The Geology and Engineering Properties of South African Calcretes

by F. Netterberg

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THE GEOLOGY AND ENGINEERING PROPERTIES OF SOUTH AFRICAN CALCRETES

(IN FOUR VOLUMES)

VOLUME III

ENGINEERING PROPERTIES AND SPECIFICATIONS FOR ROADS

THE GEOLOGY AND ENGINEERING PROPERTIES OF SOUTH AFRICAN CALCRETES

BY

F. NETTERBERG

VOLUME I: SUMMARY, CLASSIFICATION, DISTRIBUTION, COMPOSITION AND ORIGIN

VOLUME II: AGE, PROSPECTING METHODS AND ENGINEERING TEST METHODS

VOLUME III: ENGINEERING PROPERTIES AND SPECIFICATIONS FOR ROADS

VOLUME IV: APPENDICES AND MAPS

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8.1 INTRODUCTION

This chapter attempts to define the engineering (mainly roadmaking) properties of the varieties of calcrete listed and described in Chapter 1, Section 1.3.3. A summary of some of the engineering properties of the main calcrete types has been given in Chapter 1, Table 1.4. This chapter and Table 1.4 is based on test results on about 100 calcretes carried out at the NIRR and a much larger number of results and observations from road construction jobs in South and South West Africa. Photographs of typical calcretes and descriptions of typical soil profiles in which they occur will also be found in Section 1.3.3. These alone will convey a good first approximation of the engineering properties.

Most of the test methods used have already been described in detail in Chapter 7. A concise list will be found in Appendix I, together with a reference to the classification systems employed.

It should be noted that wherever the expression 'plasticity index' (PI) is used in this work without qualification, the ordinary PI determined in the usual way (LL-PL), also sometimes called herein the 'apparent PI', is intended. The true PI will be specifically referred to as such.

8.2 LITERATURE SURVEY

Gillette in 1934 seems to have been the first to publish any work on the roadmaking properties of calcretes. He discussed the types of caliche found in the United States and gave details of a number of different test methods which had been tried on caliches of known performance. Among other things he concluded that hard caliches requiring blasting or ripping gave the best service, that the quality and quantity of the fraction passing the no. 40 sieve

controlled/ ...

controlled the stability of a caliche base course, and that the quality of this binder depended on the geologic origin of the material. By this statement he apparently realised that the PI of a calcrete is largely determined by the nature of the original soil cemented by the calcium carbonate. Gillette also concluded that the plasticity index was the best indicator of the potential performance of a caliche base course and that depending on climate and drainage maximum values of 10-15 were suitable.

In 1929 Campbell briefly discussed the use of caliches in New Mexico, mentioning the importance of powdery caliches as binders and that "it reacts with water and sets up very much like lime...". Graham (1944) stated that the use of caliche for base course was a highly controversial subject and that from the few opinions obtained it appeared as though the material was not suitable for bases of roads carrying fast and heavy traffic.

In 1955 Day described the construction of a road in Texas using a caliche base course.

Uppal and Singh (1958), in a laboratory study of soft aggregates in the state of Rajasthan in India, were able to show that many of the materials tested (which included some nodular calcretes) should be perfectly satisfactory as base course if properly treated.

Woods (1960) has stated that "there is a considerable lack of knowledge relative to the influence of chemical characteristics on the properties of soil which are of greatest concern to the engineer. Those which at present appear to be of most interest are organic matter content, carbonates of calcium and calcium-magnesium sulfates, and alkalinity or acidity".

A very brief review of the information available in 1962 was made by Beaven, quoting mostly from the above

sources/ ...

sources.

In 1967 Netterberg published a classification of calcretes suitable for general engineering and geological use and listed some roadmaking properties of each group.

In general, it can be said that apart from the limited study of Uppal and Singh and the limited results published by Netterberg, no proper survey of the engineering properties of calcretes appears to have been published. This study is an extension of and provides the background and details behind the work briefly reported on by Netterberg.

8.3 CALCAREOUS SOILS

Calcareous soils are not classed as calcretes, but in order to stress this point some comments will be made as to their properties.

A calcareous soil is merely any soil (in the engineering sense) weakly cemented or completely uncemented by the few per cent of carbonate it contains. A calcareous soil is described further in Section 1.3.3 and a photograph and profile description of a calcareous clayey sand profile will be found there.

Apart from the fact that calcium saturation lowers the liquid limit and plasticity index of most clays and that a pH in the alkaline range is generally more favourable than one in the acid range, calcareous soils differ little in their engineering properties from their non-calcareous parent materials. Some test results on some calcareous soils (solonetz B and C horizons) from Ovamboland are shown in Table H.4 in Appendix H. Their grading curves are shown in Figure 8.1. The materials are mostly clayey sands and their plasticity indices range from 23 (Sample 893) to about 140 (Sample 894). Their grading moduli are all less than 1.0, their CBRs are less

than/ ...



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

FIGURE 8·1

Particle size distribution of some calcareous soils

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than 10, their group indices range from 2 to 13 and they would classify as A-2-7 to A-7-6 materials according to the revised PRA classification. X-ray diffraction analysis of Sample 894 (Table E.1, Appendix E) revealed the presence of quartz, calcite and a large quantity of montmorillonite. In short, these materials are absolute rubbish and would not be tolerated in any surfaced road foundation except perhaps (893 only) at the bottom of a high fill. They are equally unsuitable as sand-clay wearing courses according to the D.O.T. (1958) specification, though they have been used as such for want of anything better having been found at the time. As such materials have in the past been classed as calcretes by some road workers in South West Africa, it is not surprising that calcretes as a whole have received a rather worse name than they deserve.

8.4 SOIL CONSTANTS

It has already been shown in Section 7.5 that the plasticity index is likely to be an unreliable guide to the true plasticity of calcretes. Just how unreliable is as yet unknown, but it would appear that, depending on the methods employed both in the preparation of the soil fines and in the liquid limit procedure itself, results which are too high or too low might be obtained.

Properties of palygorskite and sepiolite

It has been shown in Chapter 3 that the most common clays in calcretes are probably palygorskite, montmorillonite, sepiolite and illite. The properties of montmorillonite and illite are well known and need no discussion here. The properties of palygorskite and sepiolite are relatively little know, however, and some of the unusual properties of calcretes are explained by an understanding of the properties of these two clay minerals. Palygorskite is here considered to be the same as attapulgite. The two are in fact structurally the same

mineral/ ...

mineral, though attapulgite is sometimes considered to be short fibre palygorskite. The following review was compiled mainly from Hough (1957), Grim (1962, 1968) and Haden and Schwint (1967). Little seems to be known of the engineering properties of sepiolite, but they are probably not greatly dissimilar to those of palygorskite, and much of the following probably also applies to sepiolite.

The most fundamental way in which palygorskite and sepiolite differ from the other common clays is crystal structure. Palygorskite and sepiolite possess a three dimensional chain structure as against the sheet structure of the others. The actual shape of the particles is needle-like, as against the flakes of the others. These two basic properties are the basic reasons for palygorskite and sepiolite differing in many of their properties from those of the other clays. Table 8.1 and other data mainly in Grim (1962) shows that palygorskite possesses a liquid limit, plasticity index and activity about equal to that of calcium montmorillonite. The two linear shrinkage results are moderately high. Though similar in these respects to montmorillonite, its chain structure makes it non-expansive. The shrinkage limit results are contradictory, but in view of the fact that it has the highest BET surface area and adsorptivity of all the clays it would not be surprising if it also possessed the highest shrinkage limit. It also possesses the highest plastic limit of all the crystalline clays. Only undried allophane has a higher plastic limit. Palygorskite has the highest optimum moisture content and lowest maximum dry density of all the common crystalline clays, but its shear strength at the same moisture content is the highest of all the common clays. For example, at a strength of one ton/sq ft the other clays are at a moisture content of 25-40% while palygorskite is at a moisture content of 110%. Its compression index is about the same as that of illite. Palygorskite loses its selective adsorption properties when

heated/ ...

heated above 88°C and decreases in surface area when heated above 95°C. Palygorskite suspensions are highly viscous and thixotropic and the viscosity and surface area are increased by extrusion. Palygorskite and sepiolite suspensions are not flocculated by electrolytes such as NaCl, but palygorskite (and probably also sepiolite) is dispersed by dispersants like Na_2CO_3.

It is apparent from the foregoing that palygorskite and sepiolite possess some unusual properties and it can be expected that they will be reflected in the properties of the soils containing them.

8.4.1 Liquid limit

Liquid limits of the calcretes tested will be found in Appendix H. The values range from 15 to 82 and will be further commented on when the properties of each variety of calcrete are dealt with.

The question has often been raised as to whether the carbonate fraction of calcretes is beneficial. Consideration of the nature of the mineral shows that depending on the properties considered, it may be more or less beneficial than quartz, but far more beneficial than any clay mineral. Regarding the effect of calcium carbonate on the Atterberg limits, the problem can be approached as follows: Consider a non-calcareous parent material. The clay minerals present are likely to be saturated with a cation other than calcium. Table 8.1 shows the effect of the most common cations on the Atterberg limits of some fairly pure clay minerals. It can be seen that the cation has little effect on most clay minerals but has a most marked effect on the properties of montmorillonite. Replacement of Na^+ by Ca^{++} for example, as might be expected to occur during calcification and solonetz formation from a solonchak parent material, would result in a substantial lowering of both the Atterberg limits and

the/ ...

TABLE 8.1 SOIL CONSTANTS OF SOME FAIRLY PURE CLAYS

CLAY	TYPE	LL %	PL %	PI %	SL %	LS %	ACTIVITY	NO. OF SAMPLES	SOURCE OF DATA
MONTMORILLONITE	Ca	123 - 177	63 - 79	44 - 114	-	-	0.44 - 1.34	4	White (1955), in Grim (1962)
	Mg	138 - 199	51 - 73	65 - 148	-	-	0.65 - 1.97	4	" " " " "
	Na	280 - 700	86 - 97	144 - 603	-	-	1.12 - 7.09	4	" " " " "
	Ca	510	80	430	11	-	-	1	Lambe, in Scott (1963)
	Ca??	516	168	368	-	54	-	1	Fruhauf (1946)
	Ca	187	53	134	12	20	-	1	Croft (1964)
	Ca?	170 - 227	114 - 119	56 - 108	-	18 - 29	-	3	NIRR samples 422 - 424 (1)
	Mg	410	60	350	15	-	-	1	Lambe, in Scott (1963)
	Na	710	54	650	10	-	-	1	Lambe, in Scott (1963)
	Na?	685 - 730	51 - 56	628 - 674	-	35 - 36	-	3	NIRR samples 405 - 407 (1)
	H	440	55	380	9	-	-	1	Lambe, in Scott (1963)
	Fe	290	70	220	10	-	-	1	Hough (1957)
ATTAPULGITE (PALYGORSKITE)	Ca	232	124	108	-	-	1.08	1	White (1955) in Grim (1962)
	Mg	179	109	70	-	-	0.70	1	" " " " "
	Na	212	100	112	-	-	1.12	1	" " " " "
	?	178	117	61	-	-	-	1	White (1949)
	H	270	140	130	8	-	-	1	Hough (1957)
	Ca?	227	94	133	52, 45	27	-	1	Sample 2589 (2)
	Ca?	193	113	80	-	20	-	1	W.H. Luwes Soil Lab.
ILLITE	Ca	69 - 100	36 - 42	33 - 58	-	-	0.33 - 0.58	3	White (1955), in Grim (1962)
	Mg	71 - 98	35 - 43	36 - 55	-	-	0.36 - 0.55	3	" " " " "
	Na	59 - 75	34 - 41	25 - 34	-	-	0.25 - 0.34	3	" " " " "
	Ca	100	45	55	17	-	-	1	Lambe, in Scott (1963)
	Ca	63	28	35	20	10	-	1	Croft (1964)
	Ca?	124 - 133	48 - 49	74 - 79	-	22 - 23	-	3	NIRR samples 445 - 447 (1)
	Mg	95	46	49	15	-	-	1	Lambe, in Scott (1963)
	H	100	50	50	17	-	-	1	" " " "
	Fe	110	51	59	15	-	-	1	Hough (1957)
KAOLINITE	Ca	34 - 73	26 - 36	8 - 37	-	-	0.08 - 0.37	2	White (1955), in Grim (1962)
	Mg	39 - 60	28 - 30	11 - 30	-	-	0.11 - 0.30	2	" " " " "
	Na	29 - 52	26 - 28	1 - 26	-	-	0.01 - 0.26	2	" " " " "
	Ca	38	27	11	25	-	-	1	Lambe, in Scott (1963)
	?	59 - 64	30 - 33	29 - 31	20	8 - 12	-	2	Croft (1964)
	?	39	26	13	-	7	-	1	NIRR sample 955 (1)
	Mg	54	31	23	29	-	-	1	Lambe, in Scott (1963)
	H	53	25	28	-	-	-	1	" " " "
	Fe	59	37	22	29	-	-	1	Hough (1957)

(1) Determined by Routine Tests Section, NIRR.

(2) Determined by Mr. T.E. O'Connor; SI by volumetric shrinkage and by particle specific gravity methods respectively.

the plasticity index. Only one or two per cent of calcium carbonate in a soil is required to ensure that the clay minerals are calcium saturated. Figure 8.2 shows the effect of larger amounts of calcium carbonate. This effect was studied by mixing AR quality $CaCO_3$ in various proportions with Onderstepoort black clay, a material notorious for its high plasticity and shrinkage. Partial mechanical analyses of both materials are shown in Table 7.2. X-ray diffraction analysis showed the $CaCO_3$ to be pure calcite and the black clay to be composed of quartz and montmorillonite. It can be seen that the carbonate acts as a mechanical stabilizer, reducing the liquid limit linearly with increasing carbonate content. In a 50:50 mixture the liquid limit was reduced from 70 to 45. Seed et al's (1964) work shows that in mixtures of clay and non-clay the line is not linear at low clay contents, but behaves as the dotted line in Figure 8.2. The point at which it curves away from the straight line (essentially the liquid limit of the non-clay fraction) depends on the void ratio of the non-clay fraction, increasing with increasing void ratio, as might be expected. Figure 8.3 shows an analogous trend in reverse. The black clay used would have been already calcium saturated, so that no further decrease of liquid limit could have been possible by ion exchange.

Figure 8.3 shows the effect of adding similar calcium carbonate to a washed quartz sand of Kalahari type (Sample 1965), a mechanical analysis of which is shown in Table H.4. It can be seen that the liquid limit is linearly proportional to the $CaCO_3$ content over much of the range. These mixtures were all found to be non-plastic and to have a linear shrinkage of zero (0.0). The liquid limit of the pure $CaCO_3$ was found to be 27.

8.4.2 Shrinkage limit, shrinkage index and shrinkage ratio

The/ ...

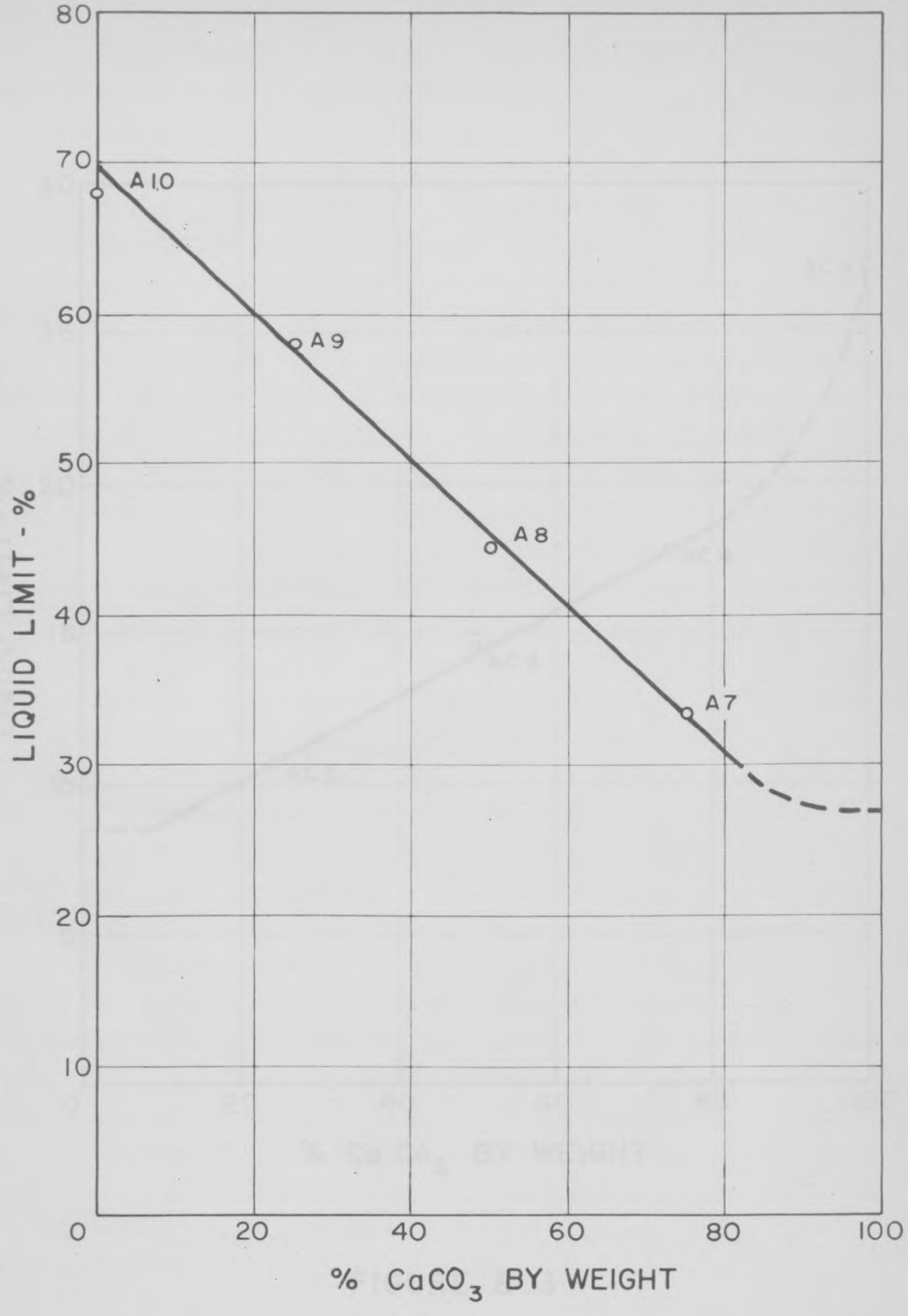


FIGURE 8.2

EFFECT OF CALCIUM CARBONATE ON THE LIQUID LIMIT OF A BLACK CLAY

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The shrinkage limit is deliberately dealt with next in order to emphasise the unusual fact that calcrete shrinkage limits are often larger than their plastic limits. The shrinkage limit is defined by Woods (1960, Section 8.20) as follows: "As the moisture content of a soil-water mixture is reduced, its volume also reduces proportionately with the reduction in moisture content. Finally, at moisture contents below the plastic limit, the mixture passes through a semisolid state and reaches a minimum volume. The moisture content at which no further reduction in volume occurs is called the shrinkage limit. At lower moisture contents air enters the voids to replace the water and the volume remains constant." The shrinkage index is equal to the plastic limit minus the shrinkage limit. It is apparent that Woods considers that the shrinkage limit is always less than the plastic limit (i.e. the shrinkage index is always positive), a statement also made by Casagrande (1932), though the latter author mentioned that the undisturbed shrinkage limit may be higher than the plastic limit. The author has not noticed any mention in any of the standard works on soil mechanics that the disturbed shrinkage limit may be larger than the plastic limit. While a thorough search has not been made, examination of all test results encountered in the literature confirmed this impression that the shrinkage limit is always lower than the plastic limit. The shrinkage limit of British soils is relatively constant at about 12-14% (Road Research Laboratory, 1952). Shrinkage limit results are hard to come by nowadays, but examination of the 30 results on United States caliches given by Gillette (1934) shows that the shrinkage limit was from 0 to 14 units higher than the plastic limit. In 28 cases out of 30 it was higher by 1 or more units and was most commonly 4 or 5 units higher. The difference seemed to be independent of plasticity index. Shrinkage limits ranged from 15 to 40.

As/ ...

As a check on South African soils, an examination was made of the Atterberg limits of 362 soils (varying from clays to sands and, rarely, fine gravels), shown on a collection of soil profiles from the Orange Free State and the Transvaal kindly made available by Mr. A.B.A. Brink (described further in Section 2.3.2). Of these, only nine samples were found to possess higher shrinkage limits than plastic limits (by 0.8 to 10.2). Of these, six were stated to be calcareous and/or to possess nodular lime. Of the other three, one was logged as gravel, another as clay plus iron concretions and the third as silt plus clay plus pebbles. Shrinkage limits of the 362 soils varied from 4 to 22%, though 6-15% was the more usual range.

The results of about 37 shrinkage limit determinations on calcretes are shown in Appendix H. The values range from 14 to 44. Some three quarters of these are over 20 and half are greater than the plastic limit. These shrinkage limits were determined by the ASTM D427-61 optional method employing the particle specific gravity (G_s). Some were carried out without the presoaking usual in the NIRR Atterberg limit method. As a check that the method was not at fault, some samples were tested after presoaking and by both the particle specific gravity and the volumetric shrinkage method. Appendix H shows that presoaked shrinkage limits are still often larger than the corresponding presoaked plastic limits. A comparison of shrinkage limits obtained by the specific gravity and volumetric shrinkage methods is shown in Table J.5 in Appendix J. Neither method appears to yield consistently higher results than the other. It appears therefore that a negative shrinkage index is a genuine property of some calcretes. Table 8.2 suggests that this property is due to the presence of amorphous silica. This is known to occur in some calcretes, mainly in the form of microfossil skeletons - chiefly diatoms (Chapter 3). Diatoms (a class

of/ ...

of the algae) possess hollow shells of amorphous silica with openings to the outside (see for example Plates 5.1 and 5.2). These shells can contain water and a mass of shells (diatomaceons earth) is known to be non-plastic, but to possess a high liquid limit (over 100). Diatoms can therefore be expected to raise the Atterberg limits of a soil, but not to affect the PI. As they possess a low specific gravity, the diatoms also lower the shrinkage ratio, though this does not seem to be indicative. The only anomalous sample in Table 8.2 is 2223. The shrinkage index in Table H.1 - 3.1 (SG method) is more reasonable and places this sample between Samples 1980 and 895,

TABLE 8.2 EFFECT OF AMORPHOUS SILICA CONTENT ON THE SHRINKAGE INDEX

SAMPLE NUMBER	SHRINKAGE INDEX (PL-SL) %		- 2 mm AMORPHOUS SILICA %	- 0.42 mm MICROFOSSILS PER GRAM	SHRINKAGE RATIO
	SG SL METHOD	VS SL METHOD			
2360	4.0	3.2	0.34	-	1.50
2114	2.2	9.4	0.14	1,200	1.68
1624	1.6	-	0.17	1,800	1.75
2116	1.0	4.6	0.09	900	1.95
1980	-1.7	-1.9	0.36	-	1.75
895	-4.2	-3.9	2.10	-	1.68
1602	-5.4	-	2.27	-	1.63
1561	-5.7	-5.5	3.13	-	1.60
2223	-7.6	-5.7	0.45	-	1.74
1601	-9.0	-	2.27	42,000	1.46

correlating well with the amorphous silica content. The effect of up to 10% added minus 200 U.S. mesh diatoms is further investigated in Table J.9. Though much less marked than might be expected, it is apparent that they do

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tend to increase the LL, PL and SL and to decrease the SI. The PI and linear shrinkage are hardly affected. The effect is surprisingly small in view of the high shrinkage limit (over 100%) of the diatoms used; less than the theoretical weighted mean value for the mixture.

The shrinkage indices using the SG shrinkage limit method of all the calcretes tested ranged between -9 and +11. Shrinkage ratios ranged between 1.36 and 1.95. Gillette's caliches ranged between 1.3 and 1.8. While the shrinkage limits of calcretes are often unusually high and the shrinkage indices often unusually low, it would seem that the shrinkage ratios are not unusual, though rather low values are sometimes obtained.

Examination of the X-ray diffraction results in Appendix E does not reveal any further reasons why calcretes should often yield negative shrinkage indices, though the samples which are particularly rich in palygorskite tend to be those in which the PL-SL relationship is normal, i.e. PL > SL. Table 8.1 shows palygorskite to possess extremely high plastic and shrinkage limits, the highest of all the crystalline clays in fact. The only other clay with similar Atterberg limits is the amorphous mineral allophane. Two undried specimens yielded liquid limits of 207 and 231, plastic limits of 131 and 136 and PIs of 76 and 96 (Grim,1962). Allophane has not so far been identified in calcretes, but the methods used were only suitable for the detection of crystalline, or at best, poorly crystalline, clay minerals. It may also be significant that the sample (1601) with the largest difference is relatively rich in montmorillonite and contains no palygorskite. Montmorillonite is common in many soils, so it does not seem possible that this is the cause, since to the author's knowledge remoulded materials possessing higher shrinkage limits than plastic limits have not been previously reported. It is strange that Gillette did not

remark/ ...

remark upon this phenomenon. It may also be significant that soils possessing 'structure' tend to have high undisturbed shrinkage limits and that the addition of cement or lime caused a marked increase in the shrinkage limits of the two calcretes tested.

It seems likely then that the presence of porous particles is the cause of the high shrinkage limits and the low or negative shrinkage indices possessed by some calcretes. Though this seems to be caused by amorphous silica (probably present chiefly as diatoms) this phenomenon need not necessarily be limited to calcretes containing amorphous silica. Calcretes are composed of many particles of variable size cemented together by calcium carbonate, a feature probably very conducive to the production of porous particles. While the effect of calcium carbonate on the shrinkage factors of mechanical mixtures of this material and clay or sand has not been investigated, even silt-sized AR quality $CaCO_3$ has been found to possess a shrinkage limit of 36, which is in excess of its liquid limit.

8.4.3 Plastic limit

The effect of the exchange of calcium for sodium during calcrete formation is to lower the plastic limit in the case of montmorillonite and to raise it in the case of palygorskite (Table 8.1). The plastic limit of illite and kaolinite is not markedly affected by the predominant cation. The effect of larger quantities of calcium carbonate on the plastic limit of the same samples used in the liquid limit experiment is shown in Figure 8.4. It is interesting to note that the relation is not entirely linear, but appears to reach a minimum at about 60% $CaCO_3$ and then rises again. This minimum represents the point at which the non-clay particles cease to float in the plastic clay matrix and also represents the point of

minimum/ ...

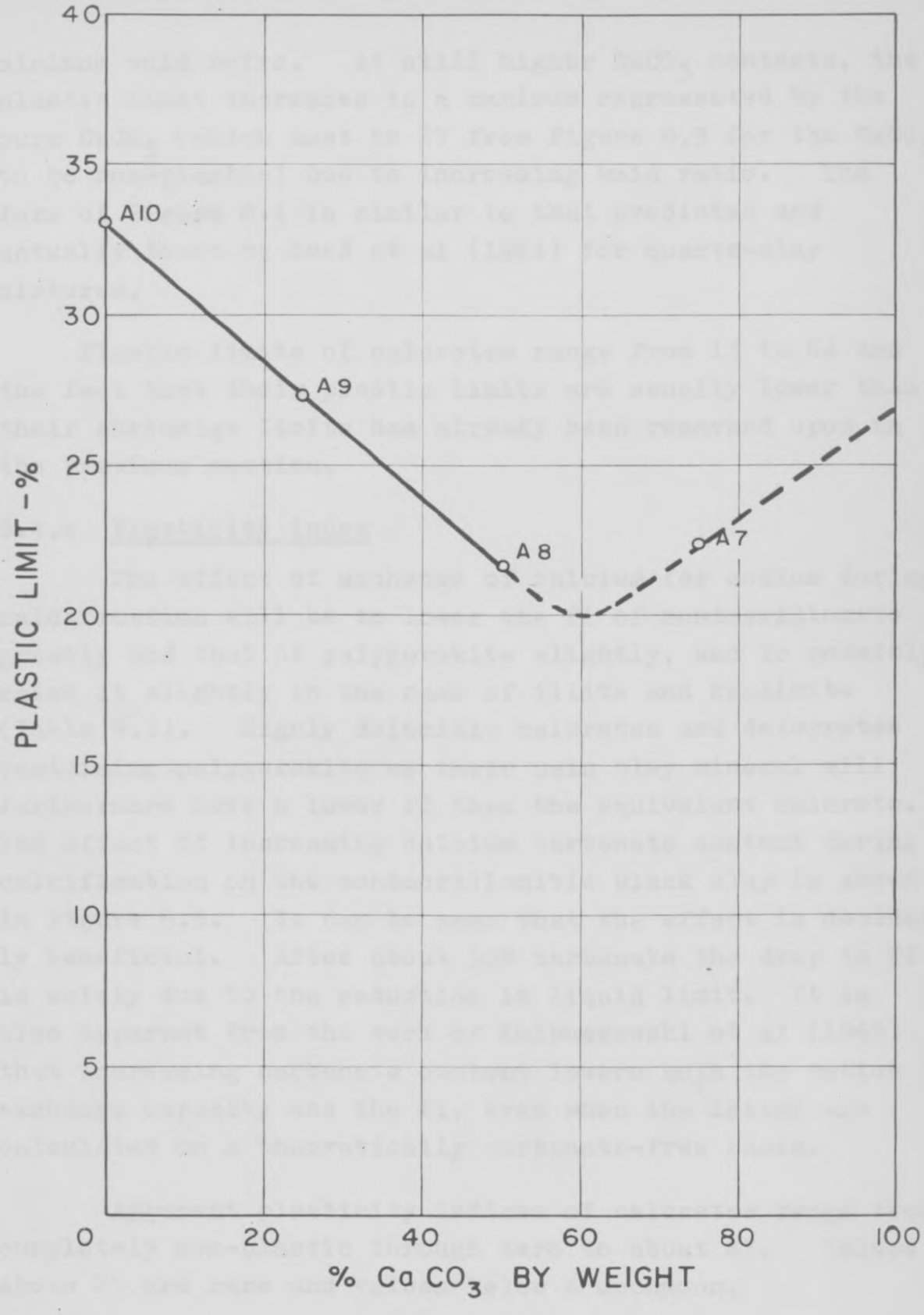


FIGURE 8·4

Effect of calcium carbonate on the plastic limit of a black clay

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minimum void ratio. At still higher $CaCO_3$ contents, the plastic limit increases to a maximum represented by the pure $CaCO_3$ (which must be 27 from Figure 8.3 for the $CaCO_3$ to be non-plastic) due to increasing void ratio. The form of Figure 8.4 is similar to that predicted and actually found by Seed et al (1964) for quartz-clay mixtures.

Plastic limits of calcretes range from 13 to 64 and the fact that their plastic limits are usually lower than their shrinkage limits has already been remarked upon in the previous section.

8.4.4 Plasticity index

The effect of exchange of calcium for sodium during calcification will be to lower the PI of montmorillonite greatly and that of palygorskite slightly, and to possibly raise it slightly in the case of illite and kaolinite (Table 8.1). Highly dolomitic calcretes and dolocretes containing palygorskite as their main clay mineral will furthermore have a lower PI than the equivalent calcrete. The effect of increasing calcium carbonate content during calcification on the montmorillonitic black clay is shown in Figure 8.5. It can be seen that the effect is decidedly beneficial. After about 50% carbonate the drop in PI is solely due to the reduction in liquid limit. It is also apparent from the work of Kolbuszewski et al (1965) that increasing carbonate content lowers both the cation exchange capacity and the PI, even when the latter are calculated on a theoretically carbonate-free basis.

Apparent plasticity indices of calcretes range from completely non-plastic through zero to about 47. Values above 25 are rare and values below 6 uncommon.

The fact that their shrinkage limits are usually higher than their plastic limits is highly significant with

respect/ ...

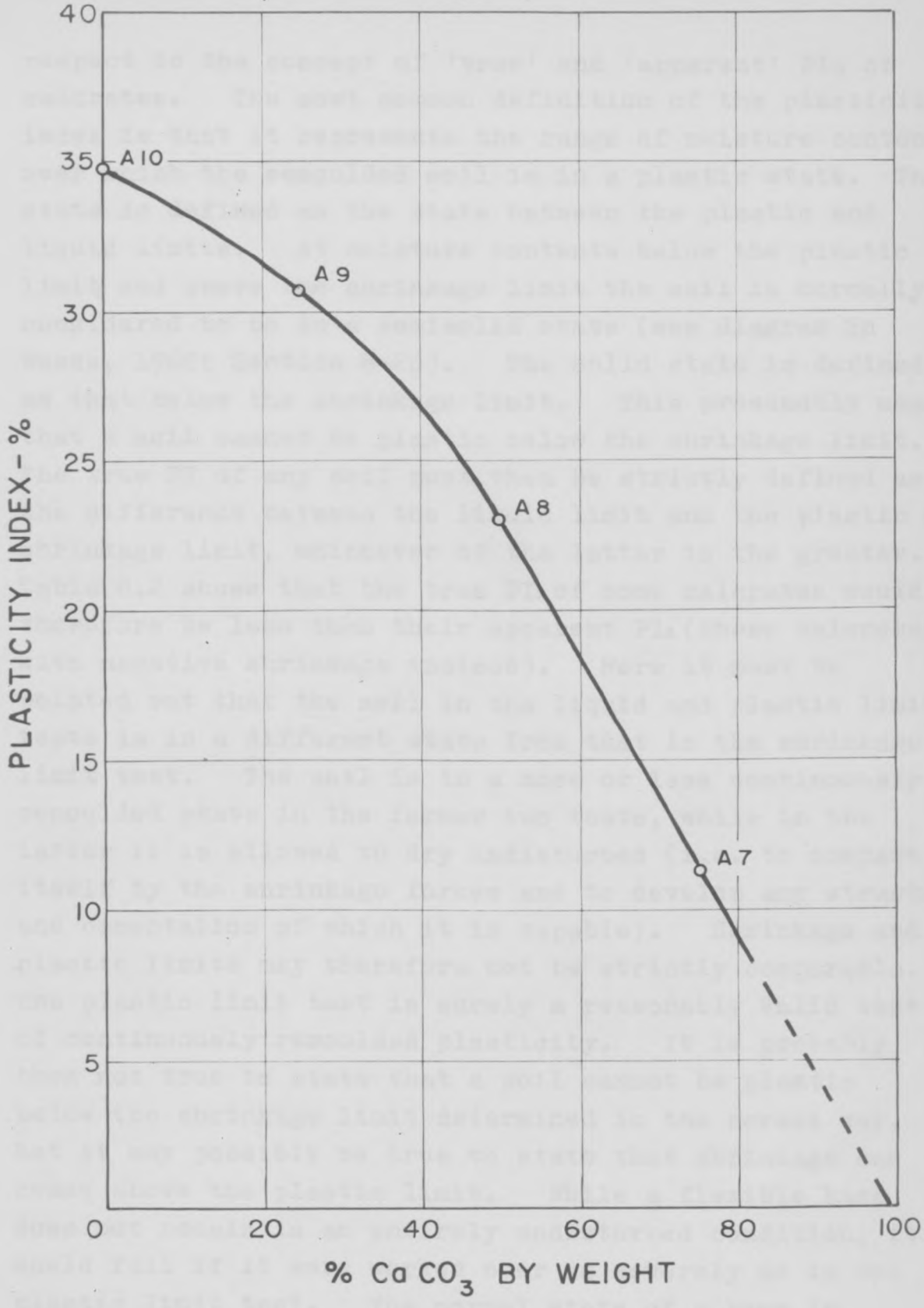


FIGURE 8·5

Reduction of plasticity index of a black clay by calcium carbonate

RG4-3197

respect to the concept of 'true' and 'apparent' PIs of calcretes. The most common definition of the plasticity index is that it represents the range of moisture content over which the remoulded soil is in a plastic state. This state is defined as the state between the plastic and liquid limits. At moisture contents below the plastic limit and above the shrinkage limit the soil is normally considered to be in a semisolid state (see diagram in Woods, 1960; Section 8-20). The solid state is defined as that below the shrinkage limit. This presumably means that a soil cannot be plastic below the shrinkage limit. The true PI of any soil must then be strictly defined as the difference between the liquid limit and the plastic or shrinkage limit, whichever of the latter is the greater. Table 8.2 shows that the true PIs of some calcretes would therefore be less than their apparent PIs (those calcretes with negative shrinkage indices). Here it must be pointed out that the soil in the liquid and plastic limit tests is in a different state from that in the shrinkage limit test. The soil is in a more or less continuously remoulded state in the former two tests, while in the latter it is allowed to dry undisturbed (i.e. to compact itself by the shrinkage forces and to develop any structure and cementation of which it is capable). Shrinkage and plastic limits may therefore not be strictly comparable. The plastic limit test is surely a reasonably valid test of continuously remoulded plasticity. It is probably then not true to state that a soil cannot be plastic below the shrinkage limit determined in the normal way, but it may possibly be true to state that shrinkage can cease above the plastic limit. While a flexible base does not remain in an entirely undisturbed condition, it would fail if it were worked over as severely as in the plastic limit test. The normal state of a base is probably somewhere between the shrinkage factor test and the plastic limit test, probably nearer the former than

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the latter in most bases in good condition in the semi-arid or arid environment. This means that shrinkage may well cease at about the shrinkage limit determined in the usual way (assuming for simplicity that no material coarser than 40 mesh is present), which would invalidate another definition of a plasticity index (as being a measure of the volume change between the liquid and plastic limits) in cases where the shrinkage limit is greater than the plastic limit. It would be interesting to attempt to measure a continuously remoulded shrinkage limit.

Apart from the possible unreliability of the test itself (Chapter 7), some doubt is therefore cast upon the validity of the plasticity index as a road performance indicator in cases where the shrinkage limit exceeds the plastic limit. It will be shown in Chapter 9 that it has been found by experience that higher than normal plasticity indices are allowable for calcrete roads.

8.4.5 Flow index

The flow index (F, or I_f) is a measure of the rate at which the shear strength of a remoulded soil changes with a change in moisture content. It is defined as minus the slope of the flow curve on the semilog plot (see for example Casagrande, 1932) and is conveniently taken as the difference between the moisture contents corresponding to 10 and 100 blows (or twice the difference in moisture contents corresponding to 10 and 32 blows). Since the number of blows of the liquid limit device required to close the groove in the soil cake is a measure of the shear strength of the remoulded soil at that moisture content, it follows that soils with large flow indices change less in strength with the same change in moisture content than do those with small flow indices.

The flow index may also be used as a relative indication of the shear strength at the plastic limit in

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that the latter is inversely proportional to the flow index for soils having the same plasticity index and proportional to the plasticity index for soils having the same flow index.

Some flow curves of calcretes are shown in Figures 7.1-7.5. Flow indices are shown in Tables H.1 to H.4 in Appendix H. Flow indices of calcretes vary widely from 0 to about 30. The flow index is not a property which is often calculated nowadays and few figures on other materials are available for comparison. Casagrande's (1932) results varied from 6.5 (a tough clay) to 32 (mica powder). Materials with low liquid limits tend to yield low flow indices and a flow index of 0 is yielded by a flow curve which is parallel to the baseline. Such materials must be extremely sensitive to moisture changes around the liquid limit.

8.4.6 Toughness index

The toughness index (T or I_t) is a measure of the strength of a remoulded soil at the plastic limit. "The toughness of a clay at its plastic limit may therefore be described as the maximum stiffness or shearing resistance which it can acquire without losing its plasticity. Hence the shearing strength at the plastic limit may be considered a measure of the toughness of a clay" (Casagrande, 1932). The toughness index is thus defined as

$$T = \log \left(\frac{S_2}{S_1}\right) = \frac{PI}{F}$$

where S_1 is the shear strength at the liquid limit (roughly constant for all soils at around 0.5 psi), S_2 is the shear strength at the plastic limit (toughness), PI is the plasticity index and F is the flow index (Casagrande, 1932).

Toughness indices of the calcretes tested are shown in Tables H.1-H.4 of Appendix H. Values vary from 0 to

infinity/ ...

infinity. Values between 1 and 5 are perhaps the most common, and the highest finite value is 31. In calculating toughness indices a slightly plastic (SP) material was assigned a PI of 1 and a non-plastic material a PI of 0. A toughness index of 0 means that the strength at the liquid and plastic limits is the same. Only non-plastic materials (for which these two limits are coincident) yield toughness indices of 0. A toughness index of infinity is yielded only by materials with a flow index of 0. Once again, few toughness indices of other materials are available for comparison. Casagrande's (1932) results ranged from 0.77 (mica powder) to 2.69 (a tough clay). It would appear from his results that any material with a toughness index of less than about 1.7 will be friable at the plastic limit. About a third of the calcretes tested possessed higher toughness indices than this. A number yielded values above 3, suggesting that they possessed shear strengths at the plastic limit more than 1000 times that at the liquid limit. While only Casagrande's results have been used for comparison, it would seem that a number of calcretes possess unusually high shear strengths at the plastic limit - possibly due to cementation. These calcretes yielding unusually high toughness indices also seem to be those possessing large amounts of sepiolite or palygorskite as the dominant clay mineral. It would be interesting to try to measure the shear strength at the plastic limit of such calcretes. That of palygorskite is high.

8.4.7 Relation between liquid limit and plasticity index

Figure 8.6 shows the calcrete liquid limit and plasticity index results plotted on the Casagrande plasticity chart. No attempt has been made to find the best straight line through the points and the A line actually represents a fair approximation for calcretes as a whole. It will be observed that the relation for nodular calcretes for example will differ from that for powder calcretes.

Calcretes/ ...

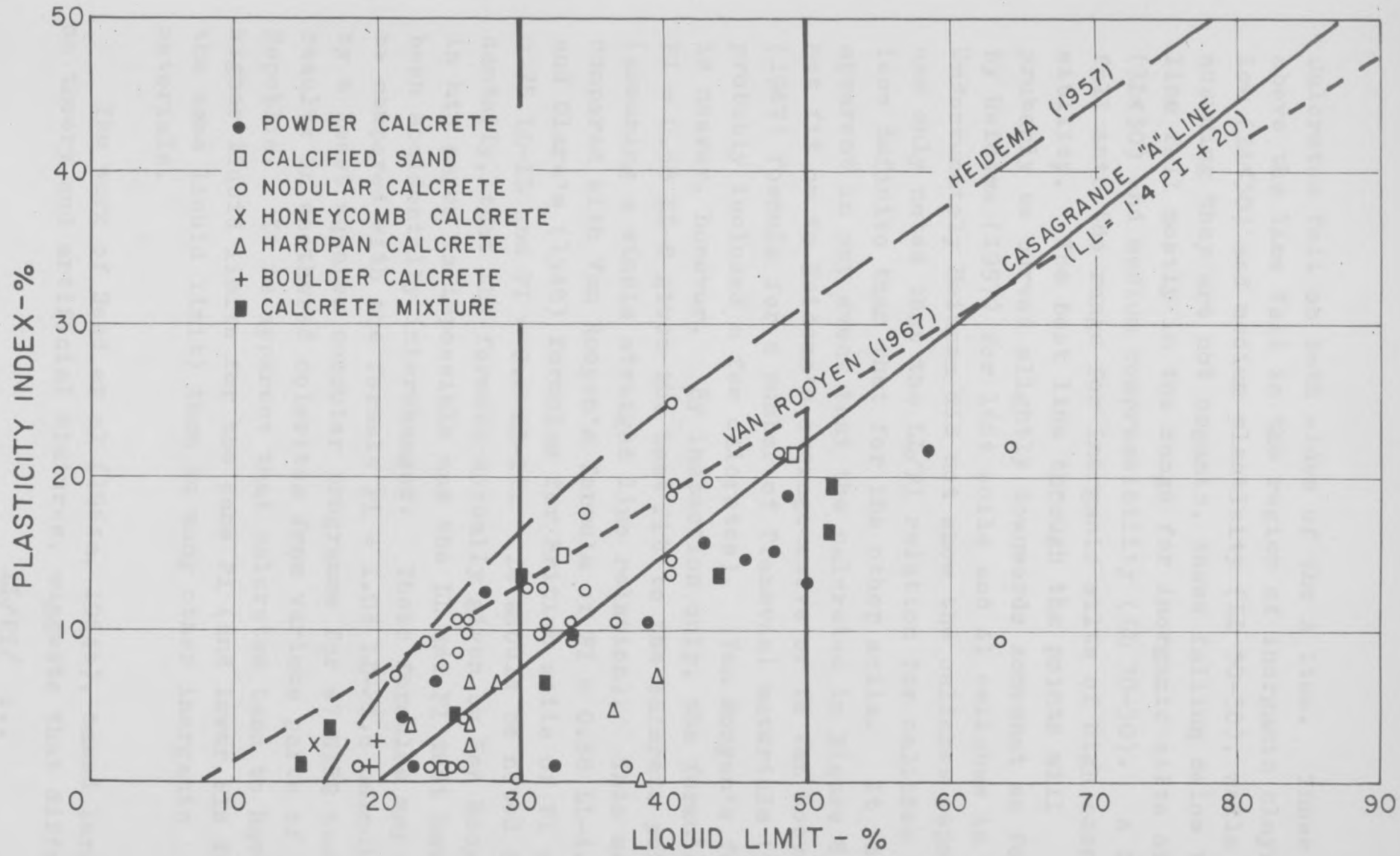


FIGURE 8.6

Position of calcretes on the Casagrande plasticity chart

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Calcretes fall on both sides of the A line. Those falling above the line fall in the region of inorganic clays of low (LL<30) and medium plasticity (LL 30-50), while assuming they are not organic, those falling below the line fall mostly in the range for inorganic silts of low (LL<30) and medium compressibility (LL 30-50). A few fall into the range for inorganic silts of high compressibility. The best line through the points will probably be curved slightly downwards somewhat as found by Heidema (1957) for 1664 soils and 41 caliches in Texas. Unfortunately Heidema did not show the caliches separately and only noted that the LL/PI relation for caliches was less definite than that for the other soils. It is apparent in any event that the calcretes in Figure 8.6 do not fit on to Heidema's average curve or to Van Rooyen's (1967) formula for a number of Transvaal materials (which probably included a few calcretes). Van Rooyen's formula is nearer, however. By inspection only, the formula PI = 0.58 LL-8 gives the best fit to the calcrete points (assuming a simple straight line relation). This may be compared with Van Rooyen's formula of PI = 0.58 LL-4.46 and Clare's (1948) formulae for British soils of PI = 0.75 LL-11 and PI = 0.8 LL-12. It should be noted incidentally, that the formula actually given by Van Rooyen in his paper is impossible and the LL and PI must have been accidentally interchanged. These formulae may also be compared with the formula PI = 1.03 LL-21.6 calculated by a least squares computer programme for 47 NIRR test results on weathered dolerites from various parts of the Republic. It is apparent that calcretes tend to have higher liquid limits for the same PI (and lower PIs for the same liquid limit) than do many other inorganic materials.

The work of Seed et al (1964a, 1964b), based largely on theory and artificial mixtures, suggests that different

LL/PI relationships are due to the presence of different clay minerals, that materials with the same activity possess the same LL/PI relationship and that the intercept (C) of the straight line equation PI = MLL-C remains roughly constant, but that it increases with activity. In 1948 Clare had stated that the LL and PL depended on the type of clay present, but that the PI was a function only of the amount of clay present.

The liquid limit, it should be remembered, is merely the water content at a certain shear strength. It is therefore influenced by the porosity of the soil paste and the porosity of the soil particles themselves. Silts, organic matter and materials with porous particles can thus be expected to yield relatively high liquid limits. The reasons for the relatively high liquid limits of calcretes are probably threefold:

1) The clay minerals present (usually attapulgite, sepiolite and/or montmorillonite) are those that tend to yield high liquid limits and plasticity indices (though the Atterbergs of sepiolite are unknown).

2) Many calcretes (particularly powder calcretes) contain a large amount of non-plastic silt sized carbonate.

3) The relatively high liquid and shrinkage limits and the fact that calcretes often possess higher shrinkage limits than plastic limits (an extremely rare feature) strongly suggest that porous particles are present. The presence of porous particles is very compatible with the origin of these materials. Some calcretes also contain diatom shells which would hold water, but be non-plastic.

As shown in Chapter 3, the organic matter contents of calcretes are small (usually less than 1%).

Factors 1 and 2 are probably relatively minor contributory factors and alone are insufficient to account for this effect. Factor three is considered to be the most important. Although diatoms are of organic origin so that calcretes containing significant amounts of them would probably correctly plot below the A line, they are harmless for most purposes and actually beneficial in lime stabilization. The effect is also too large to be entirely due to the tendency of the BS liquid limit devices in use at NIRR to yield higher liquid limits than the American-type devices probably used to obtain the results used by Van Rooyen, Heidema and Clare.

8.4.8 Shrinkage

The shrinkage of calcrete fines was investigated chiefly by means of the direct bar linear shrinkage from the liquid limit method. The term 'bar linear shrinkage' is used here to distinguish it from the linear shrinkage calculated from the volumetric shrinkage. The latter is approximately three times the linear shrinkage, but can be calculated exactly (see for example the formula in ASTM D427).

The effect of the predominant cation on the shrinkage of clay appears to be little known (see Table 8.1). If the usual rule that the linear shrinkage is numerically equal to approximately half the PI can be applied, it can be seen that exchange of Ca for Na should have similar effect to that noted in the case of the PI, that is a large lowering of shrinkage in the case of montmorillonite, little effect in the case of palygorskite and a possible slight increase in the case of illite and kaolinite. The effect of large increases in calcium carbonate content on the bar linear shrinkage of a black clay is shown in Figure 8.7. The effect is exactly similar to that of $CaCO_3$ on the PI except that it would appear that shrinkage would cease at about 86% $CaCO_3$ while the PI was still

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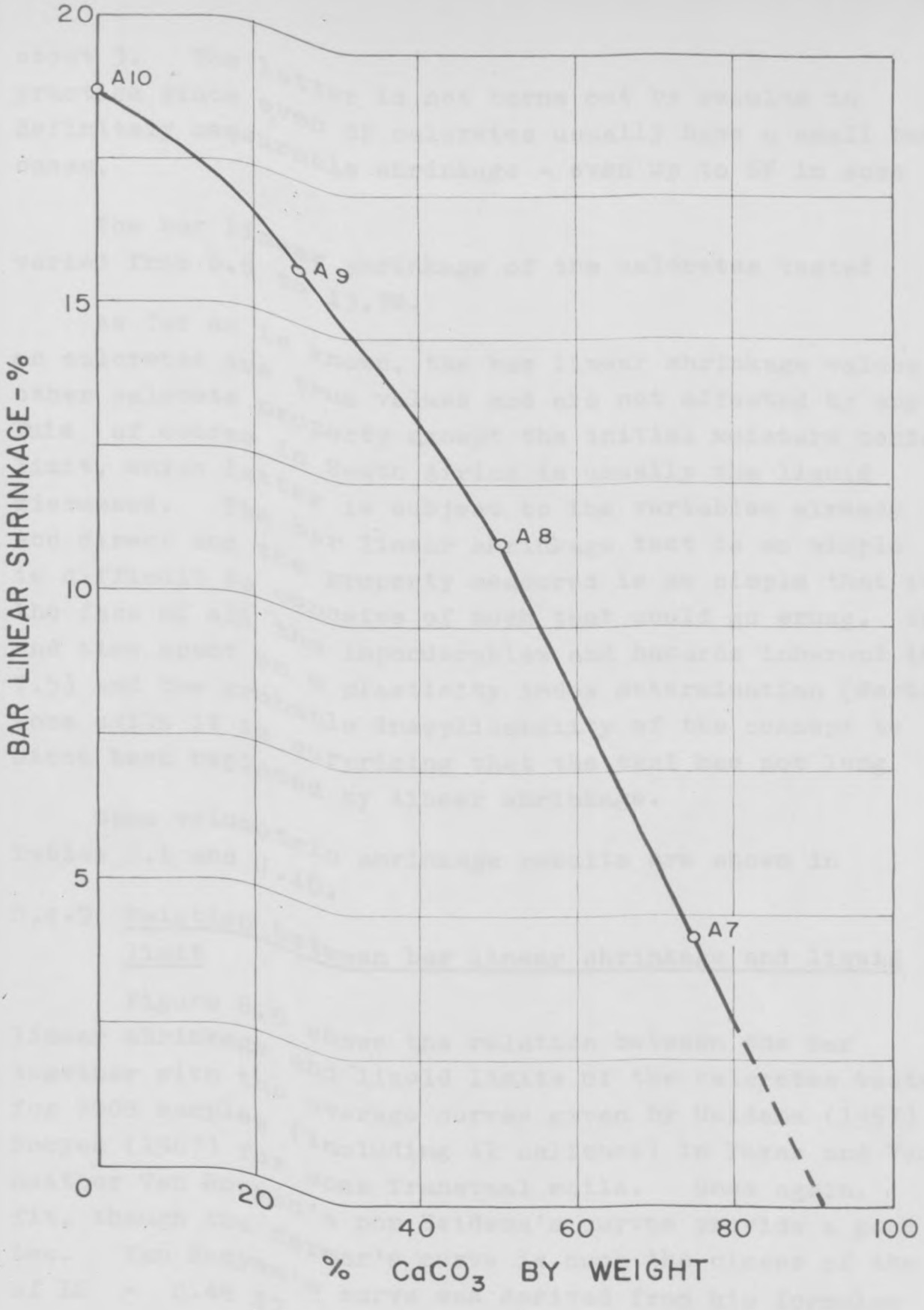


FIGURE 8.7

EFFECT OF CALCIUM CARBONATE ON THE BAR LINEAR SHRINKAGE OF A BLACK CLAY

about 3. The latter is not borne out by results in practice since even SP calcretes usually have a small but definitely measurable shrinkage - even up to 6% in some cases.

The bar linear shrinkage of the calcretes tested varied from 0.6 to 13.3%.

As far as is known, the bar linear shrinkage values on calcretes are true values and are not affected by any other calcrete property except the initial moisture content. This of course in South Africa is usually the liquid limit, which latter is subject to the variables already discussed. The bar linear shrinkage test is so simple and direct and the property measured is so simple that it is difficult to conceive of much that could go wrong. In the face of all the imponderables and hazards inherent in and time spent on a plasticity index determination (Section 7.5) and the probable inapplicability of the concept to some soils it is surprising that the test has not long since been replaced by linear shrinkage.

Some volumetric shrinkage results are shown in Tables J.1 and J.10.

8.4.9 Relation between bar linear shrinkage and liquid limit

Figure 8.8 shows the relation between the bar linear shrinkage and liquid limits of the calcretes tested, together with the average curves given by Heidema (1957) for 2005 samples (including 41 caliches) in Texas and Van Rooyen (1967) for some Transvaal soils. Once again, neither Van Rooyen's nor Heidema's curves provide a good fit, though the former's curve is much the closer of the two. Van Rooyen's curve was derived from his formulae of LS = 0.46 PI and PI = 0.58 LL-4.46.

Combining the two yields

LS/ ...

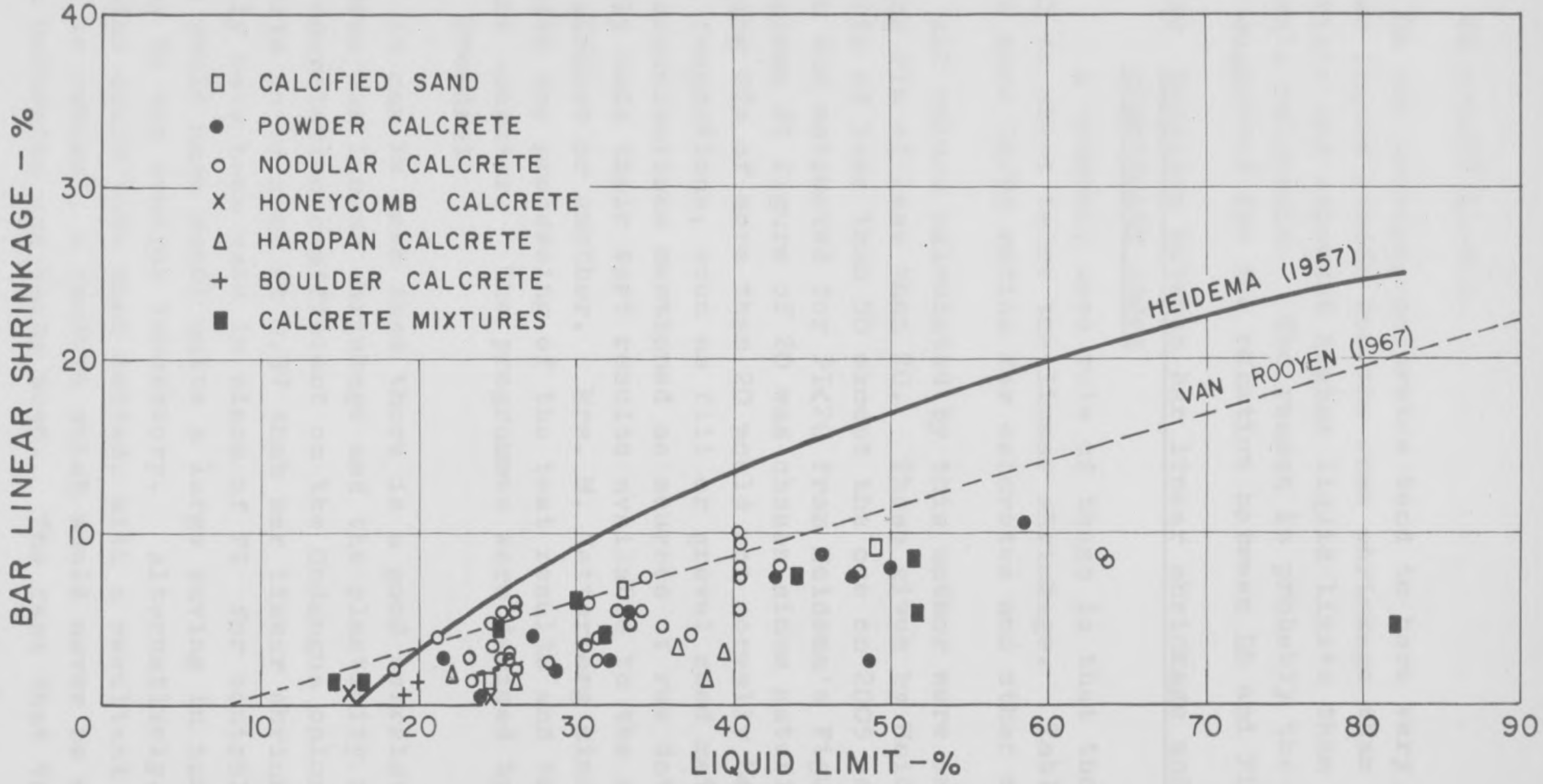


FIGURE 8.8

Relation between bar linear shrinkage and liquid limit

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LS = 0.27 LL-2.1.

On the average, calcretes tend to have very much higher liquid limits for the same shrinkage than Heidema's materials and somewhat higher liquid limits than Van Rooyen's materials. The reason is probably the same as that suggested for the relation between LL and PI.

8.4.10 Relation between bar linear shrinkage and plasticity index

A commonly used rule of thumb is that the PI is equal to about twice the linear shrinkage. Table 8.3 shows some LS/PI ratios for calcretes and other materials.

All values calculated by this author were on materials having PIs of less than 20. Those given by Heidema were for PIs of less than 50 except the one on 2005 samples which was estimated for PI<20 from Heidema's Figure 6. A maximum PI figure of 20 was chosen since materials possessing PIs of more than 20 would not normally be used in road foundations, even as fill or gravel road material. The organisations mentioned as sources of raw data had kindly made their test results available to the author for one purpose or another. Mrs. M. Netterberg kindly assisted with the processing of the test results and the feeding of the computer. The programmes were devised by Dr. M.E. Szendrei.

It can be seen that there is a good correlation between bar linear shrinkage and the plasticity index. The correlation coefficient on the Ondangua calcrete results is so good at 0.97 that bar linear shrinkage could easily have been used in place of PI for control purposes. This would have meant quite a large saving in time and money to the control laboratory. Alternatively, more samples could have been tested, with a resultant improvement in control, a feature which would never be wasted when calcretes are being used. The fact that there is a

linear/ ...

TABLE 8.3 RELATION BETWEEN BAR LINEAR SHRINKAGE AND PLASTICITY INDEX FOR CALCRETES AND OTHER MATERIALS

MATERIAL	LOCALITY	MEAN LS/PI	STANDARD DEVIATION	RANGE	CORRELATION COEFFICIENT	NUMBER OF SAMPLES	SOURCE OF RAW DATA
Calcretes	Ondangua	0.38	0.06	0.25-0.54	0.97	67	VKE
"	Runtu	0.44	0.09	0.25-0.54	0.76	14	VKE
"	Various	0.43	0.17	0.09-1.08	0.76	54	NIRR
"	Port Elizabeth	0.42	0.06	0.30-0.56	0.93	26	CPA
"	Wasser-Asab	0.50	0.10	0.16-0.70	0.86	38	Kantey
"	Cape Flats	0.43	0.05	0.36-0.50	0.83	29	CPA
"	Tsumeb-Operet	0.35	0.08	0.23-0.50	0.73	8	Shand
"	Operet-Oshikati	0.46	0.06	0.23-0.62	0.96	192	VKE
"	Katima Mulilo	0.32	0.10	0.19-0.58	0.44	14	D.O.T.
"	Bloemfontein-Winburg	0.40	0.08	0.21-0.57	0.92	43	Shand
"	Lichtenburg-Deelpan	0.51	0.03	0.40-0.63	0.98	307	TPA
Mixtures of calcretes and other materials	Wasser-Asab	0.48	0.07	0.30-0.60	0.84	68	Kantey
Topsoils	Ondangua	0.27	0.09	0.10-0.48	0.94	39	VKE
Topsoils	Runtu	0.38	0.07	0.25-0.48	0.99	8	VKE
Various materials except calcretes and weathered dolerites	Various	0.60	0.31	0.18-4.00	0.86	233	NIRR
Mostly shales	Wasser-Asab	0.48	0.08	0.25-0.69	0.85	116	Kantey
Weathered dolerites	Various	0.81	0.47	0.34-2.33	0.84	43	NIRR
Weathered dolerites	Bloemfontein-Winburg	0.43	0.05	0.30-0.58	0.77	26	Shand
		OTHER	FIGURES				SOURCE
1664 soils & 41 caliches	Texas	0.53	-	-	-	2005	Heidema (1957)
Unknown	Texas	0.53	-	-			THD (Heidema)
"Soils"	Texas	0.55	-	0.43-0.60	-	14 projects	Heidema
"Soils"	Transvaal	0.46	-	-	0.90	-	Van Rooyen (1967)
Various	Rhodesia	0.44	-	-	-	1000	MRCL

VKE = Van Niekerk, Kleyn & Edwards, Windhoek & Pretoria
NIRR = National Institute for Road Research, Pretoria
CPA = Cape Provincial Administration Roads Dept., Cape Town
Kantey = Kantey & Templer, Cape Town
Shand = Ninham Shand, Tsumeb & Bloemfontein
D.O.T. = Dept. of Transport, Pretoria
THD = Texas Highway Dept., Austin
MRCL = Ministry of Roads Central Laboratory, Salisbury (unpublished figures kindly made available by Mr. C.P. van der Merwe, 1968; private comm.)
TPA = Transvaal Provincial Administration Roads Dept., Pretoria

linear or very nearly linear relationship between linear shrinkage and PI and that such high correlation coefficients are obtained is of great significance. The latter means that both tests measure the same property, or very nearly the same property, and that property can only be shrinkage, since it is the most fundamental. The theoretical relationship between linear shrinkage and plasticity index has already been demonstrated in Section 7.6. It is obvious then that to use a PI to measure LS is a very roundabout and expensive way of going about it. Furthermore, as has been shown, PIs can be unreliable on calcretes, whereas linear shrinkages are probably not.

