	37	35	34	29										78	37					
																		98	87					
87	81	79	72	64	56	45	29	19																
48	39	37	35	33	30	26	19	14																
75	64	59	51	40	33	27	22	14	9	7	4	3						91	48					
84	74	70	62	58	51	41	25	16	12	9	6							78	25					
84	81	77	68	63	56	48	34	27	17	13	7							69	21					
																		93	51					
																		78	24					
56	45	40	30	25	23	22	19	13	11	9	7	5						93	73					
		83	67	57	50	44	36	(21)	30	26	21	11												
																		91	44					
																		100	77					
																		99	77					
68	58	54	46	40	36	33	29	24																
92	85	81	69	58	49	42	35	26																
93	84	80	65	51	43	38	34	24	22	16	13	8												
63	52	48	41	37	34	33	31	23	21	19	18	10												
82		66	61	59	50	38	25	11	10	7	3							93	66					
84	68	62	52	49	46	39	26	17	15	13	12	11	12·5	7·1	26·7		37·2 [1]	100	69	5·4		32·8	15·6	6·7
75	65	61	53	47	39	31	20	12	8	6	4	2	5·5	3·0	35·1		~50 [1]	89	39	8·4				
75	56		48	44	39	28	20	14	11	9	9	7	12·1	3·9	26·3	14·0 [1]	46·3 [1]	100	65	10·2		30·3	9·7	4·0
79	63	57	44	40	37	31	20	12	11	8	6	4												
55	43	39	33	32	29	21	12	7	6	4	4	3												
64	52	48	35	26	22	19	15	10	8	5	4	2	7·6 [2]	2·6 [2]		22·1 [2]	67·8 [2]	81 [3]	56 [3]	16·9 [2]		33·3	S.P.	3·[illegible]

...R ...TION	BULK DENSITY gms/cc	−40 # LAA FINES LIQUID LIMIT %	PLASTICITY INDEX %.	LINEAR SHRINKAGE %	MODIFIED AASHO O.M.C. %	M.D.D. lbs./cu.ft	C.B.R. %	GRADING MODULUS	EFFECTIVE SIZE mm	UNIFORMITY COEFF.	CLASSIFICATION D.O.T.	EXTENDED CASAGRANDE	REVISED P.R.A.	REMARKS
								1·70			SSC	GF	A-2-6 (1)	
												G ?		
												G ?		
												G ?		
								1·52			SSC	SF-GF	A-2-6(1)	
								1·57			SSC	GF	A-2-4(0)	
								1·54			SSC	GF	A-2-4 (0)	
								1·42			SSC	GF	A-4(1)	
								1·46			SSC	GF	A-2-6(0)	
								1·51			SSC	GF	A-2-6(1)	
								1·54			SSC	GF	A-6(3)	
								1·87			G	GF	A-2-4(0)	
												G ?		
								1·53			SSC	GF-SF	A-2-4(0)	
								2·21			G	GF	A-2-6(0)	
								2·02			SSC	GC-GF	A-1-b(0)	
								1·71			SSC	GF	A-2-4 (0)	
								1·49			SSC	GF	A-2-6(0)	
												G ?		
												G ?		
								2·34			G	GC-GF	A-2-7(0)	
								1·62			SSC	GF	A-2-4(0)	
												G ?		
												G ?		
												G ?		
												G ?		
								1·94			G	GF	A-2-6(0)	
								1·56			SSC	GF	A-2-7 (1)	
					10·6	124	80	1·68			SSC	GF	A-2-6(0)	
					7·0	135	59	2·02			G	GF	A-2-6(0)	
								1·78			SSC	GP-GF	A-1-b(0)	
		32·8	15·6	6·7				1·85			SSC	GP-GF	A-1-b(0)	
					13·6 (4)	114 (4)	80 (4)	1·96			SSC	GP-GF	A-1-b(0)	
2		30·3	9·7	4·0	~ 11	~120	~ 110	1·99			G	GP-GF	A-2-4(0)	
								2·07			G	GP-GF		
								2·31			G	GP		
(2)		33·3	S.P.	3·3				2·31			G	G	A-1-a (0)	

TABLE 17 SUMMARY OF TEST RESULTS - HONEYCOMB, HARDPAN AND BOULDER CALCRETES AND CONTROL

MATERIAL TYPE	SAMPLE NUMBER	− 40 # L.A.A. FINES								$\frac{3}{8}$"-$\frac{1}{2}$" CRUSHED AGGREGATE						$\frac{3}{4}$"-1" CRUSHED AGGREGATE					
										10 % F.A.C.T.		A.C.V.	L.A.A.			20 CYCLE SODIUM SULPHATE SOUNDNES					
																CUMULATIVE PERCENT PASSING					
		LIQUID LIMIT %	PLASTIC LIMIT %	PLASTICITY INDEX %	FLOW INDEX %	TOUGHNESS INDEX %	LINEAR SHRINKAGE %	CONDUCTIVITY m mhos/cm	pH	DRY tons	SOAKED tons	%	100 REVS %	500 REVS %	WATER ABSORPTION %	$\frac{3}{4}$"	$\frac{1}{2}$"	4 #	10 #	40 #	100 #
HONEYCOMB CALCRETES	1931																				
	1573	24.5		SP.			3.0			16	11	23.6			5.7						
	2494	15.8	13.5	2.3			0.7			23.1	16.0	16.0	5.5	24.5	4.9						
HARDPAN CALCRETES	1530			N.P.				0.19	8.4	2.5	1.5				20.0	100	98	79	64	51	32
	1562			S.P.				0.086	9.5	14.5	11.3	27.1	(20.3)	(85.8)	4.8	51	44	25	15	8	3
	1623											20.9			2.8						
	1642	36.5	31.7	4.8			3.3	0.18	10.0	2.8	1.3	52.8				100	94	84	71	38	21
	1658	26.2	23.8	2.4			2.0	0.10	9.4	22.3	12.8	19.3			4.5	48	16	10	6	4	3
	1658 X	39.5	32.5	7.0			3.0			18.2	13.0	21.0			4.4						
	1724	38.3	39.0	0			1.3	0.61	9.1	7.1	4.9	37.5	13.0	44.2	24.0	100	80	50	31	12	5
	1828							0.18													
	2200			S.P.				0.10	8.7	8.8	4.5	30.6			4.3						
	2204	22.1	18.5	3.6			2.0	0.15	8.8	18.4	12.3	20.2	7.6	34.1	4.9						
	2225	26.3	22.5	3.8			1.3			18.7		21.1	9.3	41.6							
	2226	26.3	20.0	6.3			1.3			12.7		24.9	13.7	57.6							
	2227	28.1	21.9	6.2			2.6			11.1		30.0	16.2	56.5							
	2505									12.4	6.7	24.9	11.6	48.9	5.8						
	2507									9 – 15	5 – 6		12.2	49.0	5.9						
BOULDER CALCRETES	1028			S.P.						13.3	7.2										
	1028 X							0.16	9.9	11.3	9.2	33.1			8.4	31	12	6	4	1	1
	1621											20.4			3.8						
	1622											23.1									
	1627	20.0	17.3	2.7			1.3			18.2	11.6	24.6			2.8	100	81	33	16	6	3
	1627 X	19.2		S.P.			0.6	0.057	9.2	19.4	13.7	20.8	7.8	36.1	3.4						
	2125											25.3			5.7						
	2228			N.P.						12.6		25.2	13.8	59.0							
	2302									9 – 16	6.3	26.8			7.3						
XTURE OF A, B AND C	890							0.41	9.2	23.8		17.7			3.4						
TREOUS SILCRETE	890 A																				
TREOUS SILCRETE	890 B																				
OLOMITIC SANDSTONE	890 C							0.33	9.3	19.9					5.1						
OSIB (?) QUARTZITE	1833 X							0.054	7.9	23.6	25.0	18.0	7.2	28.0	2.9						

ER CALCRETES AND CONTROL SAMPLES

$\frac{3}{4}$"-1" CRUSHED AGGREGATE — CYCLE SODIUM SULPHATE SOUNDNESS — CUMULATIVE PERCENT PASSING $\frac{1}{2}$"	4 #	10 #	40 #	100 #	SOUND CYCLES	$\frac{1}{4}$"-$\frac{5}{16}$" CRUSHED AGG. POLISHED STONE VALUE (1965)	DETACHMENT %	FLAKINESS %	HAND SPECIMENS BULK DENSITY (RAPID METHOD)	CALCULATED BULK DENSITY (WAXED METHOD)	MoHS' HARDNESS $\bar{H}w$	$\bar{H}w$ + Hmax − Hmin	EXTENDED CASAGRANDE CLASSIFICATION	REMARKS
														RIPPABLE
														RIPPABLE
98	79	64	51	32	2									RIPPABLE BY ALLIS-CHALMERS H.D.16. STIFF, SANDY, FRIABLE TUFACEOUS TYPE
44	25	15	8	3	7	0.60	0.5	5.7	2.45	2.45	4.1	7.1		TYPE 3, VERY STIFF — HARD, INTACT
														HARD, INTACT HARDPAN
94	84	71	38	21	1				1.59	1.50	1.7	(3.2)		FRIABLE TUFACEOUS TYPE
16	10	6	4	3	7	0.47	15	5.0	2.64	2.64	5.8	5.8		HARD INTACT HARDPAN AND BOULDERS. REQUIRED BLASTING. SEISMIC VELOCITY 10,000 FT/SEC
														SAID TO COME FROM THE SAME SITE AS 1658
80	50	31	12	5	4	(0.65)	100	4.7	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)		STIFF, INTACT, TYPE 6
									2.57	2.57	4.3	4.7		HARD, INTACT, MAINLY TYPE 1
						0.53		8.5	2.51	2.51				MIXTURE OF HARD TYPE 3, AND VERY STIFF TYPE 6. INTACT
						0.48		2.0	2.46	2.46				SOME TYPE 6 PRESENT, BUT MAINLY TYPE 3
						0.54		2.7	2.24	2.23				MAINLY HARD, INTACT TYPE 3
														VERY STIFF — HARD
														VERY STIFF GRAVELLY HARDPAN
													BOULDERS	
12	6	4	1	1	4	0.49	53	8.1	2.29	2.28	3.5	(5.5)	BOULDERS	
													BOULDERS	
													BOULDERS	
31	33	16	6	3	8	0.51	26	9.2	2.62	2.62	4.8	5.3	BOULDERS	
													BOULDERS	
													BOULDERS	
						0.58		2.7	2.48	2.48			BOULDERS	
													BOULDERS	APPARENTLY UNCRUSHABLE BY GRID ROLLER
													BOULDERS	
						0.47	47	2.2	2.59	2.59	7.7	8.2		
						0.43	55	5.7	2.53	2.53	7.7	7.7		
						0.59	30	7.1	2.63	2.63	7.8	8.6		
						0.56	13	5.8	2.55	2.55	7.7	9.2		

TABLE 18 SU

MATERIAL TYPE	SAMPLE NUMBER	ABSORPTION LIMIT %	LIQUID LIMIT %	PLASTIC LIMIT %	SHRINKAGE LIMIT %	SHRINKAGE RATIO	PLASTICITY INDEX %	FLOW INDEX %	TOUGHNESS INDEX %	LINEAR SHRINKAGE %	PETRIFACTION DEGREE	PARTICLE SPECIFIC GRAVITY	CONDUCTIVITY m mhos/cm	pH	3·00 76·1	2·00 50·8	1·50 38·1	1·00 25·4	0·75 19·0	0·50 12·7
		MINUS 40 MESH FRACTION (SOIL FINES)													PARTICLE SIZE DISTRIBUTION —					
CALCRETE MIXTURES	896		25·5	21·4			4·1			1·3		2·654								100
	900		82·0	64·4			17·6			4·7		2·468					100	93	92	86
	910		110·3	63·7			46·6			13·3		2·681								
	911		14·9	13·7			1·2			1·3		2·717								
	912		16·5	12·6			3·9			1·3		2·724								
	918		51·9	32·7			19·2			5·3		2·629								
	977		51·6	35·3			16·3			8·7		2·538								
	1018		31·8	23·4			8·4			4·0		2·636								
	1019		44·0	30·6			13·4			7·3		2·647					100	93	92	92
	1022		30·0	16·5			13·5			6·0		2·659							100	96
	2588																			
CALCAREOUS SOILS	891		55·7	19·4			36·3			12·0		2·700								
	893		43·0	20·1			22·9			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			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								
	2235						N.P.					2·654								
	2363	30·6					~N.P.				0·58	2·687								
	A10		67·8	33·0			34·8			18·7		2·680								
	1965		7				N.P.					2·661								

– 3590

TABLE 18 SUMMARY OF TEST RESULTS — CALCRETE MIXTURES AND CALCAREOUS SOILS

TICLE SIZE DISTRIBUTION — CUMULATIVE PERCENTAGE FINER BY WEIGHT (INCHES, MILLIMETRES, U.S. SIEVES)																HALF INCH TO THREE QUARTER INCH AGGREGATE										—
																10% FACT.		A.C.V.	L.A.A.		A.P.T.					
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	DRY tons	SOAKED tons	%	100 REVS %	500 REVS %	A.F.V. %	A.P.V. %	WATER ABSORPTION %	BULK DENSITY gms/cc	LIQ LI
			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										
																					93	61			
100	93	92	92	89	88	84	79	71	60	43	32	25	23	19	16						24	21			
		100	96	87	83	68	46	32	25	20	15	14	10	7	4						51	12			
					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	88	51	23	7	4	2	2											
							100	95	60	23	6	5	5	5	5										

ON	BULK DENSITY gms/cc	−40 # LAA FINES: LIQUID LIMIT %	PLASTICITY INDEX %	LINEAR SHRINKAGE %	MODIFIED AASHO: O.M.C. %	M.D.D. lbs./cu.ft	C.B.R. %	GRADING MODULUS	EFFECTIVE SIZE mm	UNIFORMITY COEFF.	CLASSIFICATION: D.O.T.	EXTENDED CASAGRANDE	REVISED P.R.A.	REMARKS
								1·53			SC		A-2-4(0)	MIXTURE OF CALCIFIED SAND AND NODULES, USED FOR GRAVEL ROAD
								1·92			SSC		A-2-7(0)	MIXTURE OF HARDPAN FRAGMENTS, NODULES AND CALCAREOUS SANDY CLAY, USED FOR GRAVEL ROAD
					23·0	96								
								2·69			G		A-1-a(0)	COMPACTED LUMPS (PROBABLY HARDPAN FRAGMENTS, NODULES AND CALCIFIED SAND) FROM GRAVEL ROAD
								2·77			G		A-1-a(0)	COMPACTED LUMPS (PROBABLY HARDPAN FRAGMENTS, NODULES AND CALCIFIED SAND) FROM GRAVEL ROAD
					21·0	96	60	1·14			SC		A-2-7(1)	MAINLY CALCAREOUS SANDY CLAY, SOME HARDPAN FRAGMENTS AND ODULES, USED FOR GRAVEL ROAD
								1·13			SC		A-2-7(1)	MIXTURE OF HARDPAN FRAGMENTS, NODULES AND TOPSOIL FROM UNFAILED GRAVEL ROAD SURFACE
								1·85			SSC		A-2-6(1)	MAINLY CALCIFIED WEATHERED FELSPATHIC SANDSTONE GRAVEL, SAID TO BE GOOD GRAVEL ROAD MATERIAL
														CHIEFLY NODULAR — SEE TABLE G2
					11·0	122	5	0·84			SC	SF	A-7-6(7)	CLAYEY SAND USED FOR GRAVEL ROAD
								0·89			SC	SF	A-2-7(2)	CLAYEY SAND USED FOR GRAVEL ROAD
								0·71			SC	CH	A-7-6(13)	SANDY CLAY USED FOR GRAVEL ROAD
					12·0	121	7	1·05			SC	SF	A-2-7(3)	CLAYEY SAND USED FOR GRAVEL ROAD
								0·20			SC	CH	A-7-5(20)	BLACK SANDY CLAY
								0·49			S	ML	A-4(0)	PALE BROWN SAND OF KALAHARI TYPE
								0·91			S	SU	A-2-4(0)	REDDISH BROWN SAND OF KALAHARI TYPE
								1·05			S	SU	A-3(0)	PALE BROWN SAND OF KALAHARI TYPE
					~7[1]	~132[1]	~96[1]				G	G	(0)	NODULAR FERRICRETE BASE COURSE, PROBABLY SELF-STABILISING
								0·99			S	SU	A-3(0)	REDDISH BROWN SAND OF KALAHARI TYPE

PLATE 1 : CALCIFIED GRAVEL

Note also the makondos (solution hollows) extending downwards from the surface of the calcified gravel, indicating the onset of chemical 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.

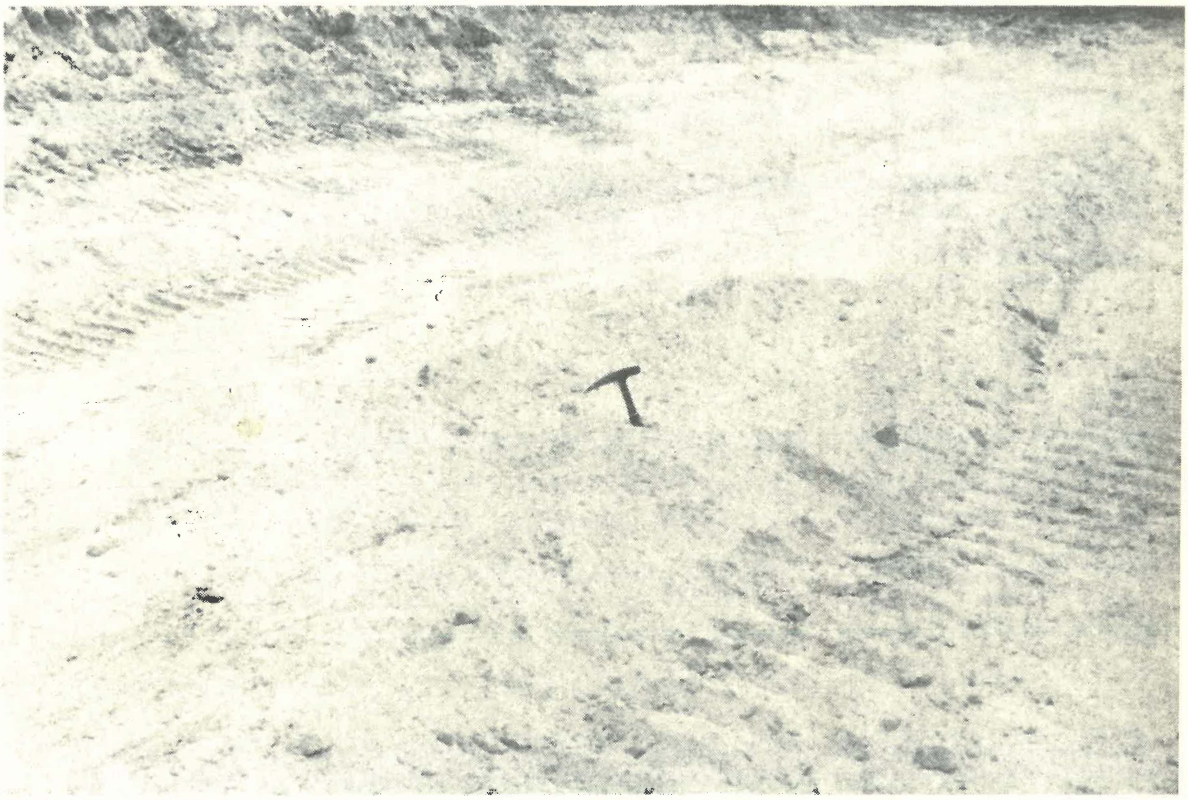


PLATE 3 : POWDER CALCRETE

Note the deficiency of coarse aggregate.

PLATE 4 : NODULAR CALCRETE

Essentially a loose gravel. Individual nodules and even compound nodules made up of several coalesced smaller nodules seldom exceed about 2½ inches in diameter.

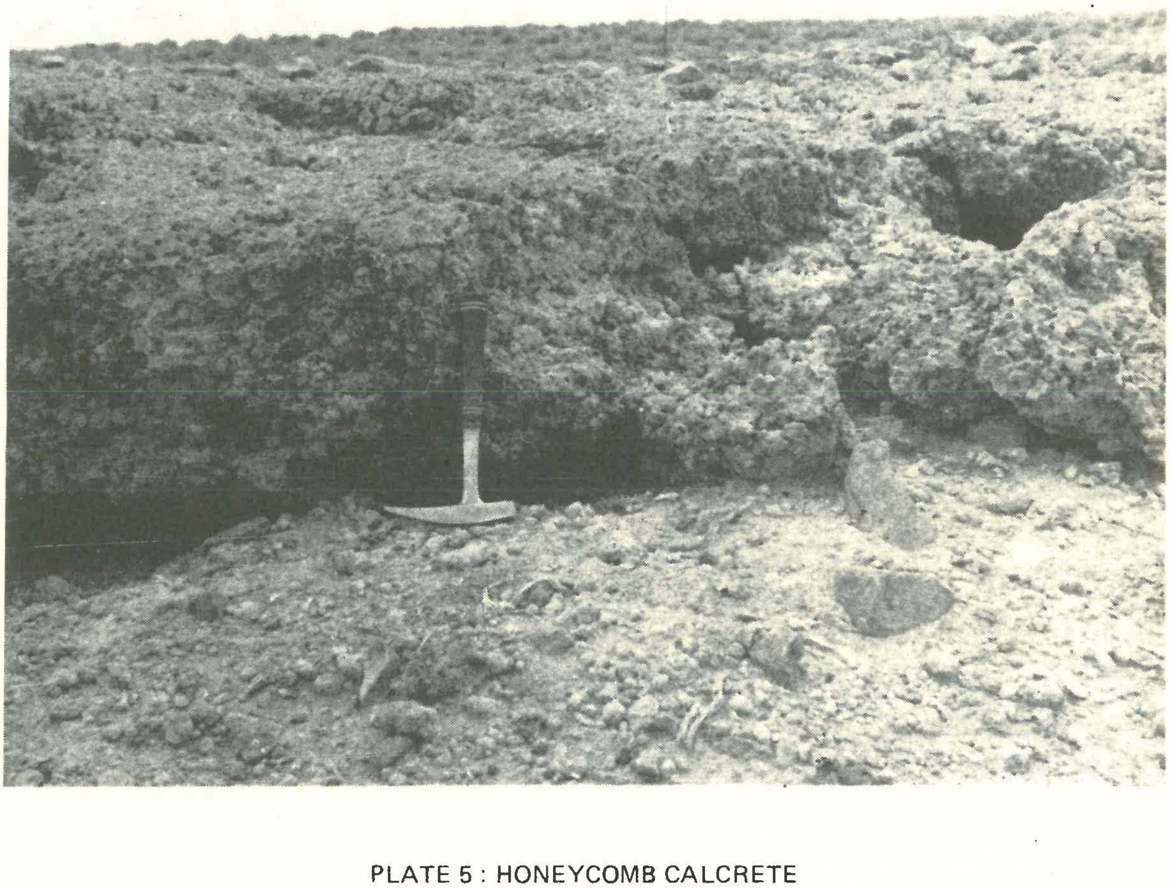


PLATE 5 : HONEYCOMB CALCRETE

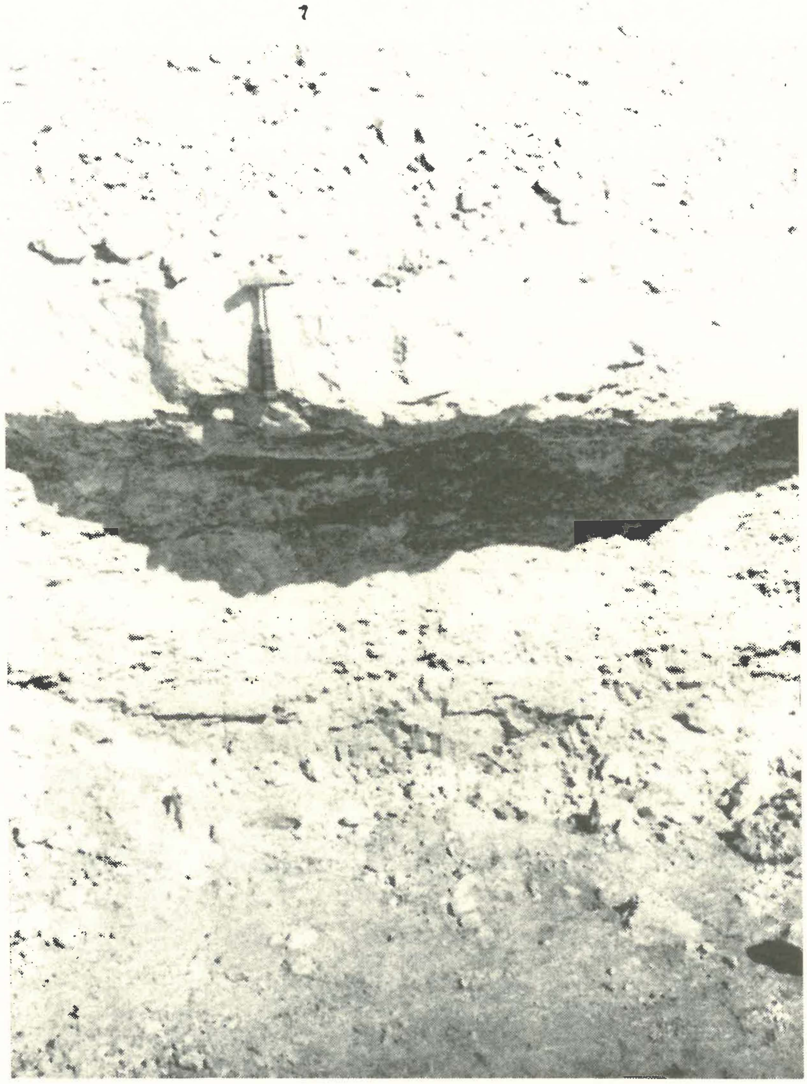


PLATE 6 : HARDPAN CALCRETE

An intact (in this case) hardpan layer overlying a faintly bedded, weakly developed nodular calcrete.