Figure 8.9 shows the results on calcretes compared with the Texas Highway Department curve (in Heidema), Heidema's U.S. Engineers Department curve in his Figure 3 and the formula LS = 0.46 PI given by Van Rooyen. Heidema's Figure 6 showing the results on 2005 samples (including 41 caliches), is virtually identical to the THD curve. The Texas Highway Department curve almost certainly includes at least some caliches. The calcretes in Figure 8.9 broken down as to stage of development are the same as those used to calculate the figure 0.43 in Table 8.3.

Both Table 8.3 and Figure 8.9 show that many calcretes tend to have lower LS/PI ratios than most other materials, or in other words, have higher PIs than other materials of the same shrinkage. If shrinkage is accepted as the more reliable test and if it is in fact the fundamental property measured by a PI, then it is apparent that LS limits rather than PI limits should be set for road materials. Furthermore, it indicates that higher PIs can be allowed for some materials than for others. Calcretes for one fall into this category. If a limit of LS of 3% is set for base course (the current D.O.T. specification) this would mean that the maximum allowable PI of the

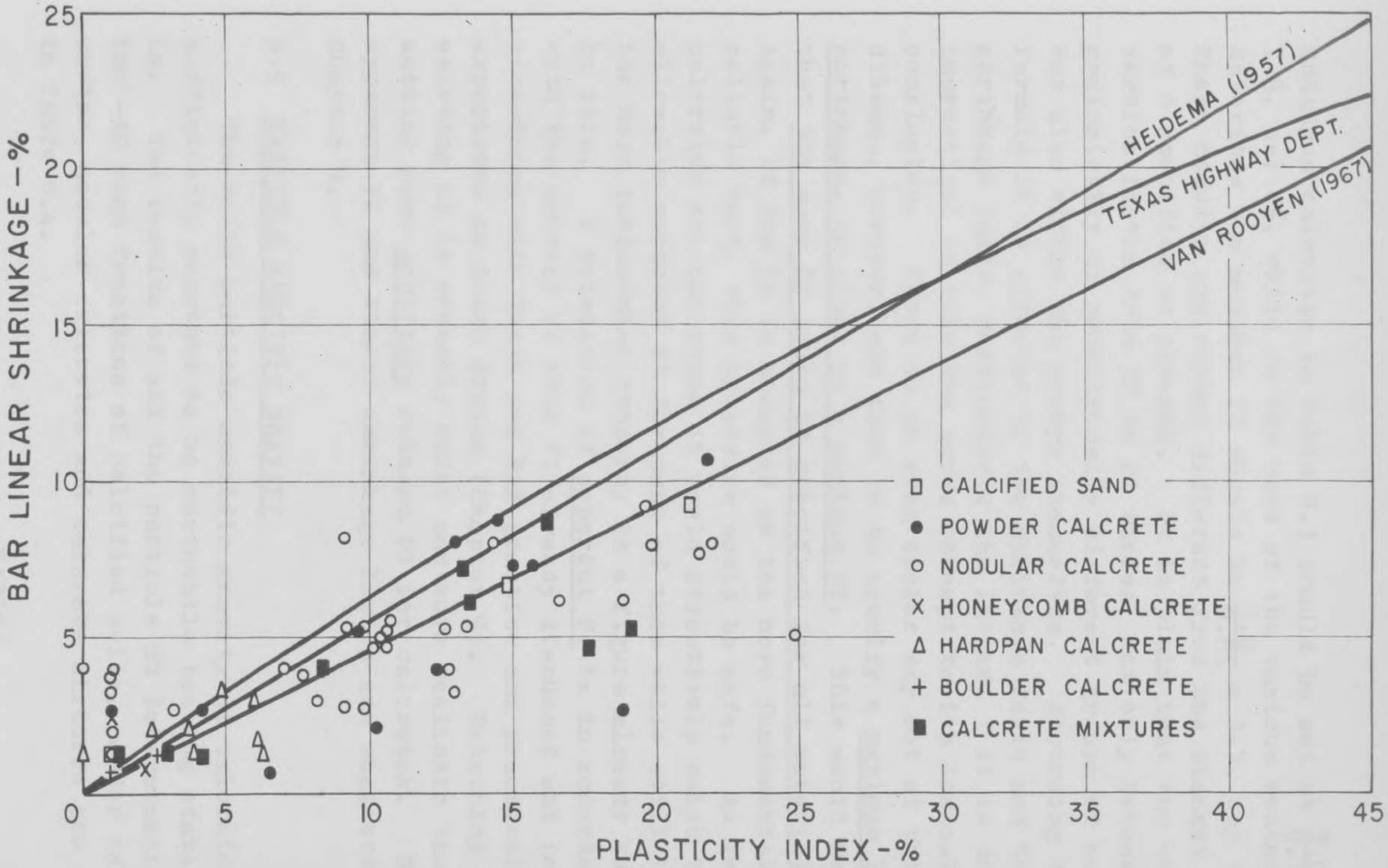


FIGURE 8·9

Relation between bar linear shrinkage and apparent plasticity index

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Ondangua calcretes in Table 8.3 should be set at $\frac{3.0}{0.38}$ = 7.9, say 8%, while in the case of the various weathered dolerites the maximum PI should be $\frac{3.0}{0.81}$ = 3.7, say 4%. These results are rather different from the blanket figure of 6 specified at present. It is plain that the conversion factor from LS to PI varies, not only between geologically or pedologically different groups of materials, but also within the groups themselves. According to the formula it is affected by the shrinkage ratio and the shrinkage index, particularly the latter. It is thus impractical to take the above concept to its logical conclusion. There is an even easier way out of this dilemma, however, and that is to specify a maximum linear shrinkage, in place of a maximum PI. This would mean that the same LS could be specified for all materials. Again, if the LS is accepted as the more fundamental and reliable test, this procedure would be safe. As far as calcretes are concerned it would effectively raise the allowable apparent PI for many of them while still limiting the more fundamental property to a figure already known to be safe. A relaxation of apparent PI is in accordance with the concept of true PI already discussed and is in accordance with Texas and FAA practice and practical experience in South Africa (Chapter 9). Retaining the existing LS is probably safer and more realistic than setting some arbitrary relaxed PI for calcretes. Both apparent PI and linear shrinkage limits are suggested in Chapter 9.

8.5 PARTICLE SPECIFIC GRAVITY

The solid particle specific gravity of calcretes is sufficiently constant to be worthwhile testing statistically. The results of all the particle SG determinations on the -40 mesh fractions of calcified soils, powder calcretes, nodular calcretes and calcrete mixtures are shown in Table 8.4.

TABLE/ ...

TABLE 8.4 SOLID PARTICLE SPECIFIC GRAVITY OF CALCRETE SOIL FINES

MEAN	STANDARD DEVIATION	PER CENT STANDARD DEVIATION	RANGE	NUMBER OF RESULTS
2.657	0.061	2.31	2.468-2.804	44

Determinations were normally carried out in triplicate and the values shown in Appendix H are means of the three determinations unless one value was in considerable disagreement with the other two, in which case it was rejected. The agreement between the triplicate results was found to be rather variable. Only considering the results which were acceptable, differences of up to 0.01 were not uncommon, while the maximum accepted of 0.1 was rare. Differences of only 0.001 were quite often attained. Dawson (1959) stated that the probable error of specific gravity determinations should be of the order of $\pm$ 0.005 for ordinary soils.

The solid particle SG reflects the mineralogical composition of the calcrete. If calcite and quartz were the only minerals present it would only vary between 2.65 (quartz) and 2.71 (calcite). Variations outside this range indicate the presence of minerals other than these. Opaline silica and most clays (kaolinite, sepiolite, palygorskite, montmorillonite and allophane) can be expected to lower the particle SG, while carbonates, some clays (some illites - especially hydrous micas - and chlorites) and the 'heavy minerals' of sedimentary petrography (magnetite, ilmenite, rutile, zircon, garnet, etc.) can be expected to raise it.

The solid particle specific gravity of a number of calcretes crushed and ground to minus 40 mesh is shown in

Table/ ...

Table J.5 in Appendix J. The values obtained are not always the same as the values for the minus 40 mesh fraction. This is probably chiefly due to the carbonate fraction not being equally distributed through all size fractions.

Various other specific gravity data is shown in Appendix J. The bulk specific gravity of calcrete particles may be much less than the specific gravity of solids and also appears prone to vary with particle size. The porosity of calcrete particles probably reduces the reliability of specific gravity data (unless very carefully carried out) and also reduces the reliability of hydrometer analyses, particle size distribution areas by weight and Atterberg limits.

8.6 CALCIFIED SOILS

Only a few results are available on these materials so that they are obviously much more variable than the results indicate. Most calcified soils are basically either calcified sands or calcified gravels. Calcified clays seem to be rare, for reasons explained under the chapter on origin.

Dr. B. Clegg of the University of Western Australia kindly allowed the author to study a paper he and Mr. B. Brooksbank are writing on Western Australian "coastal limestone". This material is actually aeolianite (dunerock), consisting of sand cemented by secondary calcium carbonate from shell fragments, and much of it would probably classify as calcified sand. The material is usually rippable by a Caterpillar D7. 10% F.A.C.T. values range from 0.5 to 3 tons and CBRs from 25 to 70. It is non-plastic and apparently makes good subbase under a thin (upwards of half an inch) crushed stone base. Clegg and Brooksbank's paper should be consulted for further details of these materials when it appears.

Test/ ...

Test results on calcified sands are summarised in Table H.1 in Appendix H.

8.6.1 Soil constants

The few values available (Table H.1) show the liquid limit to vary from being unobtainable to 49%, the PI from NP to 21% and the linear shrinkage from < 1 to 9%. All 5 samples were sands rather than gravels or clays. One shrinkage limit determination yielded the value of 29%.

8.6.2 Mechanical analysis

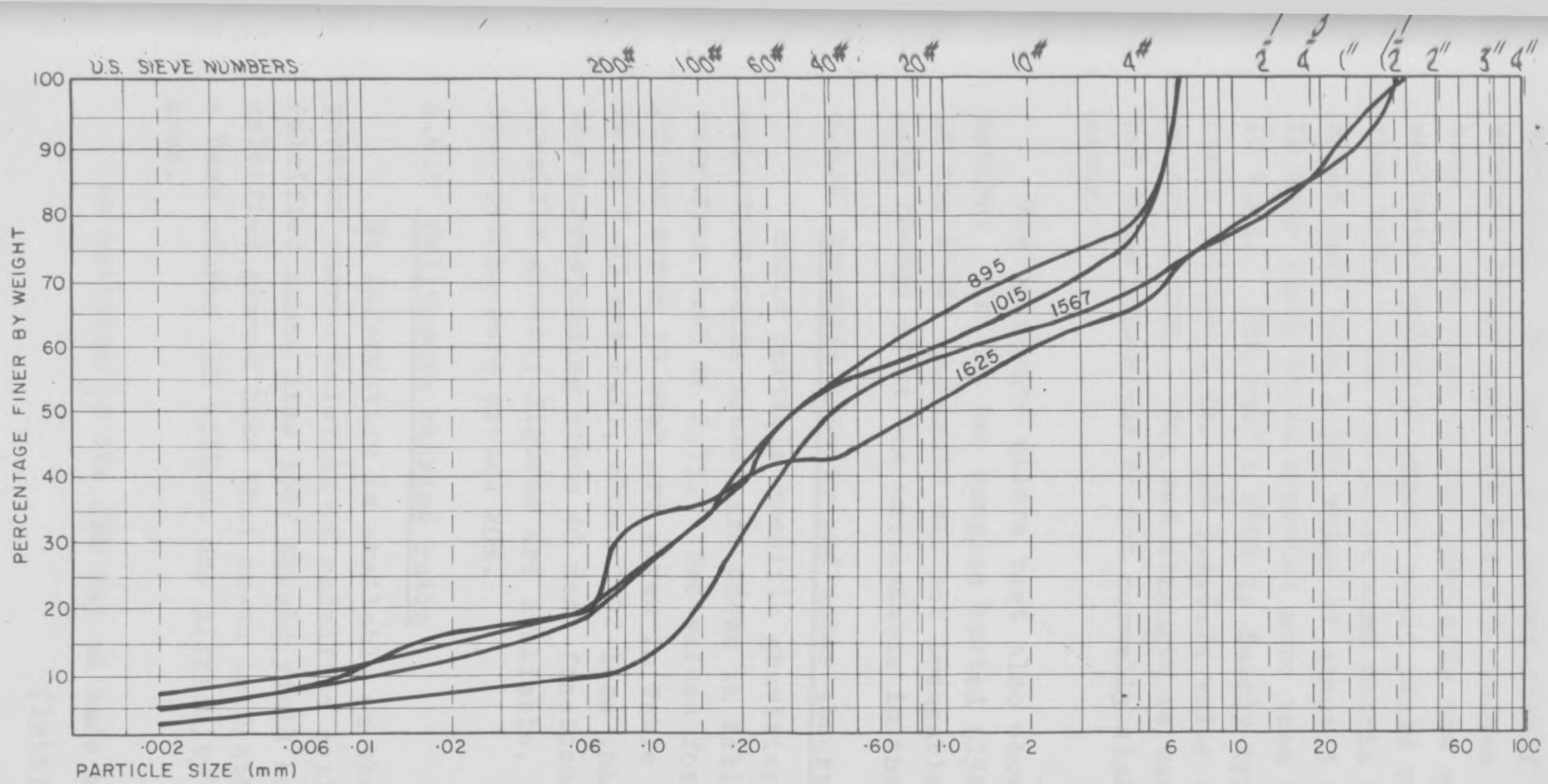
Mechanical analyses of calcified sands are probably unreliable owing to their normally friable nature. Figure 8.10 shows the apparent particle size distribution by weight of 4 samples of essentially calcified sands. Their grading moduli varied from 1.5 to 1.8, uniformity coefficients from 17 to 333 and effective sizes from 0.006 to 0.06. For comparison, a theoretically well graded material would have a uniformity coefficient of about 230 and an effective size of about 0.005 mm, while a single-sized sand would have a uniformity coefficient of around 3 and an effective size of about 0.05 mm. A grading modulus of more than 1.5, 1.6 and 2.1 is often specified for subbase, lime or cement stabilized base and natural base, respectively, for provincial-type roads in the Transvaal and South West Africa.

Calcified gravels and calcified weathered rocks can be expected to yield rather better gradings than calcified sands once they have been ripped and compacted (grid rolling if necessary).

8.6.3 Aggregate crushing strength

The crushing strength of the aggregate fraction (+ 2 mm) of excavated calcified sands and gravels is

variable/ ...



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

FIGURE 8.10

Particle size distribution of some calcified sands

RG4-3202

variable. The alluvial gravel itself in Plate 1.3 for example could have 10% F.A.C.T. values of up to about 25 tons in the case of the quartzite and quartz, while the weathered andesite gravel would yield values much less than this. The calcified sand matrix on the other hand might only have a dry value of about 3 tons (Sample 2506). In many cases it is somewhat more than this, perhaps up to 10 tons. The sample 2506 is fairly typical of calcified sands and has a dry 10% F.A.C.T. value of 2.8 tons, and an ACV of 48%. The wet strength is very low (0.3 tons), but these materials do not normally slake when placed in water.

The aggregate pliers test also shows up their weak nature. In the two samples tested (1567 and 1625), 72-94% of the $\frac{1}{2}$"-$\frac{3}{4}$" aggregate was not breakable by hand, but only 19-39% of it was unbreakable in the standard pliers.

8.6.4 Specific gravity and water absorption

Solid particle specific gravities of the four calcified sands tested are shown in Table H.1. Values vary from 2.65 to 2.73. Two values for the whole samples ground minus 40 mesh are given in Table J.6, the values being 2.63 and 2.71, about 0.02 less than the values for the corresponding minus 40 mesh fraction. No bulk specific gravity figures are available, but two water absorptions were around 20%.

8.6.5 California bearing ratio

No information is available on the CBR and compaction characteristics of calcified soils. The better calcified sands like 1567 should yield subbase CBRs, while calcified gravel like that shown in Plate 1.3 should yield a base course CBR without any difficulty if suitably broken down.

An estimate of the CBR may be made by means of Kleyn's

(1955)/ ...

(1955) method.

8.6.5 Engineering classification

The 4 calcified sands tested varied in classification from A-1-b(0) (Sample 1567) to A-2-7(2) (1625) according to the revised PRA classification. 1567 is thus quite a good material according to this classification.

According to the extended Casagrande classification they would vary from SF-GF to GP-GF, in other words they are essentially gravelly sands with an excess of fines. According to the D.O.T. classification they would all be semi-sand clays.

8.6.7 Workability

All calcified sands are pickable by hand and dozable or rippable by the smaller bulldozers. Some calcified gravels such as those shown in Plate 1.3 would require large machines such as a Caterpillar D8 or D9, with the possibility of occasional presplitting being required. Relatively little information is available on the workability of these materials though much could be gained by making the rounds of the diamond diggings along the Vaal. The impression gained at Windsorton was that most of the excavation had been carried out by hand picking.

8.6.8 Acceptability as road materials

Table 8.5 shows the probable acceptability of 4 calcified sands and one calcified weathered felspathic sandstone as roadmaking materials by some authorities in areas where calcretes occur. In deciding on their acceptability the normal specifications of the various authorities were taken more or less literally, with little or no latitude being usually allowed. Some latitude was allowed on the question of grading and a material was accepted as long as it fell within the grading envelope specified. Many specifications of course also insist

that/ ...

TABLE 8.5 PROBABLE ACCEPTABILITY OF CALCIFIED SANDS AS ROAD MATERIALS BY VARIOUS AUTHORITIES

SAMPLE NUMBER	S.A. DEPARTMENT OF TRANSPORT (1968)				TRANSVAAL ROADS DEPARTMENT			S.W.A. ROADS DEPARTMENT (OPERET-OSHAKATI CONTRACT)			U.S. FEDERAL AVIATION AGENCY			TEXAS HIGHWAY DEPARTMENT			ACCEPTABLE FOR CHEMICALLY STABILIZED BASE ACCORDING TO	ACCEPTABLE AS GRAVEL ROAD WEARING COURSE ACCORDING TO(3)	REMARKS
	BEST LAYER(1) IN SURFACED ROAD FOR WHICH SAMPLE IS ACCEPTABLE, AS INDICATED BY THE FOLLOWING CRITERIA(2)																		
	GRADING	LL PI LS	GROUP INDEX	WHOLE SPEC.	GRADING	LL PI	WHOLE SPEC.	GRADING	PI	WHOLE SPEC.	GRADING	LL PI	WHOLE SPEC.	GRADING	LL PI	WHOLE SPEC.			
895	-	-	B	Fill	S	-	Sel. subgrade	S	-	Sel. subgrade	S	-	Sel-subgrade	-	-	Sel. subgrade	-	-	
1015	-	B	B	Sel. subgrade	S	B	Subbase	S	B	Subbase	S	B	Subbase	-	B	C. sub-base?	-	-	
1567	-	B	B	Sel. subgrade	S	B	Subbase	SB	B	Subbase	S	B	Subbase	CB	B	C. base	S.W.A.	-	
1625	-	-	F	Lower fill?	S	-	Lower fill?	SB	-	Lower fill?	-	-	Lower fill?	CB	-	Fill?	S.W.A.	-	
1022	-	-	S	Fill	S	-	Fill	SB	-	Fill	CB	-	Fill?	CB	-	C. sub-base?	S.W.A.	T?,F,O.F.S.	Chiefly calcified weathered sandstone gravel

NOTES: (1) B = base, SB = stabilized base, CB = Caliche base (FAA or THD), S = subbase, F = fill

(2) Whole spec. means according to the whole specification except strength requirements except where such data is known.

(3) T = Transvaal, F = Fossberg (1963), O.F.S. = Orange Free State.

that the material must be reasonably smoothly graded within the envelope. As it was difficult to decide objectively on the amount of irregularity tolerated by each authority this requirement was dropped. In some cases then, materials shown as being probably acceptable on a grading basis to a certain authority might in practice be rejected. Others, just outside the envelope, but with a fairly smooth grading might be used if nothing else was available. Similar considerations would apply to the other criteria of acceptability, such as soil constants. Strength requirements such as CBR have been largely disregarded, since so few results are available. They have been used where available. Similarly, stabilized materials also have to meet certain strength requirements which perforce have also largely been disregarded. The column 'Whole spec.' is therefore really an assessment based largely only on the indicator test results, that is, grading and soil constants together with classifications based on them such as the revised PRA system (also known as the HRB and the AASHO classification). It should be noted that the FAA and Texas do not actually list caliche subbase, selected subgrade or fill in their specifications. The author has therefore had to rely to some extent on his own opinion as to what these authorities would do with the calcrete concerned, based on their caliche base and other subbase items. Texas do not list a subbase as such. They presumably specify a certain base item to be used as subbase on the contract documents themselves. As far as stabilized and gravel road materials are concerned, only the specifications of the South African authorities listed have been considered. The foregoing remarks apply also to the discussions as to the acceptability of the other calcrete types to follow. A summary of the calcrete specifications of the authorities concerned will be found in Chapter 9.

Table 8.5 suggests that calcified sands would only be

acceptable/ ...

acceptable as fill to selected subgrade for the S.A. Department of Transport, but that 2 of them, 1015 and 1567 would probably be acceptable in the Transvaal, South West Africa. to the FAA and to the Texas Highway Department as natural subbase. 1567 also falls into the THD specification for caliche base. Texas might also find 1022 acceptable as a subbase to a caliche base, but probable not to any other base. 1567, 1625 and 1022 should be acceptable to S.W.A. (Operet Oshakati specifications) as lime or cement stabilized base 1567 was in fact used for lime stabilized base or subbase at the Ondangua Airfield.

Only 1022 fulfills the specifications for a gravel road wearing coarse of Fossberg (1963) and the Orange Free State. It may possible also be acceptable to the Transvaal. All samples with the exception of 1567 and 1625 were actually used for gravel road wearing courses in S.W.A. 1022 was said to be the best calcrete gravel wearing course on the Tsumeb-Grootfontein road.

8.7 POWDER CALCRETES

Powder calcretes are defined as those calcretes which are normally loose and powdery or which break down to a loose powdery material on normal excavation. Powder calcretes often possess a little nodular development and the boundary between powder and nodular calcretes has been set at a grading modulus of 1.5. This has been found to give a good separation in practice and the two groups can usually be readily distinguished in the field. A grading modulus boundary of 1.5 also means that nodular calcretes would be generally acceptable on a grading basis as subbase to the Transvaal and S.W.A., while powder calcretes would not. When desirable, the powder group could further be divided into two, with the really type specimen powder calcretes having grading moduli below 1.0 and those with appreciable nodular development falling in the range 1.0 -1.5. Plates 1.5-1.7A illustrate the difference between powder calcretes and nodular calcretes.

Powder/ ...

Powder calcretes are readily distinguished from calcified sands by the fact that they contain little or no sand-sized quartz grains (readily visible to the naked eye or a hand lens).

Test results on powder calcretes are summarised in Table H.1 in Appendix H.

It is important to distinguish between true (natural) powder calcretes and those which have been powdered by traffic due to aggregate degradation.

8.7.1 Soil constants

The type specimens of powder calcrete (1515 and 2223) are composed largely of silt and fine sand sized calcite and hence may have liquid limits of around 25, but are relatively low in plasticity - normally below about 8. Since the only criterion set is grading and to a lesser extent high calcium carbonate content, other materials also fall into this category. Materials which have degraded to powder on the road must also be placed in this category for convenience and because it is not always possible to decide into which category the material would fall. The calcified sands are prone to degrade on the road and some of the materials here classified as powder calcretes may have originally fallen into that category. Others will have been nodular calcretes. It is sometimes impossible to distinguish in the field between a nodular calcrete degraded by traffic and a genuine powder calcrete.

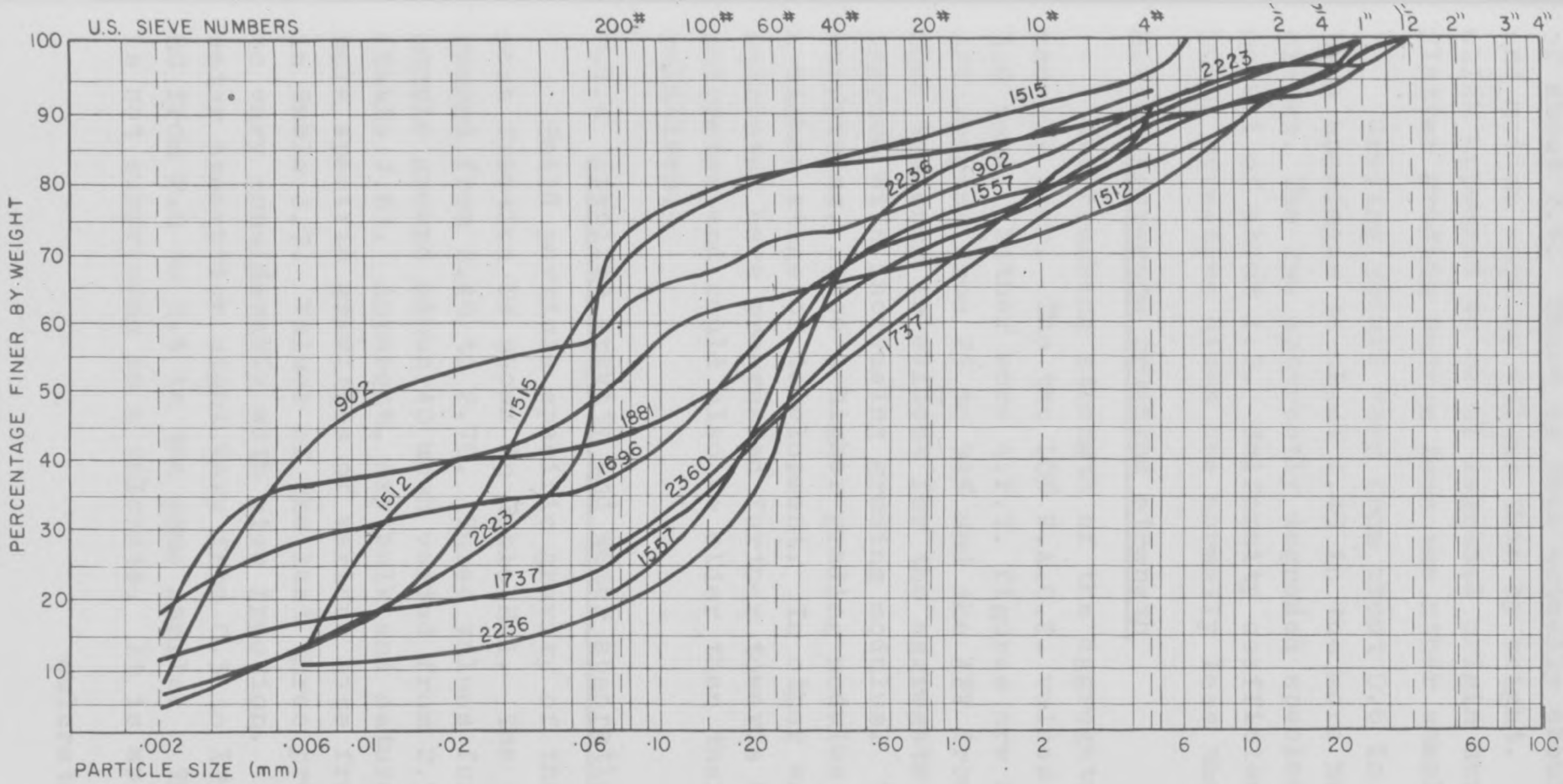
The 9 natural powder calcretes tested had liquid limits ranging from 22 to 58%. 5 of them were below 30 even though the PIs of these 5 ranged from SP to 15%. The highest PI of a natural material was 22. The 5 materials apparently powdered by traffic had liquid limits ranging from 33 to 50 and PIs from 10 to 15. Bar linear shrinkage of the 12 powder calcretes tested varied from 0.6 to 11%. Eight shrinkage limits varied from 19 to 29%. Several negative shrinkage indices were obtained.

8.7.2/ ...

8.7.2 Mechanical analysis

Mechanical analyses of powder calcretes are probably unreliable due to the friable nature of the aggregate. Mechanical analyses by weight are probably unreliable indicators of a maximum density-type grading because of the apparent variation of bulk particle density with size fraction (Table J.7). Mechanical analyses by volume would give a better indication if it were possible to obtain reliable bulk particle specific gravities on finer material. Since attempts to achieve this have been unsuccessful, an analysis by volume must at present also be regarded as unreliable. Grim (1962) also warns that, more so than for any other clay minerals, a particle size analysis on palygorskite and sepiolite only reveals "the degree of disaggregation and little concerning the size of the particles in the natural clay". The only practical way of combating the problem at present seems to be to accept that mechanical analyses on calcretes are not reliable and to adjust the specifications accordingly. Evidence will be presented in Chapter 9 to show that quite poor gradings can be tolerated. The available analyses are shown in Figure 8.11, Samples 902, 1512, 1515, 1557, 1737, 2223, 2236 and 2360 are of natural powder calcretes while 1696 and 1881 were found as such on the road and appeared to have been powdered by traffic. 1515 and 2223 are regarded as the type specimens of natural powder calcretes and are composed of about 75% silt and fine sand in the form of carbonate. A marked sand hunch is present. This excess of fine sand coupled with a deficiency of coarse sand is a characteristic of all non-indurated calcretes (i.e. calcified sands, powder calcretes and nodular calcretes). This sand hunch is more apparent than real if the bulk particle density results in Tables J.7 and J.8 can be believed. These tables suggest that the coarse and medium sand fraction of powder and nodular calcretes is composed of lower density particles in comparison with the other fractions. By volume, which is all that really matters, there would be more coarse and medium sand and less fines than is made apparent by reporting the results by weight. For example, in the case of Sample 2223 the coarse and medium sand apparently has a dry bulk particle gravity of about 2.1,

while/ ...



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

FIGURE 8.11

Particle size distribution of powder calcretes

RG4-3203

while the material finer than 60 mesh has a specific gravity of about 2.6. There is thus actually about 25% more coarse and medium sand by volume than by weight. The two samples which appeared to be of degraded origin have somewhat flatter grading curves than the other samples.

Grading moduli vary from about 0.6 in the case of the type specimens to about 1.45 in the more nodular powder calcretes. The two apparently degraded specimens both possess moduli of about 1.2. Uniformity coefficients vary from 9 to 1000. Effective sizes are normally less than 0.007 mm.

8.7.3 Aggregate crushing strength

The crushing strength of the aggregate fraction is usually poor. The two 10% F.A.C.T. values were 2.8 and 3.8 tons. Rather more A.P.T. figures are available, the AFV ranging from 25 to 94% and the APV from 8 to 63%. Some tendency is evident for the aggregate strength to improve with increasing grading modulus. This is not surprising, since a higher grading modulus merely 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.

8.7.4 Specific gravity and water absorption

Solid particle specific gravity of the minus 40 mesh fraction is shown in Table H.1. The twelve values ranged from 2.49 to 2.78. Three values for the whole sample ground minus 40 mesh varied from 2.59 to 2.71 (Table J.6). Apparent, dry bulk and saturated surface dry bulk specific gravities of various size fractions are shown in Table J.7. Values of the last three properties appear to vary considerably with size fraction. For example, water absorption might vary from 0.3 to 12% and dry bulk SG from 2.0 to 2.6 in the same sample. This phenomenon is not surprising in a calcrete. It is apparent that

calcrete/ ...

calcrete particles tend to be rather porous.

8.7.5 California bearing ratio

The general rule that CBR decreases with PI does not always hold in the case of calcretes (Mr. J. Caiger, 1965; personal comm.) and this is well shown by Sample 897, which has an apparent PI of 19, fitting it only for the lower fill of a high class surfaced road, while its CBR of 54 befits it for use as subbase in a 12,000 lb wheel load design. Most powder calcretes in fact will yield D.O.T. subbase CBRs (25-80 for a 7,000 lb wheel load design). Even 2223, with its very poor grading which would normally make it acceptable only for fill, has a CBR of 37. The powder calcrete in Plate 1.5 had a CBR of 65, which is the highest known for this type of material. Its Modified AASHO density was only 78 lbs/cu ft, its PI 7 and its optimum moisture content (OMC) 41.4%.

Little data is available on maximum density and optimum moisture content, but it is known that powder calcretes (and calcretes in general) are prone to have exceptionally high optimums. The two samples tested yielded OMCs of 8.7 and 20.0% and maximum dry densities (MDD) of 127 and 101 lbs/cu ft respectively.

8.7.6 Engineering classification

The type natural powder calcretes 1515 and 2223 classify as A-4(7) and A-4(8) materials. Other natural powder calcretes range from A-7-5(13) to A-2-4(0). On the Casagrande scale the type materials would be ML (inorganic silts with slight plasticity) while the general range of this group is from CH to GF.SF (sands with excess of fines) and ML are probably typical. In the D.O.T. classification powder calcretes would normally classify as semi-sand clays, with the type specimens as sand clays.

8.7.7/ ...

8.7.7 Workability

All damp powder calcretes are easily worked (even those which are firm or stiff in situ) by hand picking and shovelling, dozing or excavating. Ripping is unnecessary and scraper loaders can probably load without predozing.

When dry, working may present some problems due to the fine cohesionless nature of the material.

Powder calcretes sometimes underly hard, intact hardpan which may require blasting. They tend to dissipate the blast if the shot holes are drilled right through the hardpan.

8.7.8 Acceptability as road materials

Table 8.6 represents an attempt to assess the acceptability of powder calcretes as road materials to some authorities.

The type materials 1515 and 2223 would probably be rejected entirely for use in surfaced roads on account of their high group indices, even though their CBRs and soil constants would fit them for subbase for 7,000 lb designs. They might be acceptable as fill to S.W.A., the FAA and the Texas Highway Department. Their grading moduli at 0.6 are too fine to pass even the S.W.A. hardening course (selected subgrade) grading modulus requirement of 0.8 let alone to qualify as subbase to any authority. These materials would be regarded as highly frost susceptible in areas overseas where such problems are prevalent, but this is not a problem in South Africa and in view of their generally high CBRs it seems likely that they are worthy of at least a place in the fill.

Most powder calcretes fail most grading requirements, though 8 out of the 10 tested passed the S.W.A. hardening course requirement. One (1777), an A-2-4(0) material,

would/ ...

TABLE 8.6 PROBABLE ACCEPTABILITY OF POWDER CALCRETES AS ROAD MATERIALS BY VARIOUS AUTHORITIES

SAMPLE NUMBER	S.A. DEPARTMENT OF TRANSPORT (1968)				TRANSVAAL ROADS DEPARTMENT			S.W.A. ROADS DEPARTMENT (OPERET-OSHAKATI CONTRACT)			U.S. FEDERAL AVIATION AGENCY			TEXAS HIGHWAY DEPARTMENT			ACCEPTABLE AS CHEMICALLY STABILIZED BASE ACCORDING TO:	ACCEPTABLE AS GRAVEL ROAD WEARING COURSE ACCORDING TO:
	BEST LAYER(1) IN SURFACED ROAD WHICH SAMPLE IS ACCEPTABLE, AS INDICATED BY THE FOLLOWING CRITERIA(2)																	
	GRADING	LL PI LS	GROUP INDEX	WHOLE SPEC.	GRADING	LL PI	WHOLE SPEC.	GRADING	PI	WHOLE SPEC.	GRADING	LL PI	WHOLE SPEC.	GRADING	LL PI	WHOLE SPEC.		
902	-	-	-	-	-	-	-	HC	-	Lower fill?	-	-	Lower fill?	-	-	Lower fill?	-	-
1512	-	SS	LF	Lower fill	-	-	Lower fill	HC	-	Fill?	-	CB	Fill?	-	CB	fill?	-	-
1515	-	B	-	-	-	B	-	-	B	Fill?	-	B	Fill?	-	B	Fill?	-	-
1557	-	SS	B	Sel. sub-grade	-	-	Sel.sub-grade	HC	-	Fill	S	-	Fill?	-	BRGB	Fill?	-	-
1696	-	-	F	Fill	-	-	Fill	HC	-	Fill	-	-	Fill?	-	-	Sel. sub-grade	-	-
1737	-	B	B	Sel. sub-grade	S	B	Subbase	S	B	Subbase	S	B	Sub-base	-	B	C.sub-base?	-	-
1881	-	SS	S	Sel. sub-grade	-	-	Fill	HC	-	Fill	-	CB	Sel. sub-grade?	-	BRGB	Sel. sub-grade?	-	-
2223	-	S	-	-	-	-	-	-	-	Fill?	-	CB	Fill?	-	B	Fill?	-	-
2236	-	S	B	Sel. sub-grade	-	S	Sel.sub-grade	HC	HC	Hardening	S	CB	C.sub-base?	-	B	Sel. sub-grade	-	-
2360	-	-	S	Fill	Sh	-	Fill	HC	-	Fill	S	-	Fill?	-	-	Sel. sub-grade?	-	-

Notes: (1) B = base, SB = stabilized base, BRGB = bank run gravel base (THD), CB = caliche base (THD or FAA), S = subbase, SS = selected subgrade, Sh = shoulder, HC = Hardening course, F = fill, LF = lower fill. The BRGB material is a slightly higher quality specification than CB material, while B alone means that the most severe base specification of the authority concerned is fulfilled.

(2) Whole spec. in most cases means indicator tests only.

would probably just be acceptable for subbase by four of the five authorities considered if there was nothing better at hand. This material almost makes Texas caliche base, failing on its grading. Normally, however, powder calcretes would not be acceptable as subbase and their place would be in the fill or, at most, selected subgrade.

Powder calcretes would not be regarded as suitable for stabilized base nor would they satisfy the requirements for gravel road wearing courses of the five authorities considered (D.O.T., Fossberg (1963), Transvaal, O.F.S. and S.W.A.). The PI of 1737 would be too low for the D.O.T. and that of 2360 too high. 1557 was actually used in 1965 for lime stabilized base or subbase for the Ondangua airfield, though it is possible that the actual material used may have been a little coarser than 1557 itself. Apart from being heavily cracked by stabilization-type cracking, the airfield has performed satisfactorily to date, though it has been subjected to little traffic. 902, 1512, 1515, 1696 and 1881 have all been used for gravel road wearing courses. The performance of 902 is unknown, while 1512 is said to make a good road. 1515 gave very poor results, tending to powder, and is no longer used. 1696 and 1881 were also unsatisfactory and had formed powder holes. Some powder calcretes (e.g. 1881) are known to perform satisfactorily for a while if well compacted with water, upon which a hard surface crust or 'blad' tends to form, protecting the material from degradation. Once this crust is broken disintegration and the formation of large powder potholes is rapid. 2236 and 2360 have been used, but their performance and place in the road foundation are unknown.

8.8 NODULAR CALCRETES

Nodular calcretes are probably the most useful group of calcretes for roadmaking purposes. By defining a

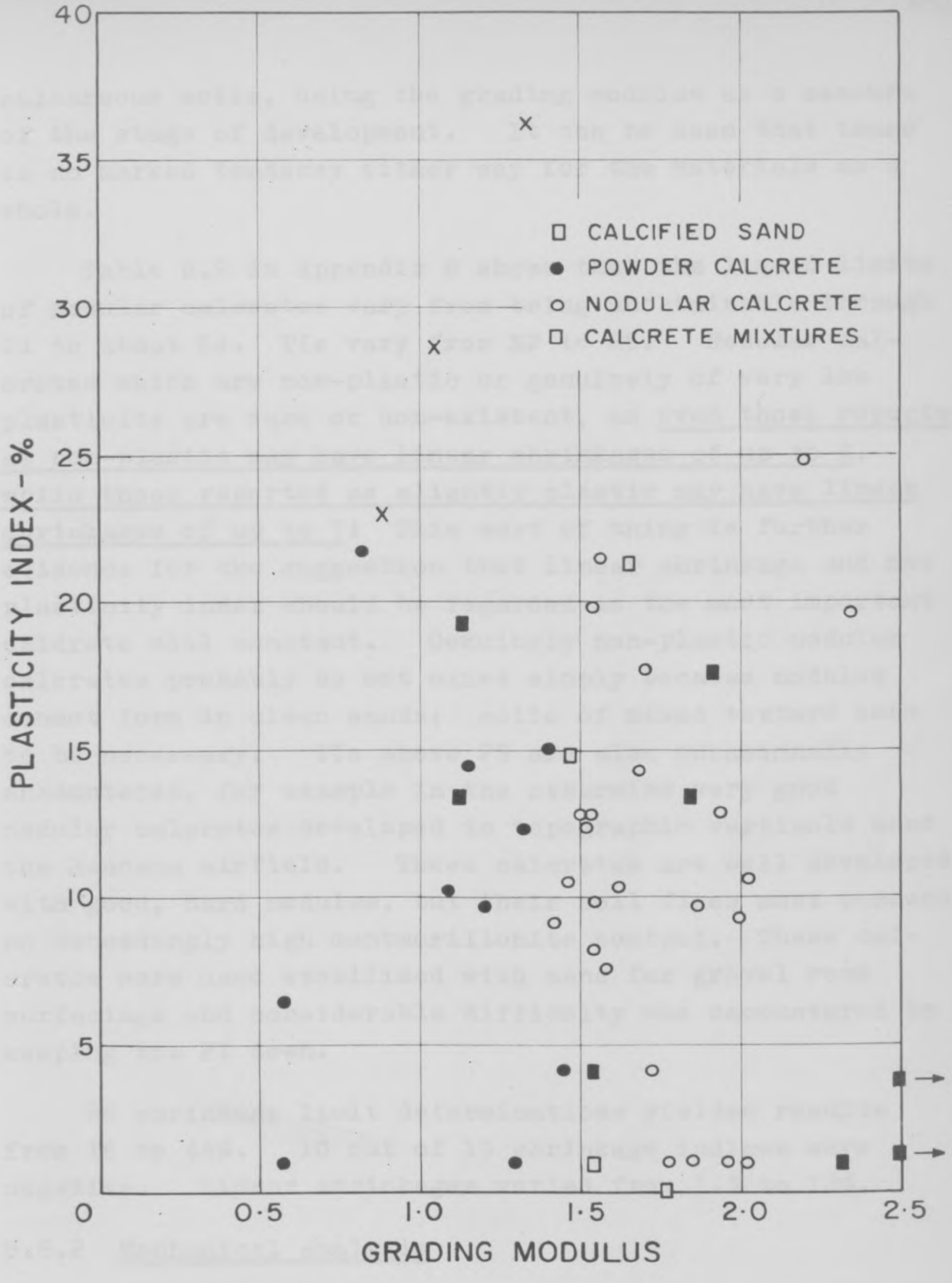


FIGURE 8·12

Effect of grading modulus on plasticity index

RG4 - 3204

calcareous soils, using the grading modulus as a measure of the stage of development. It can be seen that there is no marked tendency either way for the materials as a whole.

Table H.2 in Appendix H shows that the liquid limits of nodular calcretes vary from being unobtainable through 21 to about 64. PIs vary from NP to 25. Nodular calcretes which are non-plastic or genuinely of very low plasticity are rare or non-existent, as even those reported as non-plastic may have linear shrinkages of up to 4, while those reported as slightly plastic may have linear shrinkages of up to 7! This sort of thing is further evidence for the suggestion that linear shrinkage and not plasticity index should be regarded as the most important calcrete soil constant. Genuinely non-plastic nodular calcretes probably do not exist simply because nodules cannot form in clean sands; soils of mixed texture seem to be necessary. PIs above 25 are also occasionally encountered, for example in the otherwise very good nodular calcretes developed in topographic vertisols near the Ruacana airfield. These calcretes are well developed with good, hard nodules, but their soil fines must possess an exceedingly high montmorillonite content. These calcretes were used stabilized with sand for gravel road surfacings and considerable difficulty was encountered in keeping the PI down.

26 shrinkage limit determinations yielded results from 15 to 44%. 10 out of 19 shrinkage indices were negative. Linear shrinkages varied from 1.3 to 12%.

8.8.2 Mechanical analysis

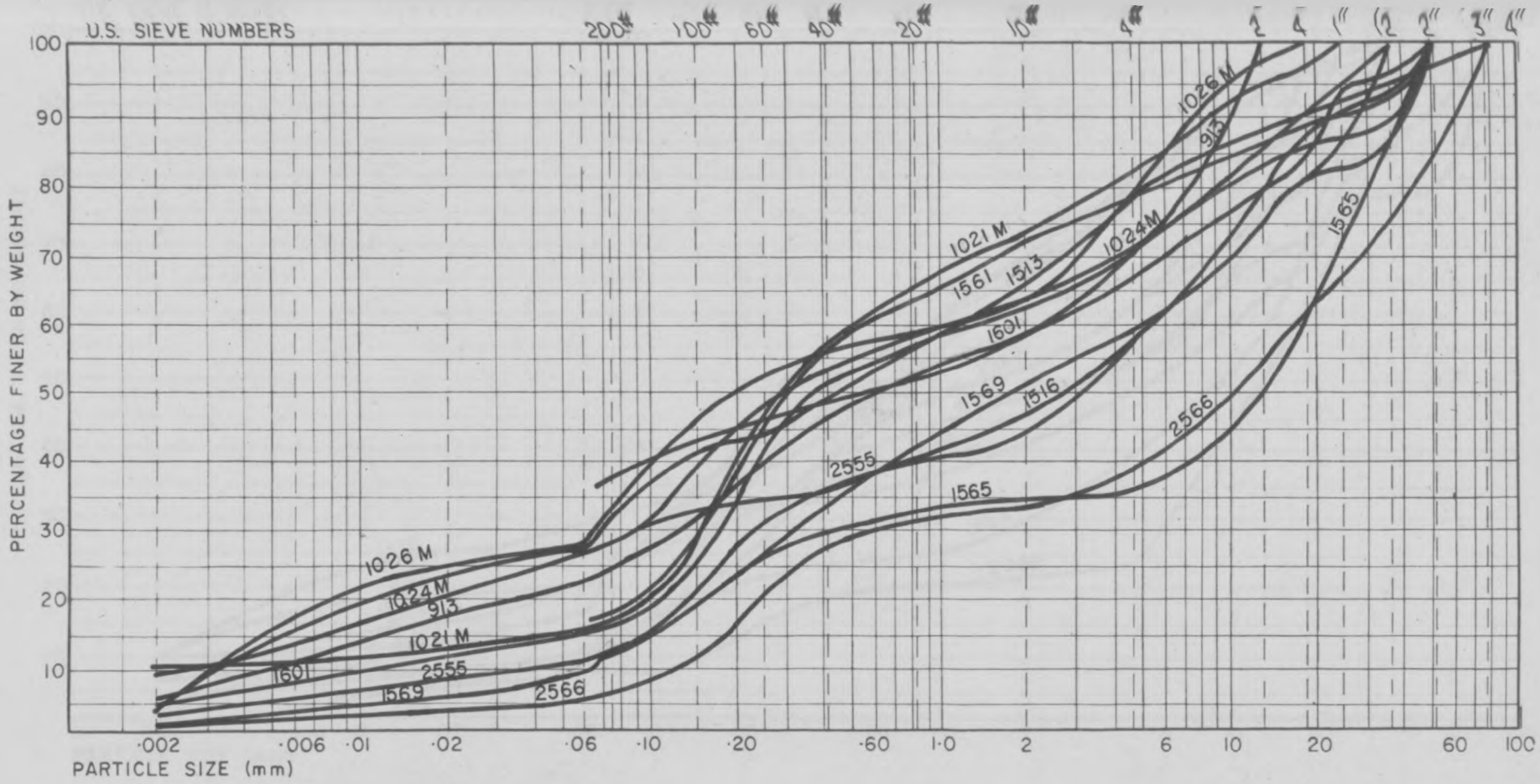
Mechanical analyses of the better nodular calcretes are probably more reliable than in the case of either powder calcretes or calcified sands. The aggregate is usually harder and Table J.8 suggests that the variation

in/ ...

in dry bulk particle specific gravity with particle size is also less, though once again, the coarse and medium sand fraction appears to be of lower SG. It will be shown in the next section that the crushing strength of nodules decreases with decreasing size, so that the finer aggregate and the coarser fines may still degrade during mechanical analyses. No strength measurements have been made on these finer particles, but examination under the microscope shows them to be composed of tiny sand and silt sized nodules composed of still finer particles cemented together by carbonate. These particles do not break down in water and can be moved about with a needle without disintegrating. Moderate pressure with a needle causes them to fracture into two or three pieces. On treatment with HCl the particles disintegrate to scores of smaller clay and silt-sized particles. The sand and silt-sized particles may well break down to some extent during soil preparation and mechanical analyses, but may possibly not break down in the road itself. The terms 'pseudosand' and 'pseudosilt' particles may possibly be justified when referring to these particles, though Dr. R. Webster (1967, personal comm.) who has experience of pseudosand and pseudosilt particles in Zambia regards the calcrete ones demonstrated to him as genuine sand and silt particles.

Figures 8.13 and 8.14 convey a good idea of the particle size distribution of nodular calcretes. There is no obvious difference between the shapes of the grading curves of fine nodular calcretes and those of the coarse nodular powder calcretes, the nodular calcretes are merely coarser all round and their curves lie to the right and below those of powder calcretes. The sand hunch with its deficiency of coarse sand (apparently deficient by weight only) and excess of fine is once more apparent. Samples 1569 and 1624 are the only ones which are reasonably well graded. The sharp step between 200 mesh and 0.060 mm is

probably/ ...

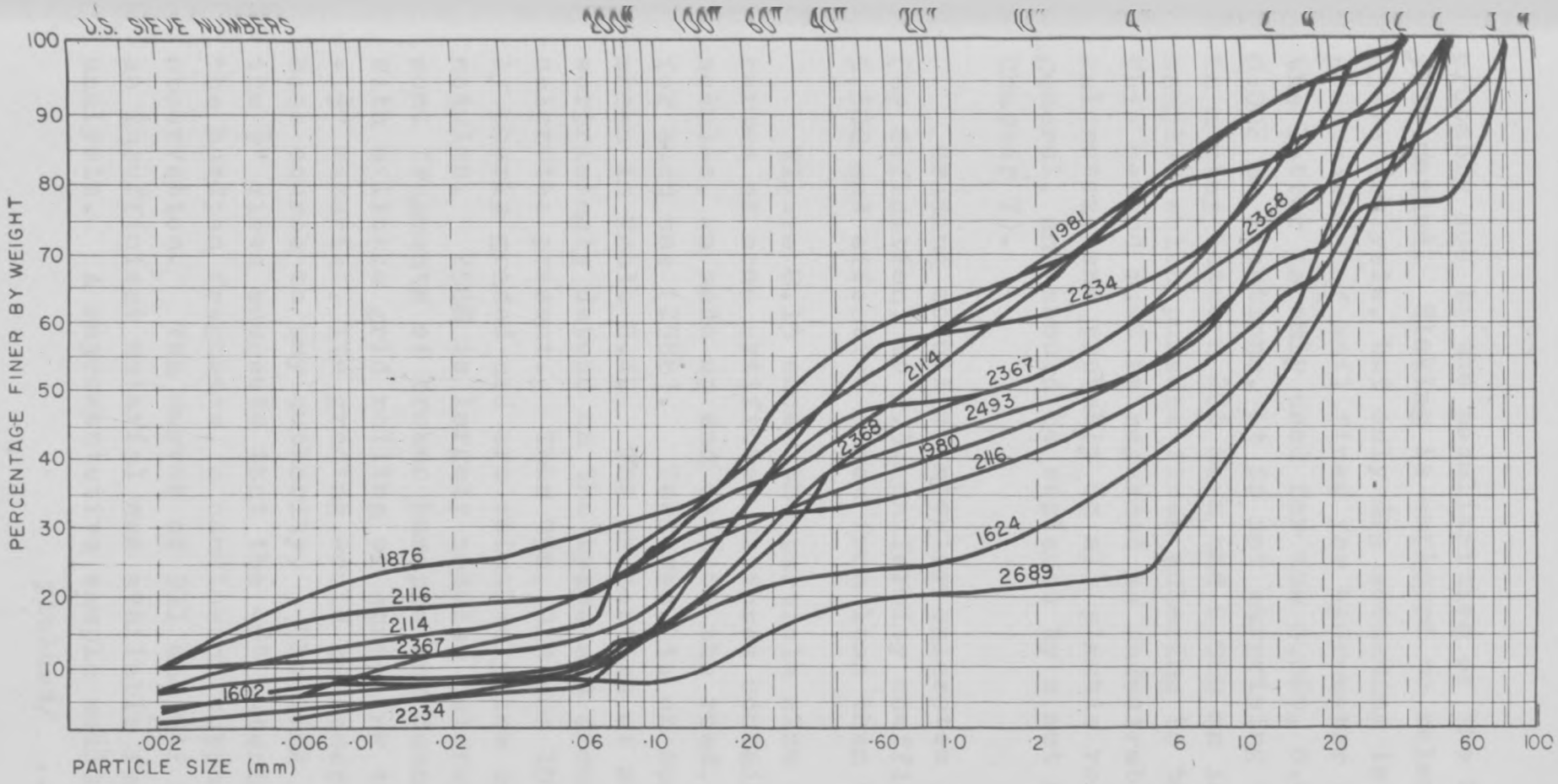


CLAY FRAC-TION	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE
	SILT FRACTION			SAND FRACTION			GRAVEL FRACTION		

FIGURE 8·13

Particle size distribution of nodular calcretes

RG4-3206



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

FIGURE 8.14

Particle size distribution of nodular calcretes

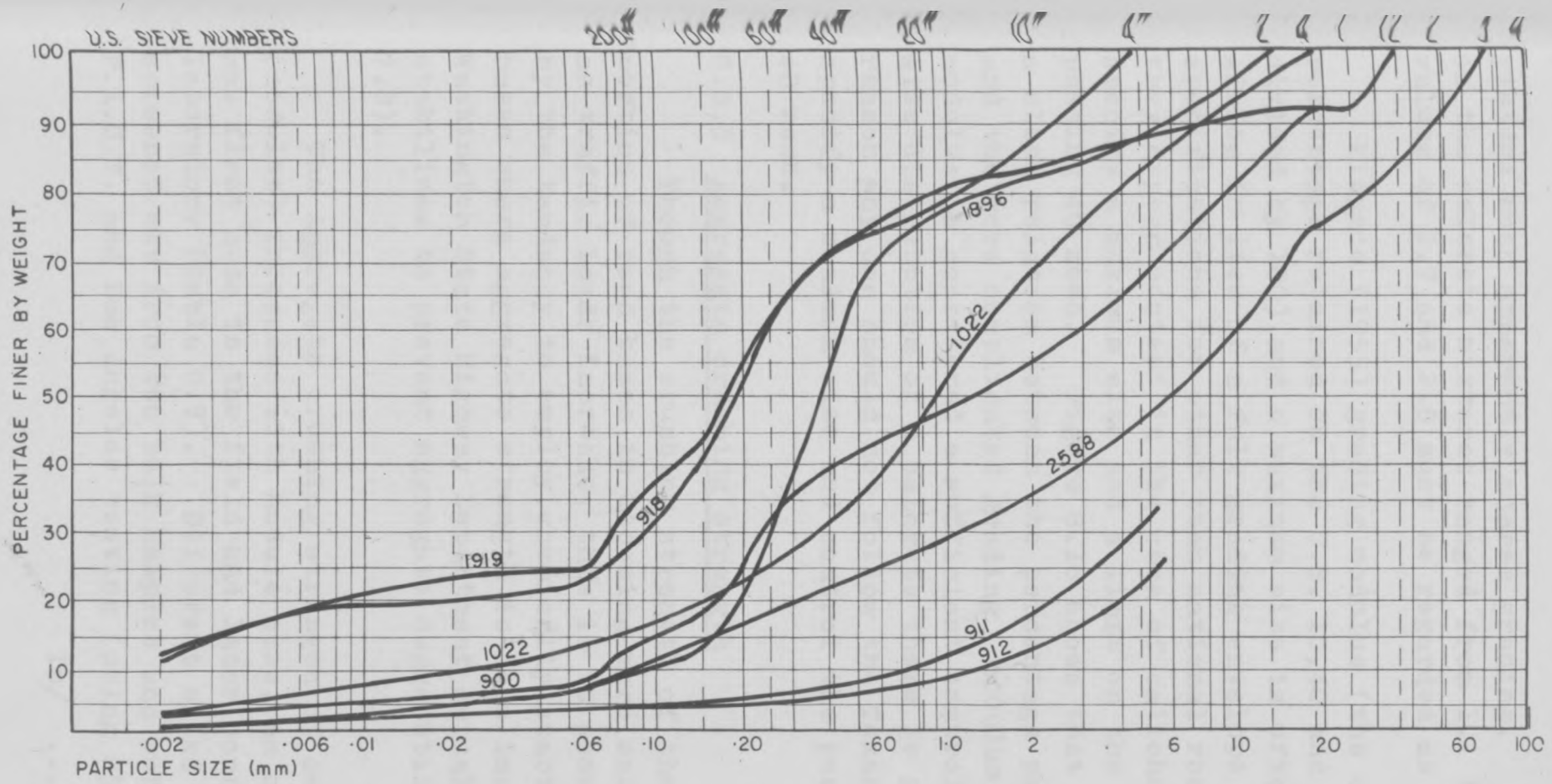
RG4-3205

probably due to the peculiarities of the NIRR mechanical analysis method. Slaking is employed to release the fines for the sieve analysis, but only dry screening is employed for the preparation of soil fines for hydrometer analysis. As the latter is only used for the 0.060, 0.020, 0.006 and 0.002 mm fractions, it is not surprising that it results in a step between 200 mesh and 0.060 mm in the case of samples which possess fines adhering to their aggregate. This method must be regarded as undesirable for use on calcretes, and probably on all plastic road materials in general, and should be replaced by a wet method (see Chapter 7).

Grading moduli of nodular calcretes range from 1.5 (by definition) to 2.3, uniformity coefficients from 22 to 5,500 and effective sizes from less than 0.001 to 0.18 mm.

Figure 8.15 shows the particle size distribution curves of some artificial mixtures containing calcrete nodules, as made up and used on the road, or as proposed for such use (2588). Test results on some mixtures are shown in Table H.4. The properties of mixtures not surprisingly depend on the types and amounts of the various calcretes present. Thus 896, 918 and 1019 are exceptionally poorly graded and are relatively low in calcrete nodules. 2588 is largely nodular calcrete, but with some fragments of broken hardpan and decomposed basalt. With a little grid rolling or screening to remove the +1½" material, its grading would be acceptable as natural base course to any authority. The kink in the curve on the ¾" sieve suggests that the +¾" material is made up of the hardpan fragments, a conclusion supported by a visual observation. The curves of 911 and 912 are incomplete, as insufficient material was available for a reliable analysis. A representative sample would not have

yielded/ ...



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

FIGURE 8·15

Particle size distribution of some calcrete mixtures

RG4-3207

yielded such apparently coarse grading. Grading moduli of the calcrete mixtures ranged from 1.1 to 2.3. The two values of 2.7 and 2.8 must be regarded as doubtful.

Kleyn's (1955) grading modulus (the cumulative percentage retained on the U.S. 10,40 and 200 mesh sieves divided by 100) and a maximum size is often used in South Africa in lieu of a full grading envelope when writing specifications for other than national roads. In Texas they appear content in the case of caliche bases to specify a maximum size and a limit on the percentage passing 40 mesh. Figure 8.16 shows that there is in fact a close relation between the percentage passing 40 mesh and the more complicated grading modulus. If a grading modulus is considered a sufficient control on the particle size distribution of a calcrete there is probably no reason why one should not follow the Texas lead and merely specify a maximum size and control the percentage passing 40 mesh.

8.8.3 Aggregate crushing strength

Though the crushing strength of the aggregate portion of road bases is becoming more and more important as traffic loads increase, this is to some extent offset by the tendency to employ chemically stabilized, semi-rigid bases where aggregate strength is less important. The Washington State Highway Department actually cement stabilizes to prevent aggregate degradation (see Section 7.8).

The aggregate crushing strength of calcrete aggregate (nodules) increases with nodule size, an observation which was first made in the field and later confirmed in the laboratory (Table 8.7). Different sized nodules were screened out from two bulk samples and subjected to 10% F.A.C.T. and Los Angeles testing (using 2500 g of material).

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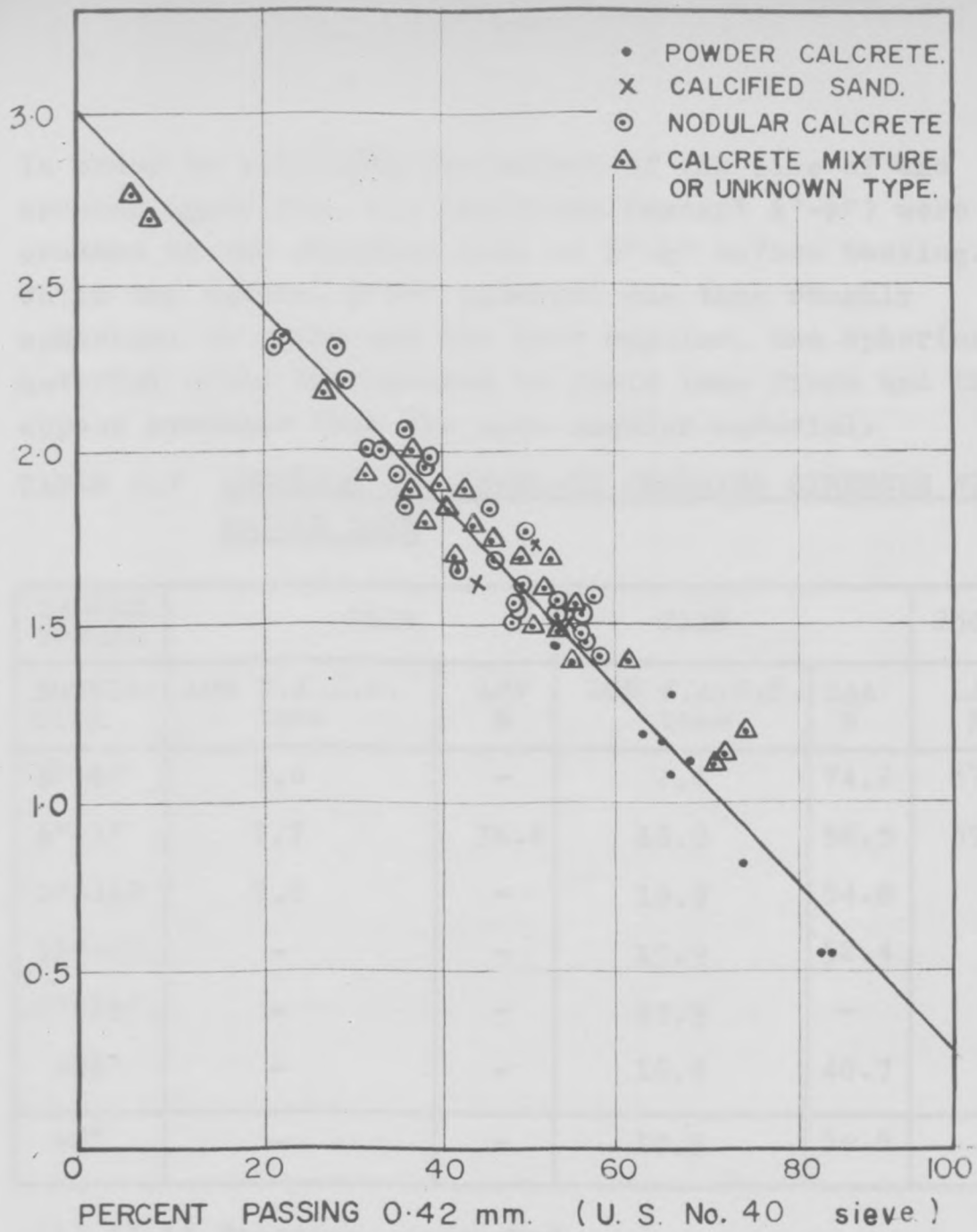


FIGURE 8:16

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

RG4-3659

In order to eliminate the effect of the size of the crushed aggregate, all fractions (except $\frac{3}{8}$"-$\frac{1}{2}$") were crushed to the standard size of $\frac{3}{8}$"-$\frac{1}{2}$" before testing. While the natural $\frac{3}{8}$"-$\frac{1}{2}$" material was thus roughly spherical in shape and the rest angular, the spherical material could be expected to yield less fines and thus appear stronger than the more angular material.

TABLE 8.7 INCREASE OF AGGREGATE CRUSHING STRENGTH WITH NODULE SIZE

SAMPLE NUMBER	2114		2116		2588
NODULE SIZE	10% F.A.C.T. tons	ACV %	10% F.A.C.T. tons	LAA %	LAA %
$\frac{3}{8}$"-$\frac{1}{2}$"	3.6	-	7.4	74.2	67.8
$\frac{1}{2}$"-1"	7.7	36.6	13.0	56.5	59.8(1)
1"-1$\frac{1}{2}$"	7.5	-	13.9	54.8	-
1$\frac{1}{2}$"-2"	-	-	15.9	52.4	-
2"-2$\frac{1}{2}$"	-	-	13.9	-	-
+2$\frac{1}{2}$"	-	-	16.8	46.7	-
+$\frac{3}{4}$"	-	-	10.5	54.8	-

(1) $\frac{3}{4}$"-1" fraction, uncrushed
Operator: Mr. T.E. O'Connor

It is apparent that the strength of calcrete nodules does indeed increase with size. Two discontinuities are, however, apparent, one at 1"-1$\frac{1}{2}$" in 2114 and one at 2"-2$\frac{1}{2}$" in 2116. Only one determination was carried out in the first case so that this may be due to some experimental error. In the second case, determinations were made in triplicate, values of 13.0, 13.3 and 15.5 tons being obtained. Either by a rare chance 15.5 tons is the correct value or the 1$\frac{1}{2}$"-2" and 2"-2$\frac{1}{2}$" fractions have been

accidentally/ ...

accidentally interchanged. It is a pity that a Los Angeles value was not determined on the 2"-2½" fraction. It is also interesting to note that the value obtained by screening out all the +¾" material and crushing it to ⅜"-½" does not lie above the expected 13.0 tons. Dr. H.H. Weinert (1968, personal comm.) has suggested that this may be due to a crushing effect of stronger aggregate against weaker. This may well indeed be the reason, for the LAA at 500 revolutions (54.8%) is rather less than the theoretical value calculated from the 100 revolution value (5 x 14.8 = 74.0), indicating that the +¾" fraction of 2116 was variable in aggregate strength. The LAA of the +¾" fraction is more or less what might be expected. In this test the stronger aggregate almost certainly does have a milling effect on the weaker. It is of interest to note that the LAA of the two uncrushed fractions of 2588 also showed a tendency to improve with size.

Not only does the aggregate strength of calcrete nodules vary with size fraction, but it also varies with the stage of development of the calcrete. Figure 8.17 shows that ½"-¾" nodules tend to increase in strength with development (using the grading modulus as a measure of the stage of development). As will be seen presently, honeycomb and hardpan calcretes tend to yield still better aggregate crushing strengths than nodular and powder calcretes.

Dry 10% F.A.C.T. values of ½"-¾" calcrete nodules range from about 1 to 17 tons, though 5-15 tons would be a more usual range. Measured ACVs range from 26 to 57%, AFVs from 0 to 100 and APVs from 0 to 77%. The latter two tests are very simple and convenient field or field laboratory tests and correlate fairly well with the dry 10% F.A.C.T. (Section 7.8.3). A few miscellaneous crushing test results will be found in Appendix J.

Some/ ...

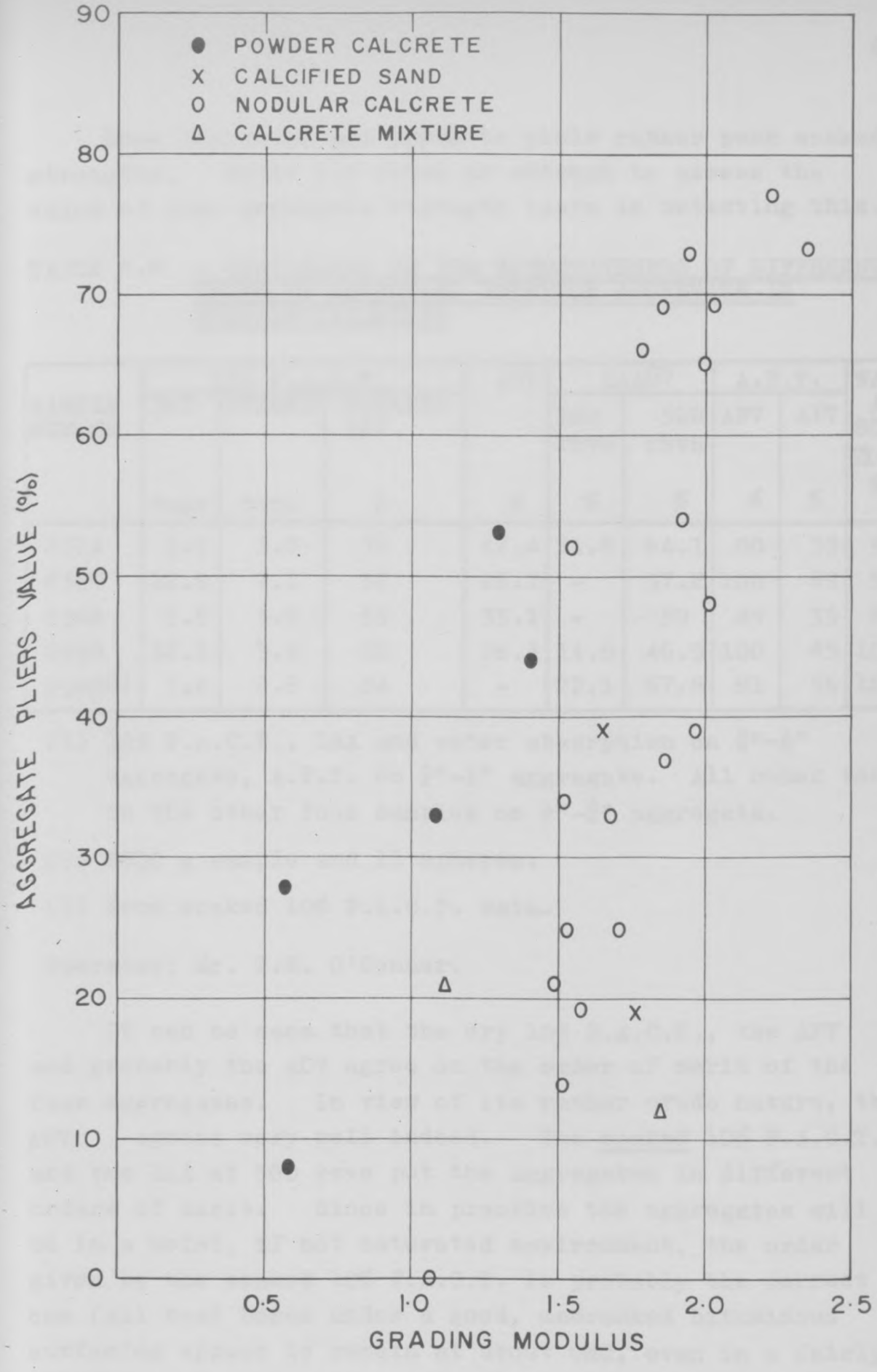


FIGURE 8.17

EFFECT OF GRADING MODULUS ON AGGREGATE STRENGTH

RG4-3767

Some calcretes are prone to yield rather poor soaked strengths. Table 8.8 shows an attempt to assess the value of some aggregate strength tests in detecting this.

TABLE 8.8 A COMPARISON OF THE EFFECTIVENESS OF DIFFERENT TESTS IN DETECTING INFERIOR AGGREGATE IN NODULAR CALCRETES

SAMPLE NUMBER	10% F.A.C.T.			ACV	LAA(2)		A.P.T.		WATER ABSORPTION(3)
	DRY	SOAKED	SOAKED/DRY		100 revs	500 revs	AFV	APV	
	tons	tons	%	%	%	%	%	%	%
2114	4.9	1.7	35	42.4	21.8	64.1	80	33	9.3
2367	12.5	7.1	57	26.7	-	37.2	100	69	5.4
2368	5.5	3.0	55	35.1	-	~50	89	39	8.4
2493	12.1	3.9	32	26.3	14.0	46.3	100	65	10.2
2588(1)	7.6	2.6	34	-	22.1	67.8	81	56	16.9

(1) 10% F.A.C.T., LAA and water absorption on $\frac{3}{8}$"-$\frac{1}{2}$" aggregate, A.P.T. on $\frac{3}{4}$"-1" aggregate. All other tests on the other four samples on $\frac{1}{2}$"-$\frac{3}{4}$" aggregate.

(2) 2500 g sample and 11 spheres.

(3) From soaked 10% F.A.C.T. data.

Operator: Mr. T.E. O'Connor.

It can be seen that the dry 10% F.A.C.T., the APV and probably the ACV agree on the order of merit of the four aggregates. In view of its rather crude nature, the APV agrees very well indeed. The soaked 10% F.A.C.T. and the LAA at 500 revs put the aggregates in different orders of merit. Since in practice the aggregates will be in a moist, if not saturated environment, the order given by the soaked 10% F.A.C.T. is probably the correct one (all road bases under a good, uncracked bituminous surfacing appear to remain at about OMC, even in a fairly dry climate). The percentage wet to dry strength is

also/ ...

also revealing. It is interesting to note that even a simple water absorption test (a measure of the porosity), while not placing the aggregates in the supposedly correct order, does at least show up aggregates with inferior wet strengths. In 1943 Walker and Stewart concluded that the two simple tests Treton and water absorption were probably all that were required for the evaluation of aggregates.

Another way of assessing the aggregate strength of nodular calcretes is by determining their hardness on the Mohs scale, as carried out by Netterberg (1967) for hardpan and boulder calcretes (see Section 7.9). The applicability of the correlation (Figures 7.15 and 7.16) to nodules has not yet been fully assessed. The difference in aggregate shape between the crushed hardpans and boulders used in the preparation of Figures 7.15 and 7.16 and the predominantly spherical shape of nodules might influence the test results to some extent. Figures 7.15 and 7.16 can, however, be presumed to apply at least approximately in the case of nodular calcretes. The hardness test has the disadvantages that a certain amount of practice and care are required to obtain good results and that it is unreliable on very sandy calcretes. It therefore has its limitations. The A.P.T. on the other hand, is virtually foolproof.

Unfortunately, no sulphate soundness tests have been carried out on calcrete nodules, but it is believed that they perform well in this test.

8.8.4 Specific gravity and water absorption

The solid particle specific gravity of the minus 40 mesh fractions of 29 nodular calcretes ranged from 2.50 to 2.80. Values for 8 of the whole samples crushed and ground minus 40 mesh ranged from 2.62 to 2.71, tending to be greater than the minus 40 mesh values, probably due to the tendency of the carbonate to be higher in the coarser fractions. Figures for the variation of apparent, dry and saturated surface dry bulk specific gravities with size fraction are given in Table J.8. These properties can be seen to be

variable/ ...

variable, though apparently less so than in the case of powder calcretes. Bulk specific gravities are still nowhere near the 2.65 normally assumed for the average road material and water absorptions are still high, values ranging between 5 and 10% being perhaps the most common.

8.8.5 California bearing ratio

Only 5 CBR results are available on the materials sampled and these range from 38 to about 110. Material from the same horizon as Sample 2555 yielded a CBR of 83, while other apparently nodular calcretes also proposed for use on the new Karasburg-Upington road yielded CBRs varying from 78 to 159. A few exceptionally high values up to 239 were also recorded (test results kindly supplied by Mr. J.A. Caiger). It is apparent that calcretes of exceptionally high quality exist in this area. There is also evidence that some of them are self-stabilizing, which might account for some of the exceptionally high CBRs (see Section 8.13). Nodular calcretes should yield CBRs from about 40 upwards, in other words, on the basis of their CBRs they are usually suitable as subbase and in some cases as base for 12,000 lb wheel load designs. OMCs of the 5 calcretes ranged from 7 to 14% and MDDs from 114 to 135 lbs/cu ft. Examination of the soil survey charts of several road contracts in the Republic and South West Africa suggests that the above ranges are reasonable for nodular calcretes as a group. When unstabilized, nodular calcretes and possibly calcretes in general normally yield higher CBRs at 0.2" penetration than at 0.1" penetration. The reverse is normally the case with cement or lime stabilized specimens. This is probably due to the lower density normally achieved in the latter case.

Calcretes on the whole tend to yield good CBRs in spite of their usually rather poor (apparent) gradings and relatively high (apparent) PIs. They often in fact pass for subbase or even base course on their CBRs, but are rejected on account of their excessive PIs and poor gradings. These latter two tests are unreliable and

more/ ...

more weight should be attached to the CBR.

In 1934 Gillette stated that "the stability of caliche base courses for bituminous surfacings depends upon the quality and quantity of the fine binder material (passing the no. 40 sieve) contained in the total volume of material in place." Figure 8.18 shows that the quality and quantity of the soil fines does indeed exert a strong influence on the CBR of South African calcretes. It would appear that merely specifying that the bar linear shrinkage times the percentage passing 40 mesh shall not exceed 200 would ensure a CBR of at least 60 in about 94% of cases and a CBR of at least 80 in about 74% of cases. Only a few calcretes with CBRs of more than 60 and very few calcretes with CBRs of more than 80 would be rejected by this procedure. Most but not all of the calcretes used to construct Figure 8.18 were nodular calcretes. Values of linear shrinkage times percent passing 40 and 200 mesh for some calcretes are shown in Table J.14.

The CBR swell of calcretes in general is usually very low. Calcretes yielding subbase or better CBRs rarely exhibit swells of more than 0.2%.

8.8.6 Engineering classification

Most nodular calcretes are A-2 materials, though they range from A-1-a(0) down to A-6(3). Of the 3 samples classified, only two were inferior to A-2-7(1) (1026 and 1514). Apart from 1514, group indices range from 0 to 1. In the Casagrande classification, most nodular calcretes would class as GF (gravel-sand mixtures with an excess of fines) while the better ones usually 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. In the D.O.T. classification most nodular calcretes would be

semi-sand/ ...

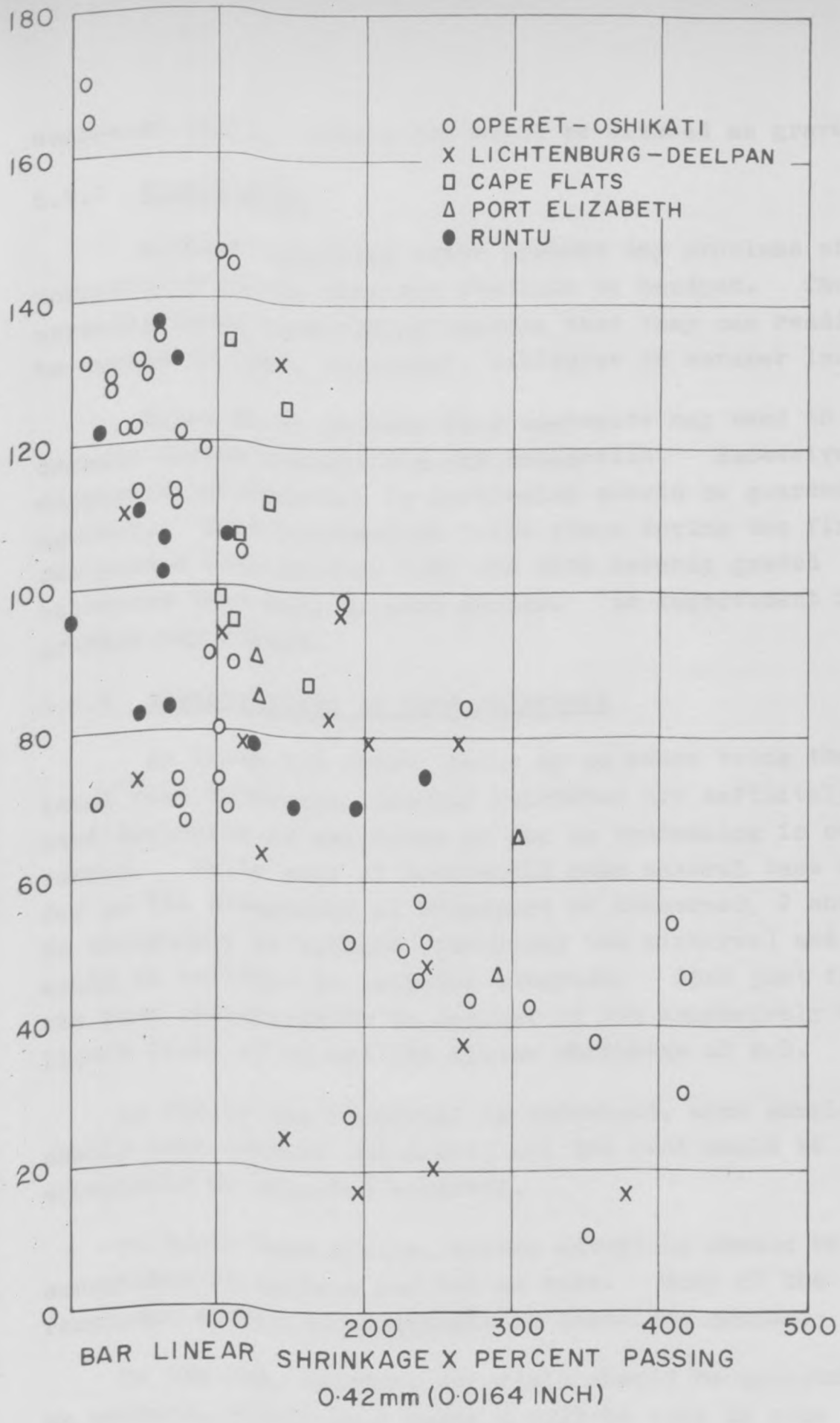


FIGURE 8·18

EFFECT OF THE QUALITY AND QUANTITY OF THE SOIL FINES ON THE CBR OF CALCRETES.

RG4–3768

semi-sand clays. Only a few would be classed as gravels.

8.8.7 Workability

Nodular calcretes never present any problems of workability unless they are overlain by hardpan. Their normally loose consistency ensures that they can readily be worked by hand, excavator, bulldozer or scraper loader.

Types which possess weak aggregate may tend to degrade during stockpiling and compaction. Excessive compaction degradation in particular should be guarded against. Most degradation takes place during the first few passes (see Section 7.8) and with harshly graded calcretes this will be most severe. An improvement in grading may result.

8.8.8 Acceptability as road materials

As Table 8.9 shows, while by no means being the ideal road materials, nodular calcretes are definitely the most desirable of calcretes as far as roadmaking is concerned. While none of them would make natural base as far as the Department of Transport is concerned, 2 should be acceptable as subbase (including the mixtures) and many would be suitable as selected subgrade. 2588 just fails the base specification on account of its excessively high liquid limit of 37 and its linear shrinkage of 4.0.

As far as the Transvaal is concerned, nine samples should make subbase and nearly all the rest would be acceptable as selected subgrade.

In South West Africa, eleven materials should be acceptable as subbase and two as base. Many of the remainder should be acceptable as hardening course.

To the FAA, thirteen materials should be acceptable as subbase, albeit only under a caliche base in some

cases/ ...

cases, and four as caliche base.

Twenty one materials should make caliche subbase in Texas, twelve caliche base, and four should be acceptable in any base.

It is apparent that the authorities are also arranged in order of their acceptance of calcretes as road materials. They are very much more acceptable in Texas than they are to the S.A. Department of Transport for example. The arrangement probably also coincides with the amount of experience each authority has of building blacktop roads or airfields of calcretes. It may be significant then that Texas and the FAA relax their grading, soil constants and aggregate strength requirements when using these materials. The arguments in the foregoing sections and in Chapter 7 are thus apparently strongly supported by overseas practice. Economic considerations also play a part of course and it is known that the FAA and Texas caliche base specifications are not normally applied to materials to be used as base for their heaviest designs, although they have successfully employed the best caliches for this purpose; their superior base specifications are also not as severe as those of the S.A. Department of Transport, for example. Specifications will be considered further in Chapter 9.

The second last column in Table 8.9 assesses the acceptability of nodular calcretes and mixtures as stabilized base to the D.O.T., Transvaal, S.W.A. and a specification combination of Fossberg and Gregg's (1963) grading requirements and an arbitrary limit of 50 on the liquid limit and 25 on the PI (denoted by F.G.H.) Soil constants before stabilization are not normally specified, but figures of maximum LL 40-50 and maximum PI 18-25 have been advised by various authorities. The higher figures appear in the Highway Research Board (1961) Bull. No. 292

and/ ...

and the lower are those of the Road Research Laboratory (1952). While it is definitely realistic to specify maximum Atterberg limits when stabilizing clays (mainly due to the difficulty of mixing and pulverising heavy clays), higher Atterberg limits than those given above would be allowable when dealing with small quantities of very active clay in gravelly soils. It can be seen that 20 out of the 30 samples tested should be acceptable for chemically stabilized base to at least one authority in each case. Nine of these should be acceptable to two or more authorities.

As far as a gravel road wearing course is concerned, about 16 of the materials should be acceptable to at least one authority in each case. A material suitable for base course is of course not necessarily suitable for wearing course and vice versa.

The actual uses to which the materials are put or have been put is shown in the final column.

TABLE 8.9 PROBABLE ACCEPTABILITY OF NODULAR CALCRETES AS ROAD MATERIALS BY VARIOUS AUTHORITIES

SAMPLE NUMBER	S.A. DEPT. OF TRANSPORT (1968) (D.O.T.)				TRANSVAAL ROADS DEPARTMENT (T)			S.W.A. ROADS DEPT. (OPERET-OSHAKATI CONTRACT) (S.W.A.)			U.S. FEDERAL AVIATION AGENCY (FAA)			TEXAS HIGHWAY DEPARTMENT (THD)			ACCEPTABLE AS CHEMICALLY STABILIZED BASE TO: (3)	ACCEPTABLE AS GRAVEL ROAD WEARING COURSE TO: (4)	PRESENT USE OR REMARKS
	BEST LAYER(1) IN SURFACED ROAD FOR WHICH SAMPLE IS ACCEPTABLE, AS INDICATED BY THE FOLLOWING CRITERIA(2)																		
	GRADING	LL PI LS	GROUP INDEX	WHOLE SPEC.	GRADING	LL PI	WHOLE SPEC.	GRADING	PI	WHOLE SPEC.	GRADING	LL PI	WHOLE SPEC.	GRADING	LL PI	WHOLE SPEC.			
913	-	-	S	Lower fill?	S	-	Fill	SB	-	Fill	S	-	Fill?	CB	-	Fill?	S.W.A.	F	Satisfactory wearing course
1021M	-	-	S	Fill	S	-	Fill	S	-	Fill	S	-	Fill?	-	-	Sel. Subgrade?	-	-	Satisfactory wearing course
1024	-	S	B	Sel. Subgrade	S	-	Sel. Subgrade	SB	HC	Hardening	S	CB	C.Subbase?	-	BRGB	C.Subbase?	S.W.A.	S.W.A.	Very good wearing course
1024M	-	S	B	Sel. Subgrade	S	-	Sel. Subgrade	S	HC	Hardening	S	CB	C.Subbase?	-	BRGB	C.Subbase?	-	S.W.A.	Very good wearing course
1026M	-	SS	B	Sel. Subgrade	-	-	Sel. Subgrade	HC	HC	Hardening	-	-	Sel. Subgrade?	-	BRGB	C.Subbase?	-	S.W.A.	Very good wearing course
1513	-	-	S	Fill	S	-	Fill	S	-	Fill	-	-	Sel. Subgrade?	CB	-	C.Subbase?	-	-	Good wearing course
1514	-	-	LF	Lower fill	S	-	Lower fill	S	-	Lower fill	-	-	Fill?	CB	-	Fill?	-	-	Excellent wearing course
1516	-	S	B	Sel. Subgrade	S	-	Sel. Subgrade	SB	S		-	B	Sel. Subgrade?	CB	BRGB	C.base	S.W.A.	S.W.A., F.	Satisfactory wearing course.
1561	-	S	B	Sel. Subgrade	S	-	Sel. Subgrade	S	HC	Subbase Hardening	-	CB	Sel. Subgrade?	-	BRGB	C.subbase?	-	S.W.A.	Lime stabilized base or subbase
1565	-	-	B	Lower fill	B	-	Sel. Subgrade	B	-	Lower fill?	CB	-	Lower fill?	BRGB	-	Lower fill?	T,S.W.A., F.G.H.	-	Gravel road
1569	S	S	B	Subbase	SB	B	Subbase	SB	B	Subbase	CB	B	C.base	BRGB	B	Base	T,S.W.A., F.G.H.	D.O.T.	Lime stabilized subbase
1601	-	S	B	Sel. Subgrade	S	B	Subbase	SB	B	Subbase	CB	CB	C.base	CB	BRGB	C.base	S.W.A.	S.W.A.	Lime stabilized base or subbase
1602	-	-	B	Fill	S	-	Sel. Subgrade	S	-	Fill	S	-	Sel. Subgrade?	-	-	C.subbase?	-	-	Lime stabilized base or subbase
1624	B	-	B	Lower fill	B	-	Sel. Subgrade	B	-	Lower fill	B	-	Lower fill?	PGB	-	Fill?	D.O.T., T,S.W.A., F.G.H.	T,F, O.F.S.	Lime stabilized subbase
1876	-	S	B	Sel. Subgrade	S	-	Sel. Subgrade	SB	HC	Hardening	S	CB	C.subbase	CB	BRGB	C.base	S.W.A.	T?,F,O.F.S.	Originally classified in field as powder calcrete

TABLE 8.9 PROBABLE ACCEPTABILITY OF NODULAR CALCRETES AS ROAD MATERIALS BY VARIOUS AUTHORITIES (CONT.)

SAMPLE NUMBER	S.A. DEPT. OF TRANSPORT (1968) (D.O.T.)				TRANSVAAL ROADS DEPARTMENT (T)			S.W.A. ROADS DEPT. (OPERET-OSHAKATI CONTRACT) (S.W.A.)			U.S. FEDERAL AVIATION AGENCY (FAA)			TEXAS HIGHWAY DEPARTMENT (THD)			ACCEPTABLE AS CHEMICALLY STABILIZED BASE TO: (3)	ACCEPTABLE AS GRAVEL ROAD WEARING COURSE TO: (4)	PRESENT USE OR REMARKS
	BEST LAYER(1) IN SURFACED ROAD FOR WHICH SAMPLE IS ACCEPTABLE, AS INDICATED BY THE FOLLOWING CRITERIA(2)																		
	GRADING	LL PI LS	GROUP INDEX	WHOLE SPEC.	GRADING	LL PI	WHOLE SPEC.	GRADING	PI	WHOLE SPEC.	GRADING	LL PI	WHOLE SPEC.	GRADING	LL PI	WHOLE SPEC.			
1980	-	-	B	Fill	S	-	Sel. Sub-grade	SB	-	Fill	S	-	Sel. Sub-grade?	BRGB	-	C.sub-base?	S.W.A.	T,F, O.F.S.	Some broken hardpan present
1981	-	-	S	Lower fill?	S	-	Fill	SB	-	Lower fill?	S	-	Lower fill?	CB	-	Fill?	S.W.A.	-	-
2114	-	-	B	Fill	S	-	Sel. Sub-grade	SB	-	Fill	S	-	Sel. Sub-grade	CB	-	C.sub-base?	S.W.A.	T,F, O.F.S.	Substandard cement stabilized base
2116	-	SS	B	Sel. Sub-grade	S	-	Sel. Sub-grade	SB	HC	Harde-ning	S	-	Sel. Sub-grade	PGB	BRGB	Base	S.W.A.	T,F, S.W.A.	Substandard cement stabilized base
2234	-	B	B	Sel. Sub-grade	S	B	Sub-base	SB	B	Sub-base	S	B	Sub-base	CB	B	C.base	S.W.A.	-	Lime stabilized base?
2367	-	B	B	Sel. Sub-grade	S	B	Sub-base	SB	B	Sub-base	S	B	Sub-base	CB	B	C.base	S.W.A.	-	Lime stabilized base
2368	-	B	B	Sel. Sub-grade	SB	B	Sub-base	SB	B	Sub-base	S	B	Sub-base	CB	B	C.base	T,S.W.A., F.G.H.	-	Lime stabilized base
2493	-	S	B	Sel. Sub-grade	SB	Sh	Shoul-der	SB	S	Sub-base	S	CB	C.sub-base	CB	CB	C.base	T,S.W.A., F.G.H.	T,F, S.W.A.	Lime stabilized base
2555	-	B	B	Sel. Sub-grade	SB	B	Sub-base	SB	B	Sub-base	CB	B	C.base	BRGB	B	Base?	T,S.W.A., F.G.H.	-	Proposed base
2566	-	B	B	Sel. Sub-grade	SB	B	Sub-base	B	B	Base	B	B	Base	BRGB	B	Base	T,S.W.A., F.G.H.	-	Proposed base
MIXTURES OF NODULAR CALCRETES AND OTHER MATERIALS																			
896	-	B	B	Sel. Sub-grade	S	B	Sub-base	S	B	Sub-base	S	B	Sub-base	-	B	Sel. Sub-grade?	-	-	Gravel road
900	-	-	B	Lower fill	SB	-	Fill?	SB	-	Fill?	S	-	Fill?	CB	-	C.sub-base?	T,S.W.A.	F,T?	Gravel road
918	-	-	F	Lower fill	-	-	Lower fill?	HC	-	Lower fill?	-	-	Lower fill?	-	-	Fill?	-	-	Gravel road

TABLE 8.9 PROBABLE ACCEPTABILITY OF NODULAR CALCRETES AS ROAD MATERIALS BY VARIOUS AUTHORITIES (CONT.)

SAMPLE NUMBER	S.A. DEPT. OF TRANSPORT (1968) (D.O.T.)				TRANSVAAL ROADS DEPARTMENT (T)			S.W.A. ROADS DEPT. (OPERET-OSHAKATI CONTRACT) (S.W.A.)			U.S. FEDERAL AVIATION AGENCY (FAA)			TEXAS HIGHWAY DEPARTMENT (THD)			ACCEPTABLE AS CHEMICALLY STABILIZED BASE TO: (3)	ACCEPTABLE AS GRAVEL ROAD WEARING COURSE TO: (4)	PRESENT USE OR REMARKS
	BEST LAYER(1) IN SURFACED ROAD FOR WHICH SAMPLE IS ACCEPTABLE, AS INDICATED BY THE FOLLOWING CRITERIA(2)																		
	GRADING	LL PI LS	GROUP INDEX	WHOLE SPEC.	GRADING	LL PI	WHOLE SPEC.	GRADING	PI	WHOLE WPEC.	GRADING	LL PI	WHOLE SPEC.	GRADING	LL PI	WHOLE SPEC.			
1019	-	-	S	Fill	-	-	Fill	HC	-	Fill	-	-	Fill?	-	-	Sel. Sub-grade?	-	-	Satisfactory wearing course
2588	B	S	B	Sub-base	B	S	Sub-base	B	B	Base	B	-	C.sub-base?	PGB	CB	C.base	D.O.T., T, S.W.A., F.G.H.	D.O.T.?	Possible lime stabilized base

NOTES: (1) B = base, PGB = processed gravel base (THD) SB = stabilized base, BRGB = bank run gravel base (THD), CB = caliche base (THD or FAA), S = subbase, SS = selected subgrade, Sh = shoulder, HC = hardening course, F = fill, LF = lower fill.

(2) Whole spec. in most cases means indicator tests only.

(3) For explanation of F.G.H. see text.

(4) F = Fossberg (1963), O.F.S. = Orange Free State Roads Dept.

8.9 HONEYCOMB CALCRETES

Relatively little is known of the properties of honeycomb calcretes and only three have been tested in the course of this project (Table H.3).

The two samples tested yielded good aggregate crushing strengths. 10% F.A.C.T. values were 16 and 23 tons (11 and 16 tons soaked) while the ACVs were 24 and 16% respectively. The 2500g LAA values show sample 2494 to be very uniform and are the best yet obtained on a calcrete, as is the 10% F.A.C.T. value of 23 tons. Water absorption at 2% is about the average for a good hardpan calcrete. Dry bulk specific gravities of the three samples tested were fairly high at 2.52 to 2.59. The PIs obtained on the fines produced in the LAA test were SP and 2.3. The corresponding linear shrinkages were 3.0 and 0.7%. The liquid limits at 25 and 16 also suggest that the PI of the first sample (1573) is higher than SP.

As judged by hardness, good aggregate crushing strengths are the rule rather than the exception. A range of about 9-23 tons is indicated. Overall consistencies of the horizons are normally stiff to very stiff. This means that these calcretes are usually rippable and can be broken down on the road by grid rolling. Materials at an advanced stage of development like that illustrated in Plate 1.10 (Sample 1931) would probably not be rippable or griddable. The soil still filling the voids in the honeycomb structure may be quite plastic (Mr. J. Caiger, 1965; personal comm.).

Sample 1573 was taken at one of the Ondangua airfield borrow pits and was not used in the airfield construction. Sample 2494 was material proposed for use on the New Geluk-Namutoni road (probably as subbase).

8.10/ ...

8.10 HARDPAN CALCRETES

Hardpan calcrete as defined in Section 1.3.3 is merely a firm to hard, sheetlike calcrete layer normally underlain by softer or looser material. In common with all the other groups of calcretes already described, this is a morphogenetic definition and no tight restriction is possible on consistency without having to create a new group of calcretes to include all the less indurated types which a hardpan definition restricted literally to hard hardpans would create. It has thus been deemed advisable to retain the geological usage of the term and to merely add suitable qualifying adjectives such as stiff, hard, tufaceous, etc. (see Section 1.3.3). Examination of Plates 1.12-1.18 and their accompanying descriptions will suggest much concerning their engineering properties to the experienced engineering geologist or materials engineer.

Since hardpan calcretes are merely the final stage of development of calcified soils, powder calcretes, nodular calcretes and honeycomb calcretes, it is reasonable to suppose that much of the following information as to the strength and other properties of hardpans and boulders can also be applied to the aggregate fraction of these non-indurated calcretes, about which less is known (the term 'non-indurated' used in this sense refers to the overall layer, not to individual nodules, which may be quite well indurated). This is thought to be true except insofar as the shape of the aggregate (i.e. spherical nodules as against angular crushed hardpan and boulders), would affect the test results concerned. Nodules of calcretes possessing calcareous fines also tend to possess a somewhat powdery outer surface and this could also affect some test results slightly.

Test results on some hardpan calcretes are summarised in Table H.3.

8.10.1/ ...

8.10.1 Soil constants

Soil constants were determined on the fines produced in the Los Angeles test in order to assess the plasticity of fines produced when using hardpan as crusher run bases. It should be noted that there is some evidence from the U.S.A. (see Section 7.8) that the fines produced in the LAA are less plastic than those produced by in-service degradation.

Liquid limits of the fines of 12 samples ranged from being unobtainable through 22 to a maximum of 40. PIs ranged from NP through 0 to a maximum of 7. Bar linear shrinkages ranged from 1.3 to 3.3%.

It is apparent that care must be taken when using these materials as crusher run, for the soil constants of the fines may exceed those allowed in some base specifications.

8.10.2 Crushing strength of crushed hardpans

Tufaceous hardpans at about firm to stiff are the weakest forms of hardpans and their 10% F.A.C.T. values range from about 2.5 to 5 tons. (Samples 1530 and 1624). Sample 1530 is actually a borderline case and it is difficult to decide whether to classify it as a hardpan or a calcified sand. It is included under hardpans in Table H.3. Soaked values are about half this and ACVs are over 50%. With continued development they become somewhat better cemented to a material like 1724, with a dry 10% F.A.C.T. value of 7 tons (5 tons under wet conditions), an ACV of 38% and a 2500g LAA of 44%. In its most developed stage the hardpan becomes slightly silicified, yielding Mohs hardnesses of up to 6 (Sample 1658) and dry 10% F.A.C.T. values of up to about 22 tons (13 tons soaked), ACVs of 19 or 20% and 2500g LAAs of 34%. Slightly better figures than these are also theoretically possible, since a honeycomb calcrete (2494) -

a/ ...

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Test results on some hardpan calcretes are summarised in Table H.3.

8.10.1/ ...

a less developed stage - has yielded a 10% F.A.C.T. value of 23 tons (ACV of 16% and 2500g LAA of 25%). Examples of hardpans yielding particularly good aggregate crushing values are 1623 (21%), 1658 (19%), 1658X (21%), 2204 (20%), 2225 (21%), 2226 (25%) and 2505 (25%). The ACVs of good hardpan and boulder calcretes compare favourably with the range of values and mean for British limestones of 11-37 and 24 respectively given by the Road Research Laboratory (1959). It is, however, doubtful whether an ACV of 11 will ever be measured on a calcrete, and of course much higher ACVs than 37 will often be measured on the tufaceous hardpans.

Attention is drawn to the correlation found in Section 7.9.1 (Figures 7.15 and 7.16) between Mohs hardness and the 10% F.A.C.T. or ACV of calcretes. The aggregate crushing strength of calcretes is plainly a direct function of their hardness and the latter can be used in the field to provide a good estimate of the crushing strength. The pocket Mohs hardness tester described in Section 7.9.2. should be particularly convenient for this purpose. Even the carrying of one blade with a hardness of 3½ on a key ring would enable the field identification of calcretes passing the common base and concrete aggregate requirements of an ACV of less than 30%. An ordinary unplated brass key is usually of about this hardness.

In addition to the effect of the degree of cementation and silicification mentioned above, the aggregate crushing strength of calcretes also depends on their texture (whether sandy, non-sandy or gravelly), density, and the crystal size of the carbonate cement. Very sandy calcretes are at earlier stages of development than slightly sandy or non-sandy ones and hence are softer, their sand grains not yet having been replaced by the carbonate cement to form the opaline cement responsible for the strengths of the harder calcretes. Cryptocrystalline calcretes (those with carbonate crystal sizes of less than 0.01 mm in size) are

normally/ ...

normally stronger and harder than Type 6 calcretes which are all microcrystalline (crystal sizes 0.01-0.2 mm) or macrocrystalline (>0.2 mm), and do not appear to get harder than 3 or 3½ on the Mohs scale. The only pure Type 6 hardpan tested (2200) only had a 10% F.A.C.T. value of 9 tons and an ACV of 31%.

Figure 8.19 shows the effect of water on the aggregate crushing strength of hardpan, boulder and honeycomb calcretes. The effect varies with individual calcretes, some having better wet strengths than others. On the average, the dry strength is reduced by about one third on soaking. It would appear that calcretes with a dry 10% F.A.C.T. value of less than about 1 ton would slake in water. A wet aggregate crushing test gives a more realistic reproduction of service conditions than does the dry test (see Section 7.8.2). It should be noted that Dr. H.H. Weinert (1969; personal comm.) considers that the most reliable way of formulating a specification is not by simple wet values, but rather by specifying a minimum dry strength and a minimum percentage of wet to dry strength.

The unconfined compressive strengths of calcretes are little known. Assuming that the relation UCS = $\frac{500,000}{ACV}$ psi mentioned in Section 7.8 also holds for calcretes, then intact hardpan calcretes can be expected to have compressive strengths ranging from about 9,000 (ACV = 53%) to about 26,000 psi (ACV = 19%). The upper figure seems reasonable, but sample 1642 (ACV = 53%) is breakable by hand and certainly does not possess a crushing strength of 9,000 psi. In all probability the relation only holds for materials falling on the straight part of Figure 8.20, i.e. dense materials with ACVs of less than 35% and porous materials with ACVs of less than about 30%. The only compressive strength determinations made on hardpan calcretes from Southern Africa known to the author are those kindly

made/ ...

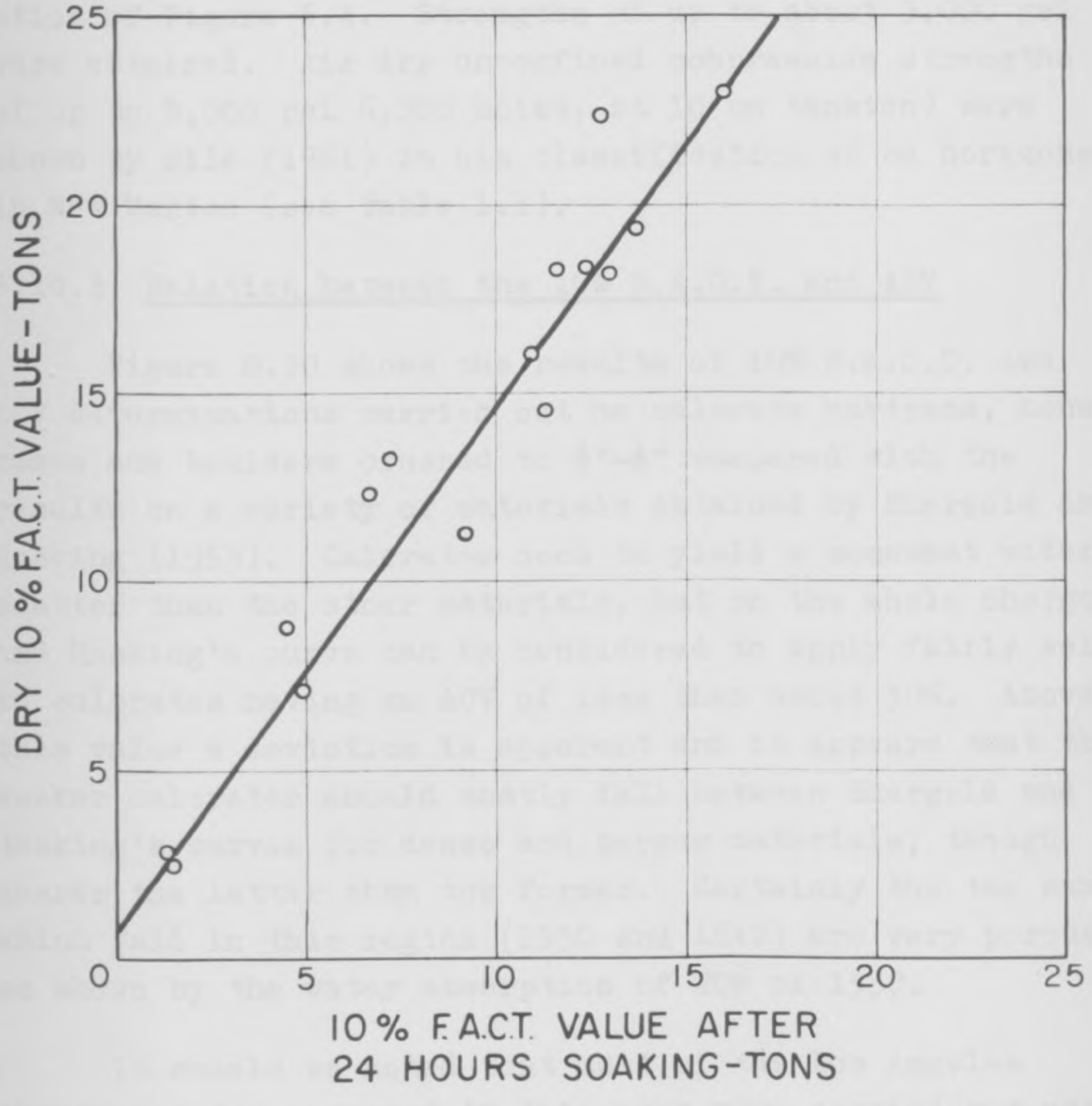


FIGURE 8·19

Effect of soaking on crushing strength of crushed calcretes

made available by Messrs. Van Niekerk, Kleyn and Edwards and White's Portland Cement Company and used in the preparation of Figure 6.4. Strengths of up to about 3,000 psi were obtained. Air dry unconfined compressive strengths of up to 8,000 psi 6,000 moist, at 10 cm tension) were shown by Gile (1961) in his classification of ca horizons in New Mexico (see Table 1.1).

8.10.3 Relation between the 10% F.A.C.T. and ACV

Figure 8.20 shows the results of 10% F.A.C.T. and ACV determinations carried out on calcrete hardpans, honeycombs and boulders crushed to $\frac{3}{8}$"-$\frac{1}{2}$" compared with the results on a variety of materials obtained by Shergold and Hosking (1959). Calcretes seem to yield a somewhat wider scatter than the other materials, but on the whole Shergold and Hosking's curve can be considered to apply fairly well to calcretes having an ACV of less than about 30%. Above this value a deviation is apparent and it appears that the weaker calcretes should mostly fall between Shergold and Hosking's curves for dense and porous materials, though nearer the latter than the former. Certainly the two samples which fall in this region (1530 and 1642) are very porous, as shown by the water absorption of 20% of 1530.

It should be noted that most of the Los Angeles Abrasion tests reported in this work were carried out using 2500g of aggregate and 11 spheres. The values obtained tend to be higher than those obtained using 5000g of the same size aggregate or grading A (Table J.4).

No Treton tests or the very similar British aggregate impact tests have been carried out on the calcretes studied. 29 Treton results on calcrete hardpans from the Tsumeb-Operet area of South West Africa kindly made available by Messrs. Ninham Shand and Partners ranged between 14 and 34%. 12 samples which were ripped with a Caterpillar D9 tractor

yielded/ ...

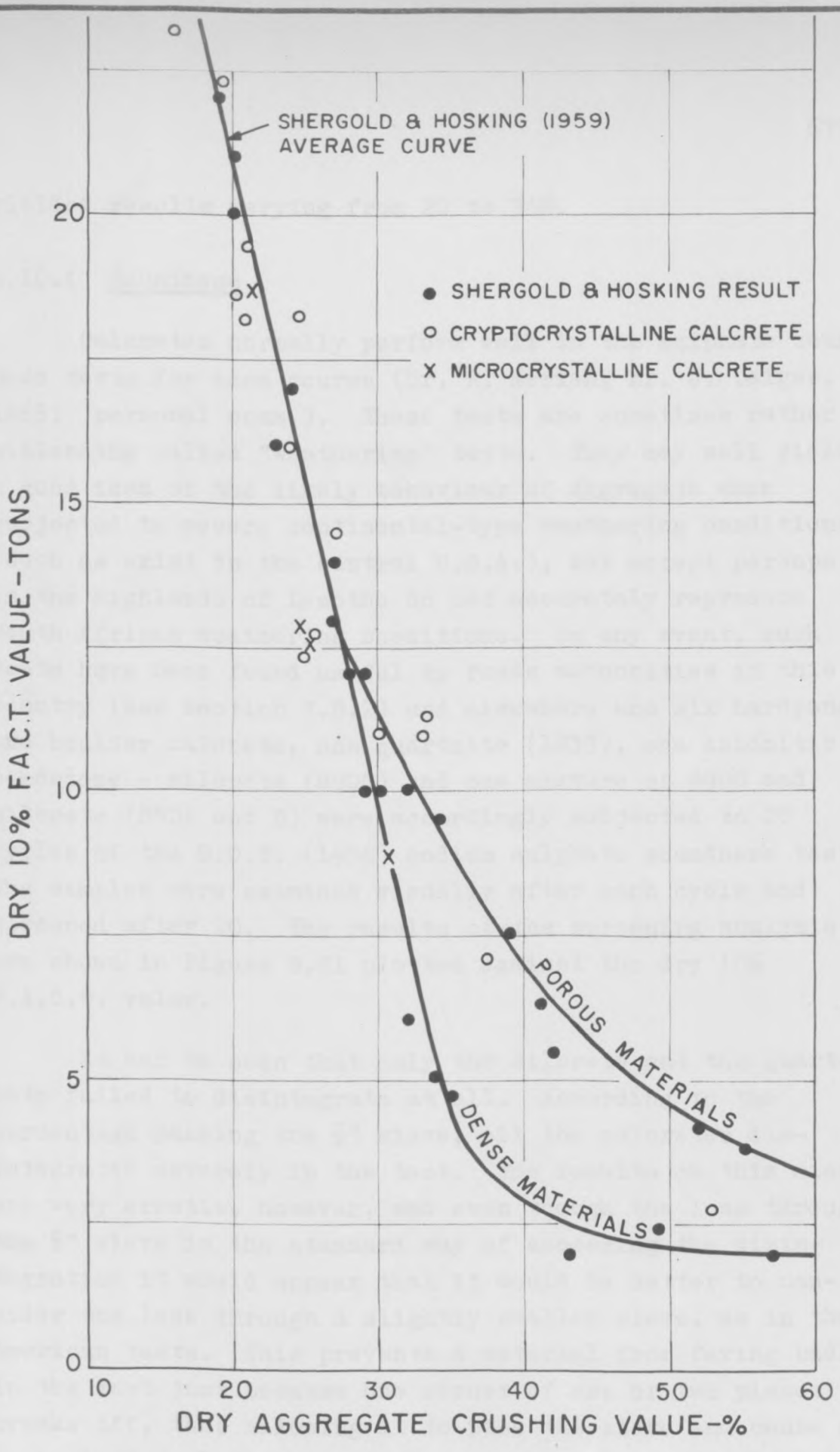


FIGURE 8.20

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

RG4-3209

yielded results varying from 20 to 34%.

8.10.4 Soundness

Calcretes normally perform well in the sulphate soundness tests for base course (Dr. R. Merian, Mr. J. Caiger, 1965; personal comm.). These tests are sometimes rather misleading called "weathering" tests. They may well yield a good idea of the likely behaviour of aggregate when subjected to severe continental-type weathering conditions (such as exist in the central U.S.A.), but except perhaps in the highlands of Lesotho do not accurately reproduce South African weathering conditions. In any event, such tests have been found useful by roads authorities in this country (see Section 7.8.2) and elsewhere and six hardpans, one boulder calcrete, one quartzite (1833), one dolomitic sandstone - silcrete (890C) and one mixture of 890C and silcrete (890A and B) were accordingly subjected to 20 cycles of the D.O.T. (1958) sodium sulphate soundness test. The samples were examined visually after each cycle and screened after 20. The results of the screening analysis are shown in Figure 8.21 plotted against the dry 10% F.A.C.T. value.

It can be seen that only the silcrete and the quartzite failed to disintegrate at all. According to the percentage passing the $\frac{3}{4}$" sieve, all the calcretes disintegrated severely in the test. The results on this sieve are very erratic, however, and even though the loss through the $\frac{3}{4}$" sieve is the standard way of assessing the disintegration it would appear that it would be better to consider the loss through a slightly smaller sieve, as in the American tests. This prevents a material from faring badly in the test just because one corner of one or two pieces breaks off, thus allowing it to pass the sieve and cause a large loss (relatively few pieces are used in the test). The per cent passing the 4 mesh or $\frac{1}{2}$" sieve correlates

quite/ ...

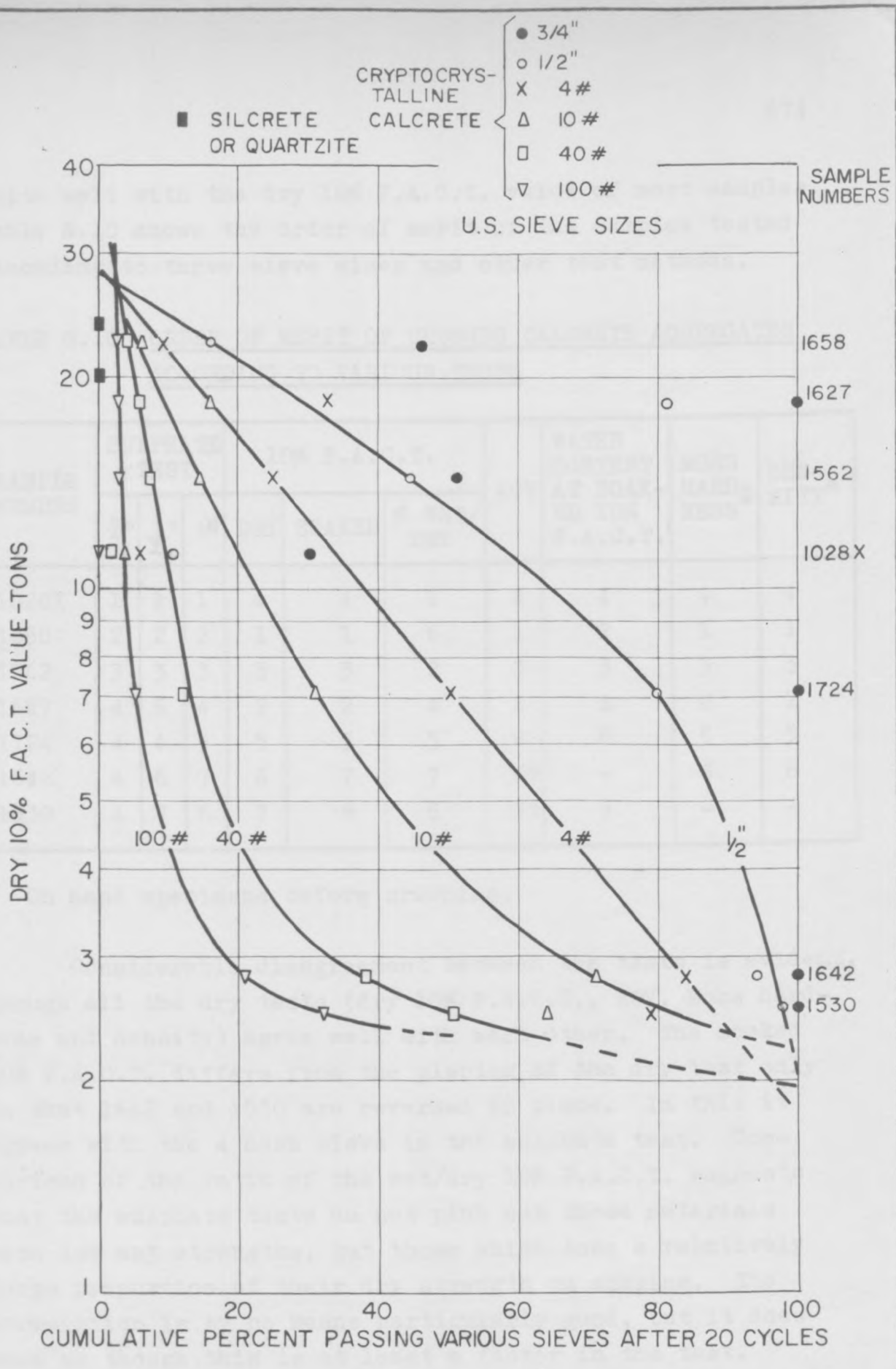


FIGURE 8.21

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

quite well with the dry 10% F.A.C.T. value of most samples. Table 8.10 shows the order of merit of the samples tested according to three sieve sizes and other test methods.

TABLE 8.10 ORDER OF MERIT OF CRUSHED CALCRETE AGGREGATES ACCORDING TO VARIOUS TESTS

SAMPLE NUMBER	SULPHATE TEST			10% F.A.C.T.			ACV	WATER CONTENT AT SOAKED 10% F.A.C.T.	MOHS HARDNESS*	DENSITY*
	¾"	½"	4#	DRY	SOAKED	% WET/DRY				
1028X	1	1	1	4	4	1	4	4	4	4
1658	2	2	2	1	1	6	1	2	1	1
1562	3	3	3	3	3	2	3	3	3	3
1627	4	5	4	2	2	4	2	1	2	2
1724	4	4	5	5	5	3	5	6	5	5
1642	4	6	7	6	7	7	6?	-	6	6
1530	4	7	6	7	6	5	7?	5	-	-

* On hand specimens before crushing.

Considerable disagreement between the tests is evident, though all the dry tests (dry 10% F.A.C.T., ACV, Mohs hardness and density) agree well with each other. The soaked 10% F.A.C.T. differs from the placing of the dry test only in that 1642 and 1530 are reversed in place. In this it agrees with the 4 mesh sieve in the sulphate test. Comparison of the ratio of the wet/dry 10% F.A.C.T. suggests that the sulphate tests do not pick out those materials with low wet strengths, but those which lose a relatively large proportion of their dry strength on soaking. The correlation is by no means particularly good, but it does seem as though this is at least a factor in the test.

Figure 8.21 suggests that calcretes would have to possess a dry 10% F.A.C.T. value of about 30 tons in order

not/ ...

not to undergo any disintegration at all after 20 cycles and that those possessing dry 10% F.A.C.T. values of less than 2 tons would disintegrate completely after 20 cycles.

Figure 8.22 shows the effect of dry aggregate crushing strength in prolonging the time before the onset of disintegration in the sulphate test. It would appear that a calcrete would have to possess a dry 10% F.A.C.T. value of at least somewhere between 11 and 14 tons in order to survive the 5 cycles usually used in testing base materials. It may be significant that 12 tons is a common base and concrete aggregate specification. A slight, but insignificant improvement in the relation is obtained by using soaked instead of dry 10% F.A.C.T. values (not shown).

8.10.5 Hardness

In engineering terminology the terms 'hardness' and 'strength' are normally considered completely synonymous. In this work the term 'hardness' is only used for the property measured by a Mohs or Vickers or some other accepted hardness test, while the term 'strength' is used for the property measured by tests such as the ACV, 10% F.A.C.T. and direct compressive strength tests. The two terms are certainly related, in the case of calcretes at least, as the valuable correlation in Figures 7.15 and 7.16 shows (and probably also in the case of materials in general), but in view of the use of both hardness and direct strength testing in this work, it is thought necessary to distinguish between the two terms, even if they do measure the same property.

The Mohs hardness of hardpan calcretes varies between about 1.5 and 6. Tufaceous types normally possess hardnesses of less than two and all other types greater hardnesses. Hardnesses of more than 3-4 (the accepted maximum hardness of calcite and dolomite and therefore the theoretical maximum hardness of the carbonate matrix of calcretes) are

common/ ...

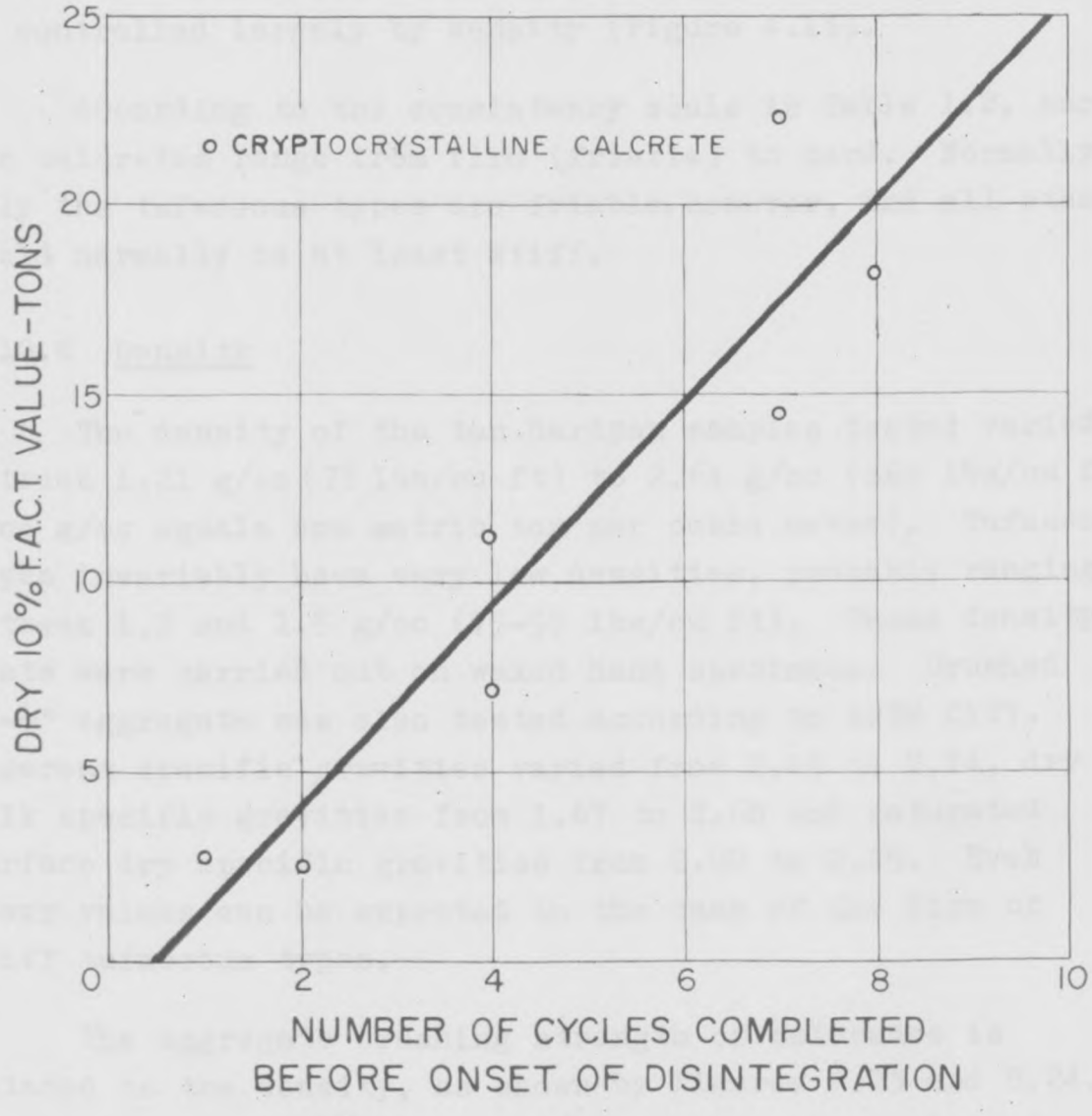


FIGURE 8·22

Relation between 10% F.A.C.T. and onset of disintegration in the sodium sulphate soundness test

RG-247
RG4-2123

common and appear to be due largely to slight silicification (Section 4.4) not normally visible in thin sections under the microscope, while hardnesses of up to about 4 seem to be controlled largely by density (Figure 4.13).