PLATE 7 : BOULDER CALCRETE

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

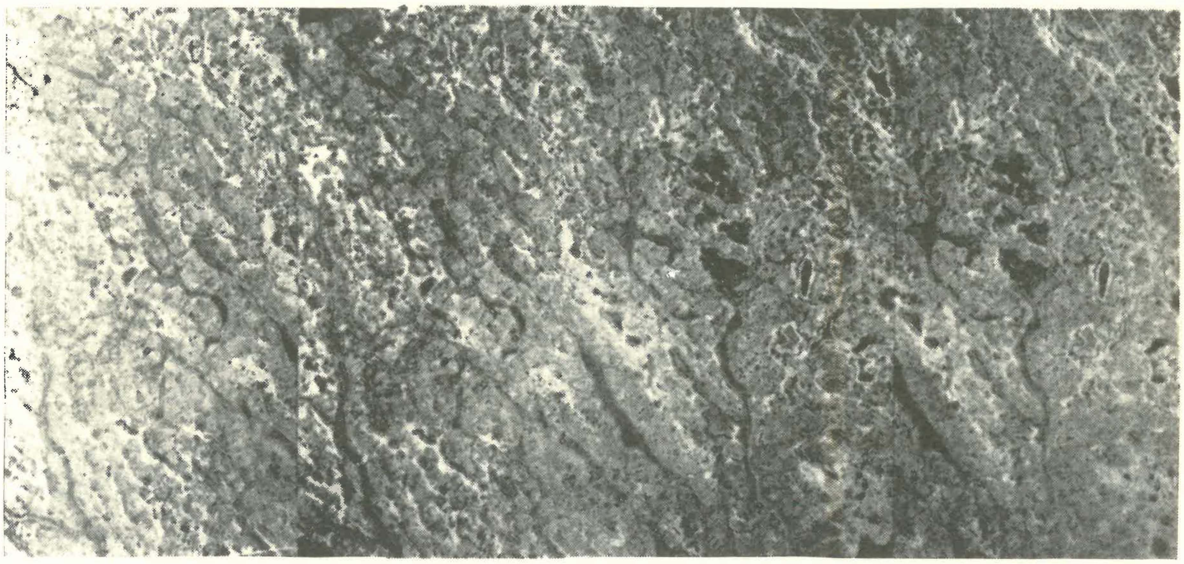
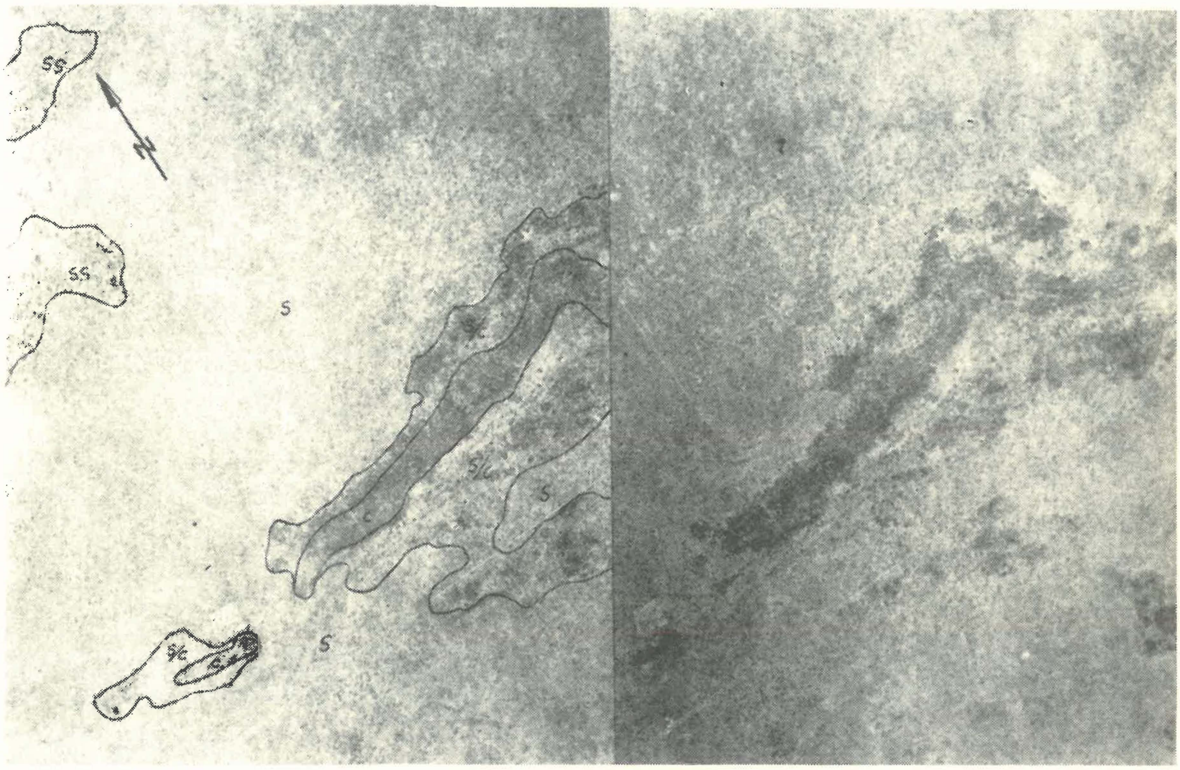


PLATE 8

Stereotriplet showing the distribution of calcrete borrow pits in the Ondangua area of South West Africa in relation to the drainage lines (oshanas).



ALTERNATIVE PLATE 8
(OR ADDITIONAL PLATE)

Calcrete (S) 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).

OTHER GOOD AIRPHOTOS SHOWING CALCRETE IN PANS IN DUNE COUNTRY ARE ALSO AVAILABLE

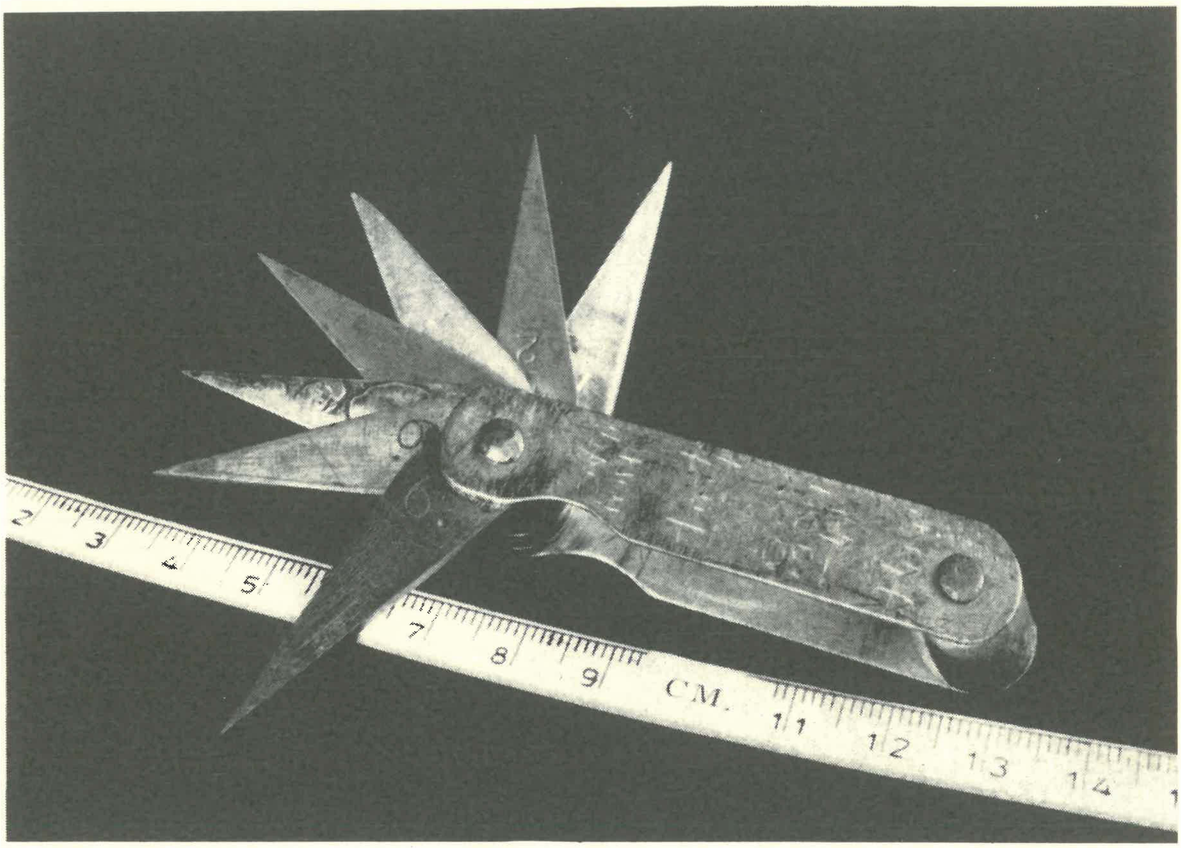
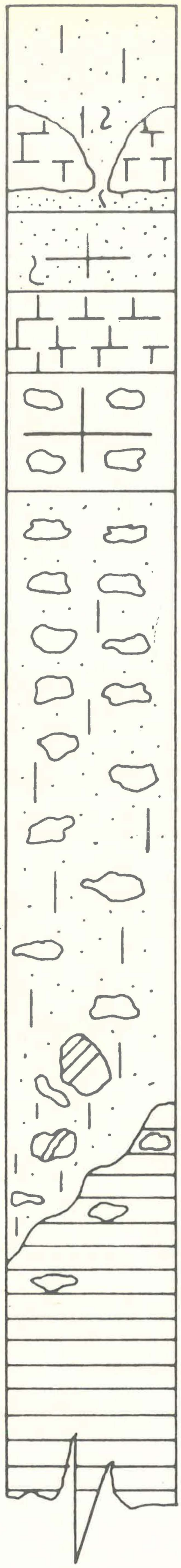


PLATE 9

Prototype pocket Mohs hardness tester. The blades are marked 1 to $6\frac{1}{2}$ (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. Length when closed 6 cms ($2\frac{1}{2}$ inches), weight 40 g ($1\frac{1}{2}$ oz). Photo: Graphic Arts, CSIR.



0 Reddish brown, loose, dry, silty SAND; roots; aeolian; non-calcareous

1'0" White, hard, dry, discrete, sandy calcrete BOULDERS in SAND as above; roots.

2'0" White, stiff, dry, shattered CALCIFIED SAND; roots

2'6"

3'0" White, very hard, dry, slightly shattered HARDPAN CALCRETE; roots in fissures; grading to

3'6" White, stiff, damp, HONEYCOMB CALCRETE; roots

5'0" White, loose, damp, silty, sandy, fine and medium GRAVEL; hard, round, intact, nodular calcrete; roots; grading downwards into

7'6" White, stiff, damp silty SAND; powder calcrete with scattered round, intact stiff nodules; grading into

8'6" Olive brown, stiff, damp, fissured and shattered weathered residual Dwyka SHALE; many fissures filled with powder calcrete; roots to water table

10'0" Water table

11'6" Bottom of pit

Date drilled : 23.11.66
Date profiled: 24.11.66
Scale :

FIGURE 1

Suggested method of recording an hypothetical calcrete profile for engineering purposes.