According to the consistency scale in Table 1.2, hardpan calcretes range from firm (friable) to hard. Normally only the tufaceous types are friable, however, and all others would normally be at least stiff.

8.10.6 Density

The density of the ten hardpan samples tested varied between 1.21 g/cc (75 lbs/cu ft) to 2.64 g/cc (165 lbs/cu ft). (One g/cc equals one metric ton per cubic metre). Tufaceous types invariably have very low densities, probably ranging between 1.2 and 1.5 g/cc (75-95 lbs/cu ft). These density tests were carried out on waxed hand specimens. Crushed $\frac{3}{8}$"-$\frac{1}{2}$" aggregate was also tested according to ASTM C127. Apparent specific gravities varied from 2.49 to 2.74, dry bulk specific gravities from 1.67 to 2.68 and saturated surface dry specific gravities from 2.00 to 2.69. Even lower values can be expected in the case of the firm or stiff tufaceous types.

The aggregate crushing strength of calcretes is related to the density, as shown by Figures 8.23 and 8.24. The former figure shows the 10% F.A.C.T. values plotted against the weight of aggregate required to fill the standard mould used in the test. Figure 8.24 shows the 10% F.A.C.T. values plotted against actual density, as determined by a rapid method which consists simply of a normal specific gravity determination on hand specimens except that the submerged weight is taken as soon as possible after submergence and after knocking off the adhering bubbles against the side of the water container. In the case of very porous specimens it results in slightly higher densities (or bulk specific gravities) being obtained than when the

wax/ ...

wax-coated method is used, the relation between the two being shown in Figure 8.25. The method was originally developed as a simple rapid test for field laboratories not equipped with a press for the determination of aggregate crushing strengths. This was before the very much better relation between Mohs hardness and crushing strength was discovered (Section 7.9). A plot of 10% F.A.C.T. value against the dry bulk specific gravity of crushed aggregate (not shown) yields a curve almost identical to Figure 8.24 - as it should. The two figures suggest that density, crystallinity and some other factor (shown in Section 4.4 to probably be silicification) control the crushing strength of calcrete. Mohs hardness seems to be measure of all these factors, since it yields a linear correlation with the 10% F.A.C.T. value.

8.10.7 Water absorption

The water absorption of hardpan calcretes crushed to $\frac{3}{8}$"-$\frac{1}{2}$" and soaked for 24 hours in preparation for the soaked 10% F.A.C.T. was found to vary between 2.8 and 24%. Only a tufaceous hardpan takes up such high amountsas the latter figure,however, and all the other hardpans tested fell within the range 2.8-5.9%, most being between 4 and 6%. Tests according to ASTM C127 yielded a range of values from 0.4 to 20%, most samples falling in the range 1 to 4%. These results are all lower than the results obtained during the soaked 10% F.A.C.T. The latter may perhaps not have been properly surface dried. For comparison, the Road Research Laboratory (1959) found the water absorption of 42 samples of British limestones to vary between 0.2 and 2.9%, with a mean value of 1.0%.

Water absorption is a useful rapid test for inferior aggregate. It would appear that a specification 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

(Figure/ ...

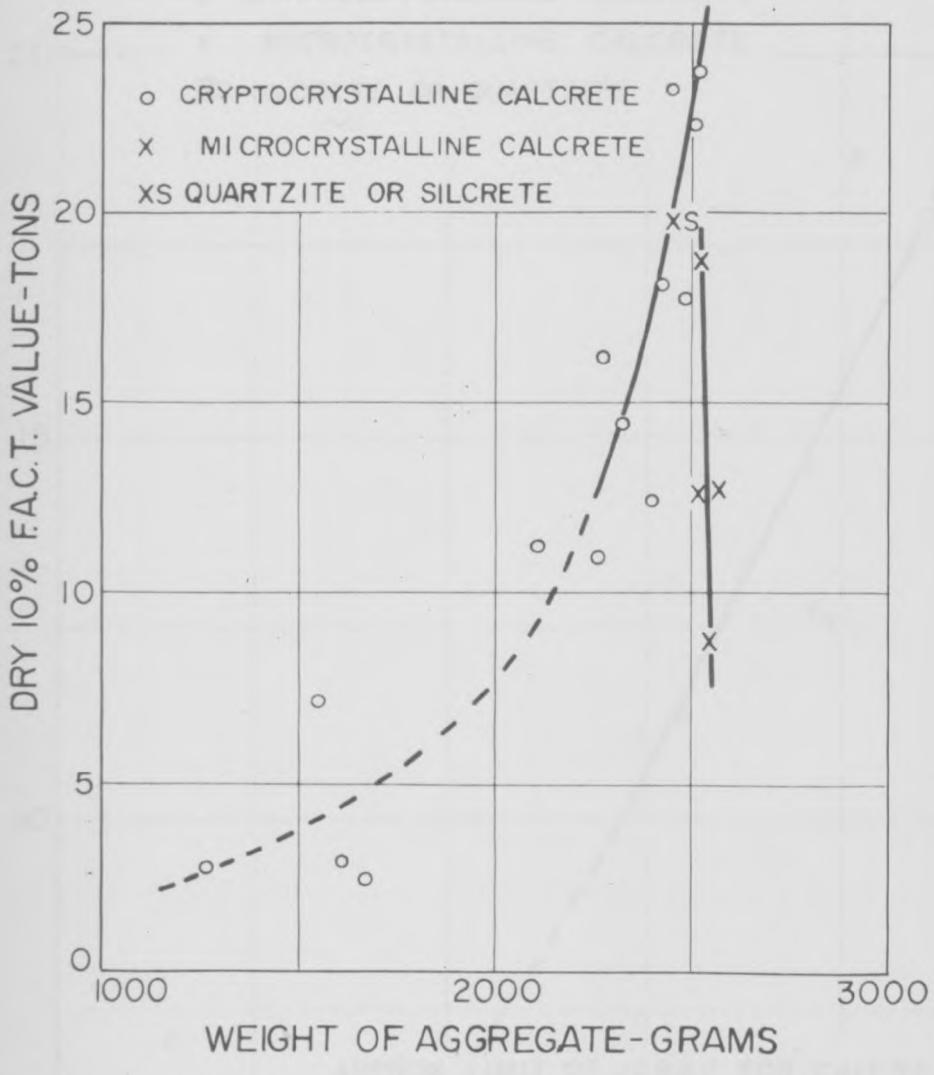


FIGURE 8·23

Relation between 10% F.A.C.T. and weight of aggregate used to fill the standard mould

RG - 243
RG4 - 2119

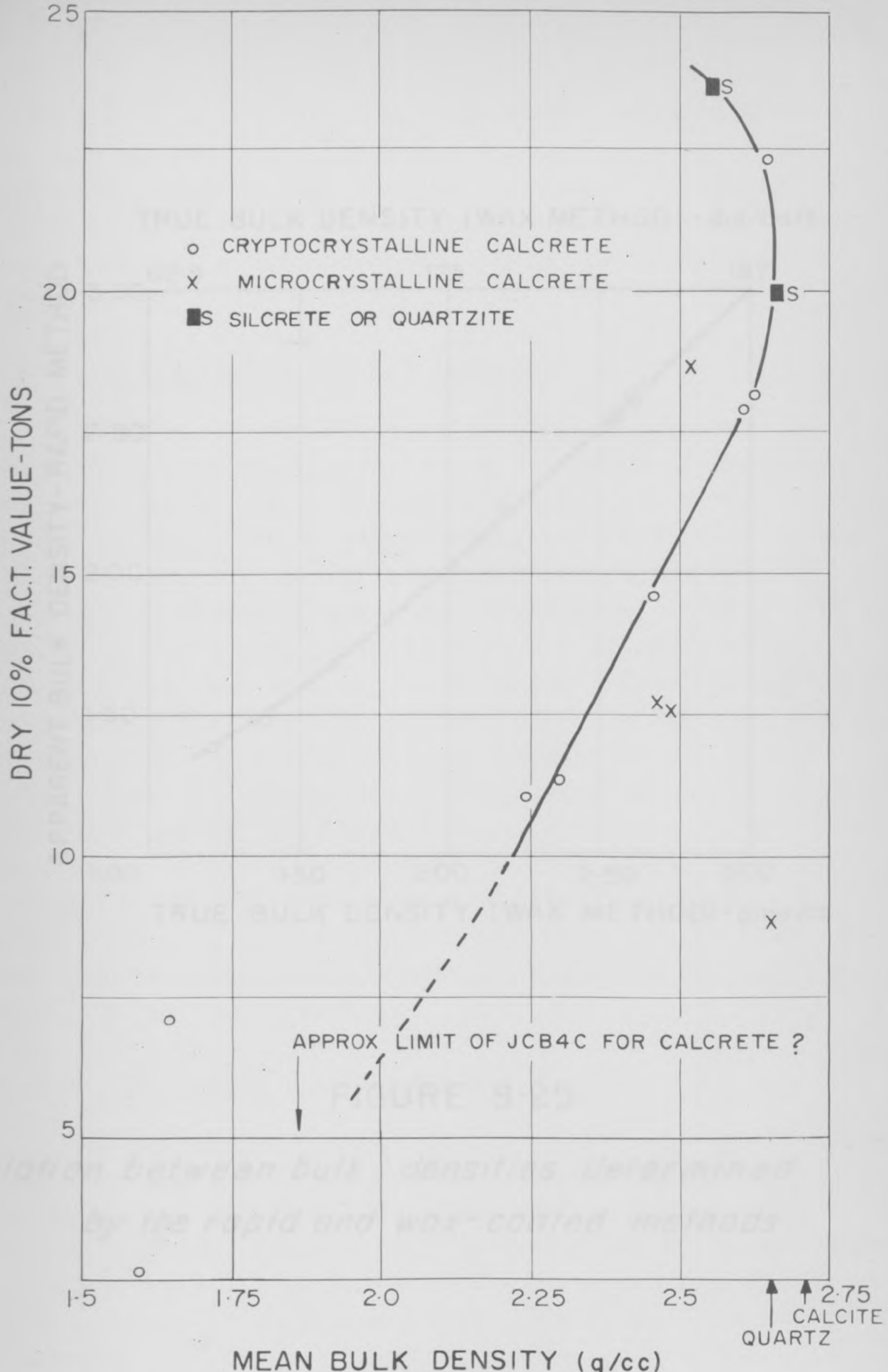


FIGURE 8·24

Relation between the 10% F.A.C.T. value and bulk density

RG-244
RG4-2120

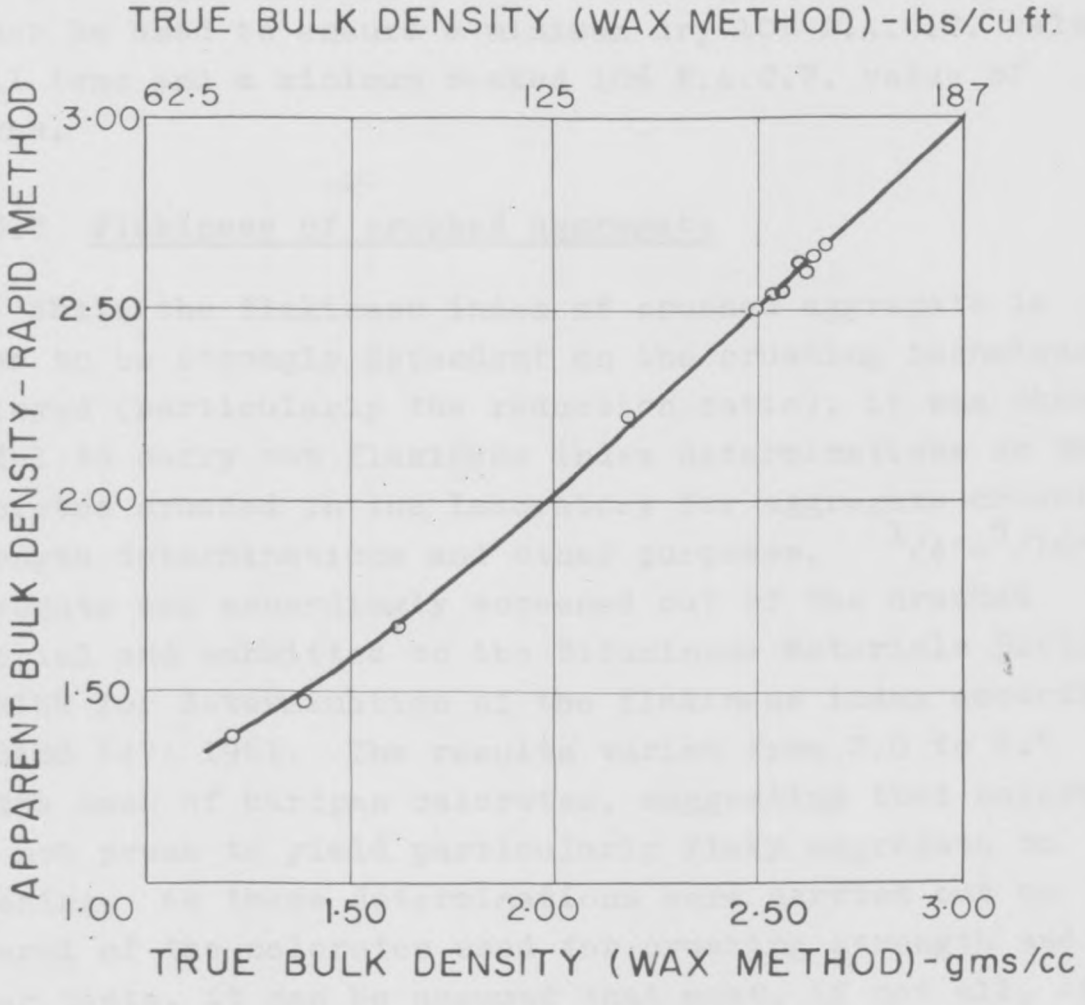


FIGURE 8·25

Relation between bulk densities determined by the rapid and wax-coated methods

RG-245
RG4-2121

(Figure 8.26). A plot of dry and soaked 10% F.A.C.T. results against ASTM C127 water absorption on crushed $\frac{3}{8}$"-$\frac{1}{2}$" aggregate suggests that a minimum figure of 5% should rather be used to ensure a minimum dry 10% F.A.C.T. value of 11 tons and a minimum soaked 10% F.A.C.T. value of 6 tons.

8.10.8 Flakiness of crushed aggregate

While the flakiness index of crushed aggregate is known to be strongly dependent on the crushing techniques employed (particularly the reduction ratio), it was thought useful to carry out flakiness index determinations on the calcretes crushed in the laboratory for aggregate crushing strength determinations and other purposes. $^{1}/_{4}$"-$^{5}/_{16}$" aggregate was accordingly screened out of the crushed material and submitted to the Bituminous Materials Division of NIRR for determination of the flakiness index according to SABS 647: 1961. The results varied from 2.0 to 8.5 in the case of hardpan calcretes, suggesting that calcretes are not prone to yield particularly flaky aggregate on crushing. As these determinations were carried out on several of the calcretes used for crushing strength and other tests, it can be assumed that most, if not all, of the crushed aggregate used for all tests was not excessively flaky in character.

8.10.9 California bearing ratio

Few figures are available on the CBR of crushed calcretes of any sort. Crusher run calcretes used for base on the Tsumeb-Operet road yielded Modified AASHO CBRs approaching 200. Some of this material was probably limestone, however. A material proposed for subbase on the road (borrow pit at chainage 1090) which was definitely a crushed hard calcrete hardpan yielded a modified AASHO CBR of 116 at 0.1" penetration and 162 at 0.2" penetration.

Modified/ ...

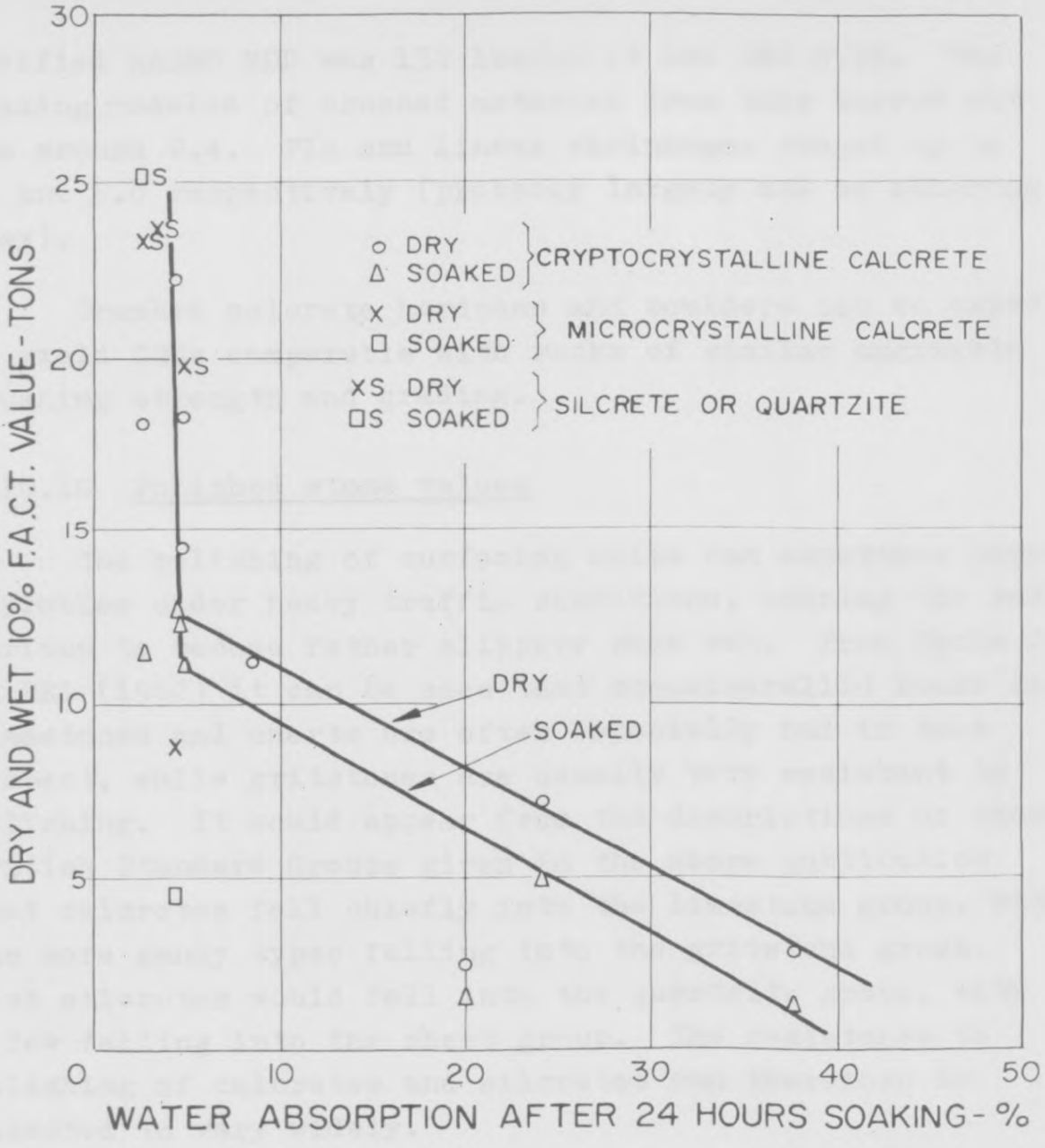


FIGURE 8·26

Relation between the 10 % F.A.C.T. and water absorption of aggregate

RG-249
RG4-2125

Modified AASHO MDD was 132 lbs/cu ft and OMC 6.5%. The grading modulus of crushed material from this borrow pit was around 2.4. PIs and linear shrinkages ranged up to 14 and 5.0 respectively (probably largely due to adhering clay).

Crushed calcrete hardpans and boulders can be expected to yield CBRs comparable with rocks of similar aggregate crushing strength and grading.

8.10.10 Polished stone values

The polishing of surfacing chips can sometimes become a problem under heavy traffic conditions, causing the road surface to become rather slippery when wet. From Table 2.2 in RRL (1962) it can be seen that monominerallic rocks like limestones and cherts are often especially bad in this respect, while gritstones are usually very resistant to polishing. It would appear from the descriptions of these British Standard Groups given in the above publication that calcretes fall chiefly into the limestone group, with the more sandy types falling into the gritstone group. Most silcretes would fall into the quartzite group, with a few falling into the chert group. The resistance to polishing of calcretes and silcretes can therefore be expected to vary widely.

Shupe and Lounsbury (1958) have stated that, when dealing with relatively pure carbonate aggregates, the skid resistence is inversely proportional to the calcite content, while Knill (1960) has stated that the polished stone value of limestones increases with the insoluble residue content and the number of coarsely crystalline carbonate particles. Maclean and Shergold (1959) suggested that an important characteristic of rocks resistant to polishing is the presence of two minerals that differ considerably in their resistance to wear. Gray and Renninger (1965) have proved the latter suggestion to be true in the

case/ ...

case of carbonate aggregates and have found a good correlation between the resistance to polishing and the amount of sand-sized (0.05-2.0 mm) material insoluble in dilute HCl. They found further that the insoluble residue alone and the differential hardness alone did not yield good correlations. The type of sand-sized minerals did not seem to matter (even micas were found to be satisfactory) and a high percentage of insoluble material less than 0.5 mm in diameter did not contribute towards the polish-resistance of the aggregate. They also found that when dealing with low residue carbonates the skid resistence was increased by the degree of crystallinity and higher dolomite contents. It would further appear from their results that by degree of crystallinity it is meant that a higher degree of crystallinity promotes a higher degree of skid resistance and that as the residue increases above about 5-10% its size distribution seems to overshadow all other effects. They also found that the binder used (asphalt or cement) did not appear to have any effect on the test results.

Measures to ensure the best use of aggregates susceptible to polishing have been discussed by Gray and Goldbeck (1959).

No work appears to have been published on the susceptibility of calcretes to polishing, so 9 samples of hardpan and boulder calcretes, 3 silcretes and one quartzite were submitted to the Bituminous Materials Division of NIRR for determination of their polished stone values. The method used was that described by Maclean and Shergold (1966) which is a modification of B.S. 812:1960. Test results obtained by the modified method are known as "polished stone values (1965)" and may under certain circumstances be about 10% lower than the values obtained by the original method. Maclean and Shergold (1966) stated that the aggregate should have been produced in a commercial plant and not by laboratory crushing since the latter has been found

to/ ...

to give erroneous results. As no commercially crushed calcretes were available the samples were merely crushed in a Pegson-Osborn 8 x 3 single toggle granulator and screened to 1/4"-5/16".

The results should therefore be used with caution, but probably serve to indicate the general resistance of calcretes to polishing. The results, in order of merit, are shown in Table 8.11. The value 'B', the polished stone value after polishing is taken as the "polished stone value (1965)". The use of the ratio B/A i.e., the ratio of the polished stone values after and before polishing was suggested by Dr. H.H. Weinert (1967, personal comm.). This ratio gives some idea of the degree of polishing undergone in the test which should give an idea of the rate of polishing in practice.

Mr. C. Marais (1967, personal comm.) has kindly made available details of the polished stone values obtained on some other South African stones, thus making a comparison possible (Table 8.12). Some values for British roadstones are also included (Road Research Laboratory, 1959).

As far as the laboratory values are concerned it can be seen that calcretes rate quite well in comparison with other materials in common usage, and better in fact than several popular South African stones. The mean of 0.53 and range of 0.47-0.60 for 8 calcretes may be compared with the mean of 0.43 and range of 0.30-0.75 for 33 British limestones given by the Road Research Laboratory (1959). It is interesting to note that even the vitreous-type silcrete (a very hard material with a conchoidal fracture) rates better than the 4 British flints tested (mean of 0.35 and range of 0.30-0.40). Areas in which calcretes occur in Southern Africa are not generally heavily populated at present so that polishing is not likely to be

a/ ...

TABLE 8.11 POLISHED STONE VALUES (1965) OF SOME CALCRETES AND SILCRETES

ORDER OF MERIT	SAMPLE NUMBER	MATERIAL	NO. OF SPECIMENS TESTED	MEAN VALUE BEFORE POLISHING	MEAN VALUE AFTER POLISHING	$\frac{B}{A}$
1	1724	Calcrete tufaceous, Type 1 & 4	4	0.86	0.65*	0.75
2	1562	Calcrete, Type 3	3	0.83	0.60	0.72
3	890C	Silcrete, quartzite type	4	0.85	0.59	0.69
4	2228	Calcrete, Type 6	4	0.82	0.58	0.71
5	LS171†	Andesite control	4	0.73	0.57	0.78
6	LS172†	Dolerite control	4	0.79	0.56	0.71
7	1833X	Quartzite	4	0.80	0.56	0.70
8	2227	Calcrete, Type 3	4	0.87	0.54	0.62
9	2225	Calcrete, Type 3 & 6	4	0.77	0.53	0.69
10	1627	Calcrete, Type 1	4	0.81	0.51	0.63
11	1028X	Calcrete Type 1	4	0.82	0.49	0.60
12	2226	Calcrete, Type 4	4	0.77	0.48	0.62
13	890	Silcrete, mixture	2	0.80	0.47	0.59
14	1658	Calcrete, Type 1	4	0.81	0.47	0.58
15	890A	Silcrete, vitreous type	2	0.85	0.43	0.51

*This value is considered unreliable owing to the rapid rate of wear during polishing.

† Sample numbers of the Bituminous Materials Division.

Operator: Mr. C. Jaeger.

TABLE 8.12 SUSCEPTIBILITY TO POLISHING OF CALCRETE AND SOME COMMON SOUTH AFRICAN SURFACING STONES

	SOUTH AFRICAN STONES				BRITISH STONES		
ORDER OF MERIT	MATERIAL	POLISHED STONE VALUE (1965)		NUMBER OF SAMPLES	POLISHED STONE VALUE		NUMBER OF SAMPLES
		MEAN	RANGE		MEAN	RANGE	
1	Tillite	0.61	0.53-0.69	2	-	-	-
2	"Shale" (Malmesbury)	0.59	-	1	-	-	-
3	Sandstone	0.58	-	1	-	-	-
4	Quartzite	0.57	0.51-0.63	7	0.57	0.45-0.65	8
5	Dolerite	0.55	0.48-0.60	8	-	-	-
6	Calcrete	0.53	0.47-0.60	8	-	-	-
7	Silcrete	0.50	0.43-0.59	3	-	-	-
8	Granite	0.50	0.49-0.50	2	0.56	0.45-0.70	13
9	Norite	0.49	0.46-0.52	2	-	-	-
10	Dolomite	0.48	0.34-0.61	2	-	-	-

a problem, and any calcrete which meets the aggregate crushing strength requirements could be used.

Figure 8.27 seems to bear out the findings of Gray and Renninger (1965) that the susceptibility of carbonate aggregates to polishing is related to the differential wear resistance of the particles it contains. It also seems to confirm Maclean and Shergold's (1959) suggestion that this is a general factor common to all rocks. A similar correlation is evident if the weighted mean hardness plus the range of hardness (greatest hardness minus lowest hardness) is plotted against B/A in Table 8.11. No correlation is evident in the case of the polished stone value determined before polishing. It is also apparent that not only is the differential hardness of importance, but a good average hardness is also required in order to hold the material together and to prevent the harder particles from being removed during polishing. Vickers hardness testing should yield better results with the finer grained rocks than Mohs hardness testing. It would also be interesting to calculate the theoretical weighted mean Mohs hardness plus range of hardness for a number of common rocks for comparison with their average and range of polished stone coefficients.

8.10.11 Adhesion to bitumen

Nine samples of crushed $^{1}/_{4}$"-$^{5}/_{16}$" calcrete, silcrete and quartzite aggregates were submitted to the Bituminous Materials Division of NIRR for evaluation in a modified version of the detachment test described by Karius and Dalton (1964). The results are shown in Figure 8.28. It will be seen that there is a fair correlation between the percentage stones becoming detached after 10 days in the test and the weighted mean Mohs hardness plus the difference in hardness between the hardest and softest component, i.e. $\bar{H}_W + (H_{MAX.} - H_{MIN.})$. It was found that calcretes

possess/ ...

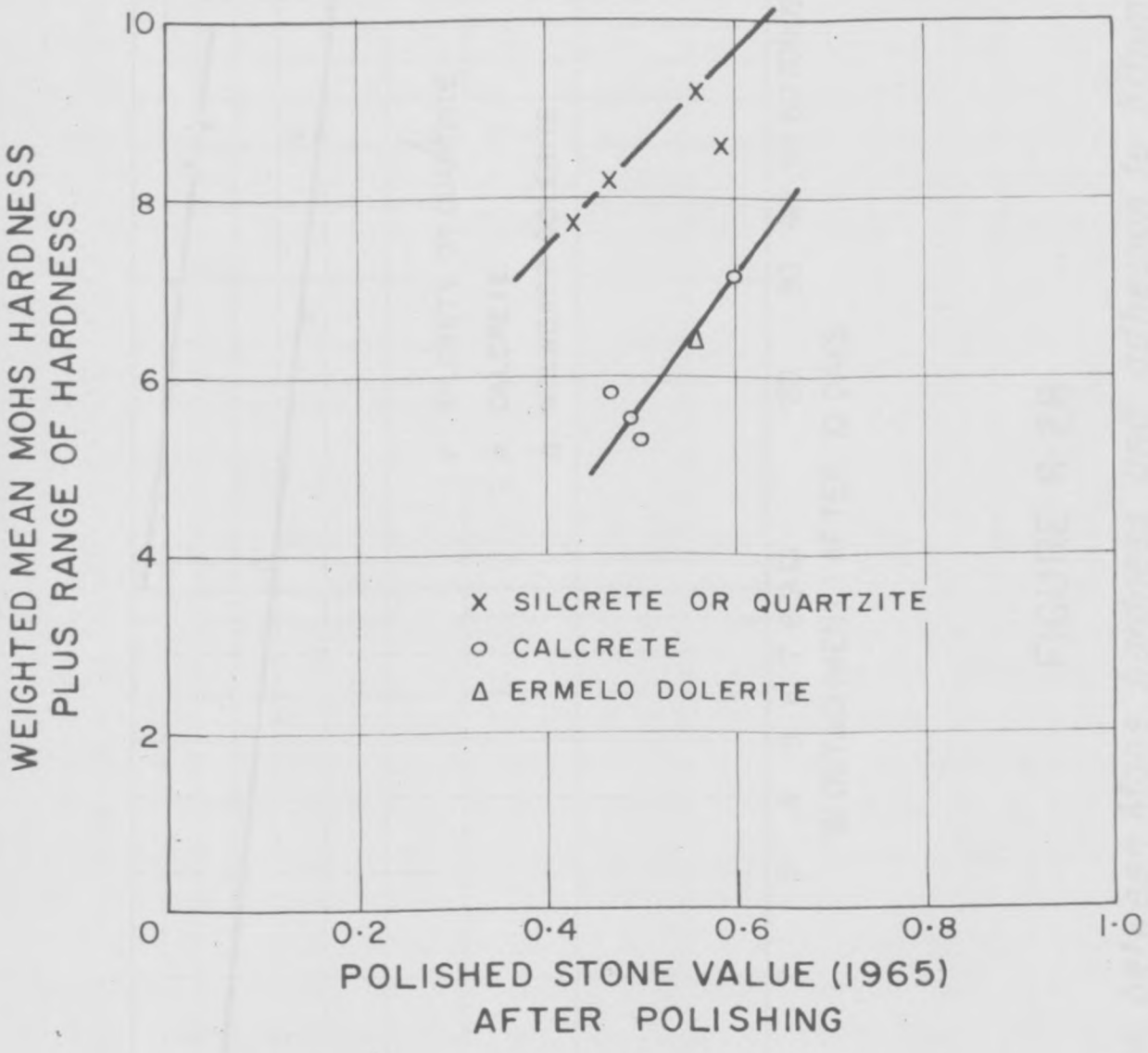


FIGURE 8·27

Relation between Mohs hardness and polished stone value

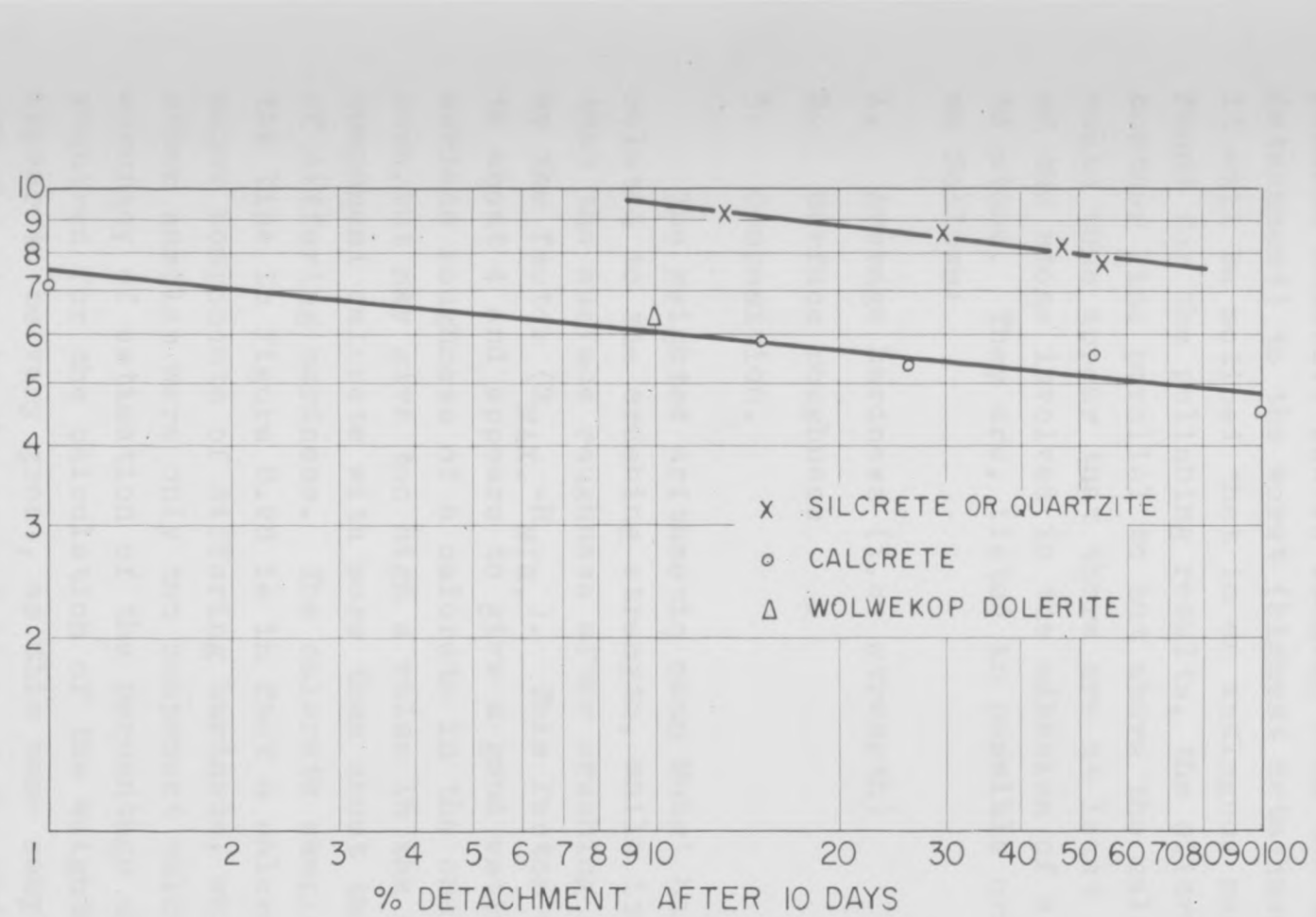


FIGURE 8·28

Relation between Mohs hardness and adhesion to bitumen

RG-251
RG4-2127

possess adhesive powers ranging from the best (lowest detachment) to the worst (highest detachment) yet found. It will be noticed that in an analogous manner to that found for the polishing results, the silcretes fall on another line parallel to but above the calcrete line. It would thus appear that there are at least three properties of the stone involved in the adhesion of a given bitumen to stone. They are, listed in possible order of importance, as follows:

1. Average hardness (i.e. strength)
2. Surface roughness
3. Composition.

The weighted arithmetic mean Mohs' hardness is directly related to the crushing strength, while it would appear that the surface roughness after crushing can be represented by the factor ($H_{MAX.}-H_{MIN.}$). This factor will vary from 0 to about 4 and appears to give a good estimate of the surface roughness of a calcrete in the one or two component case, but may give too high a value in the case of a multi-component calcrete with more than about three components of differing hardness. The calcrete sample furthest above the line in Figure 8.28 is in fact a calcrete having six major components of differing hardness, while all the other samples were only two component calcretes. The accuracy of estimation of the percentage of each component required for the calculation of the weighted mean does not appear to be very great, as this same sample did not fall off the line in the correlation of the 10% F.A.C.T. against hardness (Figure 7.15). It would appear from the position of the Wolwekop dolerite on the calcrete line and the positions of the silcretes on their own line that at least a third factor, possibly chemical (apparently not mineralogical, though this would obviously affect the average hardness and the range of hardness) composition is involved. The chemical composition of a dolerite is at least nearer

to/ ...

to that of a calcrete than to that of a silcrete.

Valton (1947) mentioned that a high water absorption is likely to lead to stripping and it is true that the sample which performed worst (1724) also possessed the highest water absorption (20%). Although the converse is not true, it may be worth carrying out further work along these lines.

Unfortunately, no testing has been carried out employing a test which is more suitable for the assessment of adhesion in premixes, such as the Riedel and Weber. In general, limestones give very good results in this test using tar or asphaltic bitumen (Valton, 1947). Valton suggested a minimum adhesion value of 3, values above 4.5 representing good adhesion and below 0.4 poor adhesion.

8.10.12 Engineering classification

Except for the tufaceous hardpans, all other hardpan calcretes can be classified as rock for most practical purposes, albeit of rather variable quality.

8.10.13 Workability

Virtually all hardpans except some of the very weak tufaceous types require ripping or blasting prior to excavation. Even the tufaceous types often possess a harder coating up to perhaps half an inch thick on their upper surfaces which can cause ripping problems. Once the tooth has penetrated the material, however, no further difficulty is normally encountered with most hardpan types. The use of seismic instruments as an aid to assessing the workability of calcrete deposits is discussed in Section 6.5.5. Calcrete hardpans possessing seismic velocities much above 8,000 ft/sec cannot apparently be normally ripped economically, even with a D9G tractor. Seismic velocities of hardpan calcretes range from about 3,000 ft/sec to about 15,000 ft/

sec./ ...

sec. When blasting is required, better results are said to be obtained if the shot holes do not completely penetrate the hardpan.

Hardpans are possibly the least variable of calcretes, though as materials go they must still be regarded as variable in quality and thickness. They are difficult and expensive to work since individual layers (and there is often only one present) are seldom more than about eighteen inches thick. Hardpans thicker than this generally only occur capping the calcretes of Pliocene age. Material less than about one foot thick is not normally regarded as a workable proposition.

8.10.14 Acceptability as road materials

Hardpans do not make particularly desirable road materials. They tend to be thin and yet require expensive ripping or blasting and grid rolling or crushing before they can be used. The weaker varieties like tufaceous hardpans would probably make reasonable gravel road materials and possibly even subbase after ripping and grid rolling, though the resulting product will be deficient in hard aggregate.

As no 5 cycle sulphate soundness tests were carried out, it is difficult to judge fully the acceptability of all of the hardpans listed in Table G.3. The following interpretations are thought reasonable, however:

Soil and waterbound macadam: On the basis of ACV alone, all except 1530 and 1642 would be acceptable to the D.O.T. (1958). Their 1963 10% F.A.C.T. requirements as well as those of the S.A.B.S. 647:1961 (amended 1966) would also eliminate 1724, 2200 and 2227. On the basis of all the tests, 1562, 1623, 1658, 1658X, 2204, 2225, 2226 and possibly 2507 would probably be acceptable at the present

time/ ...

time.

Crusher run: Samples 1658X, 2226 and 2227 would probably be rejected by most authorities (D.O.T., Transvaal, S.W.A., FAA, AASHO and ASTM) on account of their soil constants, though all samples would be acceptable in Texas on this account. 1562, 1623, 1658, 1658X, 2204, 2225, 2226, 2227 and 2505 would be generally acceptable in South Africa on the basis of their ACVs. 2200 would also be acceptable in the Transvaal on this account. 2227 might be rejected if the 10% F.A.C.T. limit of 12 tons is applied.

Some hardpan has been used and more has been proposed for use as crusher run on the new Tsumeb-Operet road at present under construction.

Premix and penetration macadam: Similar remarks to the above in connection with the ACV also apply in this case (D.O.T., 1958), but the more recent (1963) 10% F.A.C.T. requirement of 17 tons would make only 1658, 1658X, 2204 and 2225 acceptable. Of these, possibly only 1658 would pass the 20 cycle sulphate test requirement. In the Transvaal and S.W.A. the ACV limit of 30% is still in force and the nine samples 1562, 1623, 1658, 1658X, 2204, 2225, 2226 and 2227 and 2505 would be acceptable for use in premix provided their loss in the 5 cycle sodium sulphate test did not exceed 12%.

Some calcrete hardpan has been used and more is proposed for use as premix aggregate on the new Tsumeb-Operet road. Material suitable for use in premix is also normally acceptable as concrete aggregate, but this aspect will be discussed in Section 8.12.

Surface dressing: Only 1658 would pass the D.O.T. (1958) ACV requirement of 20%, . but the material would fail the 20 cycle sulphate test. 1658, 1658X, 1623, 2204 and 2225 would pass the Transvaal's limit of 22% ACV and would

probably also pass their 5 cycle sodium sulphate loss requirement of 12%.

8.11 BOULDER CALCRETES

Boulder calcretes are possibly the least desirable of all calcretes as far as their roadmaking properties are concerned and relatively little work has been carried out on them. While they are normally rippable, due to the large voids between the interconnected boulders this is very hard on the machines and once ripped the boulders may prove to be too large to be fed into the average crusher used in road construction. The quality of the boulders themselves is usually excellent, owing to the fact that they are essentially chemically weathered hardpans, and will therefore normally have passed through the full hardpan developmental and diagenetic process. Their 10% F.A.C.T. values normally range between about 10 and 20 tons and their ACVs are seldom more than 30%. Boulder calcretes have been included in the diagrams shown in the hardpan section, as crushed boulder calcretes behave in the same way as do crushed hardpans. Test results are summarised in Table H.3.

The soil constants of the four samples tested were low (PIs NP to 2.7). Bulk densities and hardnesses are always fairly high, normally ranging between 2.3 and 2.64 g/cc and being above a Mohs value of 3½-4 respectively. ASTM C127 water absorption varies from about 1 to 4% and the polished stone values (1965) after polishing of the three samples tested ranged from 0.49 to 0.58.

Although, generally speaking, decomposed rocks are usually inferior in quality to those weathered by disintegration, the reverse is true in the case of calcretes. The nearest thing to decomposition in calcretes is the removal by solution of the carbonate cement, producing rounded boulders (when covered by soil) which retain their

crushing/ ...

crushing strength. The usual criterion of decomposition, secondary mineral production, is thus absent. Disintegration on the other hand produces smaller, cracked boulders which have lost much of their strength, though the interiors of the smaller fragments may still be sound (see Section 4.5). The disintegrated material may be more usable in practice than the large rounded boulders produced by solution. The calcrete outcropping around Runtu appears to be a rare case of calcrete weathering. Here the carbonate matrix has apparently been leached out without the boulders falling apart into a loose quartz soil. The calcrete must presumably have been lightly silicified before leaching and it must be this which is holding the quartz grains together.

Most crushed boulder calcretes formed by solution are suitable for use as soil and waterbound macadams, crusher runs and premixes. The workability of these materials remains a problem, however. Some are being used at present as crusher run subbase and possibly as base on the new Tsumeb-Operet road which is at present under construction.

The suitability of boulder calcretes as concrete aggregates will be discussed in the following section.

8.12 ACCEPTABILITY AS CONCRETE AGGREGATE

No work has been found in the literature on the acceptability of calcretes as coarse aggregates in concrete, so two hardpans (1724 and 1562), two boulders (1028X and 1627), one silcrete mixture (890) and one quartzite (1833X) were submitted to the National Building Research Institute for testing. The NBRI had previously (in 1964) tested two calcified sands (914 and 917) and what seems to have been three nodular calcrete gravels from the Ondangua area and found all of then unsuitable as concrete aggregates. The two calcified sands yielded drying shrinkages of 0.26 and 0.43%, 11 day prism and 28 day cube compressive strengths

of/ ...

of 440 and 600, and 850 and 1,090 psi respectively, and 11 day transverse prism strengths of 17 and 50 psi. Two of the gravels fared better, yielding shrinkages of 0.13 and 0.09%, 11 day prism and 28 day cube compressive strengths of 1,530 and 1,420, and 1,800 and 1,600 psi respectively, and 11 day transverse prism strengths of 220 and 140 psi. The other gravel yielded even poorer strengths than the worse of the two calcified sands. Water/cement ratios varied from 1.20 to 1.71 and in all cases the whole samples were used, that is including the natural fines, which would have been quite plastic in most cases. Durability testing was not carried out, but the materials would probably have fared badly and in any event all the shrinkages and strengths cited above would not normally be regarded as acceptable.

The results on the crushed hardpans and boulders were very much better and have been reported by the NBRI in a report on contract 5031/9162 (in two parts, 1967 and 1968). Only the findings on the calcretes are briefly presented here. Initial drying shrinkages varied from 0.31-0.047% except for 1724 (0.083) and wetting expansions from 0.037-0.040% except for 1724 (0.065). 7 day prism compressive strengths varied from 3,900-5,000 psi except for 1724 which only yielded 2,000 psi. Transverse prism strengths varied from 650 to 700 psi (1724 not tested). The compressive strengths of prisms made with all samples increased after a number (50-107) of wetting and drying cycles, though the transverse strength dropped somewhat in some cases. 1562 was found to be possibly potentially alkali reactive if used with a high alkali cement. The dimensional changes on wetting and drying of specimens made with all samples except 1724 were satisfactory. The NBRI finally concluded that all the aggregates submitted (including the silcrete and quartzite) would make satisfactory coarse aggregates, though the strenghts yielded by 1627X were a little on the low side and the general behaviour of 1724 was not very satisfactory. The latter should not be used in high quality work.

It/ ...

It should be noted that strong mixes were used in the above testing and the specimens tended to break through the aggregate, indicating that the strength of the aggregate was probably the limiting factor. This is supported by Figures 8.29-8.32 which show that the concrete strengths are to some extent predictable from both the Mohs hardnesses and the 10% F.A.C.T. values of the aggregates concerned. A further improvement in the correlation could possibly be obtained by use of a differential hardness factor as was used in Figure 8.28.

The only known use of calcrete as coarse aggregate in South or South West Africa was in the construction of the road bridge over the Omuramba Ovambo at Operet on the new Tsumeb-Operet road in 1964. Sample 1028X is said to be representative of the material used. At the time of last inspection by the author and others in January 1968 the bridge was still in a perfectly satisfactory condition. The silcrete 890 and the Nosib(?) quartzite (previously thought to be silcrete) 1833X have both been used for concrete aggregate at Namutoni and Runtu respectively. Both have yielded satisfactory results.

Provided calcretes pass the normal specifications required of concrete coarse aggregate, there thus does not seem to be any objection to their use, though some of them may possibly give rise to excessive expansion when used with a high alkali cement. Alkali reactive aggregates are actually quite common in South Africa, but high alkali cements are fortunately rare. Most boulder and good hardpan calcretes would pass the normal ACV specification of 30% required of concrete aggregate and it has been shown that even the boulder calcrete 1028X (ACV 38%) has made good laboratory concrete and yielded satisfactory results in the field, while even 1724, with an ACV of 38% would be acceptable for lower quality work.

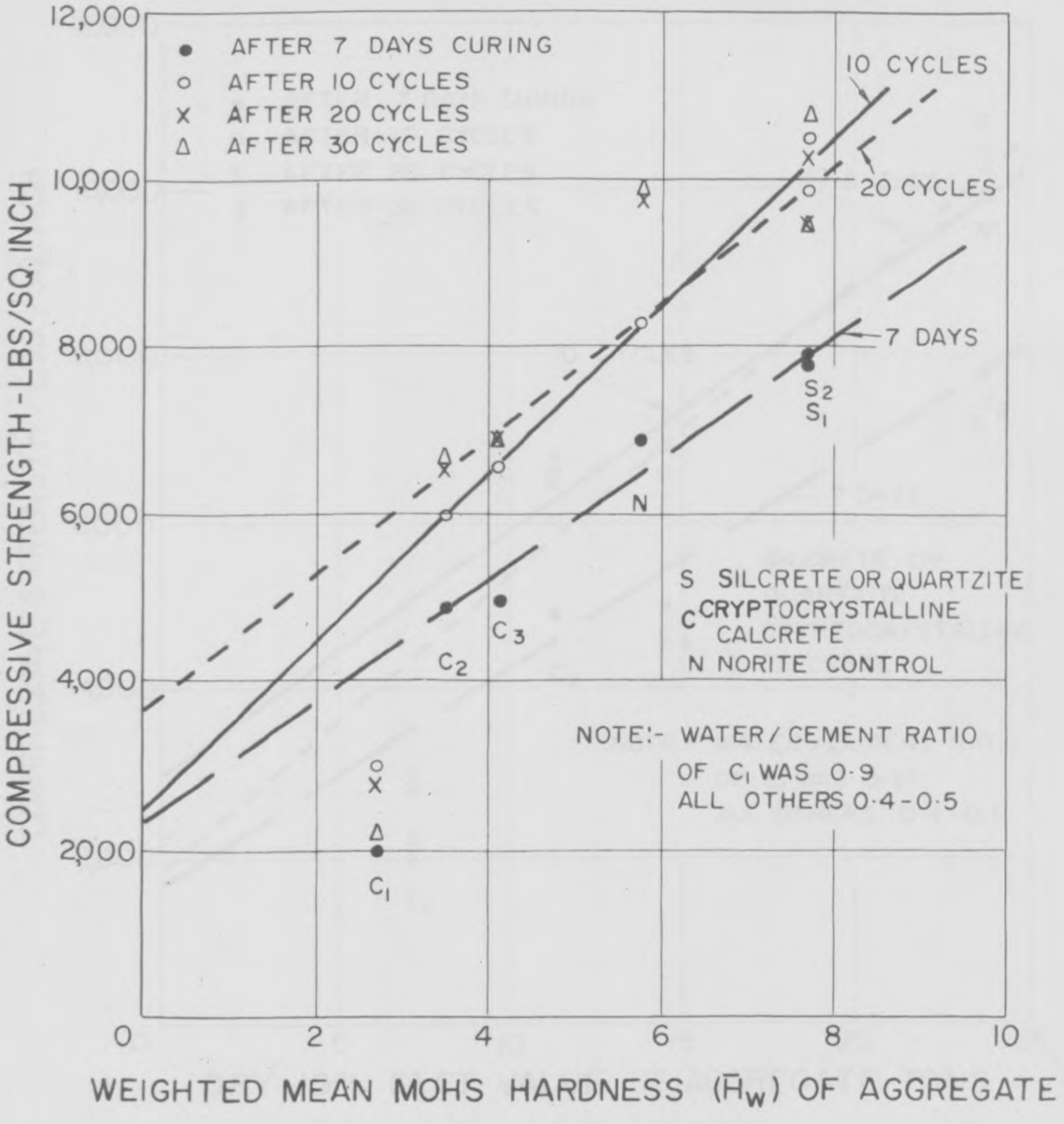


FIGURE 8·29

Relation between compressive strength of concrete cubes and Mohs hardness of aggregate

RG-252
RG4-2130

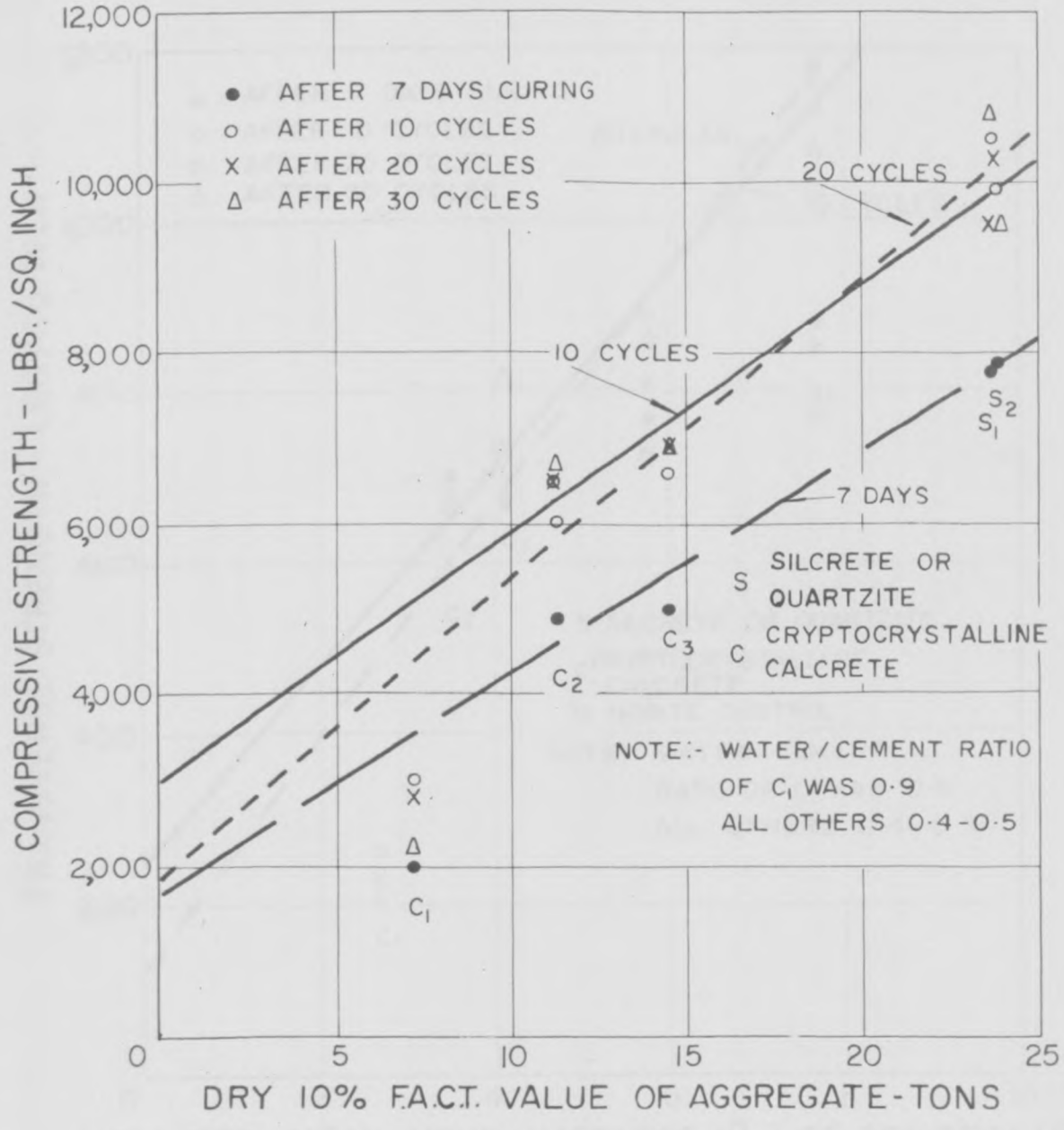


FIGURE 8·30

Relation between compressive strength of concrete cubes and 10% F.A.C.T. value of aggregate

RG-253
RG4-2131

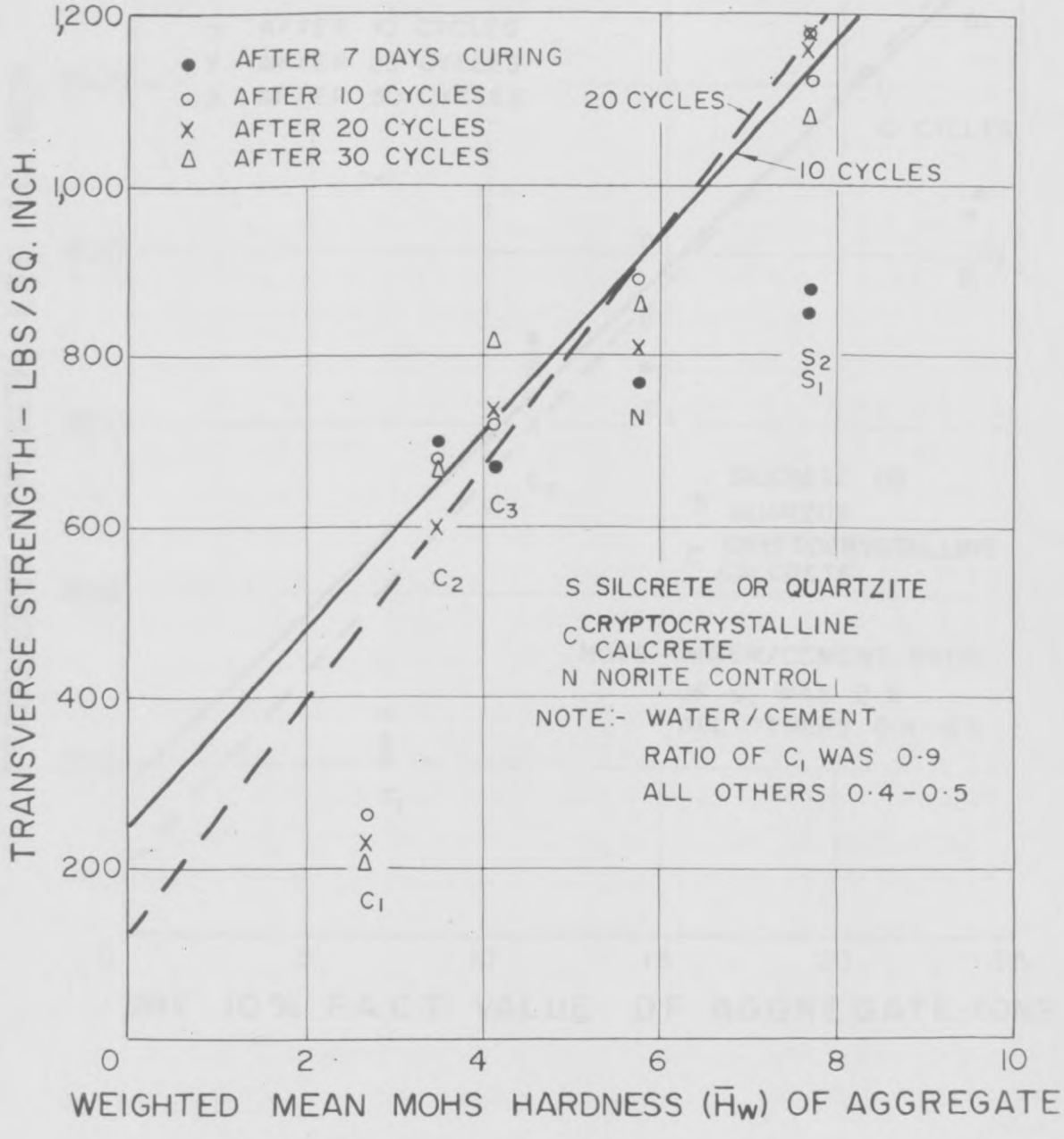


FIGURE 8·31

Relation between transverse strength of concrete bars and Mohs hardness of aggregate

RG-254
RG4-2132

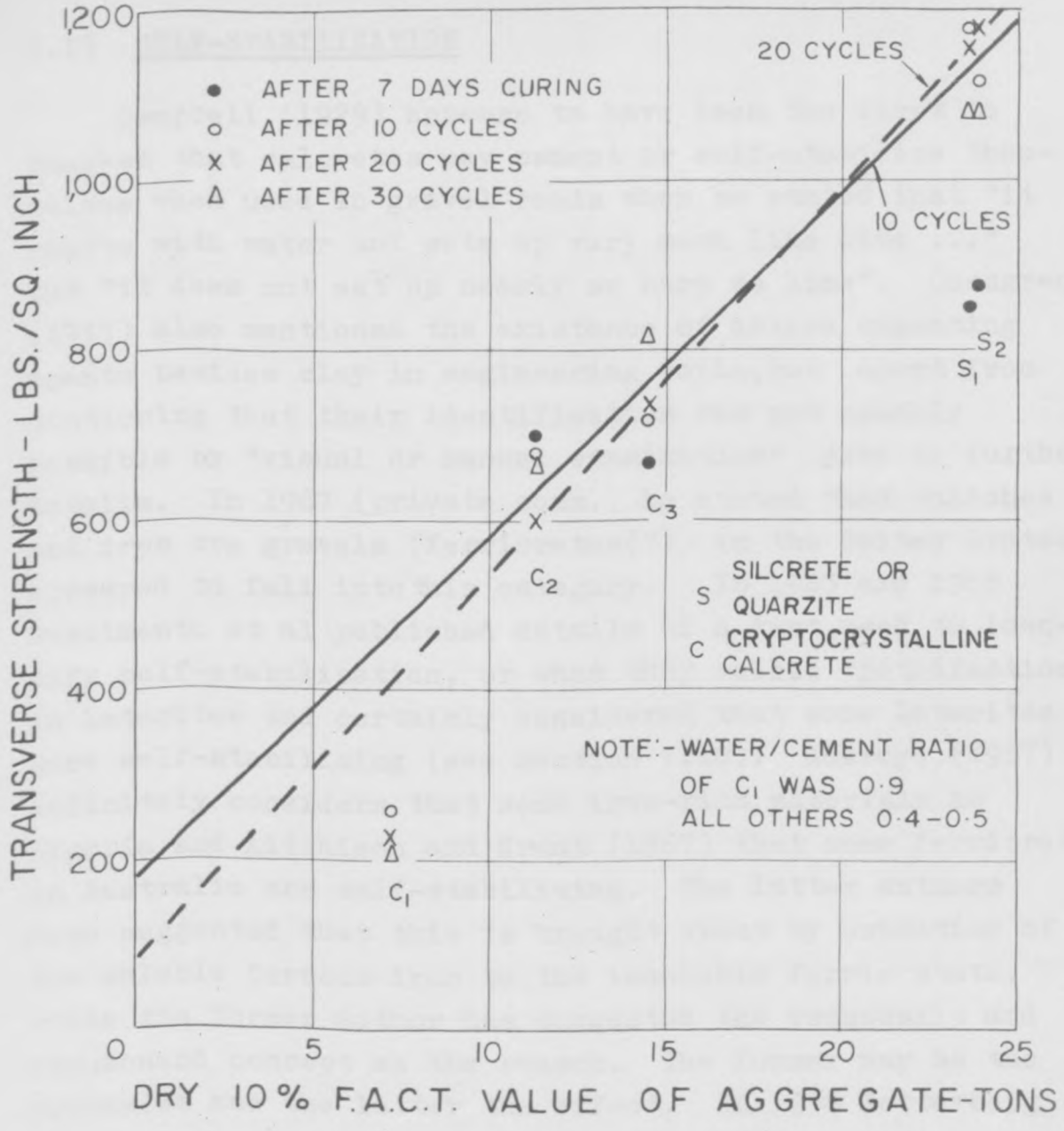


FIGURE 8·32

Relation between transverse strength of concrete bars and 10% F.A.C.T. value of aggregate.

RG-255
RG4-2133

8.13 SELF-STABILIZATION

Campbell (1929) appears to have been the first to suggest that calcretes may cement or self-stabilize themselves when used in gravel roads when he stated that "it reacts with water and sets up very much like lime ..." but "it does not set up nearly so hard as lime". Casagrande (1947) also mentioned the existence of active cementing agents besides clay in engineering soils, but apart from mentioning that their identification was not usually possible by "visual or manual examination" gave no further details. In 1967 (private comm.) he stated that caliches and iron ore gravels (ferricretes(?)) in the United States appeared to fall into this category. In 1963 and 1965 Nascimento et al published details of a test used to identify self-stabilization, or what they called "petrifaction", in laterites and certainly considered that some laterites were self-stabilizing (see Section 7.10). Ackroyd (1967) definitely considers that some iron-rich materials in Nigeria and Aitchison and Grant (1967) that some ferricretes in Australia are self-stabilizing. The latter authors have suggested that this is brought about by oxidation of the soluble ferrous iron to the insoluble ferric state, while the former author has suggested the pseudosilt and pseudosand concept as the reason. The former may be the mechanism and the latter the effect. In 1968 Netterberg recorded the occurrence of typical stabilization-type cracking in the unstabilized ferricrete base of the Ruacana airfield. This base yielded CBRs of 200 plus.

Woods (1960), in discussing soil stabilization with chlorides, has pointed out that a 5% solution of NaCl can dissolve twice as much $CaCO_3$ as pure water and that solution and recrystallisation of small amounts of limestone during moisture content fluctuations could cause cementation in a well graded mix.

Kolbuszewski et al (1965) in their study of Keuper Marl (actually only sometimes calcareous clay by all accounts) mentioned a cementing action and thought it could have been due to iron oxides. Dos Santos and De Castro (1965), in discussing erosion of roads and the effect of using different materials, mentioned that "when the carbonate content is high it seems to exert a favourable influence, possibly due to a cementing action".

Mr. L.E. Sherman, assistant road inspector at Upington reported to the author in 1965 (personal comm.) that "cementing and non-cementing limestones" (calcretes) were known in the Vryburg area, but was unable to supply any further information.

Mr. H.E. Saffrey, chief engineer of the Divisional Council of Port Elizabeth (1967, private comm.) is of the opinion that the calcrete gravel roads in his area may be self-cementing. It is also well known that the calcretes on the Cape Flats near Cape Town have performed well as natural bases in the past, though their gradings were deficient and their PIs rather high (Mr. H. Botha, chief engineer of the Cape Town Divisional Council, 1965; personal comm.) - though their CBRs were often satisfactory.

Mr. J.J. Harvey (1967, private comm.) of the Main Roads Dept. of Western Australia has written that a type of calcrete known locally as "coastal limestone" does possess some self-cementation which is effective after several years exposure in an area of high or moderate rainfall in a temperate climate and that it is considered to be due to alternate solution and precipitation of the calcium carbonate.

Right from the beginning of the field work on this project in 1964 the author too has been struck by the hard surfaces presented by many calcrete gravel roads, even those

possessing/ ...

possessing relatively low plasticity, as well as some freshly compacted layers of some surfaced roads. Many resident engineers, soil technicians, road inspectors and road foremen spoken to were also of the opinion that these roads often cemented themselves in some way. On the other hand, some experienced materials personnel do not believe that any self-stabilization of engineering significance has taken place in their areas in the materials familiar to them, chiefly laterites or ferricretes. It would appear then, that ample evidence exists that self-stabilizing road materials do exist in some areas and that in all probability calcretes can be included in this category. The problem then becomes one of providing adequate proof of the existence of this phenomenon, and establishing an indicator test and techniques to attain self-stabilization in practice.

In order to attempt to determine whether the good performance of the Cape Flats calcrete roads is due to self-stabilization, Mr. T.O.B. Bamber and the Cape Town Divisional Engineer's staff kindly took two large undisturbed samples from two of these roads. One sample came from the east (i.e. south-bound lane) of Duinefontein road about 15 ft from its intersection with the north edge of Landsdowne road and the other came from the south lane of the old alignment of Landsdowne road 0.3 miles away to the west. The samples were unfortunately cracked on arrival in Pretoria and satisfactory cores could not be obtained. Only two six inch diameter cores were eventually tested. The first was 5½" high and yielded a one day soaked unconfined compressive strength of 145 psi (moisture content 13.1%). The second was 4½" high and yielded 235 psi (moisture content 10.7%). Both specimens were capped with plaster of paris and tested without greased end plates. While the specimens must be regarded as unsatisfactory, it does seem as though cementation must have taken place to produce strengths of this order.

8.13.1 Petrification degree

Petrification degree (PD) determinations according to Nascimento et al's (1963) method were carried out in duplicate on 37 calcretes and the ferricrete gravel from the Ruacana airfield thought to be self-stabilizing (Sample 2363). This airfield shows typical stabilization type cracking and the unstabilized ferricrete used yielded CBRs of 200+ (Netterberg, 1968). Sample 2363 came from the base course windrow of a road under construction near the airfield, but was also said to yield an extremely high CBR.

The results on the calcretes are shown in Table H.1 and H.2 in Appendix H They range from 0.40 to 0.79 Samples 1567, 1601 and 1881 yielded the highest values (above 0.70) and should thus have the greatest self-stabilizing potential. In all cases, however, the absorption limits exceeded the liquid limits, while the test theory suggests that the absorption limits of self-stabilizing materials should lie between the liquid limit and shrinkage limits. Since the petrifaction degree is defined as the shrinkage limit divided by the absorption limit, the lower the latter the higher the petrifaction degree (Section 7.10). It is also apparent that an abnormally high shrinkage limit would also result in a high petrifaction degree, and this is certainly a contributory factor in calcretes. The ferricrete thought to be self-stabilizing yielded a PD of only 0.58. A reddish nodular calcrete (2555) deficient in carbonate fines from the same horizon and pit as one whose CBR increased from 83 to 149 on curing for 7 days (as for a stabilized CBR) and soaking for 4 days (Mr. J. Caiger, 1968; personal comm.) yielded a value of 0.63.

While no calcrete has yielded a petrifaction degree approaching unity after only one drying cycle, it has been

shown/ ...

shown in Table 7.12 that values approaching unity can be obtained in some cases after a number of wetting and drying cycles. All six samples tested exhibited an increase.

It is of interest to note that the petrifaction degree of AR quality $CaCO_3$ was found to be reasonably high at 0.84. The absorption limit was 43.5%, the shrinkage limit by the ASTM D427 method 35.0 and by the optional SG method 36.7 (G_s = 2.71). The shrinkage ratio was 1.37. It would be interesting to determine the effect of wetting and drying cycles on the petrifaction degree, shrinkage factors and CBR of pure $CaCO_3$.

8.13.2 CBR after wetting and drying cycles

The results of this series of experiments undoubtedly provide the most convincing evidence for self-stabilization to date. Minus $\frac{3}{4}$" material was screened out of four nodular calcretes and one powder calcrete and compacted into CBR moulds at Modified AASHO effort and OMC. Three moulds of each sample were then tested after the usual 4 days soaking and the rest were subjected to wetting and drying cycles consisting of 1 and 2 days soaking in distilled water followed by 1 and 2 days drying at 105°C. Two day soaking and drying periods were initially employed, but it was later found that one day periods were just as effective and that the moisture content variation was the same. The perforated plates and annular weights were left in place throughout this treatment. After various numbers of cycles the moulds were soaked for 4 days and penetrated as usual. The results are shown in Table 8.13. Most are the means of triplicate determinations, all are at least the means of duplicate determinations.

It can be seen that wetting and drying cycles resulted in a considerable increase in the CBR in most cases, though one sample actually decreased in CBR. The three nodular

TABLE/ ... ..

TABLE 8.13 EFFECT OF WETTING AND DRYING CYCLES ON THE CBR

		4 DAY SOAKED MOD. AASHO CBR(1) (%)						
NO. OF CYCLES(2)		0	2	5	10	0	10	20
DURATION OF ONE CYCLE (DAYS)		2	2	2	2	4	4	4
SAMPLE NO.	PENETRATION							
2114	0.1"	56	83	95	121	64	133	122
2114	0.2"	65	110	124	-	140	148	-
2116	0.1"	55	78	64	107	45	-	132
2116	0.2"	91	127	110	-	80	-	-
2223	0.1"	47	41	20	38	48	24	29
2223	0.2"	71	60	34	60	82	45	50
2555	0.1"	-	-	-	-	41	86	-
2555	0.2"	-	-	-	-	99	-	-
2689	0.1"	21	24	41	-	-	-	-
2689	0.2"	-	49	80	-	-	-	-

(1) Minus $\frac{3}{4}$" material only, no compensation.

(2) One period of soaking plus one equal period of drying equals one cycle.

Operators: Messrs. F. Netterberg and T.E. O'Connor.

calcretes all at least doubled in CBR after 10 cycles.and 2116 trebled in CBR after 20 cycles. It is difficult to ascribe such large increases in CBR to anything other than cementation. It would appear that a minimum of two cycles, each consisting of one day wetting and one day drying, is necessary for significant effect, while five cycles yielded more definite results in most of the samples tested.

The/ ...

The variation in moisture content was assessed in separate moulds of Sample 2116 subjected to the same treatment. It was found that it varied from 8.1 to 8.5% after soaking for either one or two days to 0.2 to 0.4% after drying for either one or two days at 105°C. The optimum moisture content of this material was about 7%. It was realised that this drying temperature was about 30°C higher than that which would be encountered under a blacktop surfacing in South West Africa, but it was found that the variation in moisture content when 2 day soaking and 2 day drying periods at 50°C were employed was only from 8.8 to 9.0% after soaking to 6.2% after drying. This was considered too small for rapid results, but it would be worth setting up a long-term experiement to measure the effect of say 60-70°C cycles.

The effect of cycling without the protection of the perforated plate and annular weight but with the filter paper was also assessed in the case of Sample 2116. The 0.1" CBR after 0, 4, 10 and 20 cycles was found to be 57, 50, 61 and 68, a barely significant increase. It was, however, apparent that erosion of the previously smooth surface took place during cycling, leaving an uneven surface robbed of soil mortar on which the CBR needle had to be seated. The 0.2" CBRs showed a slightly better increase, being 90, 96, 127 and 122. 0.3" CBRs were 111, 127 and 134, the 20 cycle value not being determined. The retaining of the weights during the experiment is thought to be justifiable, since a base course under a bituminous surfacing is subjected both to the weight of the surfacing (considerable when thick surfacings are used) and the loading due to traffic. It would be interesting to repeat the experiment without weights, but with some form of protection (a light perforated plate would be sufficient) for the compacted material.

Effects/ ...

Effects which also need to be assessed are whether any change of density occurs and whether there is any lowering of the soil constants. There is some evidence that the latter may take place, though variability of the samples could also have played a part. (Table 8.14). The 'before' samples were merely grab samples from the bulk samples (several cubic yards), while the 'after' samples were the composite samples taken from the penetrated moulds after 20 cycles.

TABLE 8.14 POSSIBLE EFFECT OF WETTING AND DRYING CYCLES ON THE SOIL CONSTANTS

SAMPLE NUMBER	NO. OF CYCLES	BEFORE/ AFTER	LL %	PL %	PI %	LS %
2114	20	Before After	40.4 27.5	26.1 17.7	14.3 9.8	8.0 3.8
2116	20	Before After	26.1 26.5	15.4 18.2	10.7 8.3	6.0 3.0
2223	20	Before After	24.0 20.2	17.5 -	6.5 S.P.	0.6 0.7

Operator: Mr. T.E. O'Connor

It can be seen that a decrease in PI occurred in all three cases, so that the possibility of sampling errors is reduced, though not eliminated. The liquid limit and linear shrinkage dropped in two out of three cases. It is of course known from Section 7.2.2 that the soil constants of these particular calcretes are affected by heating to 105°C.

In self-stabilization work it may be important to look at 0.2" and 0.3" CBRs as well as 0.1" values. For example, in the upward oven leaching experiment described in

Section/ ...

Section 7.10.1, the mean 0.1" CBR only increased from 55-65 to 79, but the 0.2" CBR doubled from 60-70 to 136. Sample 2116 behaved differently. After 13 days in the oven and 4 litres of water had passed through each mould, the 0.1" CBR of Sample 2116 increased from the usual value of 50-60 to 67, while the 0.2" value only went up from around 90 to 100. The higher CBR penetrations probably afford a better assessment of what is happening deeper in the mould than the 0.1" value.

8.13.3 Mechanisms

There appear to be at least 6 possible ways in which a calcrete could self-stabilize:

1) Cementation by solution and recrystallisation of carbonate.
2) Cementation by oxidation of ferrous iron.
3) Cementation by dehydration of iron and aluminium compounds.
4) Cementation by organic calcium and magnesium complexes.
5) Cementation by the formation of calcium silicates from $CaCO_3$ and opaline silica.
6) Reduction of plasticity and cementation by reaction between $CaCO_3$ and clay minerals to form calcium aluminates and calcium silicates.

These mechanisms will be briefly discussed in turn.

1) Cementation by $CaCO_3$

The two carbonates in calcretes, calcite and dolomite, are slightly soluble in water. The factors influencing their solubility have been dealt with in detail in Section 4.2.4. It will suffice to state here that changes in

moisture/ ...

moisture content, pF, salt content, temperature and the partial pressure of CO_2 in a road base will all cause changes in this solubility. In addition, there is also evidence that soluble matter is likely to migrate upwards in a soil profile (Carey, 1966) and in a base whether under a blacktop or not due to the daily temperature cycle. This will also occur due to suction gradients in gravel roads and in blacktop roads which are not perfectly sealed. Moisture content changes also occur in the base during the daily and annual temperature cycle, though there seems to be surprisingly little information on this (see for example Netterberg, 1969b). A study of data to a depth of 18" for five roads in Rhodesia kindly made available by the Rhodesian Ministry of Roads central laboratory (1961) suggests that the seasonal fluctuation in bases under blacktops in Rhodesia varies from about 0.5% to 5%. The seasonal variation as well as the average moisture content increased downwards in these shallow profiles. Average moisture contents were about half to three quarters of optimum. The writer of the report concluded that too much reliance should not be placed on these individual results determined by oven drying and that the work should be repeated, using more sophisticated techniques. The very limited work of Netterberg and Jamieson (1968) in South West Africa suggested that significant diurnal variations of moisture content were possible. In this work it is obviously of importance to distinguish between liquid and vapour water, though this is seldom done.

The net result of these effects is likely to be the preferential solution of carbonate at points of intergranular stress, its recrystallisation elsewhere as a cement and void filling, and a general migration of carbonate from depths of one to two feet to the upper few inches of the base. Both an increase in density and an increase in cohesion can thus be expected. In addition, the coarsening of the grading thus achieved will tend to

reduce/ ...

reduce the plasticity.

Stated in the simplest terms, all that happens is that calcrete simply continues to form in the road itself. Given suitable conditions and long enough, it would eventually form a rock-like hardpan crust at the top of the base. All that is required in practice to turn a compacted calcrete into the equivalent of a lean concrete base is to find a means of achieving in one year or less what takes nature 10,000 years or more. Putting a blacktop on probably already helps, but means of achieving this will be discussed in Chapter 9.

Only calcretes containing strongly calcareous fines could undergo self-stabilization by this mechanism. It can probably be detected by strength or petrifaction degree testing before and after wetting and drying cycles.

2) Oxidation of ferrous iron

Aitchison and Grant (1967) have suggested that ferricretes self-stabilize by oxidation of ferrous iron. This is a very likely mechanism and is in fact probably how ferricretes form in nature. Under natural conditions iron in the ferrous state (Fe^{++}) is likely to be more soluble than iron in the ferric (Fe^{+++}) state. Oxidation by drying, heating, or exposure to the atmosphere is likely to cause precipitation of the iron as insoluble ferric oxides or hydroxides. Investigation into the origin of ferricretes will show how this mechanism works. For example, G.W. Robinson (1949), quoting W.O. Robinson (1930) mentioned that "in the presence of organic matter, submergence is followed by a rapid rise in the solubility of iron, manganese and magnesium. Solution is affected by the presence of carbon dioxide from the microbiological decomposition of organic matter. The iron comes into solution in the ferrous state, probably as ferrous hydrogen carbonate. Where organic matter is absent, or present in

small/ ...

small amount, the solubilization of iron, manganese, calcium and magnesium is negligible".

Consideration of the iron contents of calcified sands, powder calcretes and nodular calcretes (Tables 3.1-3.3) show that up to 4.5% FeO and up to 2.3% Fe_2O_3 has been found in these types. Table C.9 shows that up to 3% FeO has been found in the minus 40 mesh fraction. How much of this is available for self-stabilization is not known, but 2 or 3% might well be enough to cause some cementation. Available ferrous iron would only be stable under wet, reducing conditions (particularly in the presence of organic matter) and may perhaps exist in solution in natural calcretes as ferrous bicarbonate. It is not known as a solid. Siderite (ferrous carbonate) concretions are known in nature, but this mineral has not so far been detected in calcretes. It probably becomes oxidised to Fe_2O_3.

There is evidence (Walkley, in R.R.L., 1952; and Jackson, 1958) that all free ferrous iron becomes completely oxidized and thus insoluble after 2 days of air drying. It is also significant that the exchangeable ferrous iron test given by Jackson requires that it be done in the field within minutes of sampling. This may be the reason why the Ruacana ferricrete and Sample 2555 (both tested months after sampling) only yielded petrifaction degrees of 0.58 and 0.63 respectively. Ferricrete (or calcrete containing ferrous iron), fresh from a damp borrow pit, and compacted before drying out, can be expected to retain any self-stabilizing properties due to iron. Material which is fossil, outcropping, and which has been oxidized by drying, cannot. This is thought to be the crux of the self-stabilization controversy.

It is also of interest to note from Hough (1957, p.471) that iron saturated montmorillonite had the lowest liquid

limit/ ...

limit and PI (220) of all and that subjecting this to 5 wetting and drying cycles reduced the PI to 74. The PI of iron illite was halved by this treatment, but it had no effect on kaolinite. The oxidation states of the iron were unfortunately not stated. It would be interesting to know whether the clays were initially saturated with Fe^{++}. Wetting and drying cycles would certainly have oxidised this to Fe^{+++}.

In conclusion, it can be stated that only calcretes with significant amounts of ferrous iron in some readily available form can be expected to undergo self-stabilization by this mechanism. These calcretes will probably possess negative redox potentials (E_H) and contain more than the usual amount of organic matter. Self stabilization due to this mechanism should be detectable by a petrifaction degree test or a strength test before and after drying on material which has not previously been permitted to become oxidised by heating, drying, or excessive exposure to oxidising agents (such as the atmosphere).

3) Dehydration of iron and aluminium compounds

Dehydration of iron and aluminium hydroxides and hydrated oxides to oxides is probably a powerful cementing mechanism. It is probably also irreversible.

Other mechanisms may also possibly be important in calcretes. For example, $Fe(OH)_2$ and $Fe(OH)_3$ or ferrous carbonate will probably be precipitated from any solution containing iron by $CaCO_3$. Insoluble calcium ferrate, $CaFeO_4$, is probably precipitated when soluble Na_2FeO_4 comes into contact with $CaCO_3$. The ferrites, e.g. $Ca(FeO_2)_2$ and Ca_2Fe_2O5, are soluble in alkaline solutions. Aluminium will behave in a similar fashion to iron, but forms only one ion, Al^{+++}. These compounds which are precipitated by $CaCO_3$ are probably already in the insoluble Ca forms in calcretes containing strongly calcareous fines. Whether the reactions are reversible/ ...

reversible has not been investigated. These reactions are probably all governed largely by E_H and pH and, quite possibly, by pF through its effect on P_{CO_2} and hence pH.

Self stabilization due to this mechanism should be detectable after one or more drying cycles, which may possibly have to be lengthy (several days) for full effect.

4) Organic dibasic complexes

Wooltorton (1954) has stated that organic matter plus divalent bases (such as Ca^{++} and Mg^{++}) form a very stable, water resisting cementing agent after drying out. Such complexes have not yet been identified from calcretes, but the methods used in the study of their composition did not differentiate between the different forms of organic matter. Organic matter may well play a more important role in the formation of calcretes and ferricretes and in their self-stabilization (probably merely a continuation of their formation) in the road than is realised. Once again a drying cycle appears to be necessary.

5) and 6) Reaction between $CaCO_3$, opaline silica and clays

If the soil suction theory of calcrete formation outlined in Sections 4.2.4 and 4.2.5 is correct, that is pF does affect the solubility of $CaCO_3$ through its effect on P_{CO_2}, then pF will also affect the pH of a $CaCO_3$-water system, probably something after the manner indicated in Figure 8.33 if the system is in equilibrium with atmospheric air (P_{CO_2} = 3.2 x 10^{-4} atmospheres). Though its measurement has not been attempted, it seems fairly certain that the pH of a calcrete base course can rise to at least 10 on heating to 70°C and/or drying out and Figure 8.33 suggests that even higher pHs, perhaps up to 11.5 will be encountered. These high pHs are due to the formation of $Ca(OH)_2$ from $CaCO_3$ and a certain amount of

reaction/ ...

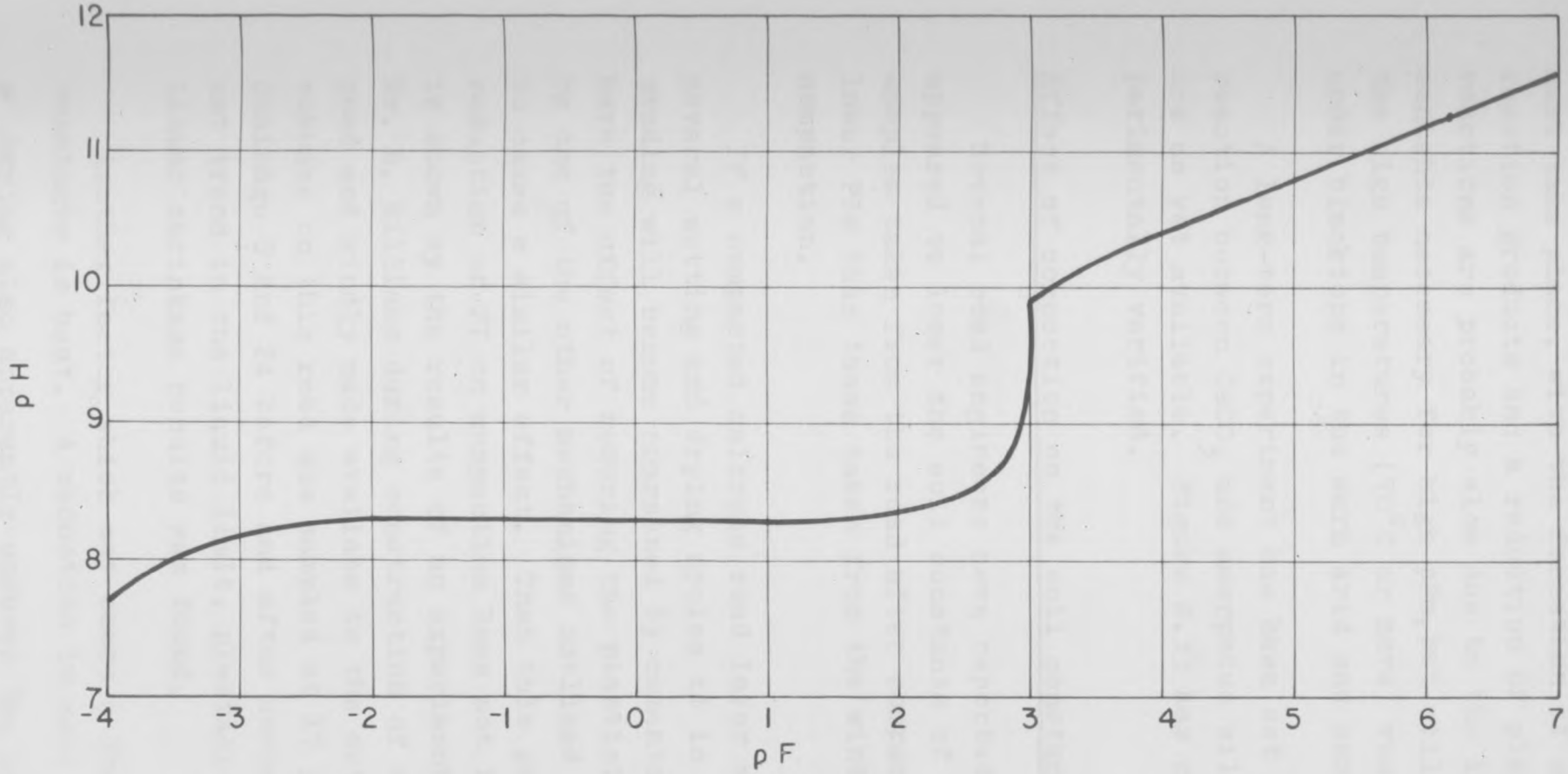


FIGURE 8·33

POSSIBLE EFFECT OF SOIL SUCTION ON THE pH OF THE CALCITE-WATER SYSTEM AT 25° C

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reaction between pozzolans like opaline silica and clays must take place, with the formation of cementitious reaction products and a reduction of plasticity.* These reactions are probably slow due to the low moisture contents necessary for high pFs, but will be speeded up by the high temperatures (70°C or more) reached by bases under blacktops in the warm arid and semiarid environment.

A long-term experiment has been set up to detect reaction between $CaCO_3$ and amorphous silica, but no results are as yet available. Figure 8.33 has not yet been experimentally verified.

Effect of compaction on the soil constants

Several road engineers have reported that compaction appeared to lower the soil constants of calcretes in that samples taken from the road after compaction often yielded lower PIs than those taken from the windows before compaction.

If a compacted calcrete road layer was subjected to several wetting and drying cycles it is likely that the grading will become coarsened by cementation and this could have the effect of reducing the plasticity. Cementation by one of the other mechanisms outlined could be expected to cause a similar effect. That this phenomenon of a reduction of PI on compaction does not invariably occur is shown by the results of an experiment carried out by Mr. R. Williams during construction of the Wasser-Asab road and kindly made availabe to the author. The calcrete subbase on this road was sampled at 17 points between chainage 0 and 24 before and after compaction. No consistent trend in the liquid limit, plasticity index or bar linear shrinkage results was found.

Another factor which may cause a lowering of soil constants is heat. A reduction in soil constants can be

* Drying also apparently ensures the attainment of full cation exchange equilibrium (Wooltorton, 1954).

expected if for any reason the samples taken after compaction are heated, while those taken before compaction were not. This might occur when material taken from density holes is used for Atterbergs after drying.

8.14 CHEMICAL STABILIZATION

As there is still much to be understood in the field of chemical stabilization and no recent review paper is available, a brief survey of the present position is given before tackling the aspects concerning calcretes.

A point which must be stressed at this juncture is that the 'lime' in calcretes is $CaCO_3$ (calcite) and/or $CaMg(CO_3)_2$ (dolomite). These two substances are very different from the 'lime', $Ca(OH)_2$ often with some $Mg(OH)_2$, normally used as a stabilizer in road construction and in building work. A road failure once resulted in Australia through the contractor using $CaCO_3$ instead of $Ca(OH)_2$. Throughout this work the term 'lime' will be used only to mean the commercial product consisting mainly of $Ca(OH)_2$. To be absolutely correct this should be called slaked lime, where the unslaked product is CaO (usually known as quicklime). Burnt and slaked calcrete (i.e. $Ca(OH)_2$) has been used to stabilize soil in India (Prasad et al, 1965) and there is no reason why it should not be used to stabilize other calcrete.

8.14.1 Introduction

The methods of stabilization in common usage may be classified as follows:

1) Mechanical (adjustment of the particle size distribution) - usually involves screening out of plastic fines and their replacement with non-plastic binder.

2)/ ...

2) Chemical - usually involves addition of about 4% of lime or cement.

3) Bituminous - stabilization by admixture of bitumen or tar (also of the order of 4%).

Gregg (1963) has also divided stabilization into stabilization by

a) modification which may be defined as "the addition of an agent to a soil in order that its existing physical properties may be controlled or limited to provide a specific range of strength" and

b) stabilization by cementation which "implies that a substantial increase is made to the natural soil strength by the addition of an agent which forms a strong bond between the soil particles". If the effects of chemical stabilization are classified in order relative to the amount of the stabilizer added, they can be listed as follows: conservation (e.g. prevention of further decomposition of a basic igneous rock), modification and cementation (Dr. K.A. Clauss, 1969; personal comm.).

Mechanical stabilization, or the screening and/or blending of different materials deficient in some respect (usually grading or PI) so as to bring the end product up to standard, is widely practised both in this country and overseas on all types of road materials, including calcretes. This method will therefore not be dealt with further here.

Maclean (1955), Woods (1960), Fossberg and Gregg (1963) and Fossberg (1963b) have described the principles of stabilization by lime, cement and bituminous materials and the methods used, the latter two authors considering mainly South African practice. Standard review articles on lime and cement stabilization are Highway Research Board (1961) and (1961a), respectively. Which method is used depends

on/ ...

on the effect which is desired, the soil type, and to a lesser extent, economic considerations. Lime is usually used mainly to bring about a reduction in plasticity, while cement is normally used to affect an increase in strength. Bitumens supply cohesion to cohesionless materials (e.g. windblown sands) and act as a waterproofer for slightly cohesive materials. A guide to the type of stabilization commonly employed is shown in Figure 8.34 (U.S. Bureau of Public Roads, in Fossberg, 1963b). Comparison of Figure 8.34 with the nodular calcrete test results in Appendix H suggests that lime and cement stabilization should be the most useful methods on calcretes. This is in fact exactly what has been found to be the case, in South Africa at least, though asphalt stabilized caliches have been used overseas.

The only available publication relevant to the cement stabilization of a calcrete road seems to be that of the Texas Highway Department (1940) which briefly describes the stabilization with 8% cement of a failed powder calcrete base course having a grading modulus of 0.85 and a PI of 15. Marais (1965) in an unpublished report has also described the use of cement to treat a failed lime stabilized calcrete base course near Naboomspruit.

Gregg (1965) has made a detailed laboratory study of the cement stabilization of a sand, including tensile and flexural strength testing. Of particular interest were the findings on fatigue strength and optimum mixing times. While a greater delay between mixing and compaction is allowed in the case of lime - soil mixtures than is allowed for cement - soil mixtures, the danger of this, as well as of the practice of allowing compaction at moisture contents higher than optimum (in order to allow for hydration) has been shown in laboratory studies by Croft (1967). In 1966 Gregg made a laboratory study of the use

of/ ...

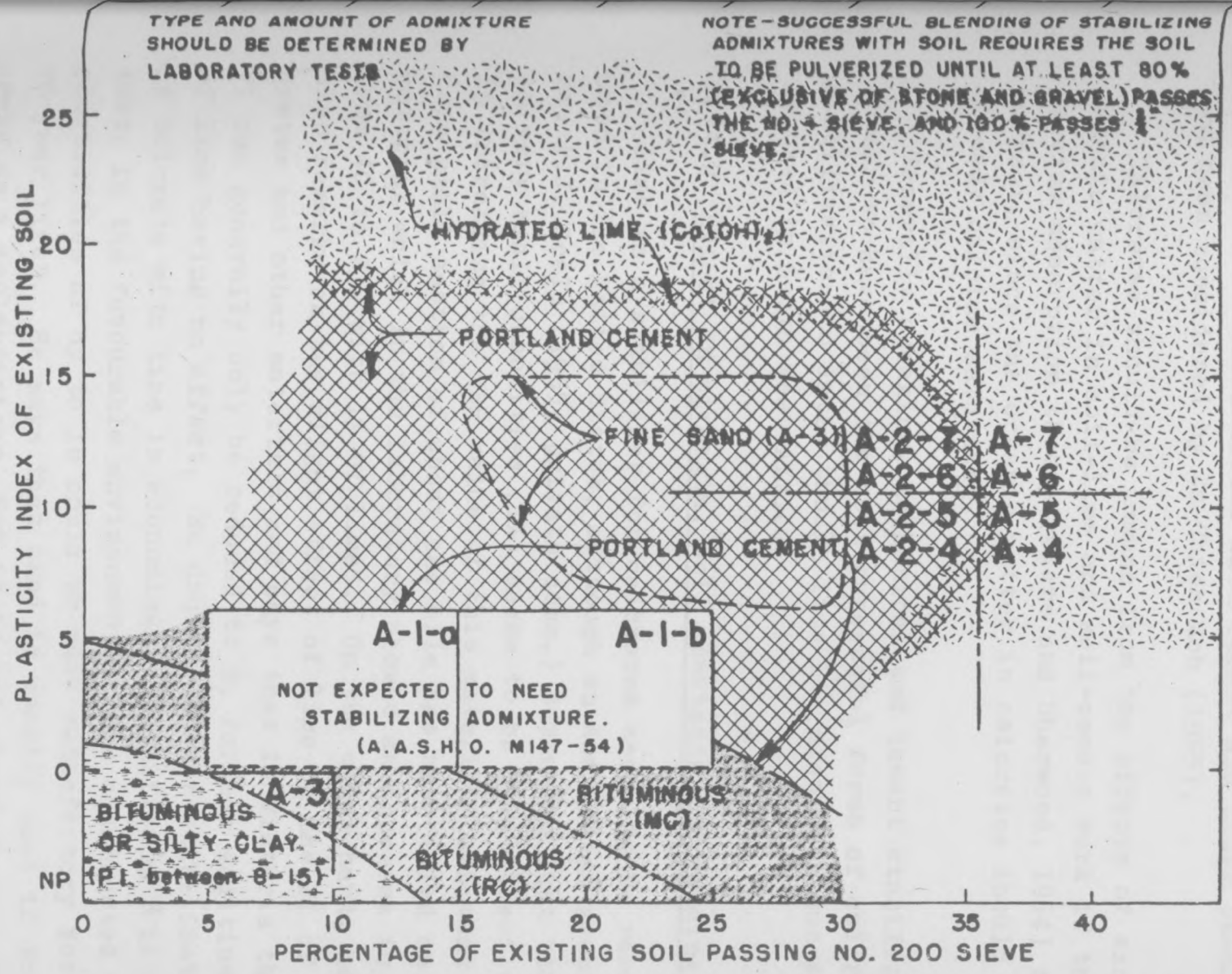


FIGURE 8·34 – SUGGESTED STABILIZING ADMIXTURES SUITABLE FOR USE WITH SOILS AS INDICATED BY PLASTICITY INDEX AND AMOUNT PASSING NO. 200 SIEVE.

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of a hydrophobic blast-furnace cement in soil stabilization. The effects of repeated loads on cement stabilized micaceous soils have been studied by Whittle and Larew (1965). The reactions of soil minerals with cement and other chemicals have been considered by Za-Chieh Moh (1965).

Calcretes should be immune from the effects of excessive organic matter in soil-lime and soil-cement work if the calcium adsorption theory (Clare and Sherwood, 1954) is correct, since the organic matter in calcretes should be already calcium saturated.

Generally speaking, mechanical and cement stabilization appear to have been the most successful forms of calcrete stabilization to date, though lime has also been successfully used in South West Africa.

8.14.2 Effect on strength, soil constants and grading

Lime as a stabilizer for calcretes seems to be rarely used in the United States, although on Australian practice, Mr. A.J. Scala (1965, private comm.) commented that "lime in very small quantities (1%) seems to be very effective in increasing the CBR value of this material" and that "one small section of about one mile has been tried on material which in the laboratory showed an increase in CBR from 40 to 200 with 1% of lime". On the other hand, he also remarks on the slight effect of lime on the PI of calcretes and other materials and says that in Victoria the PI can generally only be reduced to 9, further addition of lime having no effect. He doubts whether stabilization of calcrete with lime is economically justified "as it is felt that, in the favourable environment, roads constructed with plasticities of up to 18 could be made satisfactory for a 20 year life". He says that sand is usually used in these areas as a deplasticiser, but it is not clear whether the only experience with lime has been that mentioned.

In/ ...

In this country lime stabilization in general has been used fairly extensively in Natal, while in S.W.A. this has become a common method of stabilizing calcretes, and large increases in strength have been reported. At the Ondangua airport for example the 7 day moist cured, 4 day soaked CBR was doubled or trebled from about 100 or less to 200 to 250+ by the addition of 4% hydrated lime. CBRs as high as 390 at only Proctor compaction were attained. Laboratory tests showed that lime stabilization yielded higher CBRs than cement stabilization on this particular job. At Ondangua, the material had an average grading modulus of about 2.3 and a PI of about 4 before stabilizing (natural base course). After stabilizing, the PI dropped to 0-2. Average optimum moisture content was 11-13%. Somewhat similar results were obtained by the Department of Transport with the calcretes at the Katima Mulilo airfield. Initial CBRs of the calcretes alone were about 50-70. 1:1 calcrete-sand mixtures yielded CBRs around 60, while the same mixtures with 3% lime yielded CBRs of up to 368. Maximum dry densities were about 110 lbs/cu ft and optimum moisture contents around 13% for the mixtures. The optimums for the pure calcretes ranged up to 21%. The Grootfontein airport calcretes used for subbase also yielded CBR's of 200+ with 4% lime. Both the Ondangua and Grootfontein calcretes were good quality hard nodular calcretes, while the Katima Mulilo materials were rather powdery. Between Otavi and Okaputa, lime stabilized calcretes yielded CBR's of 240+. These too were mostly nodular calcretes.

Not all calcretes respond so well to lime from a strength point of view. The soils investigation for the Runtu airfield showed cement to yield higher CBRs than lime in this case. Calcretes on the Lichtenburg-Deelpan road also react better with cement than with lime. On this road even calcretes with modified AASHO CBRs of 17 yielded CBRs of 200+ when stabilized with 3% PBFC and were reduced to non - or slightly plastic.

It/ ...

It should be noted that while long term strength increases of possibly this order are known (Dawson and McDowell, 1961; Biszewki, 1963) it appears to be most unusual to achieve high early strengths by means of lime stabilization, though De Wet and Skinner (1966) have reported rapid strength gains when working with pure clays stabilized with large quantities (20-100%) of lime at higher temperatures (e.g. 55°C). Any reaction at 55°C would of course be expected to be about 3 times as fast as one at 25°C. The former temperature is probably often encountered during lime stabilization of calcretes in S.W.A. Ruff and Ho (1966) have found that the lime-bentonite reactions taking place above 50°C are not the same as those taking place below 50°C. The effect of lime additions to soil - cement mixtures has been studied by De Sousa Pinto et al (1962), who found that lime decreased the sensitivity to clays between mixing and compaction of soil-cement mixtures and that the highest strength gains were obtained with clayey soils and the lowest with calcareous soils. The reason for this is obscure, but it may be a feature of the particular soils used. Calcareous soils are likely to be already Ca saturated and flocculated. The "limestone" bases reported on by McDowell might have been calcretes.

Tables 8.15 and 8.16 compare the effects of some different stabilizers on Sample 2116 from the Lichtenburg-Deelpan road. This sample was known to be one which reacted very well with cement to yield exceptionally high CBRs. Cylindrical specimens 8" long and 4" in diameter were made up with various stabilizers from -3/4" material from Sample 2116, compacted to Modified AASHO density and OMC in a split-walled mould, cured for various periods at 100% relative humidity and room temperature, soaked for one day and tested at a rate of strain of 0.050" per minute in unconfined compression and Brazilian indirect tensile.

The/ ...

The latter test simply involves loading the cylindrical specimen diametrically instead of axially. The plywood packing strips advised by Wright (1955) were used. The indirect tensile strength was taken as $2P/\pi dh$, where P is the load, and d and h the diameterand height of the test specimens respectively (Frydman, 1964). The results shown are the averages of two or three specimens. Good agreement was obtained between individual specimens.

TABLE 8.15 EFFECT OF CURING TIME ON THE STRENGTH AND SOIL CONSTANTS OF A LIME STABILIZED NON-LIME REACTIVE CALCRETE

TEST	% LIME	TOTAL CURING TIME (days)				
		3	7	14	28	56
UNCONFINED COMPRESSIVE STRENGTH (psi)	4	40	51	77	71	103
	10	53	74	57	101	132
BRAZILIAN TENSILE STRENGTH (psi)	4	2.7	4.6	6.5	5.7	11.1
	10	3.5	5.5	6.1	7.7	11.2
MOISTURE CONTENT AT TESTING (%)	4	13.4-14.2	11.9-15.3	13.4-14.8	12.9-14.3	14.1-16.2
	10	11.9-14.7	11.5-14.1	13.6-17.9	14.0-16.2	13.2-16.5
LIQUID LIMIT (%)	4	27.3	26.7	26.3	-	-
	10	28.2	29.4	29.6	-	-
PLASTIC LIMIT (%)	4	24.1	-	-	-	-
	10	24.5	-	-	-	-
PLASTICITY INDEX (%)	4	3.2	SP	SP	-	-
	10	3.7	SP	SP	-	-
FLOW INDEX (%)	4	5.8	3.4	6.4	-	-
	10	8.2	5.8	3.6	-	-
TOUGHNESS INDEX (%)	4	0.6	~ 0.3	~ 0.2	-	-
	10	0.5	~ 0.2	~ 0.3	-	-
LINEAR SHRINKAGE (%)	4	1.3	1.0	1.3	-	-
	10	2.3	1.7	2.0	-	-

Operators: Messrs. T.E. O'Connor and F. Netterberg.

TABLE 8.16 EFFECT OF SOME DIFFERENT STABILIZERS ON THE STRENGTH OF A NON-LIME REACTIVE CALCRETE

STABILIZER	4% CEMENT		4% LIME + 2% SILICA GEL	4% SODIUM SILICATE
CURING TIME (days)	7	28	7	7
UNCONFINED COMPRESSIVE STRENGTH (psi)	276	451	207	208
BRAZILIAN TENSILE STRENGTH (psi)	29	45	31	-
MOISTURE CONTENT AT TESTING (%)	13.4-15.8	12.1-15.8	12.3-14.1	13.8-14.7
PLASTICITY INDEX (%)	-	SP	-	-

Operators: Messrs T.E. O'Connor and F. Netterberg

The results show that this particular calcrete does not react at all well with lime from a strength point of view. It was known, however, to react well with portland blast furnace cement and material from the same borrow pit had been used stabilized in this way for base in the Lichtenburg area. The results in the table show that it also reacts well with ordinary portland cement to yield about 276 psi at 7 days. An addition of amorphous silica in the form of -200 mesh silica gel powder yielded a large strength increase to near that attained by cement. If more silica gel had been added the strength would probably have been even higher. In the case of the 10% lime stabilized specimens, the PI dropped from 11 to 4 after 3 days and to slightly plastic at 7 days, suggesting that

the/ ...

the lime was reacting with the palygorskite clay mineral present to yield the only slightly cementitious calcium aluminate hydrates. X-ray diffraction analyses were inconclusive, but suggested that C_3AH_6 was formed at 7 days and was replaced by $C_{11}AH_{10}$ at 14 days. No reference could be found in the literature to the reaction products of lime-palygorskite or lime-sepiolite mixtures. In view of their importance as clays in calcretes (Section 3.4), some work on the subject is definitely justified. A preliminary experiment carried out for the author by Mr. J. de Wet suggests that the palygorskite-lime reaction is more rapid than that of other lime-clay reactions, possibly due to the fibrous structure of the clay. The reaction products were tentatively identified as calcium silicate hydrates.

Comparison of the soil constants in Table 8.16 with those of the original unstabilized soil (Table H.2) shows that lime stabilization raised the liquid limit and plastic limit and lowered the plasticity index, toughness index and bar linear shrinkage. The behaviour of the flow index was erratic. A detailed examination of the effect of lime and cement on the soil constants of calcretes has not been made, but it is known that the PI and LS are usually reduced and that the liquid limit may be increased. The latter is probably not deleterious under South African conditions. Information on the effect of lime and cement on the shrinkage limit is only available for Samples 2116 and 1569. In the case of 2116, addition of lime or cement raised the shrinkage limit from 15 to 23-27 and decreased the shrinkage ratio from 1.95 to 1.50-1.66. Lime had the greater effect on both properties. In the case of 1569, the shrinkage limit was raised from 20 to about 31 and the shrinkage ratio decreased from 1.72 to around 1.40. Both lime and cement had about the same effect. It would appear that the soil constants of calcretes behave normally, as Herrin and Mitchell (1961) and the Highway Research Board (1961a) have stated that the liquid limit may be in-

creased/ ...

creased (usually in the less plastic soils), and that a decrease in PI, swelling, shrinkage and shrinkage ratio and an increase in plastic limit and shrinkage limit is normal. A coarsening of the grading is also normal, and the few calcrete test results available also exhibit this. Examination of the soil survey data shows that addition of 3-5% PBFC to the calcretes on the Lichtenburg-Deelpan road about halved the percentage passing 200 mesh and almost halved the percentage passing 40 mesh.

8.14.13 Effect on compaction characteristics

Lime usually increases the optimum moisture content and decreases the maximum dry density attainable at constant effort (Herrin and Mitchell, 1961) while the effect of cement depends on the soil type and an increase or decrease is possible (Highway Research Board, 1961a). It seems obvious that the effect of both lime and cement will actually depend on the soil texture. Examination of a number of test results from the soil survey data for the Lichtenburg-Deelpan and Operet-Oshakati contracts shows that the 3-5% PBFC used on the former contract and the lime used on the latter contract decreased the maximum dry density and increased the optimum moisture content in both cases. The actual effect to be expected in any particular case must depend on the particle size distribution by volume before and after stabilization and the rate of reaction. The latter is probably particularly important with those calcretes which contain amorphous silica in the form of diatoms, since the reaction will be so fast that the delay between mixing and compaction will become a critical factor.

Figures 8.35-8.38 show the effect of lime and cement on two nodular calcretes from the Lichtenburg-Deelpan road. The method used employed the usual technique (Department of Transport, 1958) with the mixing procedure of Gregg, 1965).

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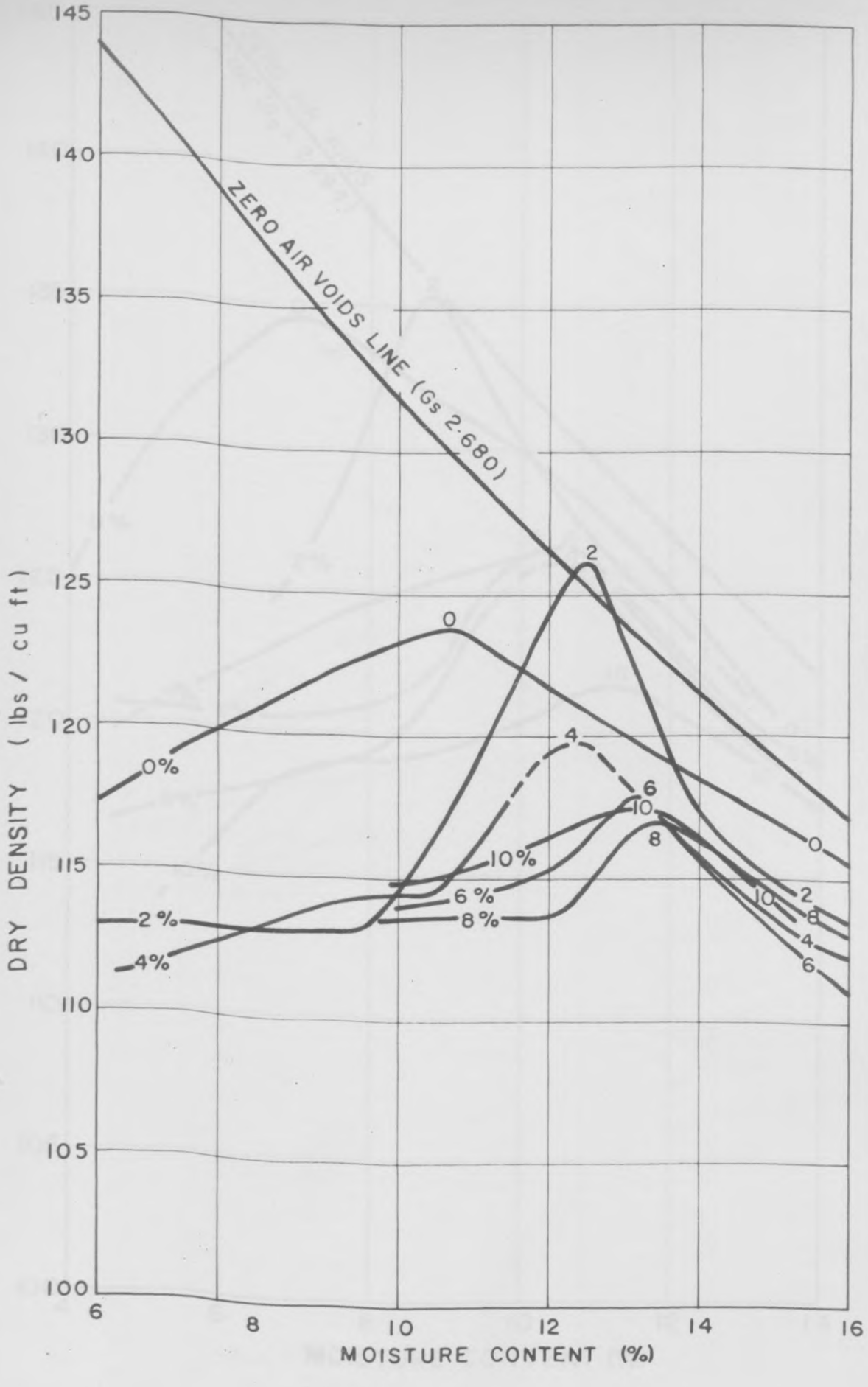


FIGURE 8·35

EFFECT OF LIME ON MODIFIED AASHO OPTIMUM MOISTURE CONTENT AND MAXIMUM DRY DENSITY (SAMPLE 2114)

RG4-3770

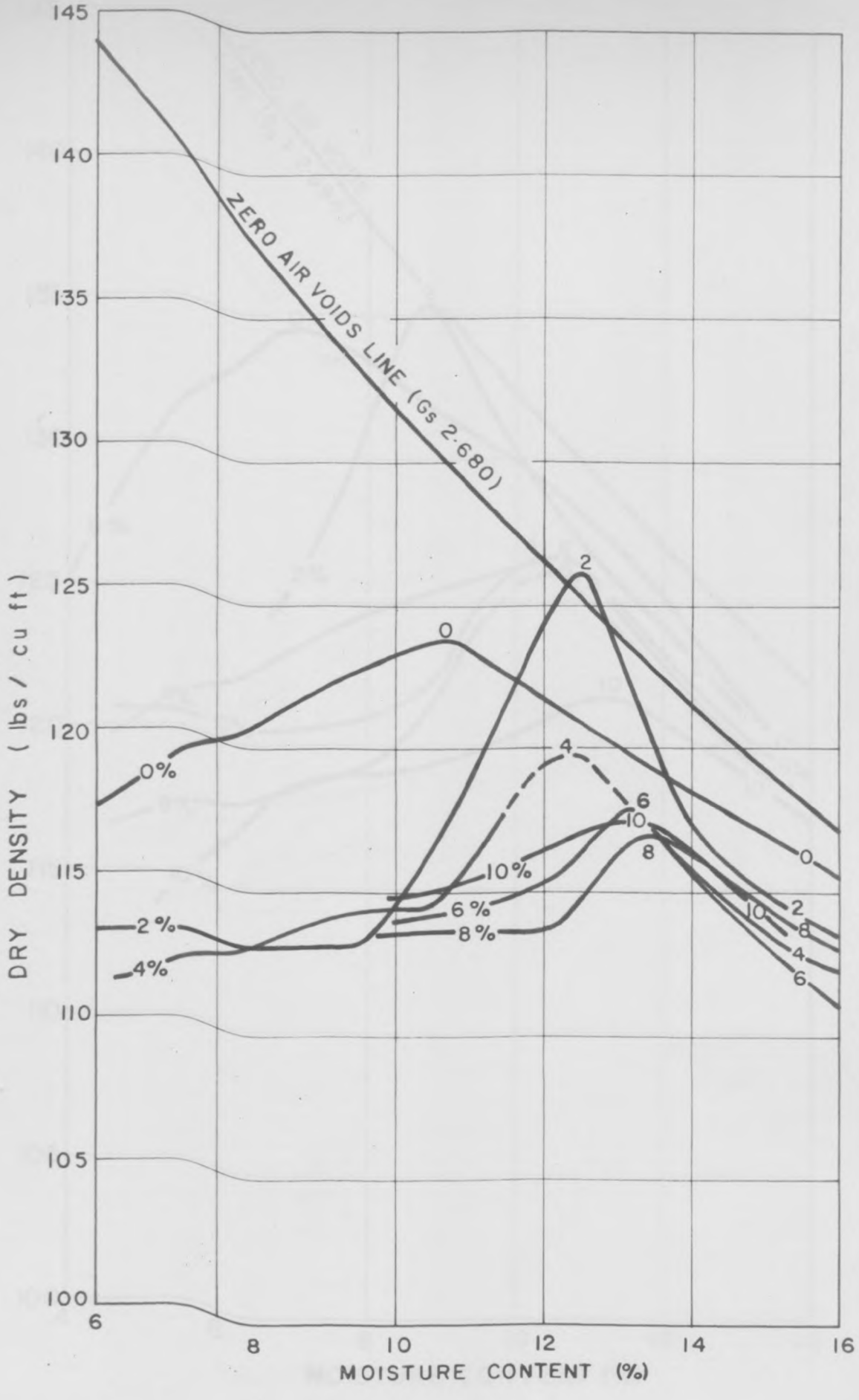


FIGURE 8·35

EFFECT OF LIME ON MODIFIED AASHO OPTIMUM MOISTURE CONTENT AND MAXIMUM DRY DENSITY (SAMPLE 2114)

RG4-3770

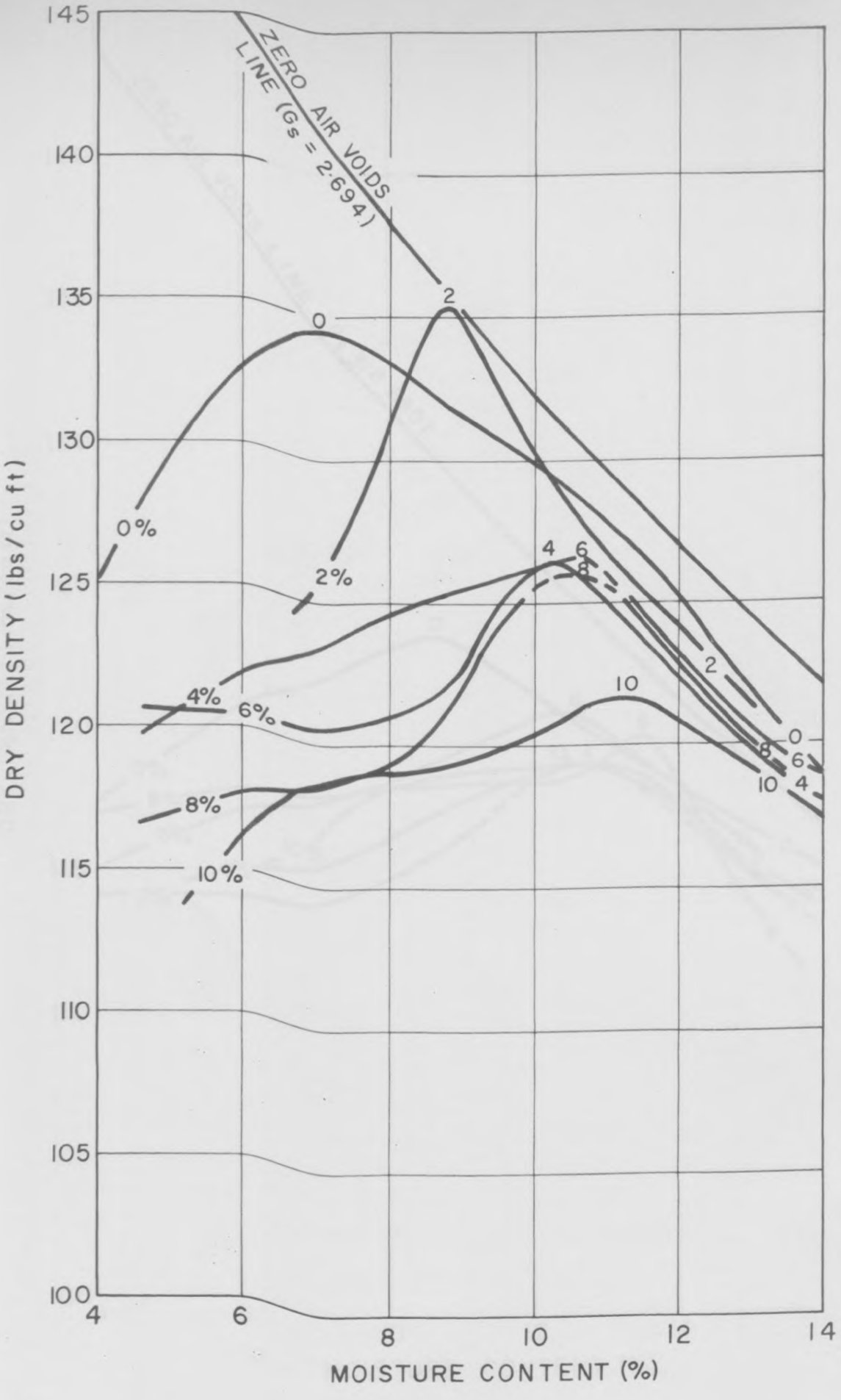


FIGURE 8.36

EFFECT OF LIME ON MODIFIED AASHO OPTIMUM MOISTURE CONTENT AND MAXIMUM DRY DENSITY (SAMPLE 2116)

RG4-3771

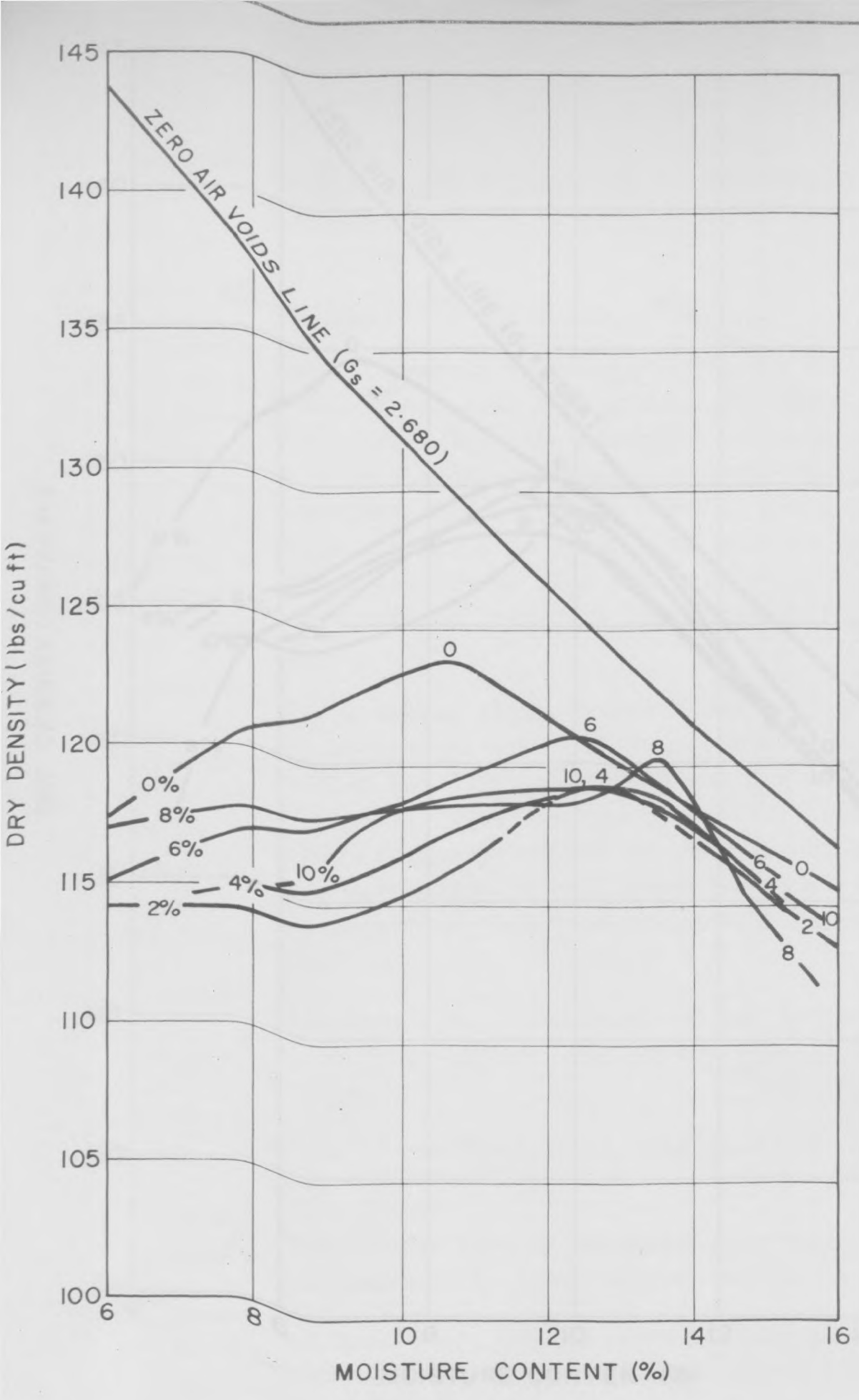


FIGURE 8.37

EFFECT OF CEMENT ON MODIFIED AASHO OPTIMUM MOISTURE CONTENT AND MAXIMUM DRY DENSITY (SAMPLE 2114)

RG4-3772

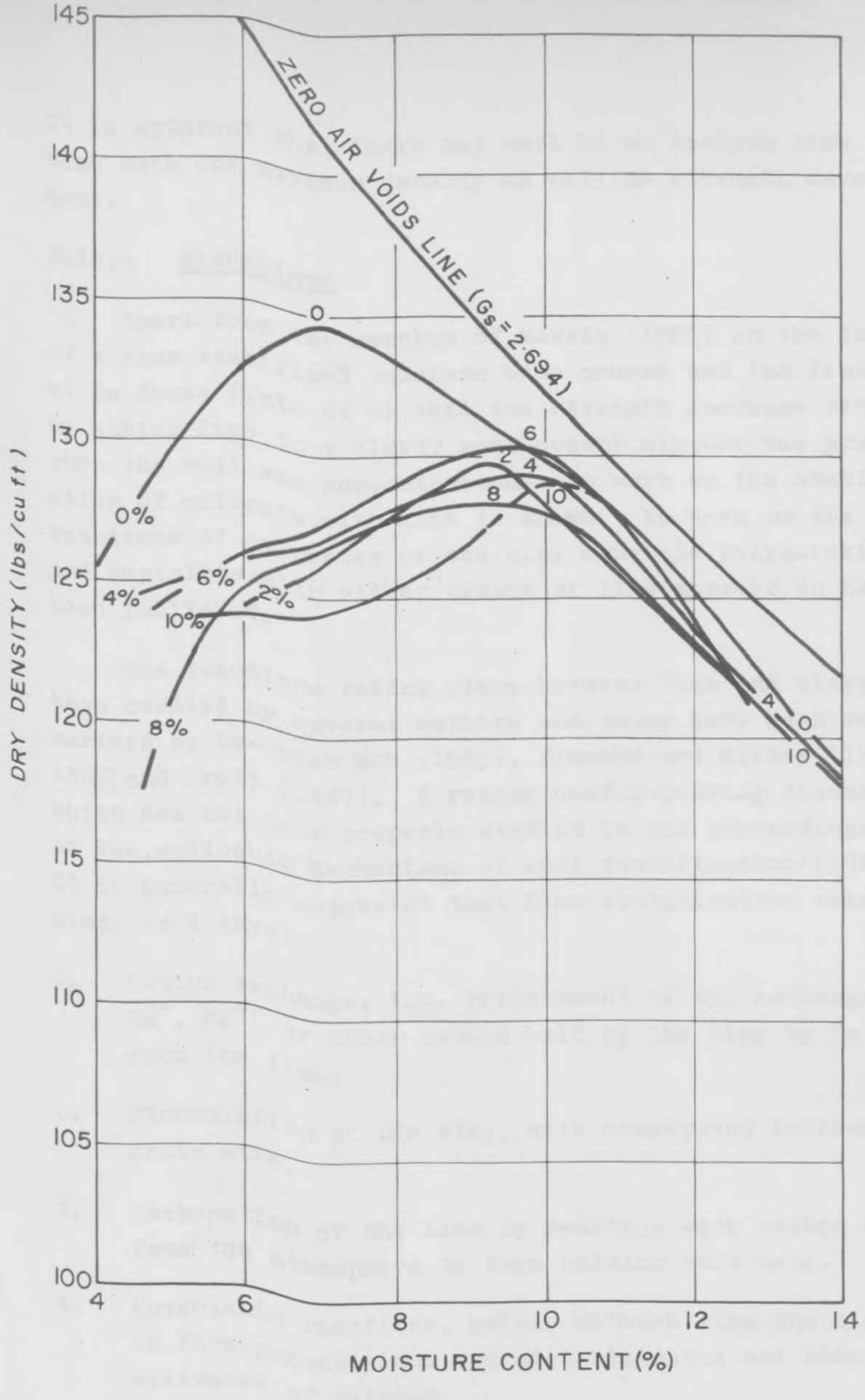


FIGURE 8.38

EFFECT OF CEMENT ON MODIFIED AASHO OPTIMUM MOISTURE CONTENT AND MAXIMUM DRY DENSITY (SAMPLE 2116)

RG 4 – 3773

It is apparent that there may well be an optimum lime content both for maximum density as well as strength development.