CALCRETE LEGEND

HA	All varieties of calcrete including hardpans always present
HC	All varieties of calcrete including hardpans common
HR	Hardpans rare, other less well developed varieties common
NA	Nodules always present Occasional nodular calcretes Hardpans extremely rare to absent
NR	Nodules rare. Nodular calcretes very rare. All other varieties absent
Cc	Gypseous calcretes common
X	Non calcareous

Δ	Unusual occurrence of hardpan or undifferentiated calcrete
●	Unusual occurrence of nodular calcrete

SOIL LEGEND

RAW MINERAL SOILS

ROCK AND ROCK DEBRIS

X	Ac	Not differentiated

DESERT DETRITUS

X	An	Sands (ergs)
X	Ar	Not differentiated

WEAKLY DEVELOPED SOILS

LITHOSOLS (skeletal soils) AND LITHIC SOILS

NR	Bb	On rocks rich in ferromagnesian minerals
X	Bc	On ferruginous crusts
HA	Bc'	On calcareous crusts
HR-X	Bd	Not differentiated

SUB DESERT SOILS

HC	Bf	Not differentiated

WEAKLY DEVELOPED SOILS ON LOOSE SEDIMENTS NOT RECENTLY DEPOSITED

HR-X	Bh	Not differentiated

JUVENILE SOILS ON RECENT DEPOSITS

NR	Bo	On riverine and lacustrine alluvium
X	Bp	On fluvio marine alluvium (mangrove swamps)

CALCIMORPHIC SOILS

HC	Ca	Rendzinas, brown calcareous soils
HC	Cb	Soils with calcareous pans (Cb); when also gypseous (Cc)

VERTISOLS AND SIMILAR SOILS

NA	Da	Derived from rocks rich in ferromagnesian minerals
NA	Db	Derived from calcareous rocks
NA	Dc	Of topographic depressions, not differentiated

HIGH VELD PSEUDO-PODSOLIC SOILS

NR	Ea	Not differentiated

BROWN AND REDDISH BROWN SOILS OF ARID AND SEMI-ARID REGIONS

BROWN SOILS OF ARID AND SEMI ARID TROPICAL REGIONS

H	Ga	On loose sediments
H	Gb	Not differentiated

RED AND BROWN MEDITERRANEAN SOILS

NR	Ia	Red mediterranean soils, not differentiated
NR	In	Brown mediterranean soils, not differentiated

FERRUGINOUS TROPICAL SOILS (fersiallitic soils)

NR	Ja	On sandy parent materials
X?	Jb	On rocks rich in ferromagnesian minerals
X,NR	Jc	On crystalline acid rocks
X	Jd	Not differentiated

FERRISOLS

X	Ka	Humic
X	Kb	On rocks rich in ferromagnesian minerals
X	Kc	Not differentiated

FERRALLITIC SOILS (sensu stricto)

DOMINANT COLOUR YELLOWISH-BROWN

X	La	On loose sandy sediments
X	Lb	On more or less clayey sediments
X	Lc	Not differentiated

DOMINANT COLOUR RED

X	Li	On loose sandy sediments
X	Lm	On rocks rich in ferromagnesian minerals
X	Ln	Not differentiated

HUMIC FERRALLITIC SOILS

X	Ls	Not differentiated

YELLOW AND RED FERRALLITIC SOILS ON VARIOUS PARENT MATERIALS

X	Lx	Not differentiated

HALOMORPHIC SOILS

HR	Ma	Solonetz and solodized solonetz
HR	Mb	Saline soils, alkali soils and saline alkali soils
HA,HR	Me	Not differentiated

HYDROMORPHIC SOILS

X,NR	Na	Mineral hydromorphic soils
X	Nb	Organic hydromorphic soils

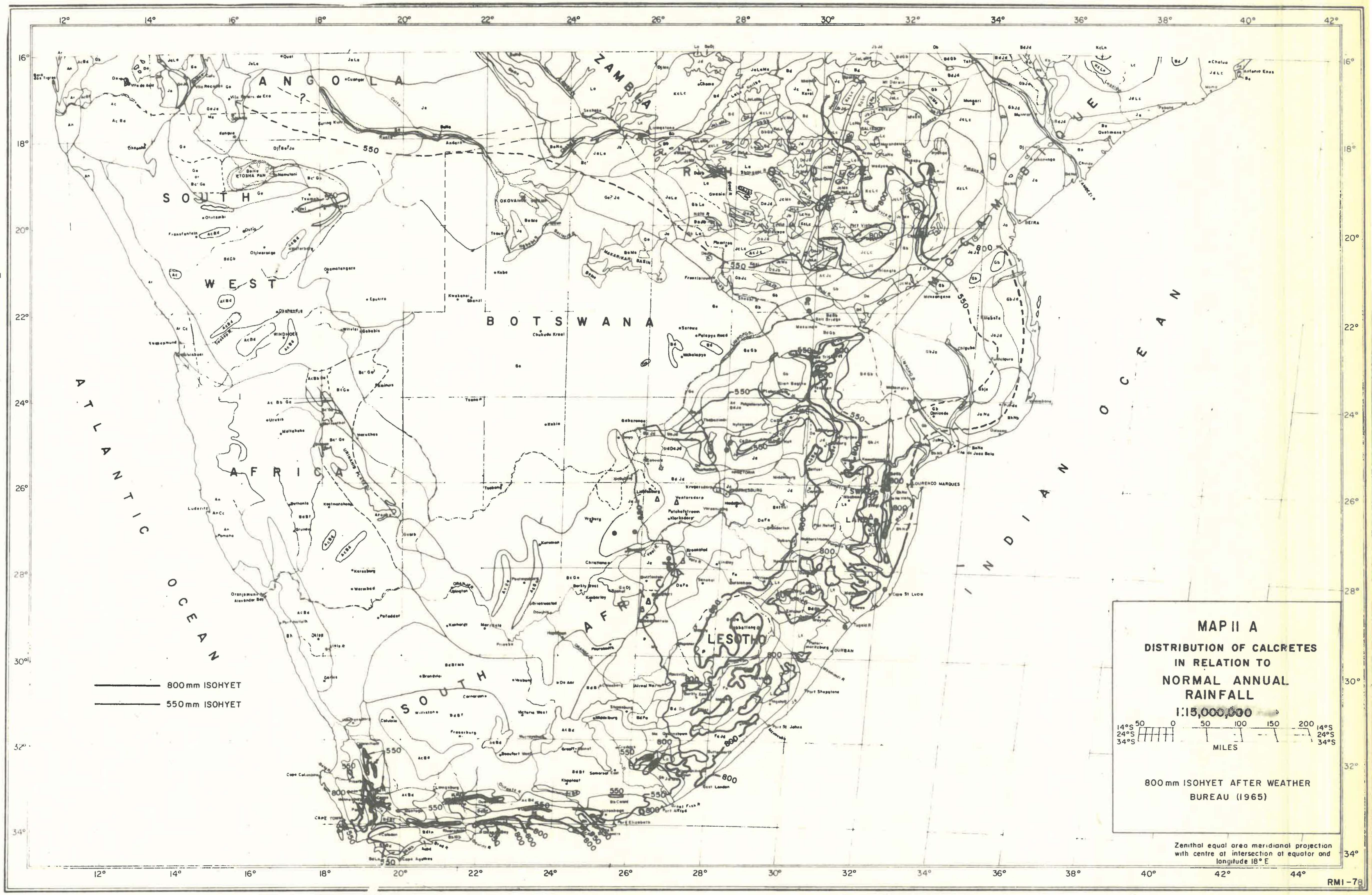


FIGURE 2.

RM4-78

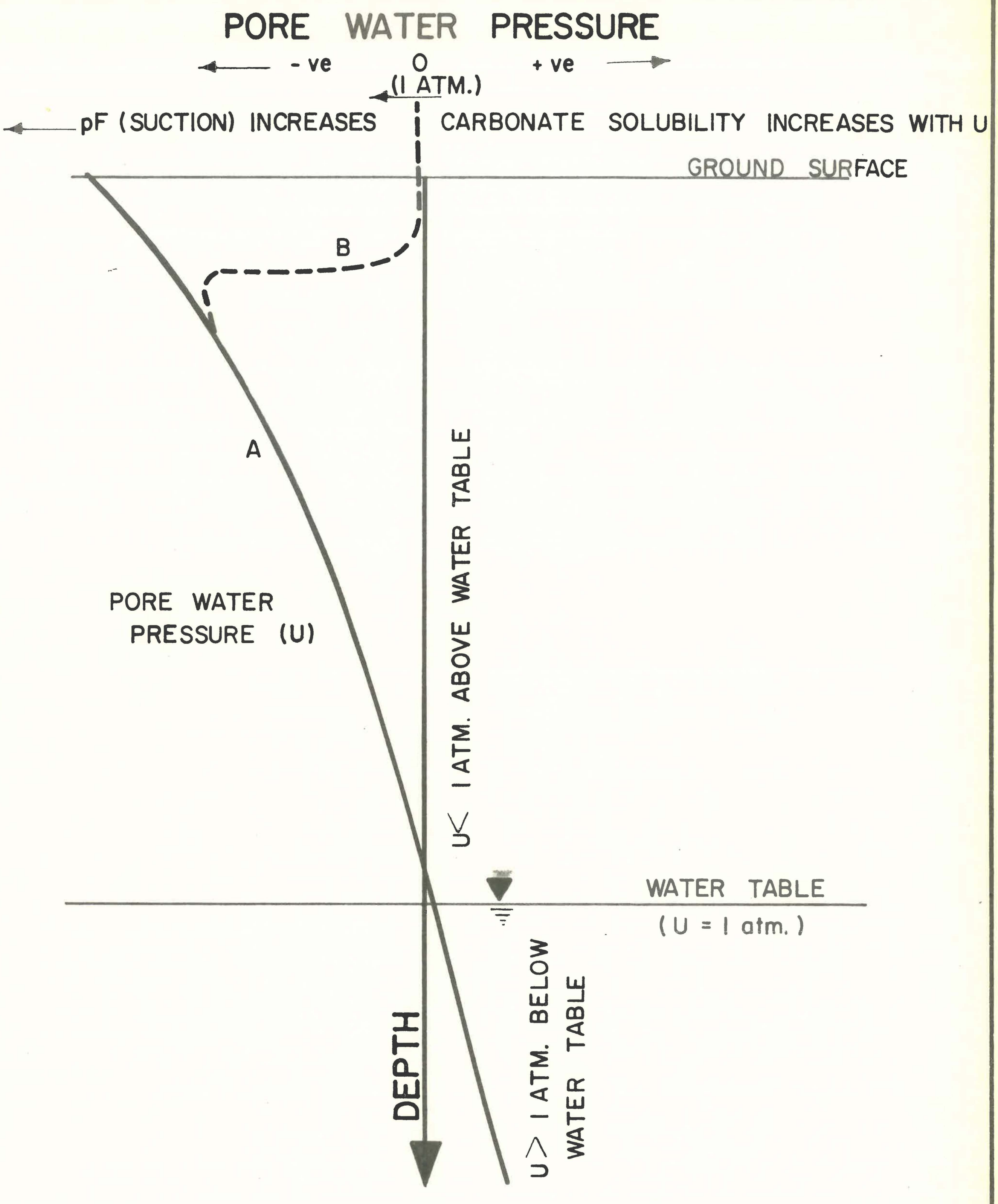


FIGURE 3

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

RG4 - 3691

SKETCH

SECTION.

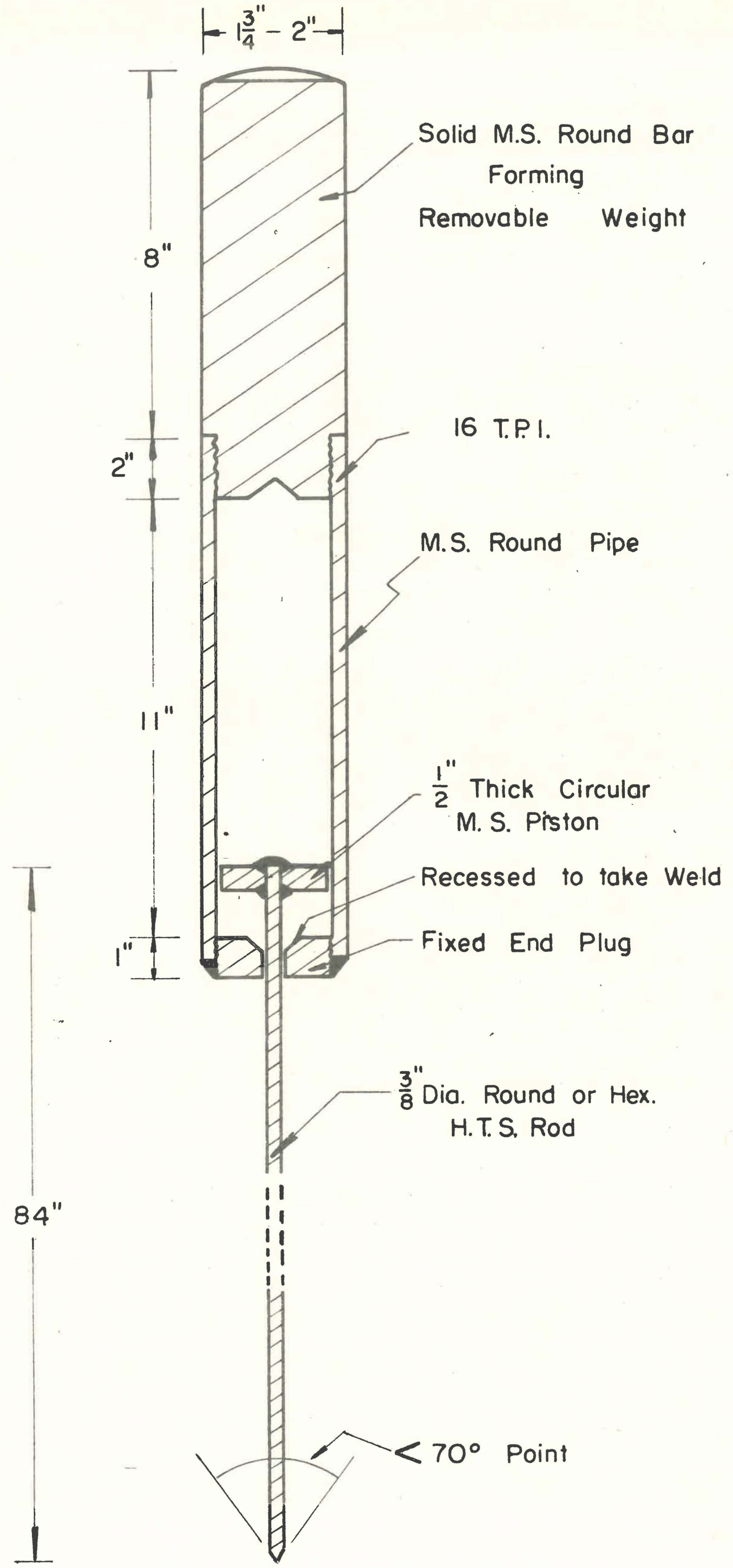


FIGURE 4

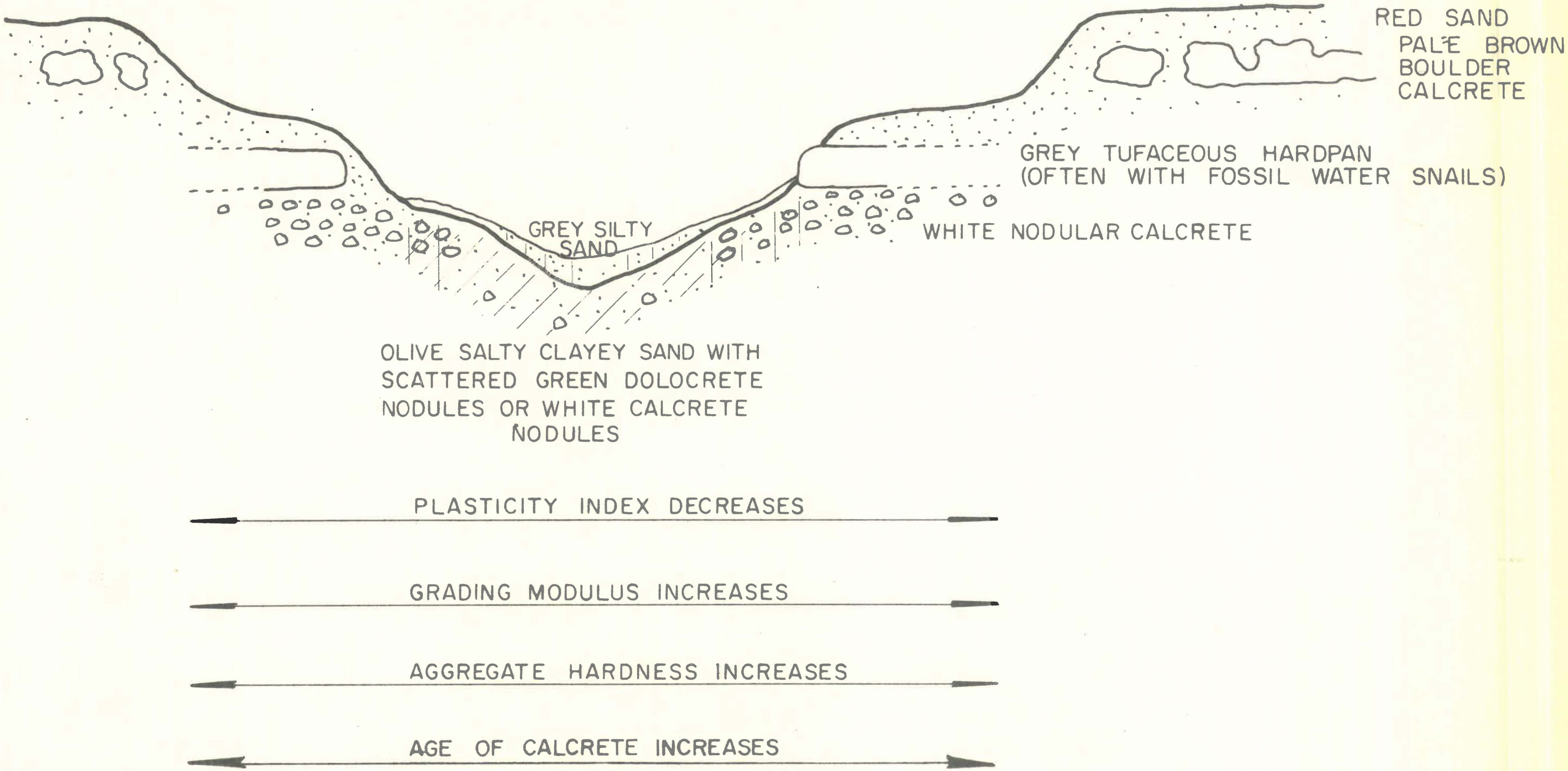


FIGURE 5

Idealised section across pan or omuramba

RG4-2473

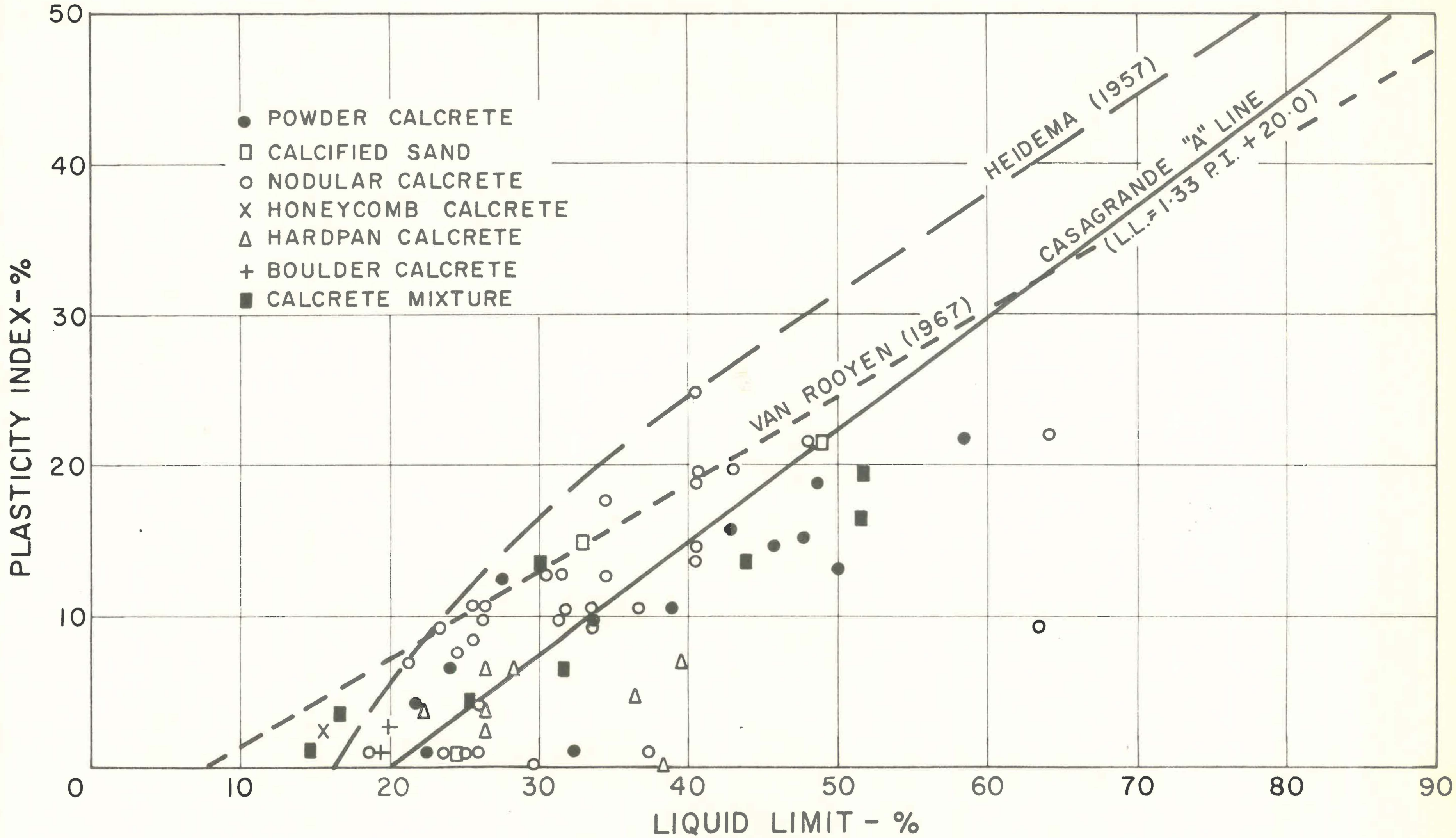


FIGURE 6.

Position of calcretes on the Casagrande plasticity chart

RG4 - 3198

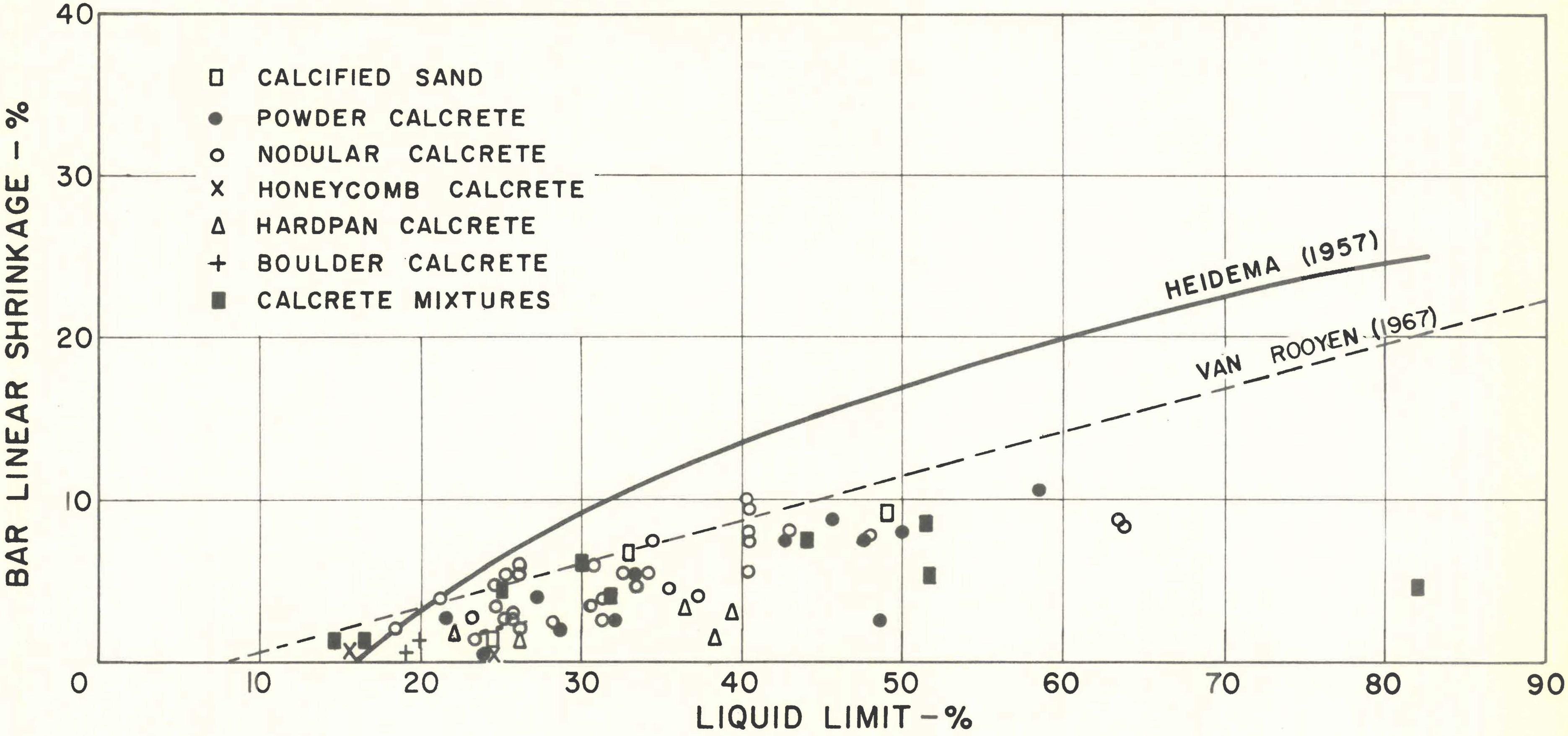
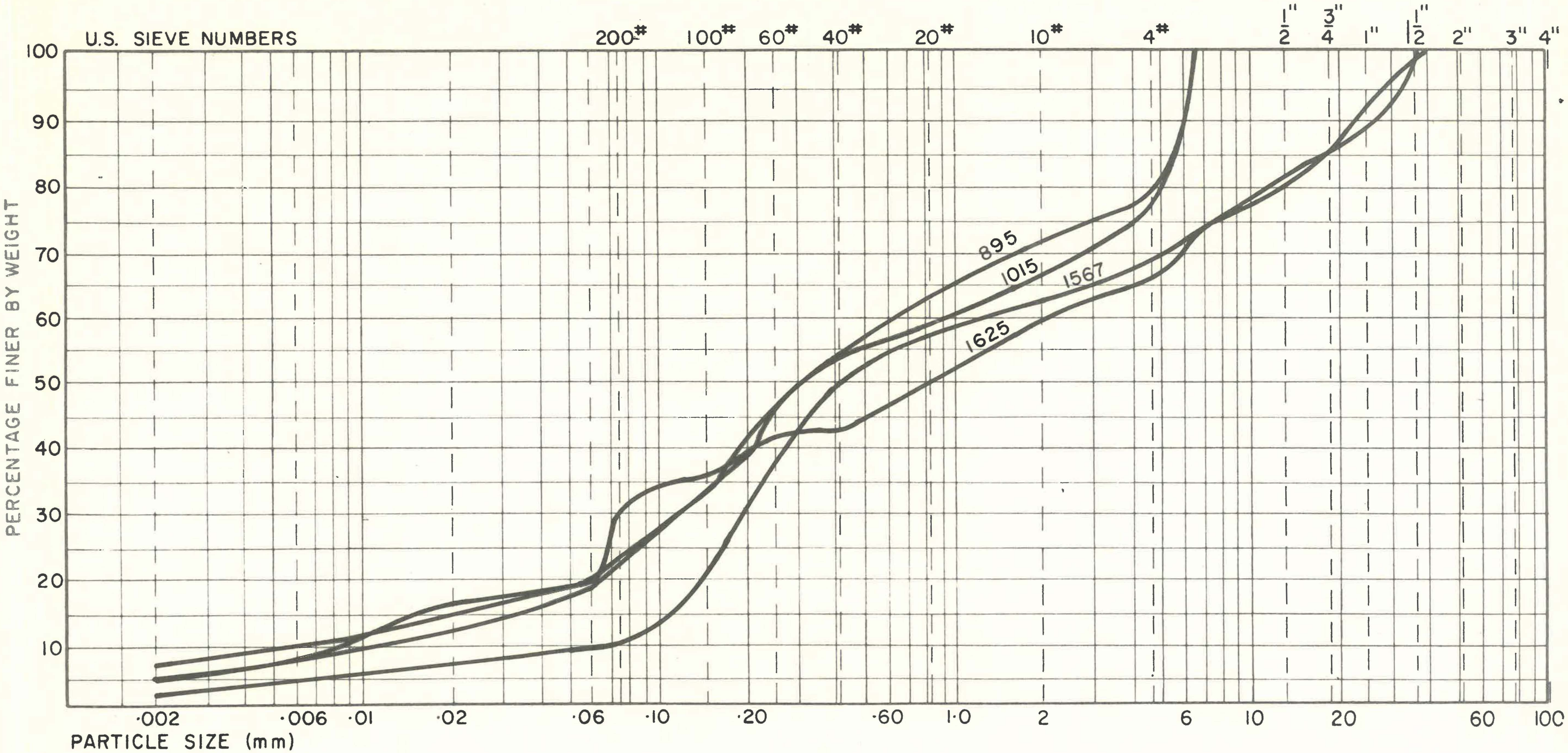