8.14.4 Mechanisms

Apart from the remarks of Marais (1965) on the failure of a lime stabilized calcrete base course and the findings of De Sousa Pinto et al that the strength increase obtained by adding lime to a clayey soil-cement mixture was greater when the soil was non-calcareous, no work on the stabilization of calcrete with lime is known. No work on the reactions of calcretes or the clay minerals palygorskite and sepiolite with either cement or lime appears to have been published.

The reactions taking place between lime and clays have been studied by several authors and these have been summarised by Za-Chieh Moh (1965), Diamond and Kinter (1965, 1966) and Croft (1967). A rather useful-looking document which has not been properly studied is the proceedings of the colloquim 'Mechanisms of soil stabilization' (1964). It is generally suggested that lime stabilization takes place in 4 ways:

1. Cation exchange, i.e. replacement of the exchangeable Na^{+}, Mg^{++} or other cation held by the clay by Ca^{++} from the lime.

2. Flocculation of the clay, with consequent increase in grain size.

3. Carbonation of the lime by reaction with carbon dioxide from the atmosphere to form calcium carbonate.

4. Pozzolanic reactions, mainly between lime and clay, to form cementitious hydrated silicates and aluminosilicates of calcium.

Reactions/ ...

Reactions 1 and 2 are held to account for the rapid decrease in PI and 3 and 4 the slow increase in strength experienced by lime stabilized soils. Diamond and Kinter (1966) have, however, pointed out that many natural soils are already calcium saturated, already flocculated, and that carbonation has been prevented by excluding CO_2 in some experiments and yet the usual effects of lime stabilization nevertheless appear. Ruff and Ho (1966) have shown that the main cementitious products are calcium silicates, not alumino-silicates. The reaction was found to be:

lime + clay → CSH[*] (gel) → CSH(II) → CSH(I) → tobermorite

and that the further the reaction proceeded the greater the strength developed. This is in agreement with a statement made by Croft (1967), who went on to say that the cementitious products of lime-clay, lime-flyash-clay and cement-clay are all similar and differ only in their structural arrangement in the soil. He considers both hardening of gels by desiccation and crystalline intergrowth to be important (the latter most so) and mentions the importance of both the inter- and intra-aggregate bond development. Diamond and Kinter (1966) suggest that the cause of the rapid reduction in PI is the result of very rapid reaction between the rapidly adsorbed (not exchanged) calcium on the clay mineral surfaces and the alumina and silica groups on the edge surfaces of the clay flocs. This is suggested as being enough to reduce the plasticity, but not to cause much improvement in strength. The long term strength gains are simply due to continued reaction of this sort taking place with the production of hydrated calcium silicates and alumino-silicates (note however results of

[*]Note: The usual notation of cement chemistry is employed, viz:

A = Al_2O_3
C = CaO
S = SiO_2
H = H_2O

Ruff/ ...

Ruff and Ho quoted above) and the consequent destruction of the clay lattice. The author agrees fully with this view of Diamond and Kinter.

Plainly, for long term strength effects to be important, a minimum lime content is required. Eades and Grim (1966) and McDowell (1966) have suggested methods for the determination of the optimum lime contents.

The following features of the lime-calcrete reaction have been observed in the field:

1. Rapid decrease in plasticity, but not always to the completely non-plastic state.
2. Rapid increase in strength within a few days to give 7 day cured and four day soaked CBRs of usually well over 200 and up to 400. The reaction is thought to be substantially complete within 8 hours by Mr. R. Turner (1968; personal comm.). However, not all calcretes exhibit this rapid increase in strength with lime.

From a study of Van der Merwe (1962), Kolbuszewski (1965) and D'Hoore (1965) it is apparent that the clay minerals in any soil containing more than about 1 or 2% $CaCO_3$ are almost fully saturated, mostly with calcium. It would appear that this fact is widely known among pedologists (see Robinson, 1949). It would therefore indeed appear that , as pointed out by Diamond and Kinter (1966) for materials in general, base exchange is <u>not</u> a potent factor in the reduction of plasticity of calcretes.

The flocculation trouble often experienced in hydrometer analysis of calcretes suggests that the clays in calcretes may well be in a naturally partially flocculated state. It was found that addition of NaCl, a much stronger electrolyte than $CaCO_3$, could aggravate the flocculation on the one artificial sample tried, so that only partial

flocculation/ ...

flocculation of the clays in calcretes can be postulated. The addition of $Ca(OH)_2$ might thus cause still further flocculation. While this would cause some coarsening of the grading and reduction in plasticity, this mechanism could hardly double the strength. It should be noted that palygorskite and sepiolite are not flocculated by electrolytes, whereas the other clays are.

Carbonation of lime has been suggested as a cementing agent by Goldberg and Klein (1952; in Za-Chieh Moh, 1965) and Eades, Nichols and Grim (1962) have demonstrated carbonation of lime to occur in the field. Diamond and Kinter (1966) have, however, pointed out that "the gain in strength from cementation of soil grains by calcium carbonate has not been conclusively demonstrated" and Woods (1960) and De Wet (1968) have concluded that carbonation is deleterious.

It seems likely therefore that any great and rapid increase in strength of any lime stabilized material is due to a pozzolanic reaction of some sort.

Microscopic examination of the insoluble residues of calcretes revealed the presence of very variable amounts of amorphous silica in the form of fossil diatoms, sponge spicules and phytoliths (plant skeletons). The Ondangua calcretes were found to be particularly rich in diatom remains, while calcretes which were known to react poorly from a strength point of view tended to be low in all forms of amorphous silica. In order to tackle the problem more objectively, rough diatom counts were carried out and compared with amorphous silica determinations and the strengths of stabilized specimens. The latter were prepared by compacting 4" x 2" cylindrical specimens from the soil mortar

fraction/ ...

fraction (-2 mm) to 115 lbs/cu ft dry density with an admixture of 4% Rotalym (a high calcium slaked lime), 4% ordinary portland cement or 4% Rotalym plus 4% added diatoms. The specimens were then placed in a humid room at 100% relative humidity for 1 day and then soaked in water for 6 days, after which they were tested. The procedure used by Gregg (1965) for specimen preparation was followed. The results of the experiment are given below (Table 8.17). All strength figures are the averages of three specimens unless otherwise stated.

It can be seen that the agreement in the results is sufficiently good to show that the opaline silica skeletons of microfossils present in the calcretes determine how well they will react with lime from a strength point of view and even exert some influence when stabilized with cement. Opaline (amorphous) silica is known to react rapidly with lime to yield the highly cementitious calcium silicate hydrates of the tobermorite group. X-ray diffraction analysis of soil mortar pastes of the reactive calcretes with various lime contents from 10 to 50% cured for periods of up to several months failed to reveal the presence of any tobermorite, or for that matter any other reaction product. This is not surprising in view of the amorphous silica contents and the fact that the detectability of even well crystalline components is only about 1 or 2% with the diffractometer used. Electron microscopy might enable tobermorite to be recognised. Some specimens were submitted to the National Building Research Institute for this purpose, but the determinations have not yet been carried out. The reason why a larger increase in lime stabilized strength was not obtained when 4% diatoms were added to samples 1624, 2114 and 2116 is unknown. The strengths of the most reactive lime stabilized samples in Table 8.17 are comparable with the strengths obtained by Gregg (1965) on a sand stabilized with 4% cement. The above conclusions are confirmed by the results in Tables 8.15 and 8.16.

The/ ...

TABLE 8.17 EFFECT OF FOSSIL DIATOM CONTENT ON THE COMPRESSIVE STRENGTH OF STABILIZED CALCRETES

SAMPLE		MICROFOSSILS PER GRAM(1)	OPALINE SILICA(2) %	UNCONFINED COMPRESSIVE STRENGTH(3) (psi)		
NUMBER	LOCALITY			WITH 4% LIME	WITH 4% CEMENT	WITH 4% ADDED DIATOMS AND 4% LIME
1601	Ondangua	42,000	2.27	65	59	92
1569	"	21,000	2.65	52	73	92
1567	"	15,000	1.93	48	7(4)	91
1624	Grootfontein	1,800	0.17	32	25(4)	42
2114	Lichtenburg	1,200	0.14	18	50	35
2116	"	900	0.09	19, 13	39	26(5)

(1) In the soil fines (-40 mesh); Analyst: Mr. F. Netterberg

(2) In the soil mortar (-10 mesh); Analysts: McLachlan and Lazar (Pty.) Ltd.

(3) Of the soil mortar.

(4) Only one specimen tested, others disintegrated while soaking.

(5) Slightly cracked on testing.

Operators: Messrs F. Netterberg and T.E. O'Connor.

The contribution of some strength by clay reaction cannot of course be discounted. Calcretes often contain montmorillonite, and Croft (1967) has stated that expansive clays are more lime-reactive and yield higher early strengths than non-expansive clays. The latter, however, yield the highest ultimate strengths. In confirmation of this empirical approach, De Wet (1968) has stated that the kaolinite-lime reaction products are mainly C-A-H while montmorillonite produces mainly C-S-H. Illite is intermediate between these two extremes and quartz is non-reactive. The contribution made by clays like palygorskite and sepiolite (both common in calcretes) appears to be unknown, but only palygorskite was present in Sample 2116 and it did not yield high early strengths.

It is also interesting to note that the sample reacted fairly well with sodium silicate. Sodium silicate reacts with the calcium carbonate to form cementitious calcium silicate hydrates and sodium carbonate, and has been used as a stabilizer for limestone roads overseas (Valton, 1947). The locally available product is unable to compete with cement from an economic point of view. The use of sodium silicate as a soil stabilizer has been dealt with by Laws and Page (1946).

8.14.5 Rate of reaction

Little proper work on this has been carried out. Experience suggests that the cementation in lime-reactive calcrete is substantially complete after the usual 7 day curing plus 4 day soaking period. In Ovamboland the 7 day curing is often omitted with relatively little effect on the strength and, as already mentioned, Mr. R. Turner is of the opinion that the cementation is complete within 8 hours. The rate of reaction is obviously of great importance in connection with the delay allowable between wet mixing and compaction.

Figures/ ...

Figures 8.39 and 8.40 and Table 8.18 represent a preliminary attempt to establish the rate of reaction by means of conductivity and pH measurements on material compacted in CBR moulds and merely cured as pastes in the soil cup conductivity cell. Sample 1569 was known to react well with lime from a strength point of view while Sample 2116 did not. The performance of 2493 was unknown, but came from an area in Ovamboland where reactive calcretes are common. The dominant clay minerals in the three samples were as follows: 1569: montmorillonite, 2116: palygorskite, and 2493: sepiolite. Figures 8.39 and 8.40 suggest that three distinct reactions are involved: a very rapid reaction probably complete within an hour (probably amorphous silica), a slower reaction complete within a week (probably the reactions responsible for a lowering of plasticity), and a long term reaction complete after about a year. Sample 2493 does not show all these trends, however, and the method used is also dependent on the initial lime content. The results also suggest that a plot of conductivity or pH against time for a period of up to 60 minutes with small quantities of lime (Figure 8.40) may be useful in distinguishing those calcretes which react to yield very high early strengths. Time has unfortunately not permitted the completion of these experiments.

TABLE 8.18 VARIATION OF pH WITH CURING TIME OF COMPACTED 4% LIME-CALCRETE MIXTURES

		pH							
CURING PERIOD (DAYS)	1½ hrs	1	3	7	14	30*	45	373	BEFORE ADDING LIME
1569	12.2	12.2	12.2	12.2	12.2	11	11.8	9.9(1)	7.6
2116	12.4	12.3	12.2	12.2	12.3	12	12.3	11.7(2)	7.8
2493	12.4	12.3	12.3	12.2	12.3	12	12.0	9.7(3)	8.0

* Considered unreliable, as measured with a different pH meter.

(1)/ ...

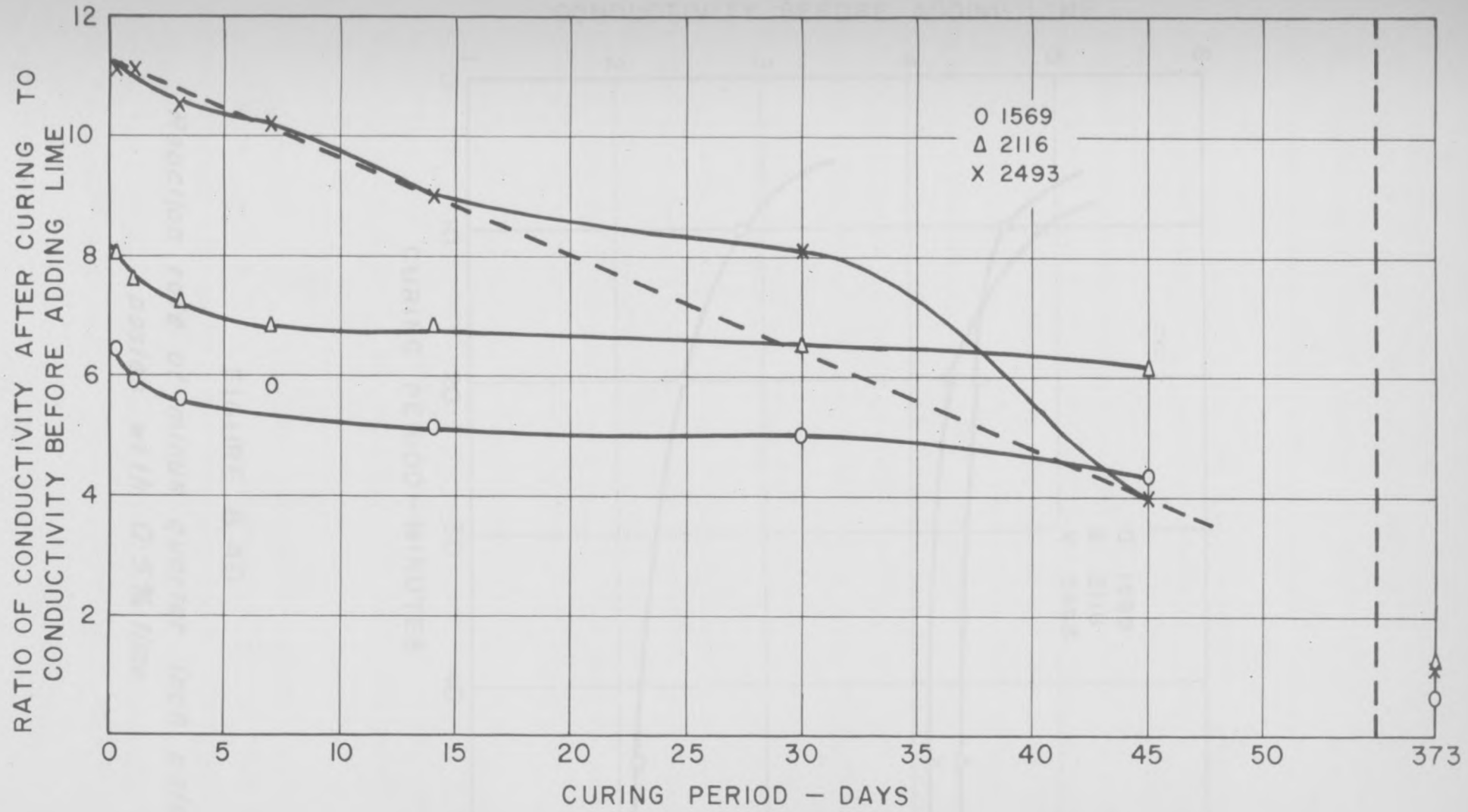


FIGURE 8.39

REACTION RATE OF MINUS QUARTER INCH COMPACTED CALCRETES WITH 4% LIME

RG4-3774

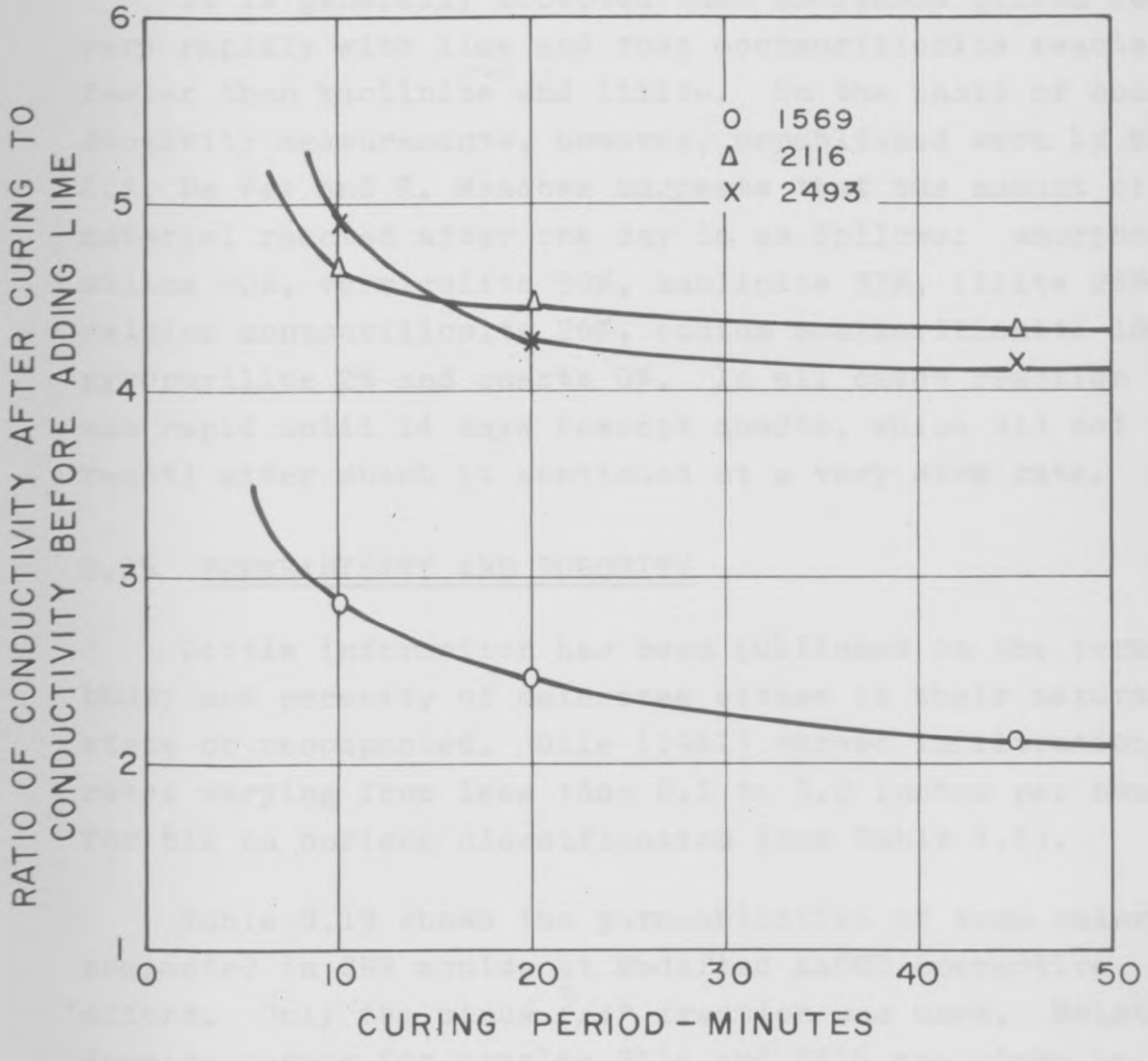


FIGURE 8.40

Reaction rate of minus quarter inch calcrete pastes with 0·5% lime

RG4-3775

(1) Less than 0.1% unreacted lime left.

(2) Probably 0.2-0.5% unreacted lime left.

(3) Probably less than 0.1% unreacted lime left.

It is generally accepted that amorphous silica reacts very rapidly with lime and that montmorillonite reacts faster than kaolinite and illite. On the basis of conductivity measurements, however, unpublished work by Messrs. J.J. De Wet and K. Meadows suggests that the amount of material reacted after one day is as follows: amorphous silica 90%, vermiculite 50%, kaolinite 33%, illite 26%, calcium montmorillonite 26%, sodium montmorillonite 18%, pyrophyllite 2% and quartz 0%. In all cases reaction was rapid until 14 days (except quartz, which did not react) after which it continued at a very slow rate.

8.15 PERMEABILITY AND POROSITY

Little information has been published on the permeability and porosity of calcretes either in their natural state or recompacted. Gile (1961) showed infiltration rates varying from less than 0.1 to 5.0 inches per hour for his ca horizon classification (see Table 1.1).

Table 8.19 shows the permeabilities of some calcretes compacted in CBR moulds at Modified AASHO compactive effort. Only the minus $^{3}/_{4}$" fraction was used. Moisture density curves for samples 2114 and 2116 are given in Figures 8.35 and 8.38.

Samples 1980, 2114 and 2116 were nodular calcretes, while 2495 was a crusher run limestone or hardpan calcrete from Quarry 21A on the new Tsumeb-Operet road used for base. Samples 2114 and 2116 have been used stabilized with 3-5% PBFC as base on the Lichtenburg-Deelpan road. It is apparent that the more plastic calcretes are comparatively/ ...

TABLE 8.19 PERMEABILITY OF COMPACTED CALCRETES

SAMPLE NUMBER	ADDITIVE	DRY DENSITY lbs/cu ft	MOISTURE CONTENT (%)				PERMEABILITY cm/sec
			OPTIMUM	COMPACTION	PRE-TEST	POST-TEST	
1980	None	126.3	~10.5	12.0	11.3	13.1	4.1×10^{-8}
1980	None	126.0	~10.5	11.2	10.7	11.7	3.3×10^{-8}
2114	None	120.0	10.6	12.6	13.3	14.8	4.2×10^{-8}
2114	None	120.6	10.6	12.1	-	14.7	6.3×10^{-8}
2114	4% cement	118.8	12.8	11.5	11.9	14.2	1.2×10^{-5}
2114	4% cement	116.9	12.8	11.4	11.9	12.4	8.5×10^{-6}
2116	None	145.0	7.0	7.9	8.1	13.9	6.2×10^{-7}
2116	None	125.0	7.0	7.9	7.8	14.3	7.7×10^{-7}
2116	4% cement	120.7	9.9	8.0	9.3	11.9	5.3×10^{-6}
2116	4% cement	122.2	9.9	8.0	8.8	11.8	2.1×10^{-6}
2495	None	135.0	6.5	7.2	7.2	10.1	6.7×10^{-4}
2495	None	133.8	6.5	7.5	7.3	10.3	1.7×10^{-5}

Operators: Dr. P.A. Loudon and Messrs. L. Clarke and T.E. O'Connor.

paratively impermeable, but that the permeability is considerably increased by adding cement. This must be due to the coarsening of the grading.

Figures 8.41 and 8.42 show the moisture content of compacted calcretes after 4 days soaking and the porosity calculated from this in relation to the optimum moisture content. The porosity was taken as equal to the water absorption in percent multiplied by the dry density in lbs/cu ft divided by 62.4. While no other figures are to hand for comparison, it will probably be found that calcretes are unusually porous (some calcretes are known to possess optimum moisture contents in excess of 20%). The unusually high porosities and optimums are probably largely due to the unusually high porosities of the particles themselves, often contributed to by diatoms.

The water absorptions of crushed hardpans and boulders and some nodules are given in Appendix H, while detailed information on the water absorption of nodules versus size appears in Appendix J.

8.16 SOLUBLE SALTS

Calcretes may contain sufficient soluble salts to cause considerable trouble when used as base and possibly even when used in the lower layers. In addition, salt can be added by the compaction water or may rise from the subgrade. The subject is still little known. The types and occurrence of deleterious salts and methods for their detection are dealt with in Appendix D, while Netterberg (1969b) has reviewed the possible effects of different types of salts on different types of bases and possible remedial and preventive measures. Results of conductivity and pH measurements on calcrete pastes are given in Appendix D, while some also appear in Tables H.1-H.4.

Soluble salts can also affect the Atterberg limits and the laboratory (and probably field) strength of lime-soil (and probably cement-soil mixtures. Sodium salts

possibly/ ...

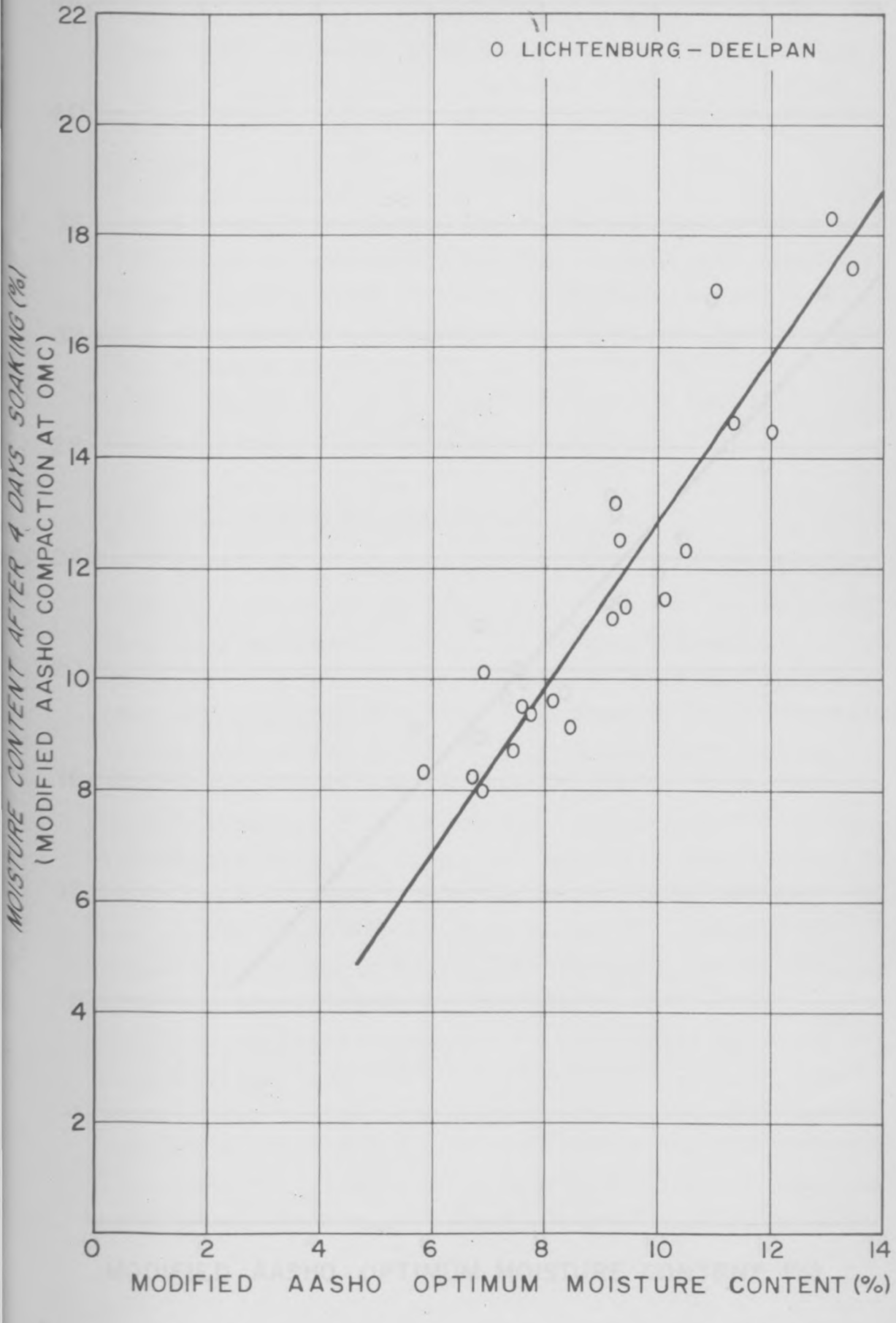


FIGURE 8.41
RELATION BETWEEN OPTIMUM MOISTURE CONTENT AND SATURATION MOISTURE CONTENT FOR SOME CALCRETES USED ON THE LICHTENBURG-DEELPAN ROAD

RG4-3776

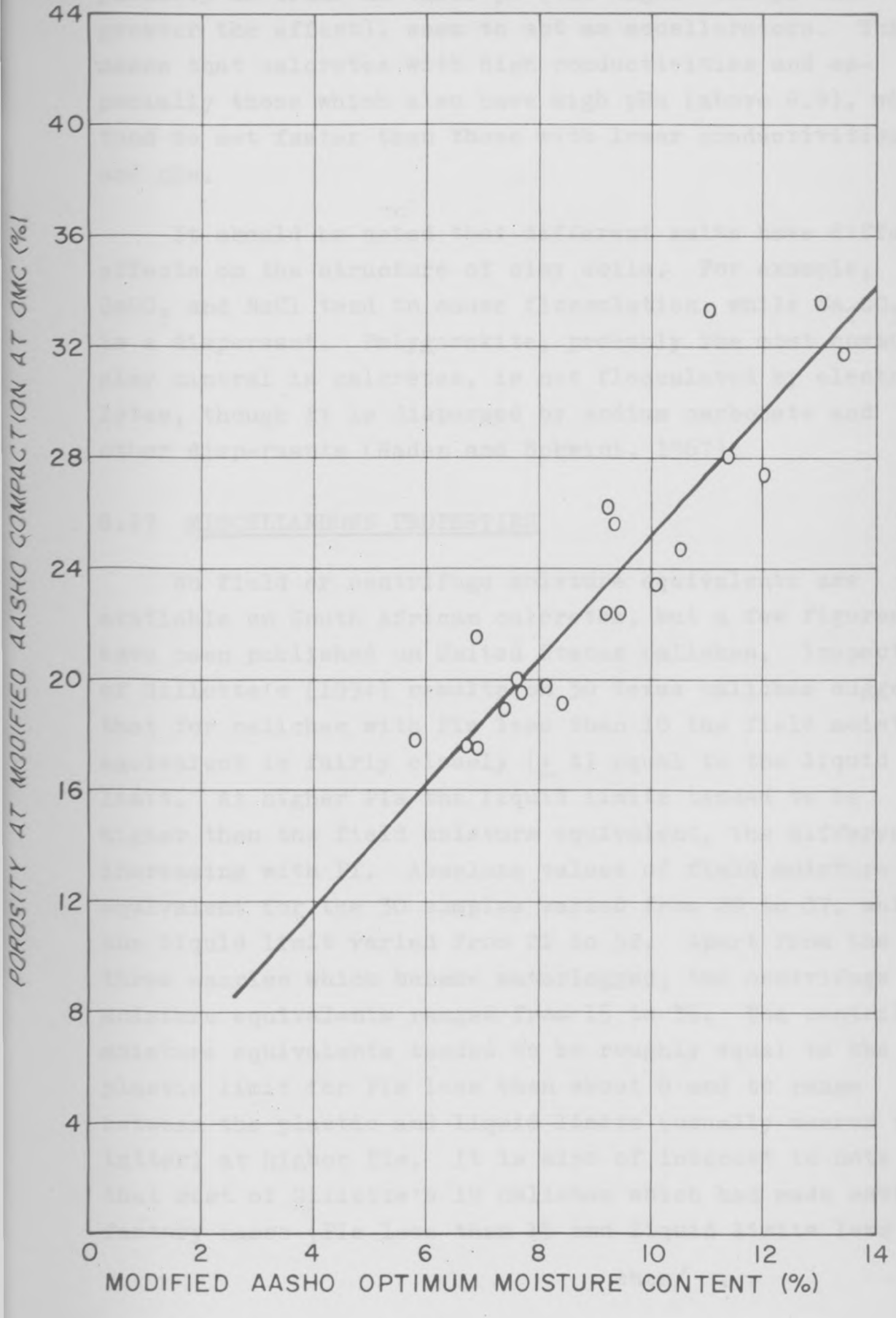


FIGURE 8.42
RELATION BETWEEN OPTIMUM MOISTURE CONTENT AND POROSITY FOR SOME CALCRETES USED ON THE LICHTENBURG-DEELPAN ROAD

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possibly in order of their pH (the higher the pH the greater the effect), seem to act as accellerators. This means that calcretes with high conductivities and especially those which also have high pHs (above 8.6), will tend to set faster than those with lower conductivities and pHs.

It should be noted that different salts have different effects on the structure of clay soils. For example, $CaCO_3$ and NaCl tend to cause flocculation, while Na_2CO_3 is a dispersant. Palygorskite, probably the most common clay mineral in calcretes, is not flocculated by electrolytes, though it is dispersed by sodium carbonate and other dispersants (Haden and Schwint, 1967).

8.17 MISCELLANEOUS PROPERTIES

No field or centrifuge moisture equivalents are available on South African calcretes, but a few figures have been published on United States caliches. Inspection of Gillette's (1934) results on 30 Texas caliches suggests that for caliches with PIs less than 10 the field moisture equivalent is fairly closely ($\pm$ 4) equal to the liquid limit. At higher PIs the liquid limits tended to be higher than the field moisture equivalent, the difference increasing with PI. Absolute values of field moisture equivalent for the 30 samples varied from 20 to 37, while the liquid limit varied from 21 to 52. Apart from the three samples which became waterlogged, the centrifuge moisture equivalents ranged from 15 to 35. The centrifuge moisture equivalents tended to be roughly equal to the plastic limit for PIs less than about 8 and to range between the plastic and liquid limits (usually nearer the latter) at higher PIs. It is also of interest to note that most of Gillette's 19 caliches which had made satisfactory bases (PIs less than 15 and liquid limits less

than/ ...

than 38) also possessed field moisture equivalents closely equal to their liquid limits. Another 7 caliches reported by Gillette in 1951 yielded field moisture equivalents ranging from 19 to 36. Liquid limits ranged from 36 to 51 and PIs from 16-22. These latter results were obtained on material prepared by the AASHO T146-49 wet preparation method. Gillette's (1934) results were probably obtained by the methods described by Wintermyer et al (1931). A large number of field and centrifuge moisture equivalents of other soils have not been obtained for comparison, but it would appear that caliches may possess unusually high moisture equivalents as it is believed that most other soils possess field moisture equivalents considerably less than their liquid limits and centrifuge moisture equivalents considerably less than their plastic limits. According to Wooltorton (1954), the centrifuge moisture equivalent is about equal to a pF of 2.7 on the drying curve.

Gillettes (1934) 30 caliches yielded flocculation factors ranging from 1.2 to 7.8. 19 out of 20 materials which yielded values less than 2.5 had made satisfactory bases. Gillette's flocculation factor is the void ratio of soil sedimented under certain conditions.

Surface area data for the minus 40 mesh fractions of 17 calcretes are shown in Table J.13. Total surface areas varied from 16 to 96 square meters per gram. Values after heating to 600°C increased in some cases, yielding apparent external surface areas of less than 3 to 194 square meters per gram.

Tufaceous hardpans have in the past been a useful source of building material. They can usually be readily quarried by hand and shaped into suitable blocks with axes and saws. Their low density makes them easy to handle, but they are of low strength, though said to harden somewhat on exposure. Their use as building stone is

probably/ ...

probably not economic where locally burned brick is available. It has been suggested that crushed tufaceous hardpan may be suitable as lightweight concrete aggregate, but it is probably too weak for this. The amorphous silica present would also give rise to excessive expansion if high alkali cement is used.

Winterbach and Weinert (1961) have recorded the control of heaving crack patterns in expansive clay by calcrete dykes formed in Dwyka mudstone. The dykes formed watertight compartments, but allowed water in along their sides between the dykes and the mudstone.

The compression index (C_c) of the minus 40 mesh fraction of Sample 1601 was found by Dr. P.A. Loudon to be 0.14. This is in reasonable agreement with the value of 0.11 predicted from the liquid limit (LL) by means of Shempton's formula (in Hough, 1957): $C_c = 0.007\ (LL-10)$.

The use of calcrete as railway ballast is said to prolong the life of railway ties (Blake, 1904).

Care should be used in founding heavy structures on calcrete hardpans as they are normally thin and underlain by weaker material. The weaker layer is not detected by the shallow seismic refraction method. King et al (1967) have recorded the occurrence of sinkholes in calcrete in the Schmidtsdrif land system. The potholes or solution hollows (makondos) often seen in hardpan undergoing solutional weathering can be regarded as minature sinkholes of a kind. Sinkholes in calcretes are likely to be small, owing to the usually thin nature of the horizons.

The presence of calcium in a soil hinders the removal of water by electro-osmosis (Woods, 1960).

Siegel (1967) has listed a number of properties of carbonate rocks in general, such as porosity, density,

thermal/ ...

thermal expansion, compressibility, elastic constants, compressive and tensile strength, thermal and electrical conductivity, dielectric constant and youngs's modulus, which will probably be applicable to the denser calcrete hardpans and boulders.

Deflection and curvature readings on a 5% PBFC stabilized substandard calcrete road (Lichtenburg-Deelpan) have been given by Buckle (1968) and curvature readings on a 4% lime stabilized calcrete road (Operet-Ondangua) by Netterberg and Jamieson (1968). Results vary widely; curvatures between 100 and 1800 ft were obtained on both roads. Results are badly affected by temperature (or suction in the base?), the radius of curvature apparently increasing with temperature (Netterberg and Jamieson).

Elephants can be dissuaded from pushing down telephone poles if a single layer of irregularly shaped pieces of hardpan or boulder calcrete about 6"-8" in diameter is placed on the ground to a radius of about six feet around the poles.

8.18 CONCLUSIONS

1) Little information is available in the literature concerning the engineering properties of calcretes.

2) Calcareous soils, which have in the past been called calcretes by some roads personnel, bear little relation to calcretes in their properties and have only succeeded in giving calcretes a poorer name than they deserve.

3) The carbonate fraction is the most beneficial mineral group present in calcretes from the engineering point of view. It lowers all the soil constants of clays, cements the finer particles into hard aggregate and softer aggregations and generally acts as a mechanical

and/ ...

and to a lesser extent, a chemical stabilizer. Silt-sized calcium carbonate has a liquid limit of 27, is non-plastic and exhibits no shrinkage when dried from the liquid limit. The presence of calcium ions lowers the soil constants of sodium montmorillonite.

4) Calcretes tend to possess higher shrinkage limits than plastic limits, a feature apparently mainly caused by the porosity of the calcrete particles (sometimes diatomaceous). Although the structure of the soil in the two tests is different, doubt is thus cast upon the reliability of a plasticity index as an indicator of the performance of a base course. The best definition of a PI is probably that it is a measure of the volume change between the liquid and plastic limits.

5) Palygorskite and sepiolite, two of the most common clay minerals found in calcretes, possess a number of unusual properties which are reflected in the properties of the calcretes containing them. Palygorskite, for example possesses the highest plastic limit of all the common clays and a PI equal to calcium montmorillonite, but unlike montmorillonite, is non-expansive.

6) Calcretes tend to have higher liquid limits for the same apparent plasticity index than other materials.

7) Calcretes tend to have higher liquid limits for the same bar linear shrinkage than other materials.

8) Calcretes tend to have higher apparent PIs for the same bar linear shrinkage than other materials. A high shrinkage limit obviously means that the material stops shrinking early on the shrinkage curve. Some calcretes reported as non-plastic may possess linear

shrinkages/ ...

shrinkages up to 4%, while those reported as slightly plastic have yielded linear shrinkage up to 7%, a further clear indication of the unreliability of a PI on calcretes.

9) The correlation between the bar linear shrinkage and apparent plasticity index of calcretes and other materials is often so high that it suggests that these two tests measure the same property. A plasticity index is in fact a measure of the continously remoulded volume change between the liquid and plastic limits, while a linear shrinkage is a measure of the volume change in one dimension while drying undisturbed from the liquid limit to the shrinkage limit.

10) The mean solid particle specific gravity of 44 calcrete minus 0.42 mm fraction was found to be 2.66, with a range from 2.5 to 2.8. Bulk particle specific gravity varies considerably.

11) The engineering properties of calcified soils, powder calcretes, nodular calcretes, honeycomb calcretes, hardpan calcretes and boulder calcretes are sufficiently different to give real engineering significance to this classification.

12) The fact that calcretes are not composed of discrete solid particles, but are composed of a host of discrete, solid particles cemented into larger, often porous, compound particles, causes them to differ significantly in their properties from other materials. This porosity and aggregate strength varies with size fraction and affects the reliability of many standard tests such as Atterberg limits and mechanical analyses. Calcrete particles tend to be unusually porous.

13) Nodular calcretes are the most desirable and useful calcretes from the roadmaking point of view. They

rarely/ ...

rarely pass existing South African base course specifications, but quite often make stabilized base, subbase, selected subgrade and gravel road wearing courses.

14) The CBR of calcretes depend to a large extent on the value of the bar linear shrinkage from the liquid limit multiplied by the percentage passing the 40 mesh sieve. There is a high degree of certainty that a calcrete with a value of LS X % minus 40 mesh < 200 will yield a 0.1" CBR in excess of 60.

15) The aggregate strength, grading, plasticity, shrinkage, soluble salt content and CBR improve with development of the calcrete.

16) The better hardpan and boulder calcretes are of sufficient quality for use in crushed bases, premixes and concrete.

17) While possessing high crushing strengths, the workability of boulder calcretes is such as to render them of little use for roadmaking purposes.

18) The aggregate crushing strengths of calcrete hardpans, honeycombs, boulders, and probably also nodules can be predicted by simple tests such as density, water absorption and particularly Mohs hardness.

19) The relation between the aggregate crushing value (ACV) and the 10% fines aggregate crushing values of calcretes is essentially the same as that reported in the literature.

20) Calcretes vary in their adhesion to bitumen from extremely good to extremely poor. Their resistance to polishing is average to good and they are not particularly prone to flakiness on crushing.

21/ ...

21) There is ample evidence that self-stabilizing pedogenic road materials do exist and that at least some calcretes can be placed in this category.

22) Emphasis is given to the probable necessity for the storing, placing and testing of potentially self-stabilizing materials at or higher than their natural moisture contents. Drying of the material before placing or testing probably results in the loss of some or all the self-stabilizing qualities.

23) The CBRs of several calcretes have been doubled to trebled by subjecting the compacted specimens to wetting and drying cycles in the laboratory. An improvement of 50% has also been reported from elsewhere on curing as for a stabilized material. The occurrence of self-stabilization of engineering significance in calcretes in practice still remains to be proven.

24) The strength properties of lime stabilized calcretes are controlled by their amorphous silica contents in the form of microfossil skeletons - chiefly diatoms. Diatoms also affect other properties such as the Atterberg limits, compacted density and optimum moisture content.

25) The testing of proposed calcrete base materials for the presence of excessive quantities of deleterious soluble salts is desirable.

26) Calcretes are probably among the most variable of materials used in roadmaking, and vary from almost useless powders to materials of rock-like quality.

27) The acceptability of calcretes for roadmaking purposes varies considerably from authority to authority.

What/ ...

What would be acceptable to the Texas Highway Department or the U.S. Federal Aviation Agency as caliche base course would normally only pass for selected subgrade in South and South West Africa. A worthwhile relaxation of grading specifications and all soil constant requirements except possibly linear shrinkage for all classes of calcrete road materials from the base downwards appears justified.

8.19 SCOPE FOR FURTHER INVESTIGATION

1) It would be interesting to determine a full shrinkage curve for a number of calcretes using the method described by the Road Research Laboratory (1952). Useful analyses of shrinkage curves are given by Wooltorton (1954).

2) The applicability of the plasticity index to calcretes possessing negative shrinkage indices needs further investigation.

3) Little or no information is available on the in situ density, permeability, porosity, compressibility, sensitivity, preload status, liquidity index, shear strength, swell, thermal and electrical conductivity, specific heat, and the general behaviour of calcretes as foundation materials for structures.

4) Several aspects of the roadmaking properties of some calcrete groups are little known. Aggregate strength for one is a subject which is becoming more and more important as traffic loads and intensities increase. The behaviour of stabilized calcrete bases is another field about which relatively little is known. The use of non-destructive test methods like curvature, Benkelmann beam and sonic testing for the measurement of in situ strength and elastic constants should be encouraged. Cracking of stabilized calcrete bases

can/ ...

can be a nuisance.

5) Much very interesting work remains to be carried out on the question of self-stabilization, its mechanism, prediction, measurement and attainment in practice. It has yet to be conclusively proven whether self-stabilization of calcretes is of engineering significance in practice. Considerable economies can be made if the CBR can be doubled or trebled in practice as it has been in the laboratory. Experiments should be carried out using water charged with carbon dioxide. Petrification and CBR tests should be carried out with fresh undried materials at the natural moisture contents as well as those which have been dried in order to assess whether this property is lost on drying. The use of redox potentials might be useful. More samples should be subjected to wetting and drying cycles, both with and without the conventional annular weights. The importance of a decrease in soil constants and an increase of density during this process should be assessed. Study of the mode of formation of pedogenic materials in nature will provide the answers as to how to make them continue to form in the road, which is all that self-stabilization really represents.

6) As few calcretes are suitable for use in surface dressings, but many are of premix quality, it may be worth assessing their adhesion properties by a test more suitable for premix stones (such as the Riedel and Weber, DIN. 1965) than the Karius and Dalton detachment test used here.

7) The CBR, grading modulus and apparent PI of a large number of calcretes should be examined after the manner of Kleyn (1955) to see if there is any correlation between the PI, grading modulus and the CBR. It is

suggested/ ...

suggested that calcretes would not agree with Kleyn's correlation and would have higher CBRs than normal. In addition to being useful in practice, this should also be a useful confirmation or denial of the proposal that grading and PI specifications can be relaxed for calcretes.

8) The question of gypsum in bases in general should be gone into.

9) The properties of calcretes should be compared with those of marl, chalk, limestone and coral. Further comparison with Texan caliches would help to assess whether their specifications can be applied to South African calcretes.

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CHAPTER 9

USE AND SPECIFICATIONS FOR ROAD CONSTRUCTION

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9.1 INTRODUCTION

This chapter describes the present extent of usage of calcrete as road material in the Republic and South West Africa and in some other parts of the world, examines specifications, and suggests revised specifications for calcrete road materials as well as methods of achieving self-stabilization in practice.

The terminology used throughout this work for the layers of the road structure is that current in South Africa at the time of writing, as described by Weinert (1968).

9.2 EXTENT OF PRESENT USAGE IN SOUTH AND SOUT WEST AFRICA

A recent survey by Dr. H.H. Weinert (1967, personal comm. and 1968) of the importance of different materials in road construction in the Republic according to the areal extent of their use has revealed that calcrete ranks second only to dolerite in its overall importance for

this/ ...

this purpose. Some comparative figures are shown in Table 9.1.

TABLE 9.1 RELATIVE OVERALL IMPORTANCE OF SOME ROAD MATERIALS IN SOUTH AFRICA ACCORDING TO WEINERT

POSITION	MATERIAL	SEMI-QUANTITATIVE RATING
1	Dolerite	1226
2	Calcrete	945
3	Shale	785

The importance of calcrete with respect to its position in the road structure as revealed by the areal extent of its uses in the Republic is as follows (Table 9.2).

TABLE 9.2 RELATIVE IMPORTANCE OF THE VARIOUS USES OF CALCRETE IN ROAD CONSTRUCTION IN SOUTH AFRICA ACCORDING TO WEINERT

USE	POSITION OF CALCRETE WITH RESPECT TO OTHER MATERIALS	AREAL EXTENT OF USE (% OF REPUBLIC)
Surfacing	-	0
Base course	7	10.2
Subbase	1	43.7
Subgrade and fill	1	45.1
Wearing course	1	44.2
Concrete	-	0
Overall use	2	45.1

The/ ...

The table shows that calcrete is the most widely used material for subbase, subgrade, fill, and gravel road wearing course in the Republic. It ranks seventh in areal extent of use as a base course and second in the extent of its general usage. It has not so far been used for concrete or surfacing aggregate in the Republic, though it has seen minor use for these purposes in South West Africa. If the necessary information was available for South West Africa and Botswana, calcrete will probably be found to rate as the most commonly used road material in all its uses except as surfacing and concrete aggregate and perhaps base course, thus displacing dolerite in Table 9.1 as far as Southern Africa is concerned. This prediction follows logically from the calcrete map (Map 4).

Figure 9.1 shows the general extent of usage of calcrete in the Republic according to an unpublished map kindly made available by Dr. H.H. Weinert, with minor modifications by the author. It was from maps such as these that the figures quoted in the previous tables were calculated. The original data was obtained from circulars sent to the Provincial Roads Departments, so that it is not impossible that in a few cases the persons concerned may not have identified the "limestone" correctly.
Figure 9.1 agrees very well with Maps 2 and 4 and on the whole also with Table 9.3. The latter was compiled from personal observations made by the author during 1964-1969 and by questioning local roads authorities and consulting engineers. It cannot be regarded as complete and may also suffer from occasional incorrect material identifications. In most cases the actual sections employing calcretes and whether they were stabilized is unknown to the author. Such information should be available from a study of the records of the authorities under whose jurisdiction the roads fall. To the author's knowledge and to the knowledge of the persons supplying the information,

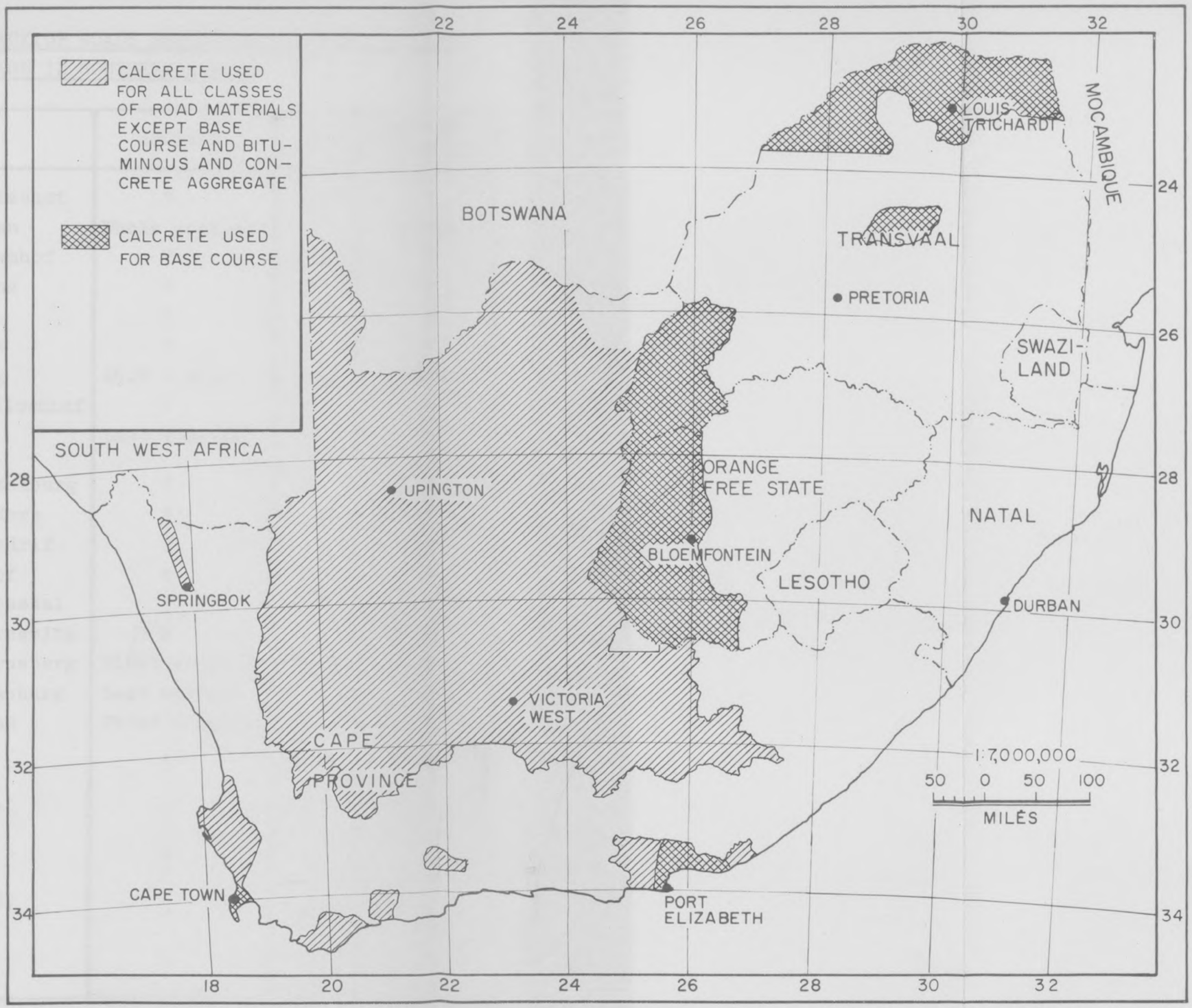


FIGURE 9·1

Regional importance of calcrete as a road construction material in the Republic of South Africa (mostly after Dr. H. H. Weinert)

no serious failures have occured on any of the roads listed. This aspect would also require verification.

TABLE 9.3 SOME BLACKTOP ROADS EMPLOYING CALCRETE AS BASE OR SUBBASE IN SOUTHERN AFRICA

ROAD	SECTION	LAYER	WHETHER STABILIZED
Delareyville-Sannieshof	?	Both	?
Lichtenburg-Deelpan	Whole contract	"	Cement
Wolmaransstad-Bloemhof	?	Base?	?
Bloemhof-Christiana	?	" ?	?
Crecy-Roedtan	?	" ?	?
Tuinplaas-Settlers	?	" ?	?
Koster-Lichtenburg	45.6 - 47.1	Base	Cement
Schweizer Reneke-Bloemhof	?	"	"
Schweizer Reneke-Delareyville	Last 7 miles	"	"
Delareyville-Lichtenburg	?	"	"
Delareyville-Geysdorp	?	"	"
Rooigrond-Buurmansdrif	?	"	"
Lichtenburg-Lusthof	?	"	"
Koffiefontein-Jakobsdal	?	Both	?
Koffiefontein-Fauresmith	?	"	?
Koffiefontein-Petrusburg	First section	Base?	?
Bloemfontein-Petrusburg	Last section	" ?	?
Bultfontein-Soutpan	First 6 miles	" ?	?
Landsdowne Road, Cape Town	?	Both	No
Duinefontein Road, Cape Town	?	"	"
Weltevrede Road, Cape Town	?	"	"
Sea View Main Road, Port Elizabeth	?	Base	"
Port Elizabeth-Grahamstown	?	"	"

TABLE 9.3 (CONTINUED)

ROAD	SECTION	LAYER	WHETHER STABILIZED
Port Elizabeth-Addo	?	Base	No
Marine Drive, Port Elizabeth	?	"	"
Piketberg-Velddrif	Last 15 miles	Base?	?
Springbok area	?	"	?
Calvinia area	?	"	?
Alexandria area	?	Base	?
Riversdale-Stillbay	?	"	?
Uitenhage-Addo	?	"	?
Vryburg-Terrafirma	?	Base?	?
Cradock area	?	Base	?
Cradock-Graaff Reinet	?	"	?
Graaff Reinet-Jansenville	?	"	?
Upington-Prieska	?	"	?
Upington-Aughrabies Falls	?	"	?
Port Elizabeth-Middelburg	Sandflats	"	Cement
Karasburg-Grunau	?	Both	?
Operet-Oshakati	Whole contract	"	Lime
Ondangua Airfield	" "	"	"
Windhoek, various roads	?	Base	?
Wasser-Asab	?	Base?	Screened
Otjiwarongo-Okaputa	First	Base	"
Mariental-Asab	?	"	?
Grunau-Riverside	?	"	?
Keetmanshoop-Grunau	?	"	?
Sukses-Otjiwaronga	Last	"	Screened
Okaputa-Otavi	Whole contract?	"	Lime
Wilhelmstal-Usakos	" "	"	Lime?

TABLE 9.3 (CONTINUED)

ROAD	SECTION	LAYER	WHETHER STABILIZED
Usakos-Trekkopje	Whole contract?	Base	?
New Years Gift-Chipinga Junction	?	Base?	No?
Schweizer Reneke-Bloemhof	?	Subbase	No
Vryburg-Pudimoe	?	"	?
Vryburg-Stella	?	"	?
Middelburg-Steynsburg	?	"	?
Murraysburg area	?	"	?
Britstown-Prieska	?	"	?
Barkly West-Koopmansfontein	?	"	?
Kuruman-Hotazel	?	"	?
Kenhardt-Keimoes	?	"	?
Graaf Reinet-Aberdeen	?	"	?
Morton Siding-Heuningneskloof	?	"	?
Kimberley-Griquatown	?	"	Lime?
Mosselbay-various roads	?	"	?
Tsumeb-Operet	Whole contract?	"	Crushed
Otavi-Tsumeb	?	"	?
Mariental-Kalkrand	?	"	?
Grootfontein Airport	Whole contract	"	Lime
Karibib Airport	Whole contract?	"	?
Bloemhof-Windsorton	?	?	?
Vryburg-Manchester	?	?	?
Addo-Coerny	?	?	?
Settlers Way, Cape Town	?	?	?
Komatipoort-Lourenco Marques	?	?	?
Magunde area, Mocambique	?	?	?

Calcrete has been used all over South West Africa except for a strip perhaps 60 miles wide along the coast where the calcretes have been partially or wholly converted to gypsum. In this strip low grade salt and gypsum deposits are widely used as gravel road materials. Whether this strip extends the full length of the S.W.A. coast is unknown to the author, but calcretes were seen reasonably close to Luderitz and it is believed that calcretes also occur in the diamond fields around Oranjemund (see also Map 4). Calcretes are relatively scarce in the Namib (Mr. J. Caiger, 1965; personal comm.), though they do occur in places (such as at Swakopmund - Mr. H.C. Brown, 1965; personal comm.), and in other parts of South West Africa. They are for example definitely scarcer and of poorer quality north of the Gaza line (the southern boundary of the Okavango territory) than south of it. They are said to be scarce in the Warmbad-Vioolsdrif area and are definitely scarce along the Oshakati (Okatana) - Ruacana Falls road until some 20 miles from the Falls. On the whole, however, calcrete occurs and is used all over South West Africa.

9.3 SURFACING CHIPS

Apart from their slight use as premix filler and chips in Ovamboland and their possible use as premix chips in the Operet area of South West Africa, calcretes do not seem to have been used for bituminous surfacing aggregates in South or South West Africa.

In South Australia, the Highways and Local Government Department (1965, private comm.) sometimes extracts screenings of $^{5}/_{8}$"-$^{3}/_{8}$" and $^{3}/_{8}$"-$^{1}/_{4}$" aggregate concurrently with the production of crusher run from hardpan calcrete and uses this for surface treatment over the screened crusher run base. A maximum LAA figure of 35% on the $^{5}/_{8}$"-$^{3}/_{8}$" aggregate and a maximum loss of 20% in the sulphate soundness

test/ ...

test (Australian Standard No. 77-1957) is required. Mr. A.J. Scala of the Australian Road Research Board (1965, private comm.) has also stated that the harder calcrete lumps overlying the crusts (hardpans) have also been used in Australia as sealing aggregate. The Los Angeles wear of the material apparently varies from 40 to 50%, but as traffic is light this is not considered a problem. The latter LAA figure is very high by normal standards.

For example, ASTM D692-59, D693-54 and D1139-61T require a maximum LAA of 40% for all types of bituminous surfacings (in addition to sulphate soundness tests). Western Australia specifies a maximum LAA of 15% for heavy traffic and 40% for light traffic and New South Wales from 20% for more than 1000 vpd to 35% for less than 200 vpd (Coulter and Punch, 1959). These authors also mentioned that the Los Angeles test "seems satisfactory for homogeneous crushed stone, but gives erratic results for aggregates such as river gravels, laterite gravels, blast furnace slag etc. where some hard and soft stones are present". Coulter and Punch gave details of a modified LAA procedure used by the New South Wales Main Roads Department on such materials. It can be expected that the LAA test will also yield erratic results on calcretes, an impression confirmed by the FAA (1968, private comm).

Calcrete is used almost exclusively for asphaltic concrete surface courses in the middle-eastern and south eastern part of New Mexico (Mr. T. Cole, New Mexico State Highway Dept., 1966, private comm.).

The Arizona Highway Department (Mr. W.G. O'Harra, 1966; private comm.) on the other hand try to keep their caliche out of bituminous mixes as it is excessively absorptive and apparently absorbs the asphalt, resulting in dry, friable mixes. This problem can presumably be overcome

by/ ...

by using more bitumen, though this of course increases the cost of the surfacing.

Caliches have not been used for bituminous surfacing aggregates in Texas by the Texas Highway Department (Mr. R.L. Lewis, 1968; private comm.).

Calcrete has also been used for surfacing chips in Nigeria (Clare and Beaven, 1962) where surface dressing aggregate is produced by sieving nodular calcrete (jigilin) The crushing strengths quoted (10% F.A.C.T. values of 2.4 -6.3 tons), are so low that the aggregate would have failed most base course specifications, let alone those for surface dressing (though BS 1984: 1967 specifies a minimum of only 8 tons for surface dressing). Not surprisingly, the results were said to be "not very satisfactory", owing to the softness of the aggregate and the fact that it became very slippery during the wet season.

It has been shown in the previous chapter that the adhesion of crushed hardpan and boulder calcretes to bitumen can be excellent, and their resistance to polishing entirely satisfactory. Their aggregate crushing strengths and performances in the sulphate soundness test are variable, but some yield good results. In view of these results and the overseas experience reported above it is apparent that there need be no fear of using the best crushed hardpans, boulders, honeycombs and nodules as aggregate for surface dressings under light traffic conditions and in premix surfacings in general, provided they fulfill the normal requirements for such materials for the traffic expected and are not too porous. A dry aggregate crushing test and a limitation on the water absorption of $^{3}/_{8}$"-$^{1}/_{2}$" aggregate of 5% should suffice in most cases. It is apparent from Australian experience that relatively weak aggregate can be used under light traffic conditions. In view of

the/ ...

the lack of experience available it is impossible to define ACV or 10% F.A.C.T. limits for calcretes under South African conditions. As a guide though, the ACV of aggregate for surface dressings could vary from about 30% (10% F.A.C.T. value of 12 tons) under light traffic to 20% (10% F.A.C.T. value of 20 tons) under heavy traffic. In the case of premix and penetration macadam these values may be relaxed somewhat, perhaps to an ACV of 35% (10% F.A.C.T. value of 9 tons on calcretes) under light traffic and an ACV of 25% (10% F.A.C.T. value of 15 tons) under heavy traffic. Under heavy traffic conditions it may also be advisable to carry out adhesion, flakiness and polishing tests. Flakiness index and polished stone value limits have been given by BS 1984:1967 and flakiness index limits by SABS 647. Variability of the material will be a problem in many cases, as well as the economics of working thin layers or of screening and crushing nodules.

As far as limestone aggregates in general are concerned, it is apparent from Valton (1947) that good limestones make entirely satisfactory bituminous surfacing aggregate. Their adhesion is normally particularly good, but some may tend to polish under heavy traffic.

9.4 BASE

Specific routes known to have calcrete base courses or subbases in Southern Africa are shown in Table 9.3. Where the calcrete was definitely known to have been used for both base and subbase this is shown. Where it was only known to be used as base, only this is shown. However, it can be fairly safely assumed that if base course calcrete was available, calcrete of subbase quality was also available and used. It can likewise be assumed that if subbase quality calcrete was available, calcrete was also used for selected subgrade and probably fill.

The/ ...

The use of unstabilized calcrete as base course in Southern Africa is rare, since it seldom meets present specifications of plasticity and grading, though the CBRs are more often satisfactory.

The only known unstabilized calcrete base courses in South Africa are in the Port Elizabeth - Addo - Grahamstown area and several roads on the Cape Flats near Cape Town. All of these roads have performed entirely satisfactorily for many years, even though the materials do not meet present Cape Provincial base standards.

In Rhodesia, calcretes or calcified weathered basic igneous rocks have been used for unstabilized bases in the Beit Bridge and Birchenough Bridge areas. Some failures have been encountred in the former area following the recent increase in traffic. It is believed, however, that the CBRs were very low and only of about South African subbase standard.

9.4.1 Overseas experience and specifications

Campbell (1929) mentioned the use of caliche in an experimental section in New Mexico prior to 1929 which appeared to be standing up well under traffic, while Graham et al (1944) were generally of the opinion that caliche in the U.S.A. had proved satisfactory as a base for parking areas and runways at airports and roads carrying light and medium traffic, but was "unsatisfactory as a base for permanent roads carrying fast traffic and heavy equipment". This could perhaps be said of most natural soil-aggregate base courses.

Gillette in 1934 discussed a number of United States caliche based roads of varying performance on the basis of laboratory test results of material removed from the bases after service and came to the conclusion that "the undesirable caliches have higher liquid limits and plasticity

indices"/ ...

indices" and that the satisfactory materials had had to be excavated by blasting or ripping. His general conclusions were:

1. "Caliches with plasticity indices less than about 10 with flocculation factors not exceeding 1.7, and without the gel-producing colloids should prove satisfactory as bases for thin bituminous surface treatments under average moisture conditions". (The flocculation factor was defined as "the ratio of pores to solids in a sediment formed in 24 hours from a mixture of 5 cc absolute volume of powdered soil solids thoroughly dispersed in 39 cc of distilled water and 1 cc of chemical deflocculant" and the colloidal gel referred to is that formed above the sediment. Gillette seemed to consider that the gel was composed of silica).

2. "That caliches with plasticity indices of 10 to 15, flocculation factors not exceeding 2.5 and without colloidal gel, should prove satisfactory for thin bituminous surfaces in semiarid or well-drained locations.

3. The better grade materials, even with a considerable amount of gel, may prove satisfactory as bases for the thicker bituminous surfaces."

4. The stability of caliche bases depends on the quality and quantity of the minus 40 mesh fraction.

5. The hard and semihard caliches are the best (those requiring blasting or ripping).

6. Powdery caliches may be used under favourable conditions, but the completed road has very high capillary action.

7./ ...

7. Though a caliche with high liquid limit and plastic limits may be satisfactory for unsurfaced roads, it will prove unsatisfactory if a surface covering, which prevents evaporation of capillary moisture, is placed on top of it or if it is placed without providing good drainage".

8. The minimum depth of base course should be 8 inches, preferably compacted in 4 inch layers.

Day (1955) mentioned the construction techniques used on a caliche base course job near El Paso, Texas, and the importance of ensuring that moisture contents during compaction were closely controlled.

Uppal and Singh (1958), in a laboratory study of soft aggregates (which included some kankars) in the state of Rajasthan in India normally considered unsuitable for base course, were able to show that although few samples tested were suitable in their natural state, most could be readily brought up to base course standard by mechanical stabilization. They also stressed the need for proper quality control during excavation and placing of low grade materials.

Clare and Beaven (1962) mentioned the use of nodular calcrete (jigilin) for road bases and one airfield (Maiduguri) base in Nigeria. These structures had given satisfactory service for up to 17 years and were still in "reasonable condition". From their paper it would appear that they do not carry very heavy traffic.

Beaven (1962) has briefly reviewed the use of calcretes in base construction, quoting chiefly from Gillette (1934).

Limestone rubble (nodular calcrete?) underlying a hard crust is used extensively for base course in Victoria, New South Wales and South Australia (Mr A.J. Scala, 1965; private comm.). The only difficulties experienced are

those/ ...

those of breaking through the hard crust and keeping the plasticity index below 12. Scala considers that PIs up to 18 could and have been used in these areas (rainfall about 10 inches) provided the drainage was good. In more recent times use has also been made of the hardpan crusts (Mr. D.F. Glynn, 1965; private comm.).

The Highways and Local Government Department of South Australia (1965, private comm.) use nodular calcretes (kunkar) as base course when not too plastic, and crusher-run hardpans have been found to make good bases.

Considerable quantities of caliche have been used in the past (apparently for base course) in Arizona (Mr. W.G. O'Harra, 1966; private comm.), but their use has been discontinued owing to the very thin deposits of good material, their variable quality, and the fact that on many occasions they had placed inferior material on the roadway before the PI was found to be too high. It is apparent that visual estimation of the quality is unreliable. No actual failures were mentioned.

In New Mexico, caliche is extensively used for base course and good quality material is comparable in all respects with a Pennsylvania limestone. (Mr. T. Cole, 1966; private comm.).

The Conference of State Road Authorities of Australia (1958) and NAASRA (1968) stated that, while the desirable limit to the plasticity index is 6, this may be increased to 9 (10 according to NAASRA) in arid climates with a rainfall of less than 15 inches, when the percentage of soil binder (that passing the B.S. No. 36 sieve) is low and when the binder has some chemical setting properties such as in the case of "some ironstone gravels and limestone rubbles containing finely divided lime". NAASRA also cautioned against accepting as a general rule that

higher/ ...

higher PIs can be tolerated by ironstone gravels and limestone rubbles. Both publications also mentioned that a higher liquid limit than 25 may be allowed for some limestone rubbles (nodular calcretes?) and for certain absorptive materials. Elsewhere NAASRA also mentioned that "gravels of limestone and, to a lesser extent ironstone origin can exhibit a form of cementing action in service and accordingly their performance may be superior to that expected on the basis of test results above".

Table 9.4 shows some overseas calcrete base specifications. As far as is known, these are the only specifications available which are specifically for calcretes or calcrete-like materials.

The specifications are discussed in order. The CAA specification appears to have been superseded by that of the FAA. The FAA (Mr M.H. Bard, 1968; private comm.) advises that "the use of caliche aggregate as a pavement construction material in the arid and semiarid southwest regions of this country is predicated on economic considerations entirely". The higher soil constants and finer gradings are allowable because of the "arid climate in which these materials occur" and also because "the majority of airports constructed of this material are of the utility category serving aircraft of less than 12,500 pounds gross weight". Only non-plastic caliches of superior quality are recommended as "high type" of base. The sulphate and LAA tests are not regarded as reliable on caliches as "experience has shown that some caliches are satisfactory for light pavement construction although sulfate and wear tests are not satisfactory". The bituminous paving thicknesses used over caliche bases are the same as for any other material. Mr. Bard concluded that "caliche construction is more art than science". All the other FAA base specifications require liquid limits of less

than/ ...

TABLE 9.4 SOME OVERSEAS SPECIFICATIONS FOR CALCRETE BASES

AUTHORITY	SPECIFICATION &/OR ITEM NO.	MAX LL %	MAX PI %	MAX SIZE IN.	PERCENTAGE PASSING				OTHER
					$\frac{3}{4}$"	B.S. 7	U.S. 40	U.S. 200	
CAA[1] (O'Fiel, 1944)	Caliche base	35	10	2"	-	-	15-35	0-20	LAA $\leq$ 80%
FAA[1] (1959)	Caliche base (P.210)	35	10	2"	-	-	15-35	0-15	-
Texas[1] (1962)	Caliche base, Grade 1 (232)	40	12	$1\frac{3}{4}$"	-	-	15-50	-	-
West[2] Australia (1967)	Coastal limestone "base"	-	-	3"	50-75	30-55	-	-	10% F.A.C.T. >1.0 ton. % $CaCO_3$ in - 7 mesh fraction $\geq$ 60%
South Australia (1965)	Calcrete crusher run	?	?	?	?	?	?	?	L.A.A. 35-50%

(1) Soil constants determined after slaking and by preparation methods involving heating at less than 140°F (60°C) i.e. F.A.A. 210-2.2 modification of AASHO T87 and Texas TEX-101-E (see Section 7.2).

(2) The coastal limestone (mostly dunerock) is effectively non-plastic and is normally used as a subbase under a crushed stone base (Dr. B. Clegg, 1969; personal comm.).

than 25, PIs of less than 6, LAA's of 45-50 depending on the base (50 for gravel bases) and a total loss in the sodium sulphate test (AASHO T104) of less than 12% (not specified for gravel bases). Maximum sizes allowable vary with the base concerned. The percent passing the 40 mesh sieve is limited to 10-30, 10-25, 15-30 depending on the base (other grading requirements also given).

The Texas Highway Department (1962) do not list a subbase specification and it would appear that all their upper construction layers are referred to as bases in the specification manual. A large number of base specifications are listed and it is presumed that the actual contract documents would state that a base of a certain item number shall be used as base and that a base some other item number as subbase. It also seems to be common practice in the U.S.A. to use the same specifications for both base and subbase. The AASHO and ASTM specifications for both base and subbase are identical in all respects, though those of the FAA and New Mexico are not. The FAA actually have a more severe soil constant specification for subbase (same as most of their bases - maximum LL of 25 and PI of 6) than for caliche and shell bases, though the remarks above concerning the Texas subbase might also apply here. The gradings of New Mexico bases and subbases are different, but the maximum LAA of 50% and PI of 6% are the same in both cases. A maximum LL of 35% is specified for subbase and 25 for base. Caliche is listed as acceptable for base or subbase, but is required to pass the normal specifications (Mr. T. Cole, 1966; private comm.), which are quite severe.

Returning to the Texas specifications, private communication with Messrs. R.L. Lewis and T.S. Huff of the Texas Highway Department reveals that the caliche base (Item 232) would not normally be used unstabilized as upper base for their "high-type highways", but rather as subbase

for/ ...

for such highways and as "upper base for lower traffic roads such as our Farm-to-Market and some State highways". Type A (crushed or broken aggregate) Grade 1 and 2 caliches only appear to have performed satisfactorily as upper base "for up to medium volume primary highways". In these cases both the liquid limit and the percentage minus 40 mesh are normally limited to a maximum of 35% (the maximum allowable PI remaining at 12). On "exceptionally low volume facilities" where the anticipated traffic is only about 250-600 vehicles per day (proportion of heavies not known, but classed as Farm-to-Market roads and not generally "through highways"), pit run (natural gravel) caliches have been used possessing liquid limits of 45 and PI's of 15. "The surfacing is usually either a one-course or two-course penetration type surface treatment under either specification Item 320 or 322". The reasons Mr. Huff gave for the high soil constants allowed were that (a) most caliches had liquid limits and PIs of less than 40 and 12 respectively and that "to enable us to take advantage of the economics of this local material, it is necessary that we not specify limits which cannot be met", (b) the dry climate, and (c) the fact that Texas (and the FAA) employ a wet sample preparation method which would release more clay and hence yield higher soil constants than the dry methods employed by others such as the AASHO, ASTM and New Mexico. It is apparent, however, that caliches with these high soil constants and poor gradings have nonetheless performed satisfactorily as bases for up to Texan medium volume highways. Caliche base Item 232 does "not require either Los Angeles Abrasion wear, wet ball mill or triaxial classification".

A very useful summary of the latest Texas specifications and design procedure has been given by Smith (1969). Taking some examples, it is apparent that Texas successfully use Grade 1 bases (LL $\leqslant$35, PI $\leqslant$10, plus other

requirements/ ...

requirements such as grading (e.g. 15-30% minus 40 mesh), aggregate strength and triaxial classification) under all traffic conditions, under surface treatments for up to 150,000 total equivalent 18^k single axle applications and under thicker surfacings for heavier traffic (e.g. $4\frac{1}{2}$" for 10 million applications). Grade 2 bases (LL $\leq$35-40, PI $\leq$10-12 and % minus 40 mesh 30-50 depending on base Type, plus other requirements) would apparently normally be used under surface treatments for up to 61,000 applications and under thicker surfacings for up to 400,000 applications (at which a $2\frac{1}{4}$" surfacing would be used). Grade 3 bases (LL $\leq$35-45, PI $\leq$12-15 and 35-55% minus 40%, plus other requirements) would normally only be used for up to 38,000 applications under surface treatments. The use of Grade 2 and 3 bases under heavier traffic than that mentioned even under thicker surfacings is "not usually recommended unless better quality base is not available". Triaxial classification is not required of Grade 3 bases. The maximum size of most of the Texas bases is about $1\frac{3}{4}$". It is apparent that the Texas specifications in general are less severe than those of most other authorities and yet successful roads are apparently built to them.

Regarding the Western Australian specification in Table 9.4, the coastal limestone varies from calcified sand to hardpan calcrete according to the classification of Netterberg (1967; see also Section 1.3.3) (Mr. F. Parrick, 1967; private comm.). The 10% F.A.C.T. limit specified of 1.0 ton was queried , but confirmed by Mr. Parrick. With the exception of the hardpan, the 10% F.A.C.T. values of the coastal limestone are normally less than 2.5 tons. The coastal limestone is what would be called dunerock (aeolianite) in South Africa (and possibly also beachrock) on which pedogenesis has acted to produce calcrete hardpans. The material is effectively non-plastic. Traffic volumes and the types of seal employed

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on these roads were not stated, but Dr. B. Clegg of the University of Western Australia (1969 , personal comm.) has subsequently informed the author that traffic varies from heavy urban conditions to lighter rural conditions and that seals vary from single coat to several inches of asphaltic concrete. An important point is that in South African terminology the coastal limestone would only be used as subbase under a crushed stone base. The latter may, however, be as little as 1/2" thick, depending on traffic. Emulsion stabilized coastal limestone has also been used for base by one authority. The climate in the coastal limestone area is of the Mediterranean type with an annual rainfall of 20-35 inches falling mostly in winter. Subgrades are freely draining sands.

It is apparent from the foregoing that calcretes with such poor specifications as those shown in Table 9.4 (and even worse) have been used and are still being used overseas (notably in the southwestern United States)for low and medium volume roads. On the Texan low volume roads (250-600 anticipated vpd), even PIs of up to 15 are allowed under thin surface treatments. There does not appear to be any reason why the Texas and the other specifications in Table 9.4 should not be applicable in the areas of Southern Africa where calcretes occur, provided that test methods are employed which yield comparable results to those employed by the authorities concerned. It is apparent that calcretes which are accepted by the FAA and Texas as satisfactory base material would normally only be used for selected subgrade in the Republic and South West Africa. Considerable savings are thus possible if their specifications can be adopted here. Limitation of wheel loads to those allowed on FAA and Texas bases of such specifications will of course also be required. It would appear that wheel load limits are more strictly enforced in the U.S.A. than in the Republic and S.W.A. (Mr. A.A.B. Williams, 1969; personal comm.).

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In connection with the Texan specifications in general, it should be noted that their other base specifications also permit high PIs and liquid limits. Depending on the grade of base, the PI might range from 10 to 15, so that it cannot be assumed without question from Table 9.4 that higher PIs are allowable for calcretes than for other materials. The Texas Highway Department employs a wet sample preparation method (see Section 7.2) and the Texan climate is on the dry side.

9.4.2 Evidence from South Africa

Table 9.5 shows some details of five roads in South Africa employing calcrete as base and subbase. Detailed test results carried out by the Cape Provincial Roads Department are shown in Table K.1 in Appendix K. These are also shown summarised in Table 9.6 for the four roads for which traffic counts are available. The details of the roads and the test results were 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 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. The borrow pits from which the calcrete bases were thought to have come from were also sampled and tested in the case of the Port Elizabeth roads and yielded similar results. 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. One calcrete with a grading modulus of 1.2 (74% passing 40 mesh) yielded a CBR as high as 107. All five roads have performed entirely satis-

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TABLE 9.5 SOME DETAILS OF FIVE SOUTH AFRICAN ROADS EMPLOYING CALCRETE AS BASE AND SUBBASE

ROAD	YEAR COMPLETED	SURFACING	TOTAL THICK-NESS OF IMPORTED (CALCRETE) LAYERS	NATURAL SUBGRADE	12 HOUR TRAFFIC COUNT, vpd		RAIN-FALL mm	N VALUE	THORN-THWAITE'S MOISTURE INDEX	REGIONAL DESIGN FACTOR
					CARS	HEAVIES				
PORT ELIZABETH MARINE DRIVE (MAIN ROAD 4)	Before 1936	¾" surface treatment to 2" (penetration?)	6"-10"	Sand	220* 800*	5* 18*	~540 ~770?	~2.6 ?	~-18 ?	0.50 0.50?
PORT ELIZABETH - ADDO (MAIN ROAD 1)	Before 1936 to 1940	¾" surface treatment to 2" penetration	6"-8"	Brown soil	290* 350*	70* 40*	~500 ~400	~3 ~5	? ~-30	0.50 0.50
LANSDOWNE ROAD, CAPE FLATS	1926, surfaced 1952	2 coat surface treatment	11"-24"	Sand or grey soil	3000**	760**	~500	~3.7	~-7	0.50
DUINEFONTEIN ROAD (NO. 122), CAPE FLATS	1950	?	8"-14"	Sand	960+ 1100+	520+ 630+	~500 ~ "	~3.7 ~ "	~-7	0.50
WELTEVREDE ROAD (NO. 156), CAPE FLATS	1960	?	12"-16"	Sand	?	?	~500	~3.7	~-7	0.50

* Average of 6 day period (Monday to Saturday) in July 1967, from 6 a.m. to 6 p.m., near each end of road. Vehicles, excluding cars, with payloads in excess of ½ ton counted as heavies.

** On 12/1/67, from 7 a.m. to 7 p.m. All vehicles over 3 tons counted as heavies.

+ At the same point on 15/1/67 and 24/1/66 respectively, from 7 a.m. to 7 p.m. All vehicles over 3 tons counted as heavies.

factorily 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 probably be acceptable on a grading basis to the Transvaal if screened minus $1^1/2$". 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 <u>some</u> of them for use as stabilized base or subbase, presumably also only on other than national roads and other roads not intended for heavy traffic.

It is also most interesting to apply the FAA and Texas caliche base specifications in Table 9.4 to these materials. The detailed analysis is shown in Table K. On a grading and soil constant basis only one base course sample out of 30 (3%), sample DE.7418, would pass the FAA specification and only 3 out 30 (10%), DE.7382, V.2329

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and V.2333 would pass the Texas specification. Many fail only on the maximum size criteria, but if these are waived the figures are still only raised to 2 (7%) and 15 (50%) passing the FAA and Texas specifications respectively.

Table 9.6 summarises the data on the four roads for which some idea of the traffic conditions is available. The information is grouped according to total traffic counts, although the Port Elizabeth-Addo road and the Port Elizabeth Marine drive actually carry about the same traffic on an equivalent 18,000 lb single axle load basis. Landsdowne road carries about $1\frac{1}{2}$ times the traffic of Duinefontein road on this basis.

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

The Texan specification in Table 9.4 is probably applied to roads in the 500-1000 vpd traffic category. Here their limit of 50% passing the 40 mesh sieve agrees well with the limit of 56% possessed by most of the roads in Table 9.6 (only one sample out of the 26 exceeded this) and their maximum PI of 12 is not exceeded by the worst in Table 9.6 (9). Considering the different preparation and testing methods employed by the Cape and Texas, they agree very well indeed. It is apparent from Sections 7.2-7.5 that the Texas methods are likely to yield less coarse minus 40 mesh material and possibly more clay (and hence higher soil constants) than the Department of Transport (1958) methods used by the Cape.

The last two groups in Table 9.6 would presumably fall into the Texan medium volume primary highway category. The 43-56% passing 40 mesh in Table 9.6 is somewhat higher

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TABLE 9.6 SUMMARY OF TEST RESULTS ON 26 SAMPLES OF CALCRETE BASES

PROPERTY	TRAFFIC CATEGORY (vehicles per 12 hour day)							
	350-400 (10-20%>½ ton payload		220-820 (2%>½ ton payload)		1,500-1,700 (35%>3 tons)		3,800 (20%>3 tons)	
	P.E. - ADDO RD.	NO. OF SAMPLES	P.E. MARINE DRIVE	NO. OF SAMPLES	DUINEFONTEIN RD.	NO. OF SAMPLES	LANDSDOWNE RD.	NO. OF SAMPLES
MAX. SIZE (inches)	0.75-1.5	8	1.5-2.0	8	1.5->2.0	5	1.5->2.0	5
GM	1.4 -1.8	8	1.5-2.2	8	1.2-1.7	5	1.5-1.9	5
% - 40 MESH	39-56	8	28-56	8	48-74	5	43-56	5
LL (%)	28-33	8	20-24	8	18-20	5	17-20	5
PI (%)	11-15	8	4-9	8	4-7	5	6-8	5
LS (%)	5.5 -6.5	8	2.5-4.0	8	1.5-3.0	5	2.5-3.0	5
LS X % - 40 MESH	254-330	8	83-168	8	111-159	5	107-168	5
GI	0.2 -0.5	8	0.0	8	0.0	5	0.0	5
REVISED PRA CLASS.	A-2-6(0) to A-2-6(1)	8	A-1-b(0) to A-2-4(0)	8	A-1-b(0) to A-2-4(0)	5	A-1-b(0) to A-2-4(0)	5
0.1" LAB. CBR (%)	47,66	2	62,90	2	87,107	2	112-134	3
0.2" LAB. CBR (%) @	61,83	2	81,92	2	108,133	2	133,142	2
REL. LAB. COMPACTION (%)	100,100	2	100,99	2	100,100	2	102-98	3
CBR SWELL (%)	0.1, 0.2	2	0.0, 0.1	2	0.0, 0.0	2	0.0	3
REL. FIELD COMPACTION (%)	?	0	?	0	96-104	5	97-102	5
ESTIM. 0.1" FIELD CBR (%)	-	0	-	0	70,92	2	60-118	3
ESTIM. 0.2" FIELD CBR (%)	-	0	-	0	74,120	2	77-142	3

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

than the 35% allowed by Texas and the maximum liquid limit and PI of 20 and 8 less than the Texan limit of 35 and 12, but once again the difference in preparation and test procedures could account for at least some of this discrepancy (the discrepancies are still in the correct direction). The maximum sizes in the last three roads in Table 9.6 also vary on either side of the Texas figure of 1.75".

9.4.3 Conclusions and suggested specifications

It is apparent from both overseas and local experience that calcretes which would fail the normal South African base course requirements have nevertheless made perfectly satisfactory bases for light and medium trafficked roads under relatively thin surfacings. This is true in the case of South African experience in the case of material which has failed both grading and CBR, material which has failed grading, CBR and soil constants requirements and material which has failed only grading requirements. Reasons for this will be considered in Section 9.11.

Table 9.7 shows the suggested specifications derived from the study described.

Until further information is available, it may be wise to apply the specifications in Table 9.7 only in cases where the road is well drained, the water table not shallow, the subgrade favourable, and the soil fines strongly calcareous. Unless excess soluble salts are present, moisture passing upwards through the base and evaporating near the surface and moisture cycles are likely to be favourable, however. There must be some limit of carbonate content only above which a material will behave as a calcrete. The usual type of low PI specification (such as the AASHO) was derived from studies of the behaviour of such leached materials as bases. These leached materials

TABLE/ ...

TABLE 9.7 SUGGESTED INTERIM SPECIFICATIONS FOR CALCRETE NATURAL GRAVEL OR CRUSHED NATURAL GRAVEL BASES

PROPERTY	TRAFFIC CATEGORY (vpd), <20% <3 TONS			
	<500	500-1000	1000-2000	2000-4000
MAX. SIZE (inches)	0.75-1.5	1.5-2.0	1.5-2.0	1.5-2.0
MIN. GM	1.5	1.5	1.5	1.5
PERCENTAGE PASSING 0.42 mm BY WEIGHT	15-55	15-55	15-55	15-55
MAX. LL (%)	40	35	30	25
MAX. PI (%)	15	12	10	8
MAX. BAR LS (%)	6.0	4.0	3.0	3.0
MAX. -0.42 mm COND., Cl^- (m mhos/cm)	1.8	1.8	1.8	1.8
MAX. -0.42 mm COND., SO_4^{--} (m mhos/cm)	0.7?	0.7?	0.7?	0.7?
MAX. GROUP INDEX	0.5	0.0	0.0	0.0
WORST REVISED PRA CLASS.	A-2-6	A-2-4	A-2-4	A-2-4
MAX. BAR LS X % -0.42 mm	320	170	170	170
MIN. DRY 10% FACT. VALUE (tons)	-	9?	12?	12?
MAX. DRY ACV (%)	-	35?	30?	30?
MIN. SOAKED/DRY 10% F.A.C.T. VALUE (%)	-	60?	60?	60?
MAX. WATER ABSORPTION (%)	-	-	5?	5?
MIN. DRY APV	50?	60?	70?	70?
MIN. 98% M.AASHO 0.1" CBR (%)	60	80	80	80
MIN. 98% M.AASHO 0.2" CBR (%)	80	80	100	100
MAX. M.AASHO CBR SWELL (%)	0.5	0.5	0.5	0.5
MIN. REL. FIELD COMPACTION (%)	98	98	98	98

Notes: 1) Department of Transport (1958) routine test methods to be used.

2) Traffic counts are per 12 hour (daylight) day.

3) Conductivity is measured on the saturated paste at 60°F at a consistency between about 1 and 30 blows of the liquid limit device (see Section D.5).

4) All aggregate strength tests are carried out on $\frac{1}{2}$"-$\frac{3}{4}$" aggregate and soaked tests after 24 hours soaking.

would probably fail if the specifications in Table 9.7 were to be applied to them. This limiting carbonate content is not known with any certainty, but is probably of the order of 10% calcite plus dolomite. Limiting the total carbonate content of the soil fines (minus 0.42 mm fraction) to a minimum of 8% calculated as CO_2 (equivalent to 18.2% $CaCO_3$) should be safe, though it may exclude some good materials. This figure of 8% would exclude most calcretes containing kaolinitein their soil fines (also a sign of leaching). If the total carbonate content of the soil fines drops below 5-8% CO_2 it would probably be wise to specify the authority's normal specifications. A qualitative test with hot dilute hydrochloric acid (both dolomite and calcite effervesce strongly) would probably suffice in most cases. A number of rapid carbonate methods are available should more quantitative determinations be desired. The weight loss on acid treatment, acid neutralization and Collins' Calcimeter are particularly rapid. The method should be one that determines calcite plus dolomite, as there does not appear to be any reason to discriminate against either of the two carbonates at present.

The question of surfacing thickness is not too clear. Table 9.5 suggests that a surface treatment is adequate, but it will be noted that the surfacings do vary up to 2" of penetration macadam in some cases. Until more

definite/ ...

definite information is available it may be advisable to apply the authority's usual surfacings for the traffic expected. The surfacings used in Texas (see Smith's(1969) Table II) are somewhat heavier than those suggested by the information in Table 9.5. The few caliche bases included in McDowell's(1966) study carried up to 7,000 vpd, but were fairly good materials (PIs less than 10) under 4-5" of asphaltic concrete.

The data in Table 9.5. suggests that the minimum cover on the good sandy subgrades should be 6, 6, 8 and 11 inches respectively for the four traffic groups, increasing with traffic severity. Gillette (1934) suggested a minimum depth of "base course" of 8 inches, preferably compacted in two 4 inch layers. Gillette's base course was probably actually total cover. The usual CBR cover design method should be satisfactory. Loose Kalahari-type sands probably require a good cover owing to their uniform grading, rounded grains and lack of cohesion when dry.

The items in Table 9.7 will now be briefly discussed in order of their appearance. The maximum sizes were derived simply from Table 9.6 and could be reduced if desired (which would also have the effect of making the percentage passing 40 mesh more conservative, according to maximum compacted density concepts), but should not be increased. The grading modulus is the most common minimum value of the 26 samples and is closely equivalent to 55% passing the 40 mesh sieve (Figure 8.16). The minimum minus 40 mesh figure is the Texas Grade 1 (Item 232) caliche base minimum. The grading modulus is a quite well accepted method of providing a simple control on the grading in the Transvaal and South West Africa. Common requirements are minimum values of 2.1, 1.6 and 1.5 for natural base, cement or lime stabilized base and natural subbase, respectively. If a grading modulus is accepted as adequate, Figure 8.16 suggests that merely specifying a maximum size

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and the percent passing 40 mesh after the Texas manner will also be adequate in the case of calcretes. It would obviously not be adequate as a blanket specification. Detailed grading requirements are probably not warranted.

It will be observed that the liquid limits and PIs in Table 9.7 are not always those suggested by Table 9.6. The former are often higher and were decided on after consideration of the values allowed by Texas, the probable unreliability of the two tests and the range of liquid limit and PI possessed by calcretes with the linear shrinkages shown. Table 8.3 suggests that even higher PIs are allowable if the linear shrinkage is regarded as the most reliable calcrete soil constant. Although the liquid limit is also an indicator of the compressibility of most soils (in calcretes it also includes the particle porosity), moderately high liquid limits are probably not deleterious in granularbases under South African semiarid or arid conditions, provided the linear shrinkages and PIs are not excessive. The linear shrinkage is intended to catch the calcretes with excessive clay contents and is regarded as a reliable and essential test. It will be observed that it is not relaxed above that suggested by Table 9.6. Except for the values of 6 in the lightest traffic group, the figures of 4 and 3 suggested are in line with what is usually employed in South West Africa (4) and in the Republic (3) for all classes of surfaced roads. If the designer wishes to be more conservative, he should specify maximum liquid limits of 35, 30, 25 and 25 and maximum PIs of 15, 10, 8 and 8 for the four traffic groups.

The soil paste conductivities were derived from Table D.2 and are designed to eliminate those calcretes containing deleterious amounts of highly soluble salts like NaCl, Na_2SO_4 and Na_2CO_3. The limits assume that the compaction water is non-saline. If it is, the limits

should/ ...

should either be adjusted according to the calculated amount of salt to be added by the compaction plus any curing water or applied to unstabilized material taken from the full dpeth of base course after compaction.

The group index and revised PRA classification were derived from Table 9.6 and need no further comment. The figures for linear shrinkage times percent passing 40 mesh were also derived from Table 9.6. Figure 8.18 shows that they provide some indication of the CBR and of course are also measures of the potential volume change from the shrinkage limit to the liquid limit of the whole sample for the case where the plus 40 mesh fraction is floating in the minus 40 mesh fraction. Linear shrinkage is here preferred to PI because of its probable greater reliability.

No information is available on the aggregate strengths required of calcrete bases and the figures given are intended as rough guides only. The dry 10% F.A.C.T. and equivalent ACV requirements in the 1000 vpd plus categories are those of the South African Department of Transport for national roads. The APV of 70% is roughly equivalent to this (Figure 7.13). The percentage ratio of soaked to dry strength is about 6% less than the average for crushed calcrete hardpans and boulders. Very few results are available for nodules. It is designed to detect those calcretes which possess unusually low wet strengths. The data in Appendix H suggests that a limit on the water absorption of 5% will eliminate all material with a dry 10% F.A.C.T. value of less than 11 tons and a soaked value of less than 6 tons. The rest of the figures are merely guesses. It is not intended that all the aggregate strength tests should be specified. A number are merely shown in order to give some choice. The combination dry and soaked 10% F.A.C.T. is probably the best test, while the APV and water absorption are the simplest. The APV can also be carried out on soaked aggregate, but no results are as yet available.

The/ ...

The Treton is another useful simple test, while some will desire a sulphate test, for comments on which see Section 7.8.2. The AASHO and ASTM limits for this test are probably too severe for South African conditions, particularly those under which calcretes occur. It is of interest to note that AASHO M147-67 does not require a sulphate test for aggregate and soil-aggregate subbase, base and surface courses. The $\frac{1}{2}$"-$\frac{3}{4}$" aggregate is usually tested for base courses in South Africa, but it is possible that this requirement will be altered to $\frac{3}{8}$"-$\frac{1}{2}$" aggregate in the near future. This would be in line with BS 812 and SABS 647. The smaller calcrete nodules are likely to yield lower strengths than the larger. Although no failures of surfaced calcrete roads due to aggregate degradation are known to the author, calcrete aggregate is of very variable strength and is often rather weak, and strength testing of some sort is advisable. The subject is discussed in Section 7.8.

The CBR values are derived from Table 9.6 and the usual South African 0.1" requirement of 80 (see also Section 7.12). NAASRA (1968) states that while a value of more than 80 is desirable the minimum is 60 and that in arid climates a value as low as 30 can be used. It will be observed that Table 9.7 is conservative in that the CBR is always above 80 if the 0.2" CBR is also used (see Section 7.12). The CBR requirements are not relaxed due to the possibility of self-stabilization because the latter cannot be guaranteed at this stage. The CBR swell is the usual Department of Transport requirement. A value of swell obtained from oven dry to soaked conditions might be more indicative of the behaviour of a base under warm semiarid and arid conditions.

In using these specifications it is considered that emphasis should be placed on the CBR, compaction, swell, conductivity, linear shrinkage, the maximum size and the

percentage/ ...

passing 40 mesh. These are thought to be the most important indicators of the performance of a calcrete base.

As far as mechanically stabilized calcrete bases are concerned, Table 9.7 can probably be applied to all cases where the soil fines remain strongly calcareous. The authority's normal specifications should be applied when the carbonate content of the minus 0'42 mm fraction falls below 5-8% CO_2.

Crushed bases of all kinds derive a large proportion of their strength from mechanical interlock. For this to be effective a high density structure consisting of strong angular particles is essential, so that in the absence of any information to the contrary, it is advisable to apply the authority's usual grading criteria. The soil constants of the highest traffic category in Table 9.7 can probably be applied in the case of a straight crusher run. The authority's usual specifications should be applied in cases where other than calcrete fines are to be employed and where the carbonate falls below the 8% suggested. An aggregate strength test is essential. The figures given in Table 9.7 for the highest traffic group should be adequate for crusher run and waterbound macadams, but in the absence of information to the contrary it would be wise for the authority to specify its usual aggregate strength requirements. The requirements of the Department of Transport since October 1963 have been a minimum dry 10% F.A.C.T. value of 17 tons for penetration macadam (formerly a maximum ACV of 30%) and 12 tons for soil and waterbound macadam (formerly a maximum ACV of 45 on $\frac{1}{2}$"-$\frac{3}{4}$" aggregate. SABS 647-1961 (as amended) requires 17 tons for penetration macadam and 12 tons for waterbound macadam. Calcrete hardpans and boulders are not normally saline, but the conductivity limits could be applied to the crushed fines as a check. The grading may have to be

corrected/ ...

corrected for bulk particle specific gravity if a very sandy calcrete is used or other material added to improve the grading. Valton (1947) has shown that limestones can make perfectly satisfactory macadam bases.

9.5 SUBBASE

The use of calcrete for subbase in South Africa and other countries is quite common, although even for this purpose it often has to be stabilized to bring the PI within the usually accepted South African requirement of 10 if more than 30% passes 2 mm and 12 if less than 30% passes 2 mm. Figure 9.1 shows the areas in South Africa where calcretes have been used for all classes of road materials. Examples of specific roads which have used calcrete subbases are shown in Table 9.3. This table is not claimed to be complete.

In the United States, the use of caliche for subbase on a particular job in Texas was mentioned by Day (1955) who stated that under most conditions caliche makes a good subbase. He also emphasized the need for proper moisture control during compaction.

Uppal and Singh (1958), in their laboratory study of soft kankar (calcrete nodules) aggregates, concluded that most of them were suitable for subbase "even under worst subgrade moisture conditions".

In South Australia, the Highways and Local Government Department (1965, private comm.) use much nodular calcrete (kunkar) and scalpings (taken before crushing hardpans for crusher run) for subbases. If the fines are not too plastic, the scalpings apparently make almost as good a base as the crusher run.

Caliche is also used, apparently for subbase, in parts of New Mexico, Texas and Oklahoma (Mr. T. Cole, 1966;

private/ ...

private comm.).

Less information is available on specification for calcrete subbase than for base. It would appear from Texas experience that their Item 232 Caliche Base (Table 9.4) is suitable as subbase for any of their "high type" highways. The FAA have a very wide grading specification for subbase in general, while requiring low soil constants.

Tables K.1 and K.2 are of little help, as the subbase is in most cases of comparable quality to the base. It is apparent that the same base specification in Table 9.7 can also be used for subbase. This appears to be common United States practice (though McDowell (1966) suggested maximum liquid limits and PIs of 35 and 12 and 40 and 20 for the top 6"-8" of base and for subbase respectively in Texas). More information comes from the Western Transvaal, where the Schweizer Reneke-Bloemhof road has employed a calcrete subbase between mile 15.1 and 17.7 under a cement stabilized alluvial (diamond) gravel base and surface treatment. Seven test results carried out during construction were as follows: LL NP through 23 to 56, PI 0-19, LS 0-10, percent passing 40 mesh 20-45%, LS X % minus 40 mesh 0-320 (mostly 200-300), Modified AASHO densities 109-112 lbs/cu ft, and optimums 14-16.5%. Relative field compaction was 94-103% and field CBRs all 25 plus. The road was constructed in 1962 and apart from some stabilization cracking due to the base and some rare crocodile cracking, the calcrete-subbased sections have performed satisfactorily to date.

In view of what is known, it seems reasonable to suggest that the lightest duty base specification in Table

9.7/...

9.7 should prove adequate as subbase under any flexible base. CBR can of course be relaxed to 25 or 44 in place at 95% Mod. AASHO compaction depending on whether a 9,000 lb or 12,000 lb wheel load design is called for. Group index can also be relaxed to the usual figure of 1, or possibly even more. The percentage passing 0.42 mm should perhaps be tightened by 5 or 10% if the minimum size allowed is 2½ inches. The conductivity requirements can possibly be dispensed with, though until it can be shown that salts will not migrate from the subbase to the top of the base it would be safer to apply the base course limits to subbase as well. If the completed base will be completely non-saline (minus 40 mesh conductivity less than about 0.1 mmhos/cm) the conductivity limits in Table 9.7 can probably be relaxed somewhat (perhaps doubled). Aggregate strength requirements are probably not critical, and for light and medium trafficked roads a specification is probably unnecessary. The West Australian authorities successfully employ "coastal limestone" (aeolianite or dunerock, mostly equivalent to a non-plastic calcified sand) with dry 10% F.A.C.T. values down to 1.0 tons as subbase under thin crushed stone flexible bases and surfacings.

9.6 SELECTED SUBGRADE AND FILL

Although calcrete has been used for selected subgrade and fill in all the areas in which it has been used for subbase (and probably also in other areas as well), no information is available on its requirements.

If calcretes with apparent PIs of up to 15 can be used in light duty bases and as subbases in Texas and at Port Elizabeth, it seems reasonable that such materials can also be used anywhere in Southern Africa where calcretes occur for those purposes as well as for selected subgrade, though such calcretes would fail the present D.O.T. selected subgrade PI requirement of 12. It is probably

reasonable/ ...

reasonable to relax the group index requirement by one unit, making calcretes with group indices of up to 1.0 acceptable for selected subgrade. Material with up to 72-80% passing 0.42 mm (GM of 1.0 to 0.8) and linear shrinkages of up to 7 should be adequate. The group index requirements for the rest of the fill could presumably be similarly relaxed, but in all cases the usual CBR, swell and compaction requirements should be maintained.

In the absence of information on the depth of moisture migration in roads due to temperature changes and other causes, it is difficult to decide whether it is worth limiting the soluble salt content of the subgrade. Until more is known on the subject it may be advisable to apply the base course limits to both the subbase and the selected subgrade. In the unusual case of a very shallow water table, upward salt migration will certainly take place if the surfacing is permeable. It would probably be advisable to apply subbase conductivity limits to the upper subgrade until more is known of salt migration in roads.

9.6.1 Calcrete fills under seepage conditions

It may be necessary to approach the use of all strongly calcareous soils in fills subject to seepage conditions (e.g. earth dams) with caution. The carbonate portion of a calcrete is slightly but definitely soluble in water and this may lead to loss of carbonate by solution, subsequent increase of permeability and decrease of density of the fill, with consequent failure of the structure, either completely or as an effective water barrier.

The seepage rate can be readily calculated from the permeability of the compacted soil and the hydraulic head at that point. The solubility of calcium carbonate (it will be advisable to check whether the cement is dolomite or calcite or both) at any partial pressure of CO_2 at any

temperature/ ...

temperature and at any porewater pressure may be found or calculated (see Sections 4.2.4 and 4.2.5) so that theoretically it would be a simple matter to calculate the rate of loss of carbonate from the fill. It may be difficult to decide on a suitable CO_2 concentration to use. The CO_2 content of dam water may well be quite high.

If any dam wall or road fill likely to be subjected to seepage forces is to be constructed of calcareous or calcified soil or calcretes it would be advisable then to calculate the rate of carbonate loss from the seepage loss (determined from the permeability of suitably compacted samples of the soil and the flow rate for the design of dam or fill) and the likely solubility of carbonate at that point (determined from the temperature, pressure, existing carbonate content of the water and the CO_2 content of the water). With regard to the solubility problem it may be advisable to actually measure the solubility of the particular calcareous material in the particular river water and also to measure CO_2 contents of dam water at different depths (taking precautions against CO_2-loss when bringing the sample to the surface) near the wall of one or two similar dams. This procedure should result in a very accurate estimate of the carbonate loss being obtained.

It may be mentioned that some of the carbonate may be precipitated (depending on degree of pressure drop and evaporation taking place) soon after the exit of the seepage water from the dam will. The solubility should in fact be affected by the pore water pressure prevailing at any particular point on the flow net. This could theoretically be estimated from Figure 4.8.

Without a knowledge of the potential solubility of carbonate in dam water it is difficult to decide whether solution of the fill is likely to be a problem or not.

This/ ...

This discussion is merely presented as a warning that it should be considered.

9.7 BINDER

Nodular powder calcretes, powder calcretes and calcified sands of suitable grading and plasticity should make excellent binders for base courses. Such materials have already been used as fillers in premix.

Wherever calcretes at an early stage of development such as the above are to be used, attention should always be given to their soluble salt contents. The normal base course limits suggested in Table 9.7 should be applied to the final mixture. Disintegration problems could also arise in a premix if a salty calcrete is used.

9.8 GRAVEL ROADS

Calcretes have also been extensively used for gravel road wearing courses wherever they occur, in fact this is probably their major use. In South Africa and South West Africa they have been used in the areas shown in Figure 9.1 and it can be assumed that they have been used wherever they occur in usable quantities. In practice this means the areas marked on Maps 2 and 4 as those in which hardpans occur. Deposits in the nodule zone, i.e. that area receiving between about 22 inches (550 mm) and 32 inches (800 mm of rainfall) (N = 5 to 2), are usually too poorly developed to be worthwhile. Nodular calcretes in this zone are usually very thin and the occurrence of no or only scattered nodules in the profile is more normal.

A good calcrete gravel road will possess a hard surface crust, (probably formed in a similar manner to hardpan) which enables the road to last for some months without grading. This hard surface crust is called a "blad" (Plates 9.1 and 9.2) by Afrikaans-speaking road-

workers/ ...



PLATE 9.1

Fairly good choice of material. Hard crust or "blad" on a calcrete road near Grootfontein. Note little loose surface material, lack of corrugations and incipient potholes. Failure by potholing rather than by corrugation is typical of calcrete gravel roads. In the above example this is possibly at least partly due to insufficient crossfall.



PLATE 9.2

Closeup of calcrete gravel road surfacing shown in Plate 9.1.

workers and can be obtained even on a calcrete wearing course of relatively low plasticity. The crust does not corrugate or raise excessive dust, but does sometimes tend to pothole. When this occurs the potholes must be patched without delay as the crust breaks up rapidly once potholing has commenced. Grading damages the crust and hence this should be resorted to only when really necessary. When available, a thin(1") layer of Kalahari-type sand may be graded over the surface as protection for the crust until it can be watered and rerolled - in dry areas use should of course be made of rainfall for this purpose.

Few authorities in South Africa test materials for gravel roads and the economics of gravel roads are such that they are often even compacted without the assistance of water. However, the reliance on the experience of road workers in calcrete gravel selection can lead to trouble in some cases. Plate 9.3 shows the result of using a poorly developed nodular calcrete mixed with black clay, while Plate 9.4 shows the result of insufficient aggregate strength. Blinding with sand, diatomaceous earth or broken tufaceous hardpan (the weaker types are very porous and absorbent)before rainfall will help in the first case, while a "blad" can be developed in the second case by recompaction with water. In some cases gravel is scarce and there is not much choice, but even experienced roadworkers do not always expect such poor performance.

Mr. G. Blunt (1965, personal comm.) has suggested that, in order to overcome the problem cheaply, the ideal calcrete gravel road wearing course be established, samples of this made up (artificially if necessary) and circulated among those responsible for the selection of gravel road materials. In this way they would learn the appearance and feel of a good calcrete the easy way, without having to try it out on the travelling public first.



PLATE 9.3

Poor choice of material, shrinkage and PI too high. Result of use of material with too high a shrinkage – a soft nodular calcrete mixed with black clay. Near Maroelaboom.



PLATE 9.4

Poor choice of material, AFV and grading modulus too low. The result of using a powdery material with insufficient aggregate of too low a strength, but which also lacks a suitable grading and soil constants (too low) for a sand-clay wearing course. The above material is said to form a "blad" and to last reasonably well if compacted with water. Near Okaukejo.

9.8.1 Some existing specifications

The performance of and specifications for gravel roads have been discussed for example by Fossberg (1962, 1963a, 1963c). The following conclusions may be drawn from this author's work.

1. There is no universally applicable design method for gravel road wearing courses.

2. Unlike a bituminous or concrete surfaced road, a gravel road is profoundly effected by the weather.

3. Specifications vary widely, many are wanting and the allowance made for climate (if any) also varies widely.

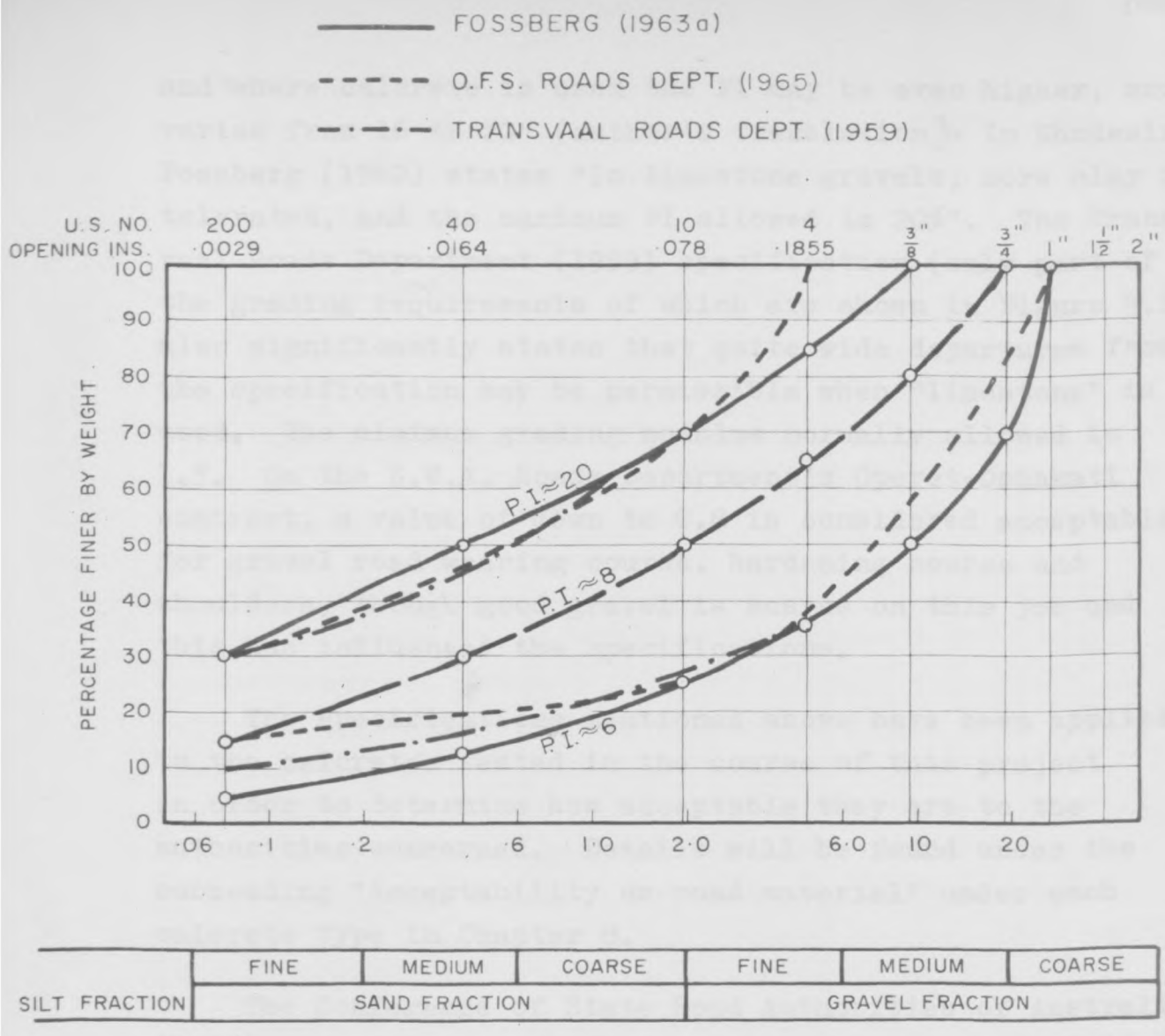
4. Performance data and suitable test methods for wearing courses - which differ in their requirements from base courses for surfaced roads - are lacking.

5. Most authorities place great reliance on indicators, i.e. grading and Atterberg limits.

6. The importance of proper compaction is stressed.

Fossberg further considers that the PI should always be considered in relation to the grading, and not separately, as is commonly done. His suggested specifications are shown in Figure 9.2., together with the recommended grading envelope of the Orange Free State (1965) and Transvaal (1959) Roads Departments. The O.F.S. Roads Department are known to have included calcretes in their experimental wearing courses. They state that the percentage passing the 0.0164" sieve should preferably be 32 - 37%, but that in the case of calcretes this may be 40% or higher. They further specify PIs depending on the rainfall and grading of the gravel in question, for example "in dry areas the PI shall be 14 when the % minus 0.0164" is 45%, but may be 18 when it is only 20%

and/...



THE PARTICLE SIZE DISTRIBUTION CURVE SHOULD BE SMOOTH AND AS CLOSE AS POSSIBLE PARALLEL TO THE ENVELOPES.
FOR GRAVEL ROADS, UNDER DRY CONDITIONS THE GRADING CURVE SHOULD TEND TOWARDS THE UPPER ENVELOPE (PLASTICITY INDEX TO BE IN HIGHER RANGE), UNDER WET CONDITIONS IT SHOULD TEND TOWARDS THE LOWER ENVELOPE (PLASTICITY INDEX TO BE IN LOWER RANGE).
FOR SURFACED ROADS, THE GRADING CURVE SHOULD FALL BETWEEN THE DASHED LINE AND THE LOWER ENVELOPE, AND THE PI SHOULD BE BELOW 8 (FOSSBERG, 1963a)

FIGURE 9·2

Suitable particle size distribution envelopes for gravel road wearing courses. After Fossberg (1963a), the O.F.S. Roads Dept. (1965) and the Transvaal Roads Dept. (1959)

and "where calcrete is used the PI may be even higher, and varies from 16 to 20" (author's translation). In Rhodesia, Fossberg (1962) states "In limestone gravels, more clay is tolerated, and the maximum PI allowed is 20%". The Transvaal Roads Department (1959) specification (only part of the grading requirements of which are shown in Figure 9.2) also significantly states that quite wide departures from the specification may be permissible when "limestone" is used. The minimum grading modulus normally allowed is 1.5. On the S.W.A. Roads Department's Operet-Oshakati contract, a value of down to 0.8 is considered acceptable for gravel road wearing course, hardening course and shoulders, though good gravel is scarce on this job and this has influenced the specifications.

The specifications mentioned above have been applied to the calcretes tested in the course of this project in order to determine how acceptable they are to the authorities concerned. Details will be found under the subheading "Acceptability as road material" under each calcrete type in Chapter 8.

The Conference of State Road Authorities of Australia (1958) stated that while the use of a maximum liquid limit of 35 and a PI of 2-9 was normal practice in Australia for unsurfaced gravel roads, these limits may be increased when dealing with certain limestone rubbles (nodular calcretes?), when the rainfall is less than 15 inches per annum (PI can be increased to 12), when the percentage of soil binder is low, or when it has some chemical setting qualities (certain limestone rubbles and ironstone gravels - nodular calcretes and ferricretes?). NAASRA in (1968) repeated these statements (though a PI of 4-9 was mentioned, of the order of 4-6 being considered optimum) adding that the usual linear shrinkage figure of 3 for gravel roads could be increased to 6 in arid climates. Usable calcretes rarely occur where the rainfall is more than 22", so that

an/ ...

an increase in PI and LS is justified merely on climatic grounds; however, a significant point is that a higher PI is also allowed for calcretes than for other materials in the same area.

9.8.2 Specifications for calcretes

A full treatment of the subject of gravel road wearing courses in general is beyond the scope of this work, but some comments will be made which are thought to be of value in connection with the calcrete specifications to be proposed presently.

All road materials can be divided into two basic types, those which are relatively coarse in which the coarser material forms a relatively rigid structural framework and those in which the coarser material floats in the finer. In the first case the aggregate strength, degree of interlocking and the compacted density control the stability of the mass and the nature of the fines is of secondary importance. The most important property of the fine fraction under these circumstances is that it should not be able to expand sufficiently to force the coarse material apart, thus destroying the stability (Wooltorton, 1954). According to Wooltorton (1954), the dividing line between these two groups of materials is 30-45% passing 0.42 mm by weight for sand-clays and 30-45% passing 2 mm for coarse aggregates. This of course assumes a constant bulk particle specific gravity. If the bulk particle specific gravity of calcrete fines is higher than that of the coarser material (which should theoretically be the case) the aggregate would only begin to float in the fines at higher percentages of fines by weight. It would appear that minus 200 mesh material does not form a structural framework (Wooltorton). Most specifications for surfaced base courses are derived from maximum density criteria, but an increase of fines is desirable in wearing

courses/ ...

courses to provide cohesion when dry. This occurs when for example the AASHO M147 B-F gradings are used (Woods, 1960). A most useful discussion of concepts has been given by Wooltorton (1954).

In dealing with specifications for calcretes, the properties of the coarse aggregate will be dealt with first.

Aggregate strength requirements

While working in the field in the Okavango during 1965, it was found that the calcretes which made good gravel road wearing courses were those with the stronger aggregate. Those with weak aggregate tended to break up and form powder potholes (e.g. Plate 9.4) even under light traffic. A pair of pliers from the vehicle toolkit was brought into play and it was found that the two types of calcrete gravels (mostly reasonably coarse nodular calcretes) could be readily distinguished by means of this simple test, called the aggregate pliers test (A.P.T.). The test has been described by Netterberg (1967) and in detail in Section 7.8.3 where it has also been correlated with the 10% F.A.C.T.

A number of calcrete gravel road wearing course gravels were sampled and tested in the field or later in the laboratory, and an interpretation of the aggregate strength tests alone (disregarding all other properties of the material entirely) on 25 samples of known performance is given in Tables 9.8 and 9.9.

It can be seen that good wearing courses generally possess an AFV of at least 60% and an APV of at least 20%. The figures for Samples 1017 and 1020 are so high because the material contained so little plus $\frac{1}{2}$" aggregate, all the softer aggregate having already been degraded. It can also be seen that, as might be expected, the aggregate

TABLE/ ... .