FIGURE 7

Relation between bar linear shrinkage and liquid limit

RG4-3200

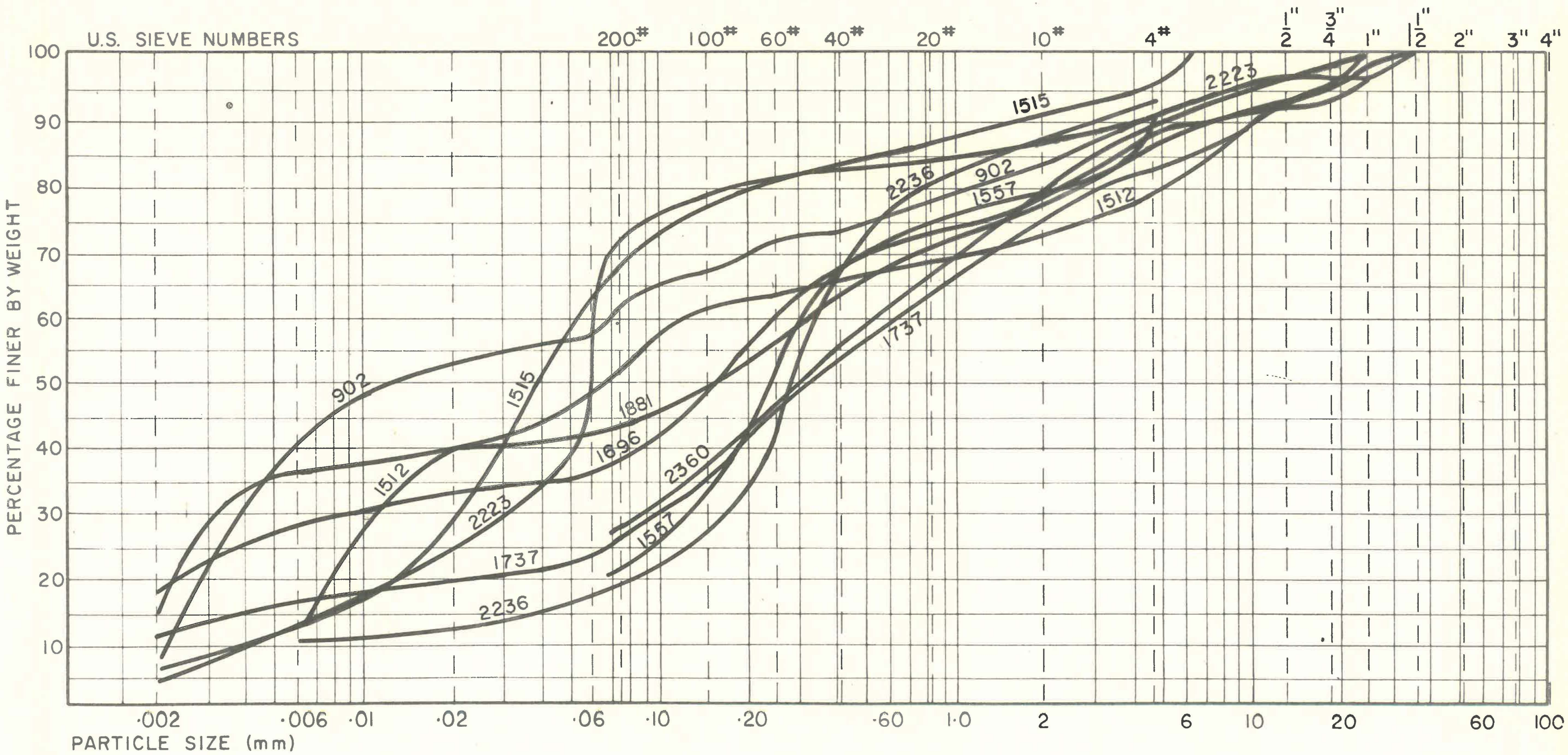


CLAY FRACTION	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE
	SILT FRACTION			SAND FRACTION			GRAVEL FRACTION		

FIGURE 8.

Particle size distribution of some calcified sands

RG4-3202

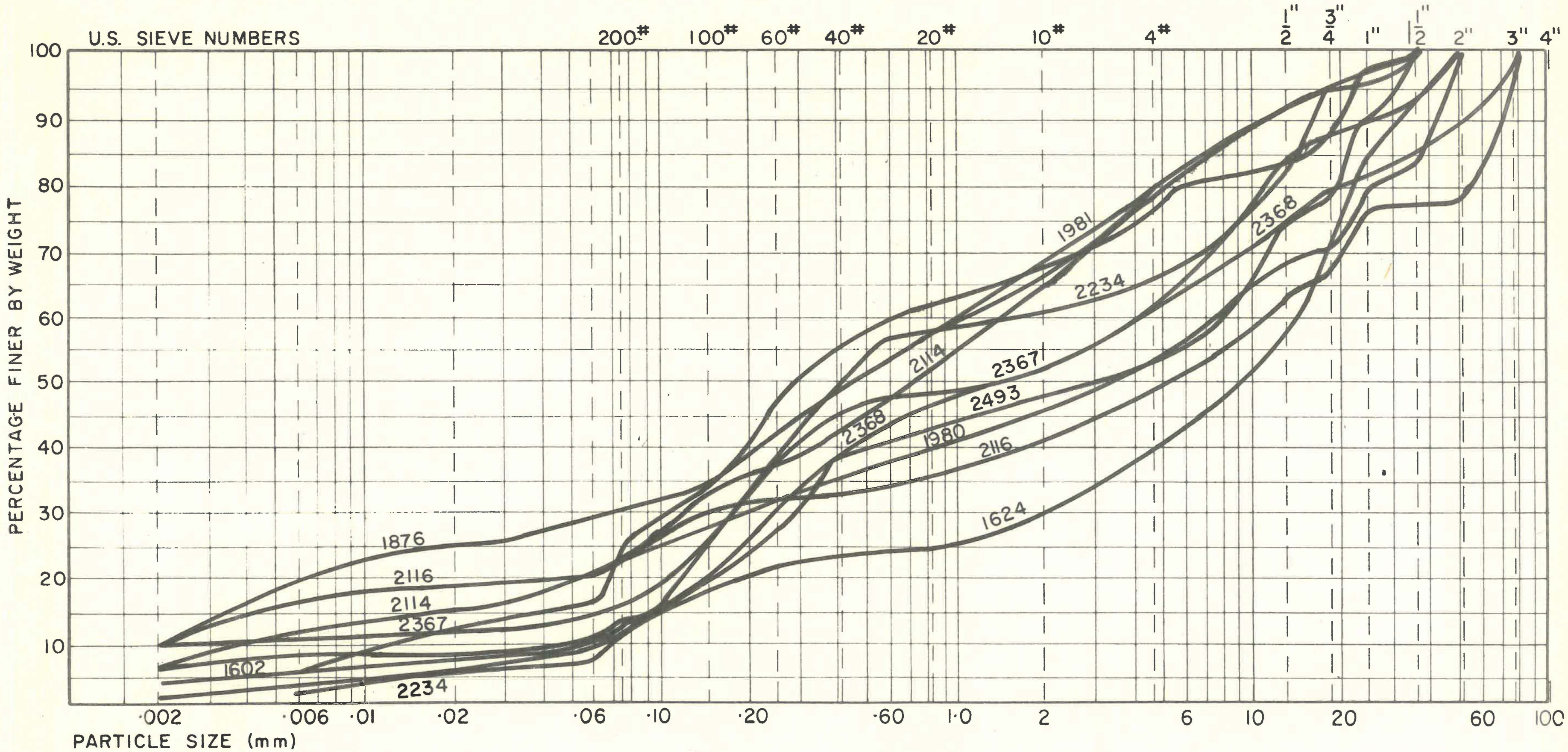


CLAY FRAC-TION	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE
	SILT FRACTION			SAND FRACTION			GRAVEL FRACTION		

FIGURE 9.

Particle size distribution of powder calcretes

RG4-3203



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

RG4-3205

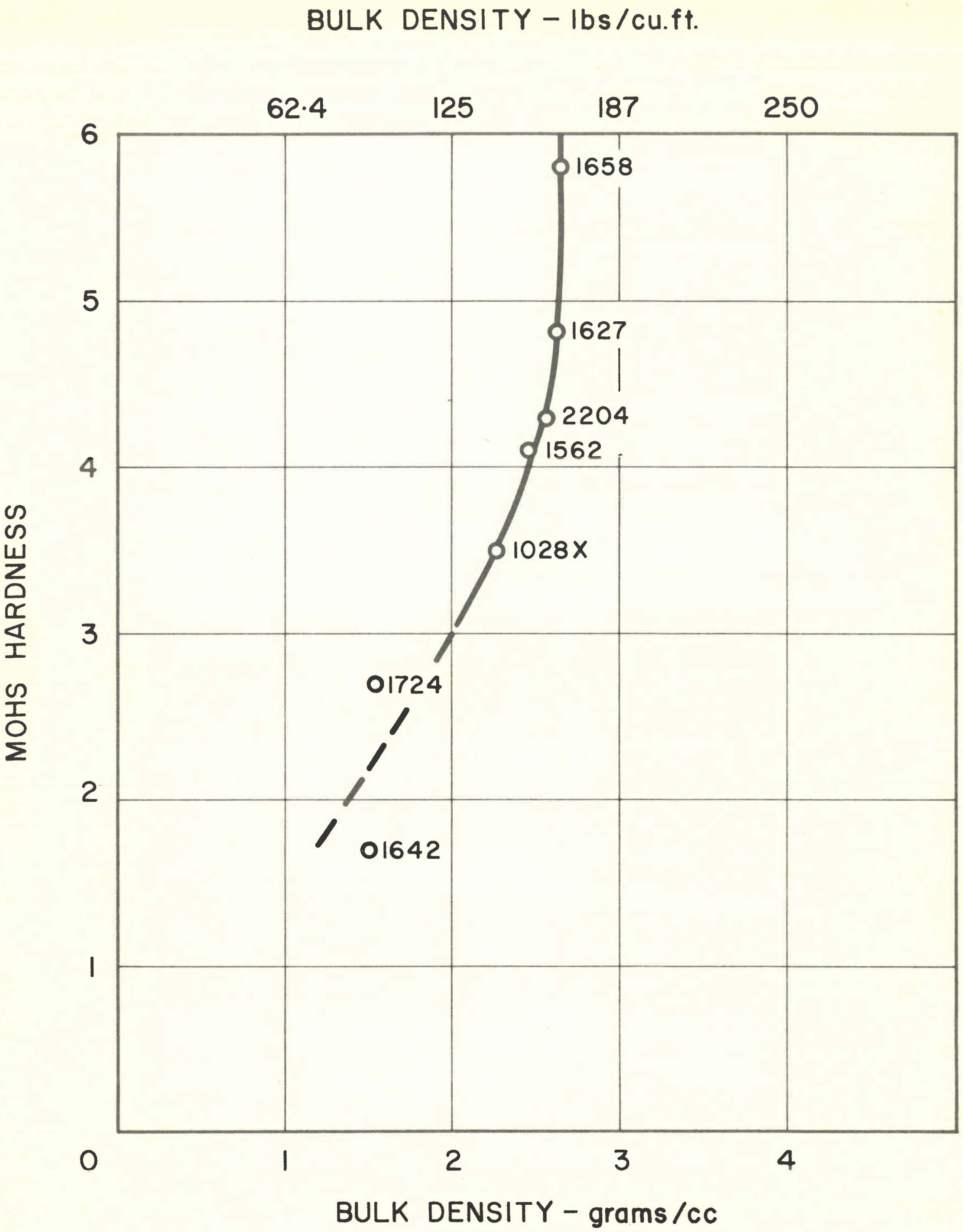


FIGURE 11.

Effect of bulk density on hardness of cryptocrystalline calcretes

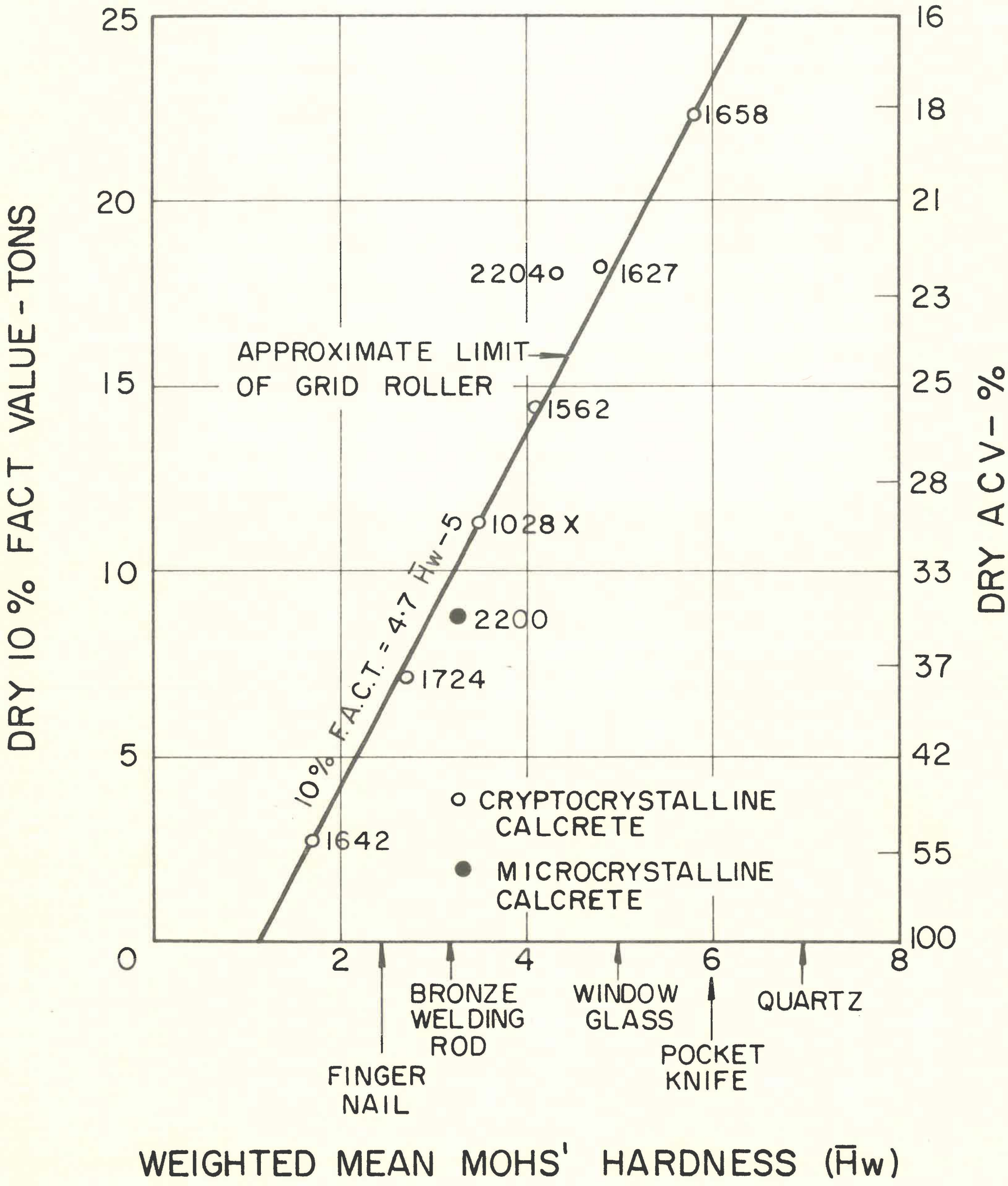


FIGURE 12.

Relation between aggregate crushing strength and Mohs' hardness.

OZALID-C. (S.A.) 64277 (6/67)

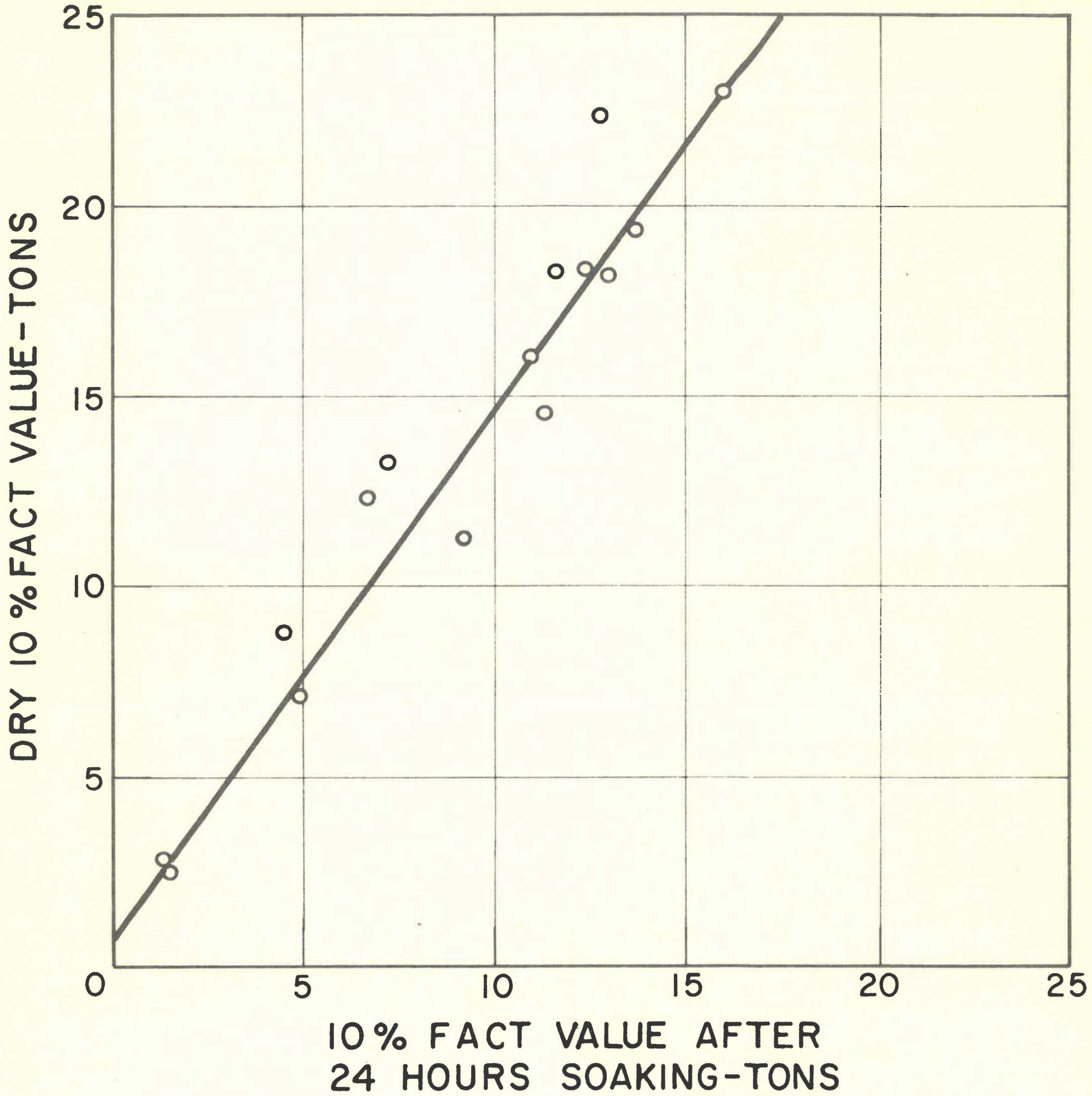


FIGURE 13.

Effect of soaking on crushing strength of crushed calcretes

OZALID 525

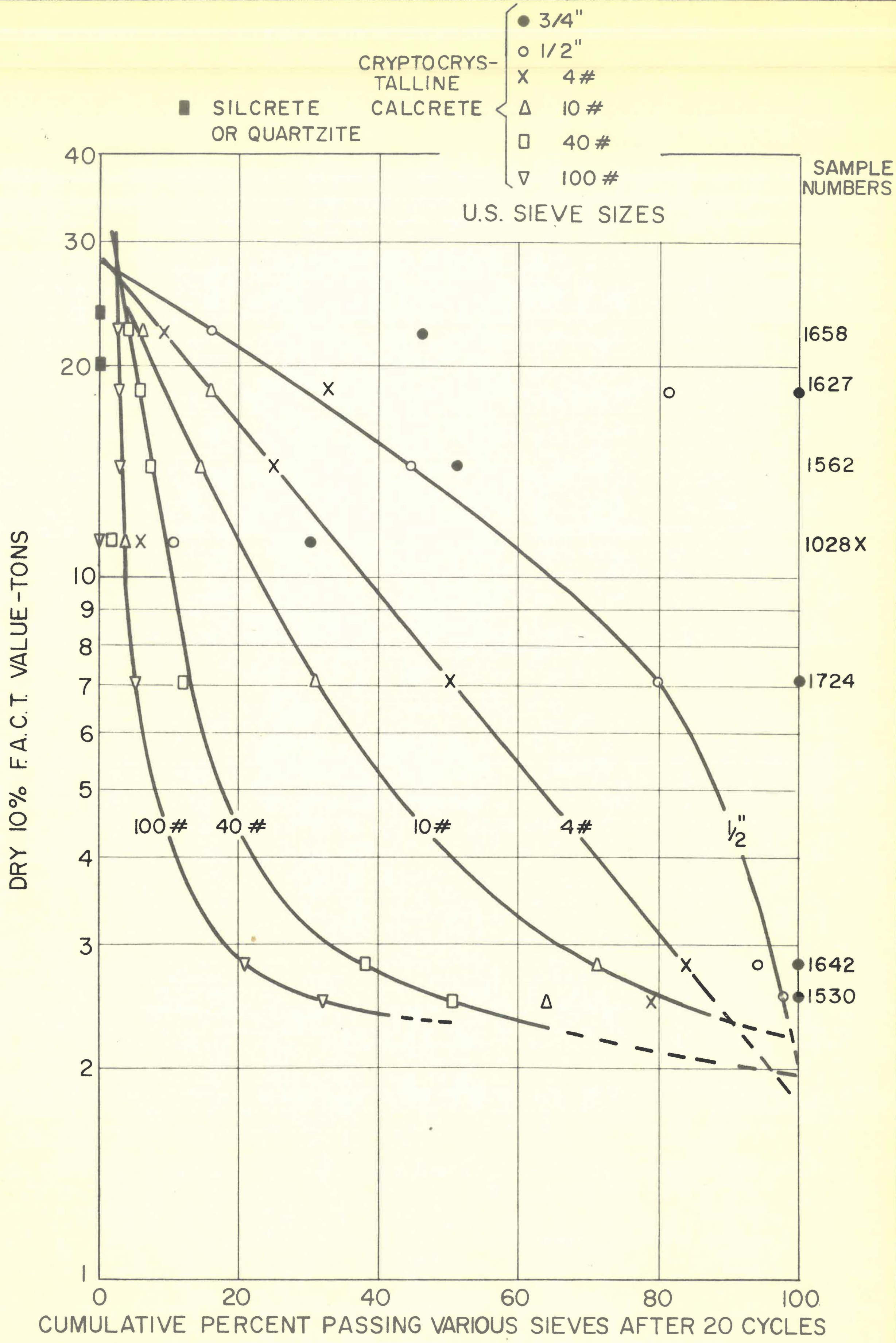


FIGURE 14.

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

RG-246
RG4-2122

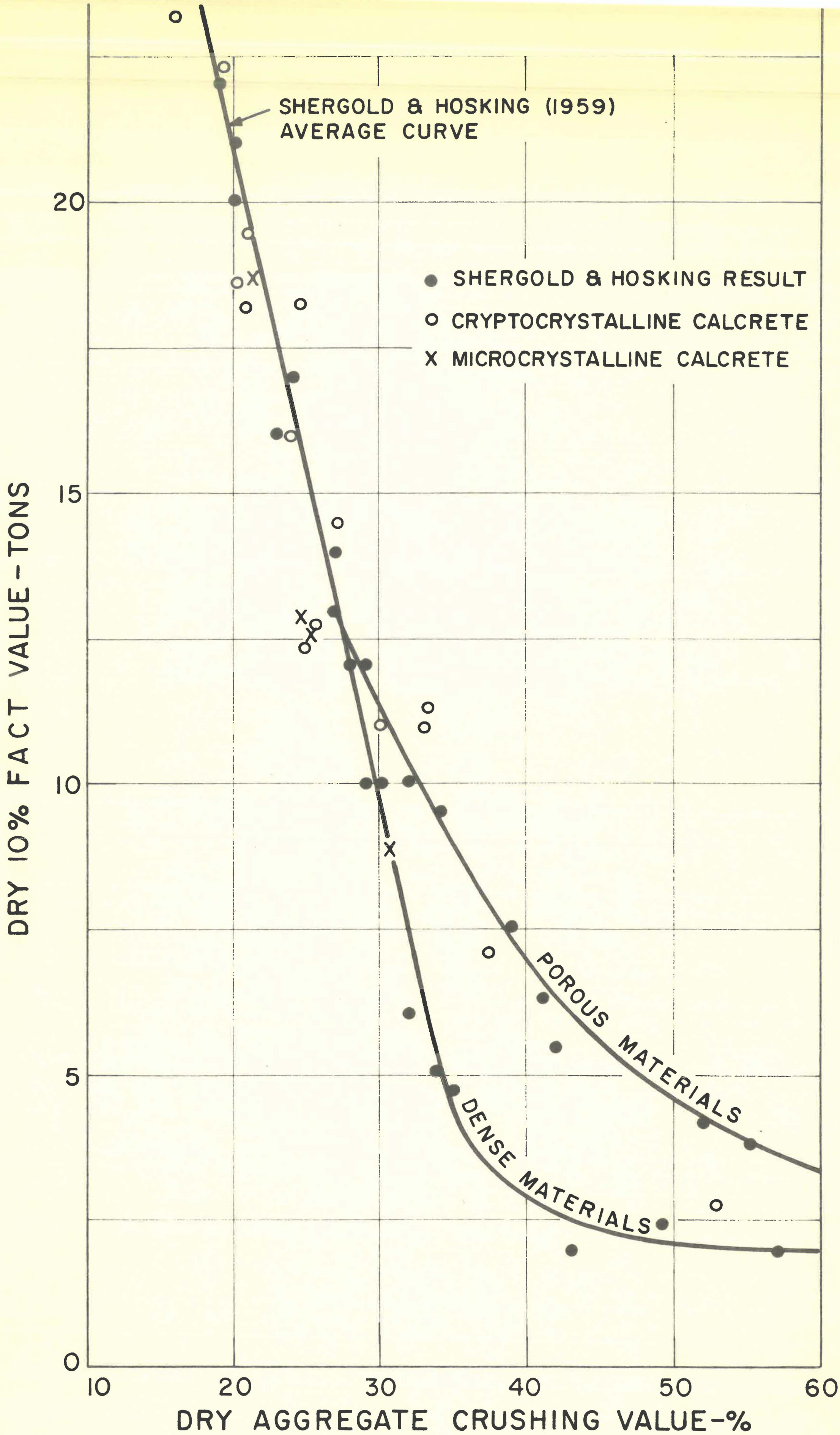


FIGURE 15.

Relation between the 10% FACT and the ACV for crushed calcretes and other materials

RG4-3209

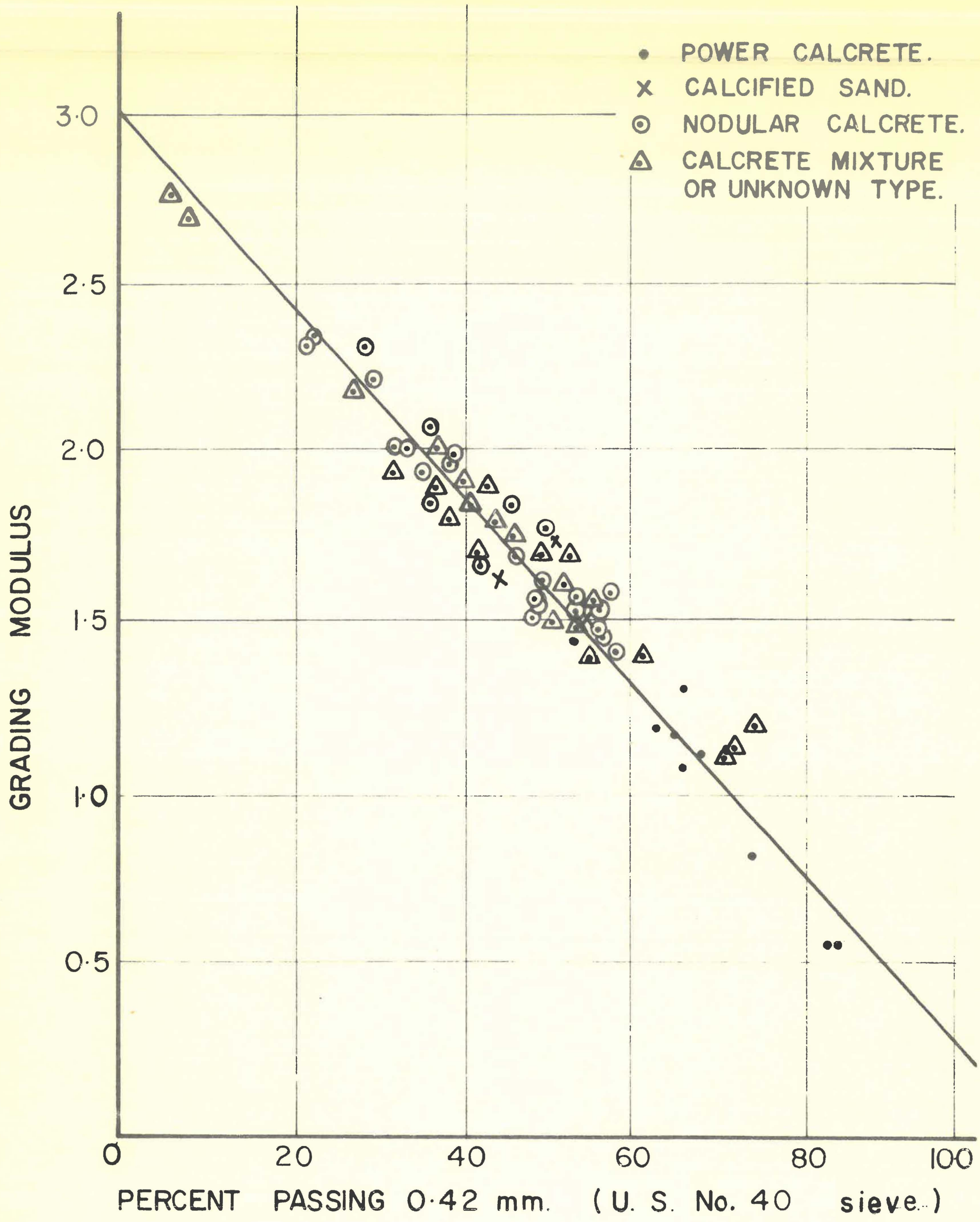
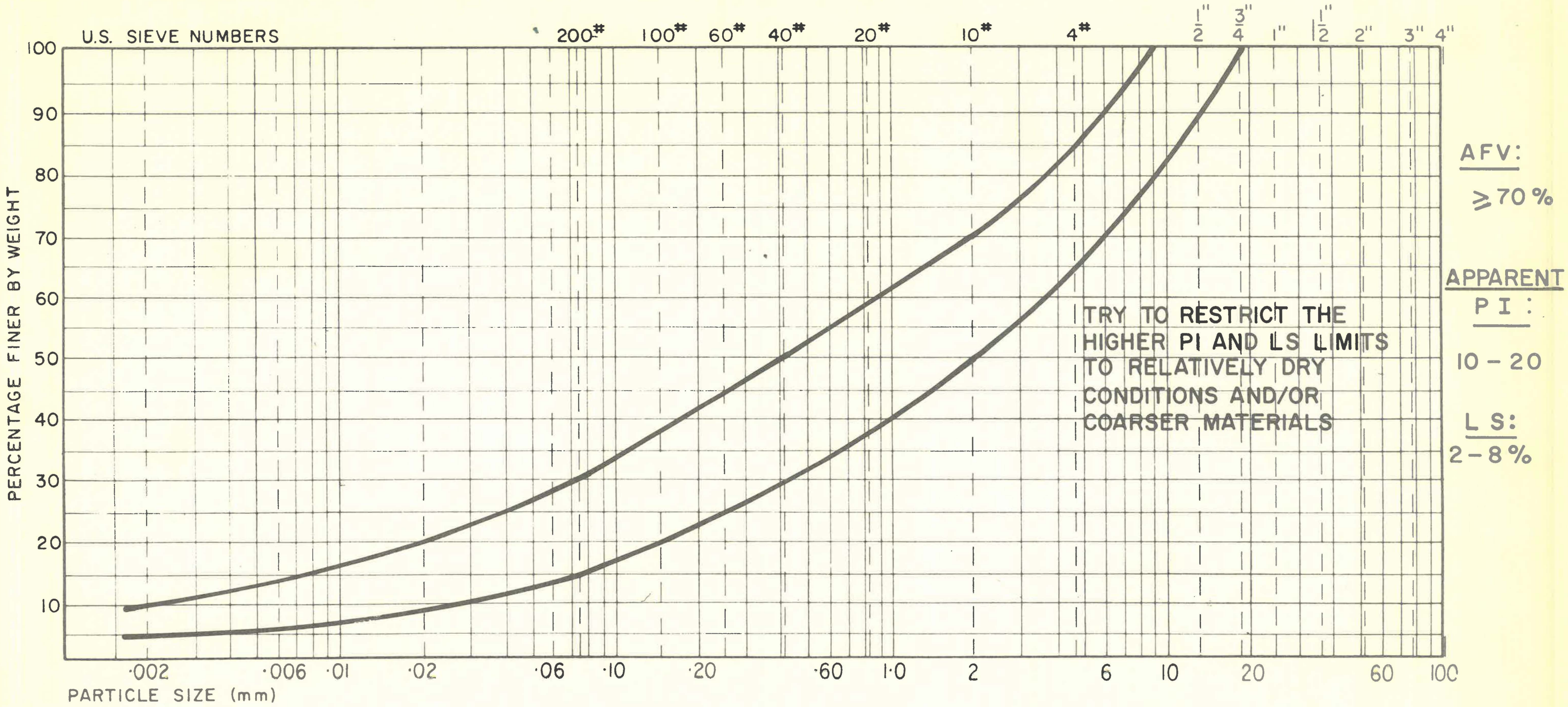


FIGURE 16.

RELATION BETWEEN GRADING MODULUS AND PERCENTAGE PASSING 0·42mm.

RG4 - 3659



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

FIGURE 17.

Suggested specifications for calcrete wearing courses

RG4 - 3415A