TABLE 9.8 AGGREGATE STRENGTH OF GOOD CALCRETE WEARING COURSES

SAMPLE NUMBER	% MINUS 0.42 mm	AFV (% PASSING FINGERS) %	FAIL PLIERS, BUT PASS FINGERS %	APV (% PASSING PLIERS) %	NUMBER OF PIECES TESTED	REMARKS
977	Low	67	16	51	140	Satisfactory?
978	38	86	32	54	148	Satisfactory
1016	38	96	22	73	161	Good
1018	Low	93	32	61	145	Corrugated slightly
1019	71	24	4	21	146	Good
1021	58	70	36	34	106	Good
1022	32	51	39	12	409	Very good
1024	54	39	20	19	167	Good
1512	66	75	42	33	52	Good
1513	49	92	78	14	51	Good
1514	49	77	52	75	69	Very good
1516	37	78	41	37	78	Fairly good
1525	Low	80	50	70	148	Good?
1533	Low	98	11	87	313	Good
1618	Low	97	42	51	57	Fairly good
1620	Low	80	56	24	59	Very good
1879	Low	100	23	77	137	Good
1880	Low	99	22	77	153	Good
-	Low	100	10	90	164	Good
-	Mod.	62	15	47	106	Good borrow pit
RANGE	-	24-100	4-78	12-90	51-409	Satisfactory-very good
SUGGESTED LOWER LIMIT		60	30	20	-	
% OBEYING LIMIT		85	55	85	-	50% obey all 3 limits, 95% obey one limit

Note: All road samples, unless otherwise stated

TABLE 9.9 AGGREGATE STRENGTH OF CALCRETE WEARING COURSES TENDING TO POWDER

SAMPLE NUMBER	% MINUS 0.42 mm	AFV (% PASSING FINGERS %)	FAIL PLIERS, BUT PASS FINGERS %	APV (% PASSING PLIERS) %	NUMBER OF PIECES TESTED	REMARKS
976	Mod.	0	0	0	120	Powder hole
1017	High	94	42	53	127	Powder hole
1020	High	75	12	63	161	Powder hole
1515	84	25	17	8	36	Bad borrow pit
-	Mod.	53	35	18	60	Bad borrow pit
RANGE	-	0-94	0-42	0-63	60-161	All powdered

Operators: Messrs T.E. O'Connor and F. Netterberg

strength required for successful performance also tends to rise with coarseness of the material. The fit of the data is improved if the test is only applied to materials with less than about 50% passing 40 mesh.

It will be noticed that only aggregate strength has been considered so far and that it appears that it may only be necessary to specify aggregate strength in the case of the coarser samples. Even specifying that at least 60% of the $\frac{1}{2}$"-$\frac{3}{4}$" aggregate shall be unbreakable by hand (AFV) would apparently ensure a reasonably successful road in more than 85% of the cases studied. The AFV and APV will, however, increase on the road, (it has been found that loose surface calcrete gravel which has been on the road for at least a year usually possesses an AFV of around 100 and an APV of at least 75%) so that it is essential that sufficient aggregate be present at the start. Failure of the road must

result/ ...

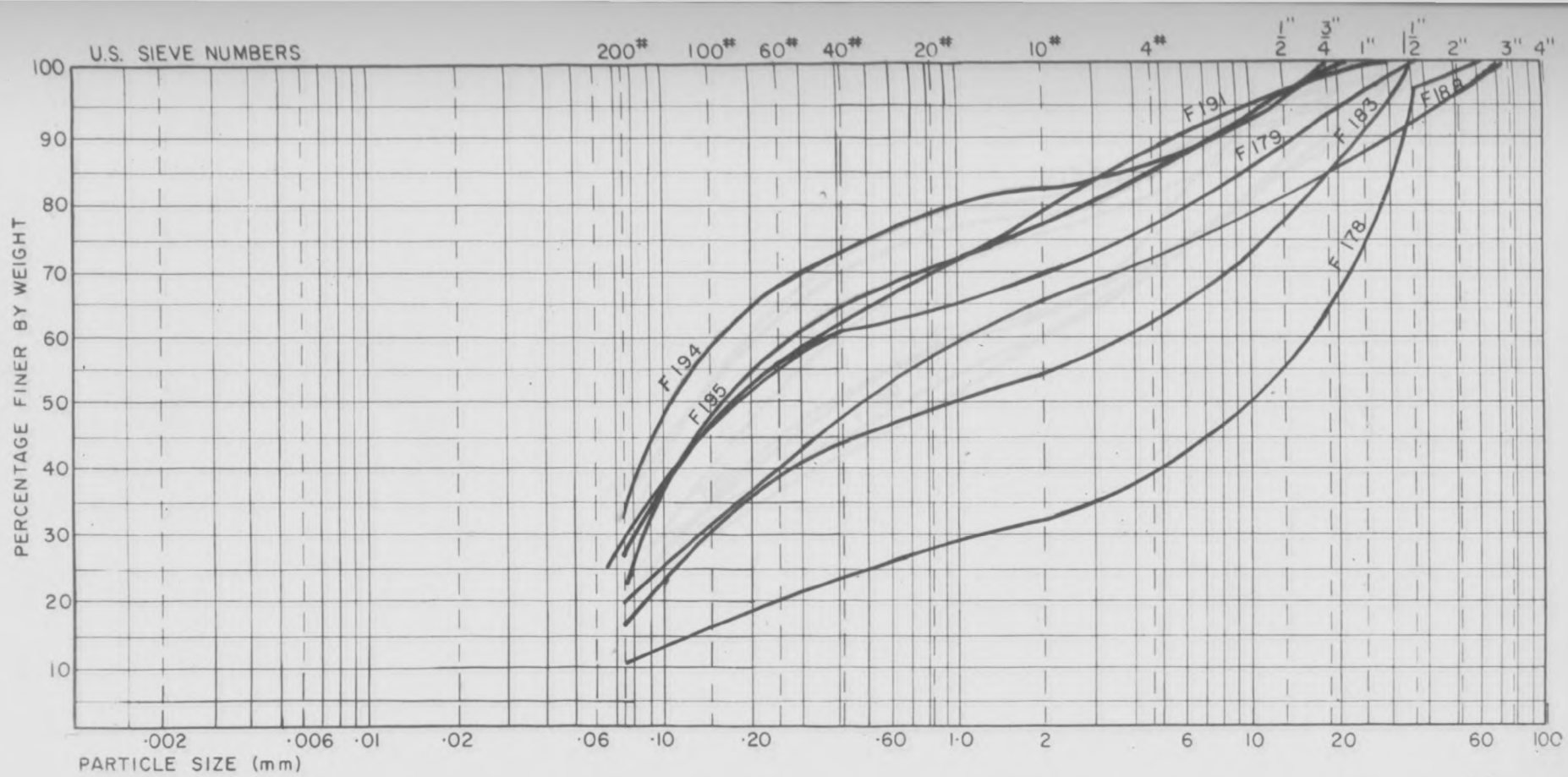
result if the aggregate degrades to a material unsuitable for a sand clay wearing course.

Calcretes which formed powder holes were found to possess liquid limits ranging from 23 to 50, PIs from SP to 27, and linear shrinkages up to 8.0. Their minus 40 mesh contents were moderate to high. Clay content alone does not therefore appear to be sufficient to prevent the formation of powder potholes in calcrete roads during the dry season. Recompaction with water usually ensures an at least temporary cure.

Particle size distribution

Figures 9.3-9.5 show the particle size distribution of some satisfactory and unsatisfactory calcrete wearing courses from the Western Transvaal. These roads were sampled and tested by the Transvaal Roads Department as part of a recent joint inspection of calcrete roads inthe Western Transvaal by the Department and the NIRR. Further details of the samples will be found in Appendix K. Apart from one sample, it can be seen that particle size distribution curves do not afford a good means of distinguishing good from bad material. The Transvaal and Orange Free State wearing course specifications (including soil constant requirements) do not afford a very good separation between good and bad calcretes. They do eliminate all the bad materials, but only at the cost of rejecting about half the good. Figure 9.6 shows the probably desirable particle size distribution of calcrete wearing courses in comparison with those that have actually been found satisfactory. The probably desirable range was arrived at after consideration of Figure 9.2, a general consideration of wearing course requirements, an attempt to obtain the best fit between the good and bad samples in Figures 9.3-9.5 and other earlier samples, and a conviction that the sand hunch common in so many calcretes is at least partly

due/ ...

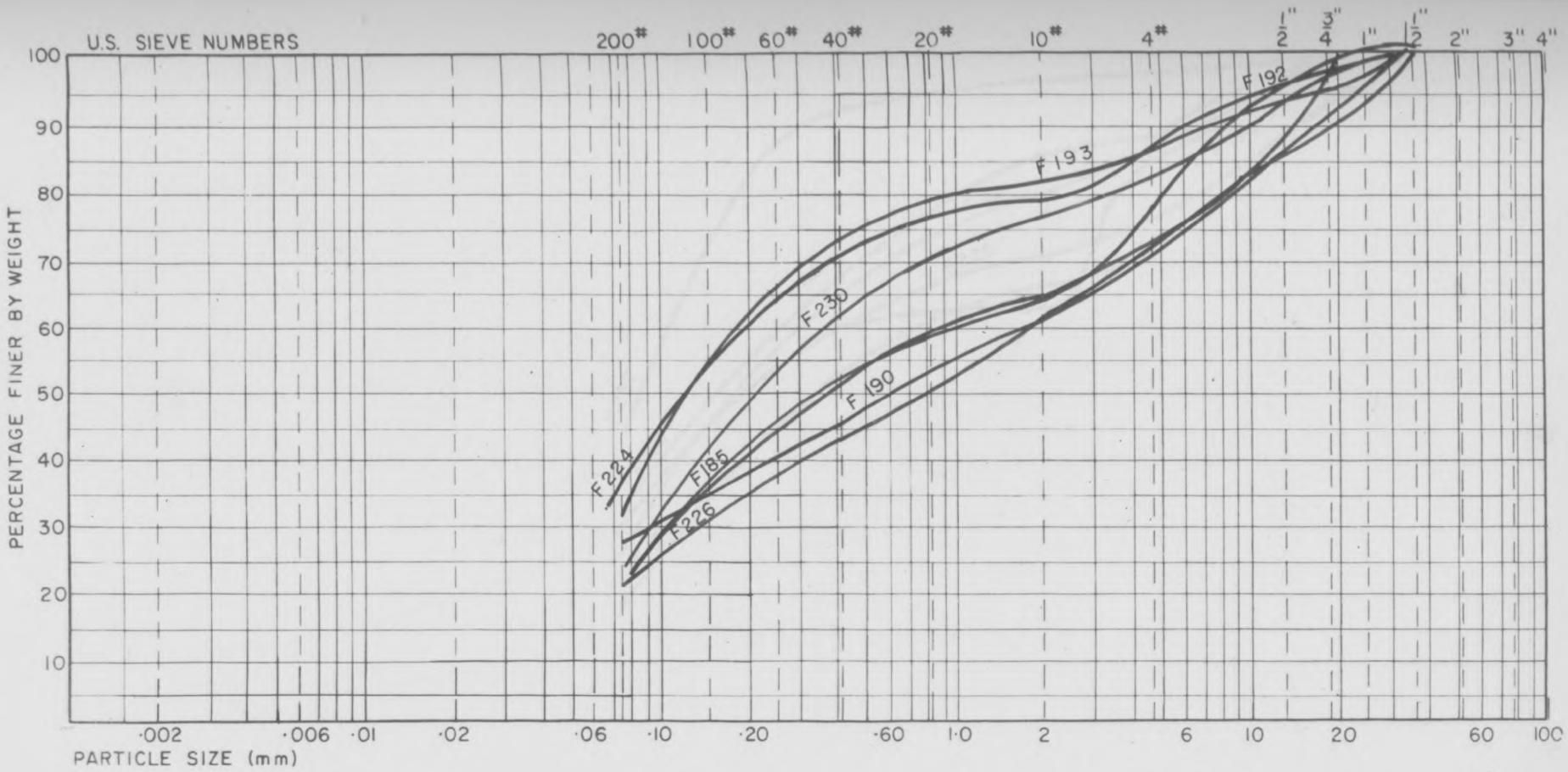


CLAY FRACTION	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE
	SILT FRACTION			SAND FRACTION			GRAVEL FRACTION		

FIGURE 9.3

GRADING ANALYSES OF WEARING COURSES FROM GOOD ALL WEATHER CALCRETE ROADS

RG4 3782

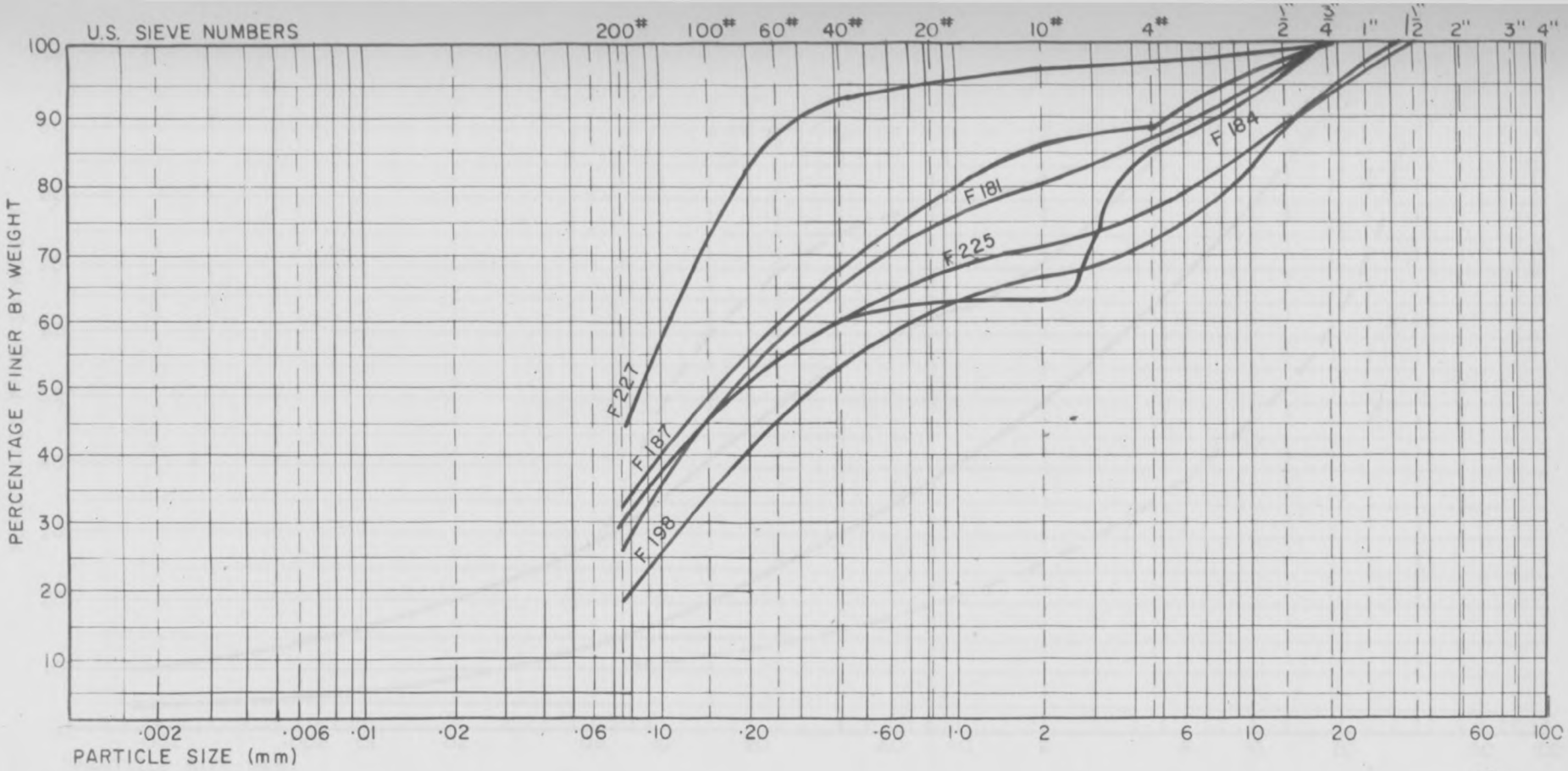


CLAY FRACTION	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE
	SILT FRACTION			SAND FRACTION			GRAVEL FRACTION		

FIGURE 9.4

GRADING ANALYSES OF WEARING COURSES FROM GENERALLY SATISFACTORY CALCRETE ROADS

RG4-3783

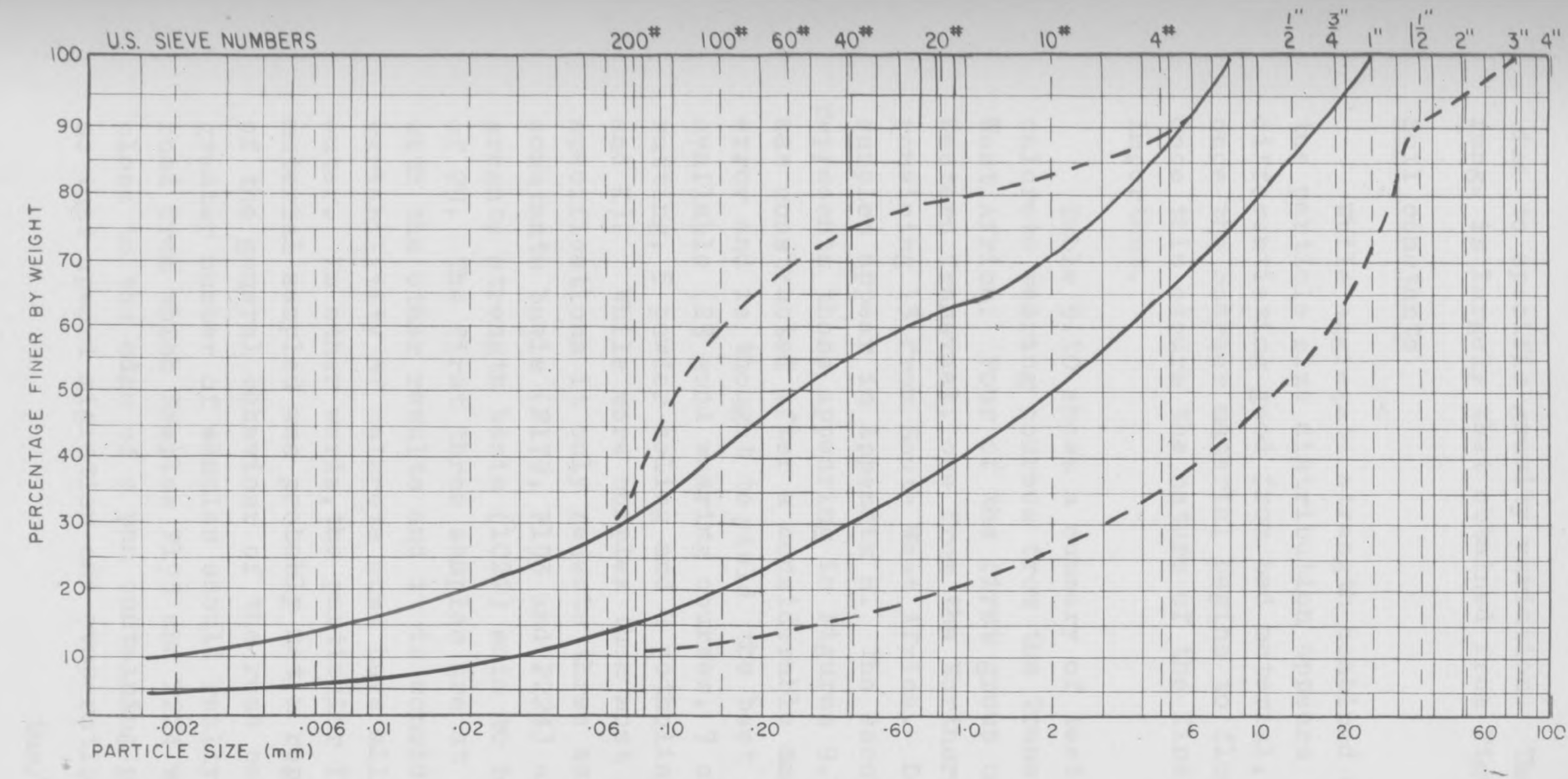


CLAY FRAC-TION	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE
	SILT FRACTION			SAND FRACTION			GRAVEL FRACTION		

FIGURE 9.5

GRADING ANALYSES OF WEARING COURSES FROM SLIPPERY CALCRETE ROADS

RG4 – 3784



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

FIGURE 9.6

PROBABLY DESIRABLE PARTICLE SIZE DISTRIBUTION OF CALCRETE WEARING COURSES (SOLID) AND THE KNOWN SATISFACTORY RANGE (DASHED)

RG4 – 3785

due to specific gravity variation. The known satisfactory range is largely that obtained from Figures 9.3-9.5

Soil constants

While aggregate strength combined with a limit on the particle size distribution appears to be useful in differentiating good from bad material, it is inadequate once the coarser material begins to float in the finer. Once this occurs the nature of the fines becomes all important.

Table 9.10 shows a summary of test results on 29 calcrete wearing courses from the Transvaal and South West Africa. Four of the first group came from the Western Transvaal, one from the Northern Cape and the remaining 13 from South West Africa. Details of these samples appear in Appendix H. The second group of samples represents those appearing in Figures 9.3-9.5. Table 9.10 was constructed after a considerable amount of trial and error and is thought to yield the best fit to the data available (29 good wearing courses, 7 slippery and/or rutting, 5 powder holing and 2 potholing - Appendices H and K). While more complex than most wearing course specifications it only rejects three samples on a soil constants basis (F179, F193 and F224) and one on an aggregate strength basis (1022) said to be satisfactory out of 29. The first three samples are at complete variance with the other results and it is considered that the great variabilitity of calcrete must have allowed error to enter. In other words, the particular few pounds of material sampled was probably not a representative sample of the general behaviour of the road concerned. A much greater number of samples should really be taken. The road from which Samples F193 and F224 were taken ran close to the edge of a pan containing perennial water, so that upward migration and evaporation of moisture from

the/ ...

TABLE 9.10 SUMMARY OF TEST RESULTS ON SATISFACTORY CALCRETE WEARING COURSES FROM SOUTH WEST AFRICA AND THE TRANSVAAL AND SUGGESTED SPECIFICATIONS

PROPERTY	PERCENT PASSING 40 MESH (0.42 MM) BY WEIGHT								SAMPLE LOCATION	TESTED BY
	15 - 45	NO. OF SAMPLES	45 - 55	NO. OF SAMPLES	55 - 65	NO. OF SAMPLES	65 - 75	NO. OF SAMPLES		
REVISED PRA CLASS.	A-2-7 to A-2-4	4	A-7-6 to A-2-4	3	A-2-6	1	A-2-7	1	Chiefly S.W.A. & W.Transvaal	NIRR
GI	1-0	4	3-0	3	1	1	1	1	" " " " "	
LL	64-31	4	43-26	3	37	1	29	1	" " " " "	"
PI	22-9	4	20-10	3	8	1	10	1	" " " " "	"
LS	8-3	4	8-5	3	3	1	2	1	" " " " "	"
LS X % - 40 MESH	331-100 (1)	4	392-260 (3)	3	307	1	132	1	" " " " "	"
AFV	100-51	12	92-39 (4)	4	70	1	75-24	2	" " " " "	"
APV	90-12 (2)	12	75-14	4	34	1	33-21	1	" " " " "	"
REVISED PRA CLASS.	A-2-7 to A-2-6	3	A-2-6 to A-2-4	4	A-2-6	3	A-2-4	1	W.Transvaal	TPA
GI	1-0	3	1-0	4	0	3	0	1	" "	"
LL	48-29	3	32-22	4	36-24	3	19	1	" "	"
PI	23-14	3	22-5	4	12-11	3	8	1	" "	"
LS	9-5	3	8-2	4	5-4	3	3	1	" "	"
LS X % - 40 MESH	374-168	3	416-104	4	305-262	3	199	1	" "	"
AFV	97-71	3	92-88	4	86-43	3	60	1	" "	"
APV	61-30	3	41-26	4	28-4	3	14	1	" "	"
SUGGESTED SOIL CONSTANTS AND AGGREGATE STRENGTH REQUIREMENTS FOR CALCRETE WEARING COURSES										
LL	65-25	7	45-20	7	40-20	4	30-18	2	Valid for Southern Africa ?	
PI	23-8 (6)	7	22-5	7	13-8 (7)	4	13-5	2	" " " " "	
LS	9-3	7	8-2	7	5-3	4	5-2	2	" " " " "	
LS X % - 40 MESH	380-100 (7)	7	420-100 (7)	7	330-100 (7)	4	330-100 (7)	2	" " " " "	
AFV	>65	15	>60 (5)	8	-	4	-	2	" " " " "	
APV	>25	15	>20 (5)	8	-	4	-	2	" " " " "	

(1) Only one under 78

(2) Only one under 24

(3) Only one under 62

(4) Only one under 19

(5) These limits need not be applied in the 50-55% range if desired.

(6) Linear shrinkage down to 2 allowable if some looseness and dust tolerable

(7) Values down to 70 allowable if some looseness and dust tolerable.

the shallow water table under the road might have caused some cementation, causing the samples to behave better than they would have under more normal conditions. In addition, this road was said to pothole, so that it is doubtful if it can really be regarded as satisfactory. The reason for Sample 1022 not obeying the limits is obscure. The specification would also just accept Sample F198, said to be unsatisfactory, but which yields quite reasonable test results.

Table 9.10 shows that, depending on the grading, apparent PIs from 5 to 23 are tolerable and that aggregate strength testing is probably only necessary when the minus 0.42 mm fraction is less than about 55%.

The soil constants tolerable will vary somewhat with the climate, but in general calcretes appear to be more tolerable of variations in constants and grading than most materials (reasons for this will be discussed in a later section). The results have not been properly analysed with respect to rainfall, but inspection suggests that the maximum allowable PI in Table 9.10 should be lowered by one or two units and perhaps the linear shrinkage by 0.5 to 1.0 in areas receiving more than 20 inches of rainfall. Well compacted calcrete roads are not usually prone to corrugate, but if the soil constants are less than those shown in Table 9.10 they are liable to be dusty. If the soil constants shown are exceeded, they will be slippery and/or rut when wet. It is suggested that rutting when wet and potholing when dry might be expected when the percentage passing 40 mesh is greater than 55 and the value of LS X % minus 40 mesh is greater than about 400.

CBR

CBR test results are not available on any of the materials, but field testing of in situ CBR at the natural moisture content using the hand operated dynamic cone

penetrometer/ ...

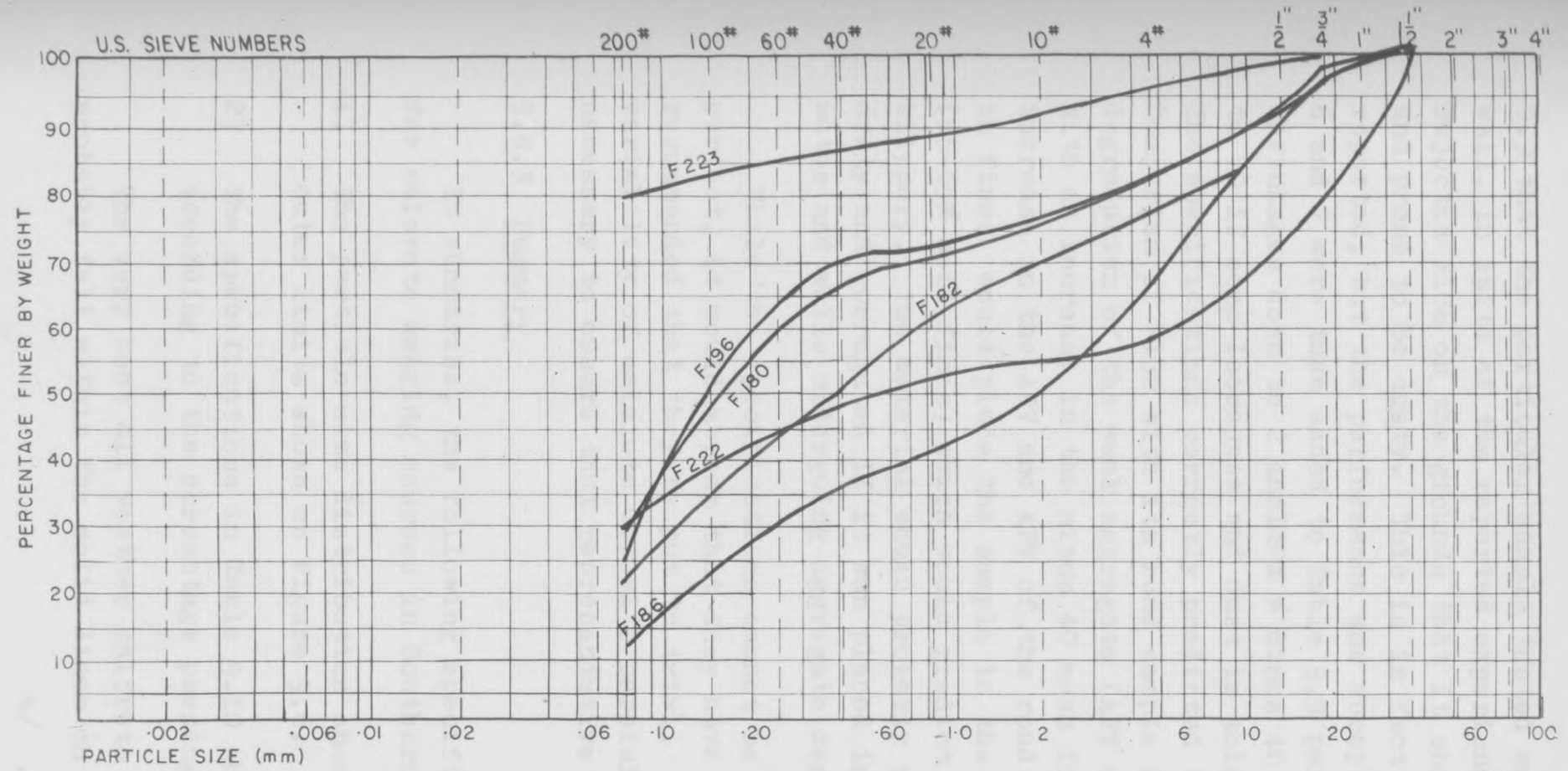
penetrometer of Mr. D.J. van Vuuren of the NIRR showed that the satisfactory materials tended to yield moderate to high CBRs. Two unsatisfactory materials yielded low CBRs and one said to be unsatisfactory (F198), but which must be regarded as doubtful, a high CBR. A Mod. AASHO CBR of more than 60 is desirable for heavily trafficked gravel roads (Yoder, 1959) and this is also what is required in the Transvaal specification. The cone of the penetrometer unfortunately broke early in the investigation, preventing a more quantitative assessment, but it was nevertheless found most useful.

Attention is drawn to the rough correlation found in Figure 8.18 and CBR There is a high probability that a calcrete with a value of bar linear shrinkage multiplied by the percentage passing 40 mesh of less than 200 will possess a Mod. AASHO CBR of at least 60.

Trial of specifications against 6 borrow pit samples

After construction of the specifications based mostly on road samples in Table 9.9, they were tested by applying them to 6 samples from borrow pits in the Western Transvaal, details of which are shown in Table K.4 and Figure 9.7.

The specifications passed F222 and rejected F223, agreeing with the known performance of these two samples. Although F223 powdered on the road, it would probably have lasted for a while if compacted with water. The specifications rejected F196 and F180, suggesting that they should be slippery and rut, especially F196. This is at variance with reported experience, both materials being said to yield satisfactory performances. The PI, linear shrinkage and LS X % -40 mesh requirements in the 65-75% minus 40 mesh group in Table 9.9 were then relaxed to from their original 10, 3 and 200 to 13, 5 and 330 in order to take in F180. It was felt that the results on F196 are so at variance with the other results in Table



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

FIGURE 9.7

GRADING ANALYSES OF SOME CALCRETE BORROW PIT SAMPLES YIELDING VARYING PERFORMANCES AS WEARING COURSES

RG 4-3786

9.9 that the particular sample tested would not perform well, in spite of the reported experience. The specifications rejected F186 on the grounds that it should be too loose and prone to be dusty. This is in fact the experience reported, but the performance was acceptable. Footnotes 6 and 7 were then added to Table 9.9 permitting linear shrinkages down to 2 and LS+X % minus 40 mesh values down to 70 if some looseness and dust is tolerable in dry weather. The specifications correctly predicted the behaviour of F182. Comparism of F182 with its road sample F181 suggests that degradation of the weak aggregate (AFV only 30%) took place with an increase in the minus 40 mesh fraction and a slight increase in the AFV and APV of the road sample. The increase in fines. would place the sample in the next highest group (55-65% soil fines) which would predict that it would be slippery. The material would probably only become loose, dusty and corrugated if it was placed in the winter without water and while undergoing aggregate degradation.

While the specifications cannot be claimed to be perfect, it would appear that they have merit and it is recommended that they be put to trial. In view of the great variability of calcrete deposits careful work will be necessary to ensure that representative samples are obtained.

9.8.3 Summary

To summarise, the following specifications are suggested for calcrete wearing courses in Southern Africa.

1) The particle size distribution should be within the outer limits shown on Figure 9.6.

2) The specifications in Table 9.10 should be applied according to the percentage passing 40 mesh.

The very best all weather calcrete wearing course will probably fall within the solid lines on Figure 9.6, possess

a maximum size of ¾", a linear shrinkage between 3 and 6, an AFV in excess of 65%, an APV in excess of 25%, and a value of LS X% minus 40 mesh of between 100 and 200. The latter will ensure a good CBR.

Soluble salts are not deleterious in gravel roads and in fact are actually beneficial, exerting a cementing and dust palliative effect.

9.8.4 Dust palliatives

When a hard crust or "blad" has not been obtained, calcrete gravel roads can be among the most dusty of roads. (Plates 9.5 and 9.6). The problem of dust laying has been discussed for example by the Road Research Laboratory (1963). Not only do clouds of dust constitute a hazard to driving and make journeys on heavily trafficked gravel roads rather trying, but dust can also kill or damage roadside vegetation and crops by settling on the leaves and thus preventing transpiration. Dust also increases wear on motor vehicles and its effect when inhaled by mineworkers over long periods is also well known.

The methods of dust-laying all involve binding the fines to the coarse material to prevent their being loosened and lost as dust. "Estimates of the amount of material lost vary between 100 and 500 tons per mile of road per year" (Road Research Laboratory, 1963). This obviously also effects the grading of the wearing course. The Road Research Laboratory has classed dust palliatives as follows:

i) Water, fresh or sea.

ii) Deliquescent and hygroscopic chemicals - e.g. calcium chloride.

iii) Other inorganic chemicals - e.g. sodium silicate.

iv) Organic, non-bituminous binders - e.g. sulphite lye.

v)/ ...

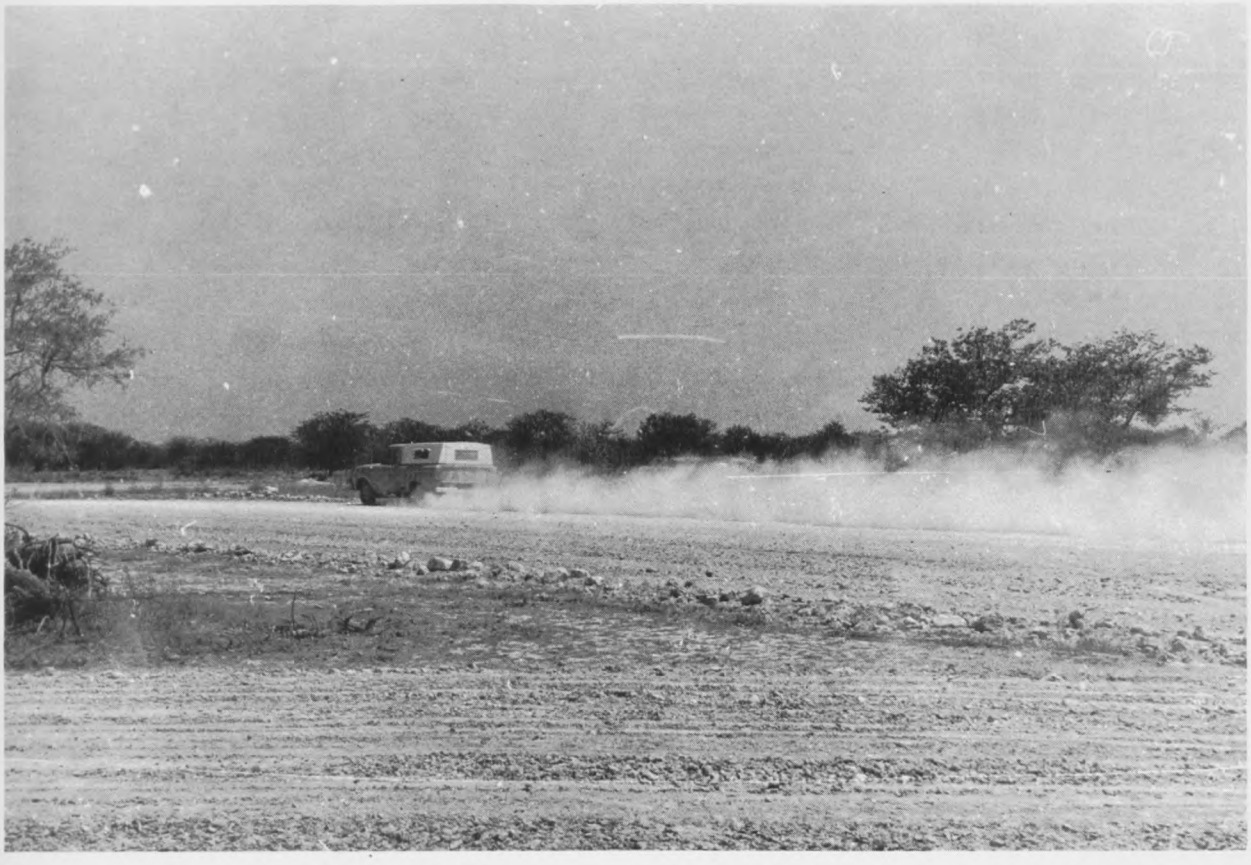


PLATE 9.5

The dust problem. Near Welwitschia.



PLATE 9.6

The dust problem. Near Welwitschia.

v) Tar and bituminous materials.

Water, though the cheapest, is of course also only normally temporary in effect. Bituminous materials, tars, road oils, etc., are probably the most effective and last the longest, but are also probably the most expensive. In fact, generally speaking, all dust palliatives, even industrial waste products like sulphite lye, become so expensive due to transport costs that their uses are restricted to the vicinity of the source of supply. Even the salt roads along the S.W.A. coast near Swakopmund are economically limited to the immediate coastal strip (as well as requiring certain humidity conditions).

The effectiveness of calcium chloride depends both on the relative humidity and the temperature, and the relation is shown in Figure 9.8 (plotted from data supplied by the Calcium Chloride Institute, 1953). From the figure it is easy to calculate for any place whether and for what parts of the year calcium chloride will be effective as a dust palliative. From the tables of mean hourly relative humidity and temperature (Weather Bureau, 1954) for each month and using Figure 9.8 the effectiveness of calcium chloride was calculated for Windhoek and Wingfield. The results are shown in Table 9.11.

The table shows that calcium chloride dust laying is possible all the year round at Wingfield as the temperature - humidity relationship lies to the right of the curve in Figure 9.8 the whole year round. During the winter and spring months in Windhoek, however, for part or the whole of the day the relation lies in the field to the left of the curve and calcium chloride dust laying is not effective at these times. In December for example, calcium chloride would not be effective from 11.00 hours to 17.00 hours. The method would be effective for only 53% of the year in Windhoek. In truth the curve in Figure 9.8 represents the lowest relative humidity and

corresponding/ ...

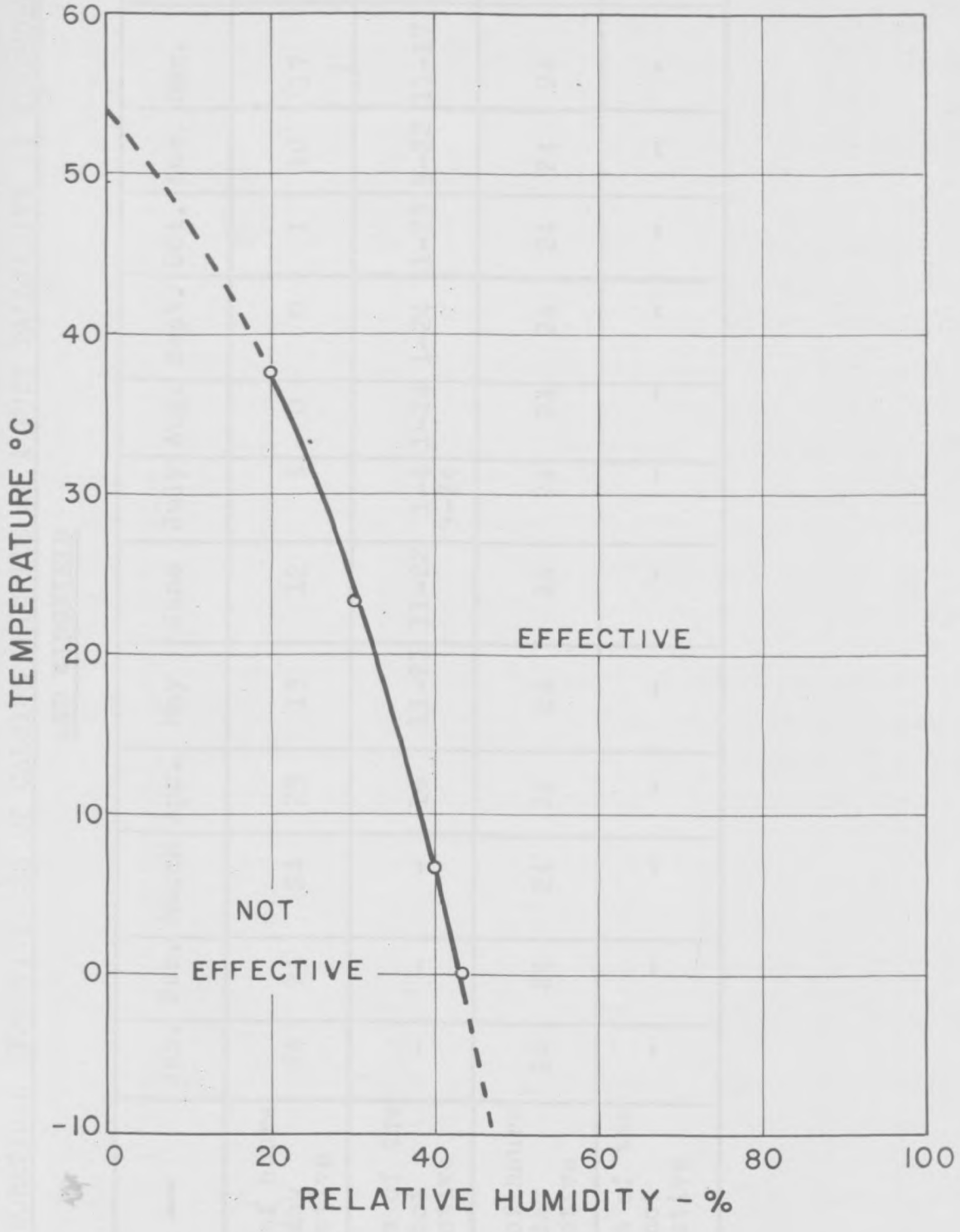


FIGURE 9·8

Effectiveness of calcium chloride as a dust palliative in relation to the relative humidity and temperature. (From manual H.M.-1 Calcium Chloride Institute)

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TABLE 9.11 THEORETICAL EFFECTIVENESS OF CALCIUM CHLORIDE AS A DUST PALLIATIVE AT WINDHOEK AND WINGFIELD

—	—	Jan.	Feb.	March	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	% of year effective
WINDHOEK	No. of hours per day effective	24	24	24	23	13	12	4	0	0	1	10	17	53
	Hours of the day not effective	-	-	-	16	11-21	11-22	1-4 9-24	1-24	1-24	1-23	9-22	11-17	-
WINGFIELD	No. of hours per day effective	24	24	24	24	24	24	24	24	24	24	24	24	100
	Hours of the day not effective	-	-	-	-	-	-	-	-	-	-	-	-	

corresponding temperature at which calcium chloride will completely dissolve in its own water of hydration. To the left of the curve the vapour pressure of the hydrate would be higher than that of the air and it would no longer be deliquescent. In order to attain equilibrium it would effloresce until its vapour pressure was equal to the partial pressure of water vapour to the atmosphere at that time.

Calcium chloride is spread at a rate varying from ½ to 2½ lb/sq yd depending on the climate and other factors (Road Research Laboratory, 1963). Additional treatment during the year after the rains may also be necessary. At a cost of 2.0 cents in 5 ton batches for about 71% $CaCl_2$ c.i.f. Durban (Imperial Chemical Industries, 1966; private comm.) the cost of treating a 21 ft wide road with 2 lbs/sq yd would be about R500 per mile exclusive of railage and application costs. Subsequent annual applications of 1 lb/sq yd would cost R250 per mile per annum.

The presence of large amounts of calcium carbonate in a calcrete road suggested that it might be cheaper to use a very dilute solution of hydrochloric acid for compaction instead of water. Hydrochloric acid reacts with calcium carbonate as follows:

$$2HCl + CaCO_3 \rightarrow CaCl_2 + CO_2 + H_2O$$

yielding 1.52 lbs of pure $CaCl_2$ for every lb of pure HCl used. (2.9 lbs 32% HCl would yield 2.0 lbs 71% $CaCl_2$).

The cost of commercial HCl (about 32%) f.o.r. Sasolburg is about 1.65 cents/lb in 5 ton batches (African Explosives and Chemical Industries, 1966; private comm.) At a cost of 1.65 cents/lb then, the cost of treating a 21 ft wide road with 2.9 lbs sq yd would be about R600 per mile exclusive of railage and application costs.

Subsequent/ ...

Subsequent annual application of 1.45 lb/sq yd would cost R300 per mile per annum.

Excluding railage and application costs then, calcium chloride dust-laying at Durban would cost about R500 per mile and hydrochloric acid dust-laying on a hypothetical calcrete road at Sasolburg would cost about R600 per mile. As far as is known, the hydrochloric acid method has not been previously tried. Due to the carbon dioxide evolved during the reaction it may also aid the calcrete to self-stabilize by temporarily increasing its solubility. A cost-benefit study of both methods should be made before either is used to a large extent in this country. Fossberg and Gregg (1964) mention the possibility of calcium chloride being manufactured in this country and suggest suitable grading envelopes and plasticity indices for calcium chloride stabilized gravels in South Africa. If calcretes are used, the specifications can presumably be relaxed as per calcrete gravel road wearing courses. It is apparent,however, that calcium chloride (and hydrochloric acid) dust laying is both expensive and of limited usefulness in the drier areas of the country where it is most needed.

The possible use of sulphite lye,(a waste product of the paper-making industry,) in South Africa has been the subject of a detailed investigation by Fossberg (1966). An addiition of 1% sulphite lye was demonstrated to be both a successful dust palliative and to reduce the amount of road maintenance required. Especially good results were obtained on the one calcrete gravel road tested at Lichtenburg, possibly due to a reaction between the alkaline carbonate and the acidic sulphite lye. The author suggests that the resulting calcium lignosulphonates would probably be less soluble than the lignosulphonic acid in the lye and that this would be the major factor involved - i.e. less leaching by rain water. Fossberg also found

that/ ...

that the grading and soil constants were very important and that best results can be expected when the annual rainfall is between 10 and 25 inches. Raw sulphite liquor can be obtained from SAICCOR at Umkomaas at a nominal loading charge and thus the only major costs are those of transport and application. In the above report Fossberg gave details of these for the experimental sections studied. He concluded that owing to transport costs, it was uneconomic to use the raw sulphite liquor at a greater radius than about 20 miles from the source of supply. Concentration of the liquor to a 50% concentrate (costing about R10.00 per ton) was suggested as being the most economical solution, but even this becomes uneconomic again at a still greater radius. It is believed that at present only the raw liquor is available in South Africa and that concentration in any form is not now envisaged. Sulphite lye powder "Dustex" is, however, available from Herzberg Chemical Corporation at 4.5 cents/lb f.o.r. Cape Town (1965, private comm.) and assuming a ratio of application of 0.6 lb/sq yd (Wargöns, undated leaflet) treatment of a 21 ft wide road would cost about R330 per mile (excluding transport and application costs). Further applications of 0.3 lb/sq yd per annum would cost about R170 per mile per annum. Sulphite lye powder produced in South Africa would cost about 1 cent/lb. (Fossberg, 1966) so that the cost would then be about R74/mile with retreatment costing about R37/mile/annum.

Fossberg (1964) has described an experiment to test a bituminous dust palliative on a calcrete road at Geysdorp in the Western Transvaal. Details of the subsequent performance of the road are unfortunately lacking.

Sodium silicate holds out good possibilities as a dust palliative and stabilizer for calcrete, but cannot compete economically with cement.

In/ ...

In general, it would appear that dust palliatives are only economic for use on long streches of road within a short distance of the source of supply. In South Africa the sulphite lye method is at present the cheapest and has been shown to work exceptionally well on a calcrete road. Even this method will probably only be economic in special cases where the distance from the source of supply is not great.

The proper selection of gravel and careful compaction with water of calcrete roads so as to form a hard surface crust or "blad" appears to be the only economic solution at present. With proper treatment, the presence of a crust can be as effective as a dust palliative.

9.9/ ...

9.9 STABILIZED CALCRETES

This section deals chiefly with specifications for lime and cement stabilization; mechanical stabilization and stabilization by means of other additives is hardly touched upon. The effects of additives on calcretes have been dealt with in Section 8.14.

9.9.1 General experience

The roads in South Africa which are known to have employed lime or cement stabilized calcrete as base or subbase are shown in Table 9.3. To date, no (or possibly one- Marais, 1965) failures have been reported which can be ascribed to the failure of the cement or lime stabilized material. The difficulties experienced with some roads in the Western Transvaal have been ascribed either to faulty construction or the failure of a layer other than the calcrete layer (largely due to drainage), while those on the Operet-Ondangua road in South West Africa are due to salt action, though some of the salt was in the calcrete. All other things being equal, the high pH environment in any calcrete is favourable to the preservation of silicate and aluminate reaction products and the destruction of clay minerals. Lime is also less likely to be leached out of a base in calcrete country than it is in more humid areas.

The fact that calcretes do not obey the usual rule of thumb that lime should be added to reduce plasticity and cement to improve strength has been noted in Section 8.14.

South African experience with bitumen stabilized calcrete is minimal. An improvement in most properties and in road performance was reported by Gregg et al (1967) when calcrete fines were used as a filler in experimental bitumen-sand bases in Ovamboland.

Mechanical/ ...

Mechanical stabilization of calcrete is widely practised in South West Africa. As with other materials, plastic fines may sometimes adhere to the aggregate, making it very difficult to reduce the plasticity to usually accepted limits by screening.

Regarding overseas experience, Mr. T. Cole (1966, private comm.) has stated that the cement and bitumen stabilized caliches of the New Mexico State Highway Commission have been very successful, while they have not had occasion to lime stabilize any caliches. Mr. W.G. O'Harra (1966, private comm.) has stated that some of the cement stabilized caliches of the Arizona Highway Department have been satisfactory, while they do not believe that caliche can be bitumen stabilized owing to its absorptive nature. They have also not attempted to stabilize any caliches with lime. It is believed that cement, lime and bitumen stabilized caliches have been employed in Texas.

9.9.2 Placing and testing of chemically stabilized materials

This subject has been discussed for example by Maclean (1955), Woods (1960), the Highway Research Board (1961, 1961a), Fossberg and Gregg (1963), Gregg (1965) and Metcalf (1966).

Points of particular importance are:

1. Delay in compaction after mixing is much more critical with cement stabilized soils than with lime stabilized soils.

2. The CBR method, widely used in this country for design and testing purposes, was intended for flexible bases, while stabilized materials lie somewhere between flexible and rigid pavements. Unconfined compressive testing may be more realistic.

3./ ...

3. The use of densities as a means of quality control is plainly unreliable.

4. Thorough mixing is of great importance.

5. The use of strongly stabilized bases can lead to failure by slab action, i.e. in tension. The importance of adequate tensile strength is stressed, as well as the necessity of proper support for rigid and semi-rigid bases.

6. The fatigue effect of repeated loads may reduce the effective strength to half the unconfined compressive strength.

7. The rate of strength development is greatly affected by temperature. For example, cycling between 65°C during the day and 25°C during the night increased the 7 day strength by 80% over that cured at a constant 25°C.

8. Field mixed material should yield a strength of at least 70% of the laboratory mixed material. It may be desirable to specify a maximum, in addition to a minimum strength.

9. A stabilized pavement should be considered and designed as completely cracked and should be provided with construction joints if to be of very high strength.

The delay between mixing and compaction will be more important for lime stabilized calcretes than for other lime stabilized materials because of the frequent rapidity of the reaction. The reaction rate with lime (Section 8.14.5) of calcretes containing diatoms is at least equal to that of cement-soil mixes. For purposes of delay between adding water, mixing and compaction it is suggested that, when lime is added to calcretes for the purpose of increasing their early strength, they be treated as cement stabilized materials and compacted as

soon/ ...

soon as possible after watering and mixing. A United States specification is that soil-cement base courses shall be compacted to the specified density within two hours of watering and mixing (THD, 1962; FAA, 1964; and AASHO, 1968) and within 6 hours of placing the cement (THD and FAA). AASHO also specifies a maximum delay between starting mixing and starting compaction (60 minutes) and the maximum time that uncompacted mixed material may be left undisturbed (30 minutes).

Further remarks on the testing of lime and cement stabilized calcretes will be found in Sections 7.11 and 7.12.

9.9.3 Specifications for lime and cement stabilized base courses

Specifications in general have been discussed for example by Maclean (1955) and Fossberg and Gregg (1963) and Figure 9.9 shows the gradings usually required according to Fossberg (1963) and Fossberg and Gregg (1963). It will be seen that somewhat finer material is usually allowed. The upper limit for stabilized gravels shown is identical to that used by the Transvaal Roads Department. The S.W.A. Roads Department do not stabilize materials for base which they would not have used unstabilized for subbase. On the Operet-Andoni contract they required a maximum size of 2 inches (subbase 2½") and a grading modulus of not less than 1.6 (subbase 1.5). The D.O.T. requires all natural base standards except grading, which may be as for natural base or their 1" subbase grading.

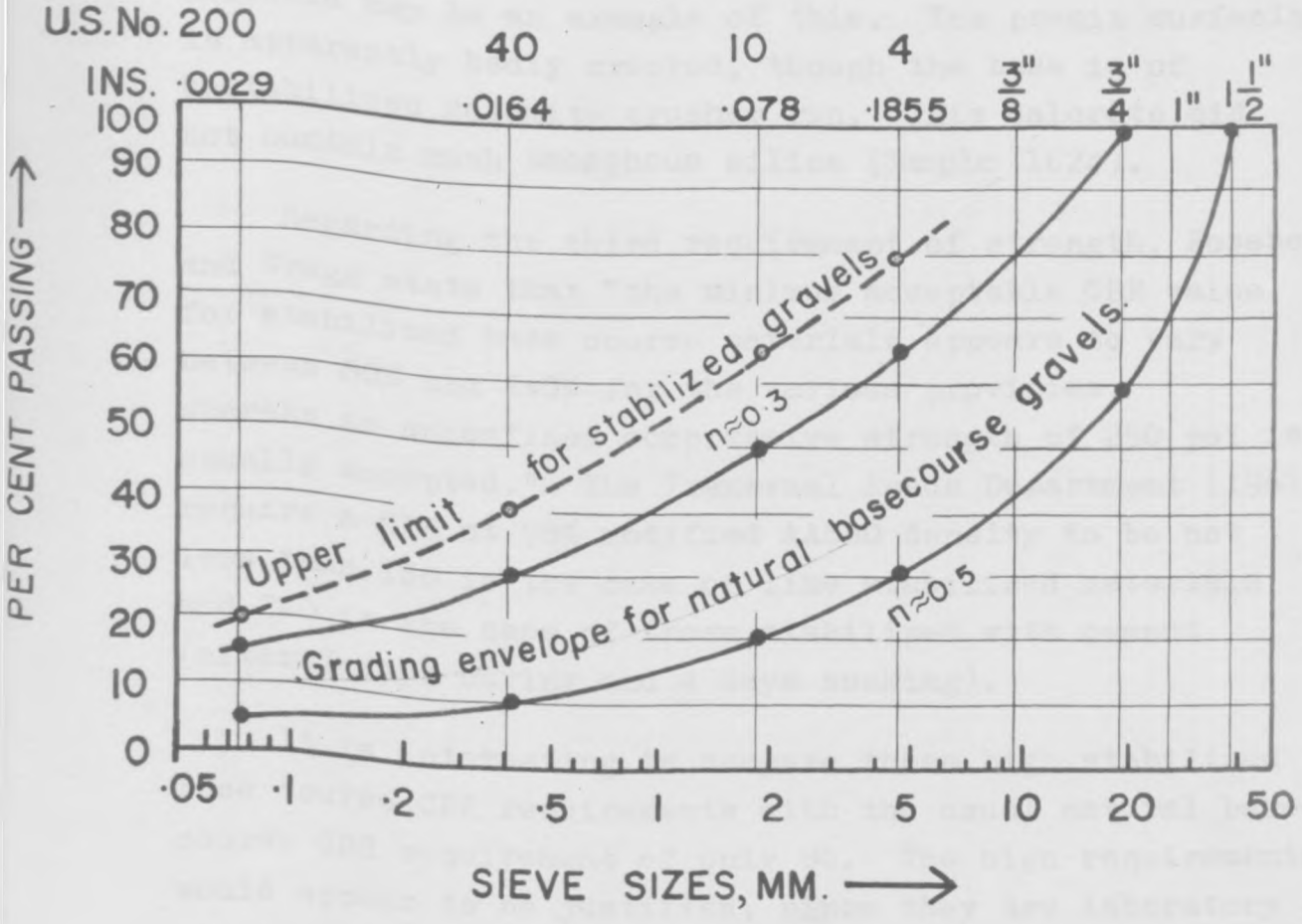
The second requirement is usually that of plasticity, it being usually required that the stabilized material shall at least meet with the authority's usual base course specifications (usually LL $\leq$ 25 & PI $\leq$ 6). Some authorities specify a slightly lower PI than the normal base requirement, for example S.W.A. (4 and 8

respectively/ ...

FULLER'S FORMULA: $p = \left(\frac{d}{D}\right)^n$

WHERE p = PERCENTAGE PASSING SIEVE OF SIZE d

D = LARGEST PARTICLE SIZE



SILT FRACTION	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE
	SAND-FRACTION			GRAVEL FRACTION.		

FIGURE 9·9

Typical grading envelopes for natural and stabilized basecourse materials (After Fossberg and Gregg, 1963)

respectively. This is presumably to ensure that the PI will not rise again to above the normal limit as the stabilizer becomes depleted (perhaps due to aggregate degradation). Soil constants requirements before stabilization are not often specified, but clayey soils with liquid limits above 50 and PI's above 25 will be very difficult to handle (HRB, 1961). Gravelly soils with such high constants are presumably still suitable, though shrinkage and cracking might possibly be aggravated. The lime stabilized calcrete subbase at the Grootfontein airfield may be an example of this. The premix surfacing is apparently badly cracked, though the base is of unstabilized dolomite crusher run. This calcrete did not contain much amorphous silica (Sample 1624).

Regarding the third requirement of strength, Fossberg and Gregg state that "the minimum acceptable CBR value for stabilized base course materials appears to vary between 80% and 240% for the various provinces, whereas an unconfined compressive strength of 250 psi is usually accepted." The Transvaal Roads Department (1961) require a CBR at 98% modified AASHO density to be not less than 160 in the case of lime stabilized materials and 240 in the case of those stabilized with cement (after 7 days curing and 4 days soaking).

It is interesting to compare these high stabilized base course CBR requirements with the usual natural base course CBR requirement of only 80. The high requirements would appear to be justified, since they are laboratory strengths on semi-rigid materials. Maclean (1955) has pointed out that the field strengths (mixed in place method) can be only about two thirds of the laboratory strengths, while Fossberg and Gregg (1963) found that the fatigue strength of a cement stabilized windblown sand was only about 60% of its unconfined compressive strength as determined by the usual static method. Non-cemented

materials/ ...

materials of low PI will plainly not be subject to a loss of strength due to fatigue, though they could undergo aggregate degradation. If these figures can be generally applied, it would mean that the effective strength actually obtained in the field under moving traffic conditions is only 0.4 of the laboratory strength of laboratory mixed materials. To obtain the equivalent of a CBR of 80 then, the laboratory CBR of stabilized material should be 200. Such a high strength introduces the problem of flexural failure. The elastic modulus (E) increases with the unconfined compressive strength and CBR (E = 110 CBR according to Heukelom's formula) and it is plain that this must result in higher stress concentration in the stabilized layer. Thompson (1966) has shown that the elastic modulus of five soils increased with unconfined compressive strength in a regular manner and that the modulus of elasticity of the lime stabilized soil was 3 to 25 times that of the unstabilized soil. He concluded therefore that the flexural stress was probably the controlling factor in design.

In the design of roads stabilized by cementation then, attention must be given to the flexural strength of the base, preferably under dynamic loading conditions. The flexural strength is proportional to the compressive strength, but the relation between the latter and the CBR depends greatly on the soil type. A material with a CBR of 200 may not even have an unconfined compressive strength of 100 psi or it may be as high as 400 psi. (See for example Maclean, 1955). It is plain that the CBR design design method should not really be used for strongly cemented soils (above a CBR of about 150?)

British practice has been to specify an unconfined compressive strength of at least 250 psi (equivalent to a CBR of anything from 100 to 500), but recent practice

is/ ...

is to use higher strengths (400 psi), while several states in the U.S.A. specify 750 psi (Fossberg and Gregg, 1963). The latest S.A. Department of Transport specifications for major roads require a 7 day strength of 1,000 psi in an AASHO mould for base and 150 psi for cement stabilized subbase. A CBR of not less than 50 at 95% modified AASHO compaction is required for lime stabilized subbase.

The flexural strength of a stabilized base course can be increased in the following ways:

1. Increase of compressive strength of base course.

2. Proper support by the subbase - i.e. an increase in stiffness (elastic modulus).

3. Increase of thickness of the base course.

4. Proper choice of soil to be stabilized.

By reducing the difference between the elastic modulus of the stabilized base and the unstabilized subbase the flexural stress in the base is reduced. This concept appears to be of great importance - it may be unwise for example to use even a rigid or semi-rigid subbase under an unstabilized crusher run base even if they had the same CBR or even if the CBR of the subbase was less than that of the base. Since the soil is to be stabilized by cementation and will probably fail in flexure, the fourth requirement of the tensile strength of the aggregate is most important. As a strongly cemented base is no longer a flexible base, the abrasion of the aggregate due to flexure must be almost eliminated. In a completely rigid base the aggregate strength can probably be reduced to that desired of the overall layer. It is well known that reasonable concrete can be made with quite weak aggregate and Figures 8.29 - 8.32 show that the ultimate strength of the concrete is controlled by

the/ ...

the strength of the aggregate. Even Sample 1724, with a dry 10% F.A.C.T. value of only 7 tons and a soaked value of 5 tons, yielded a 7 day cube compressive strength of 2,000 psi and a transverse strength of about 200 psi Durability of the concrete was also adequate. Since the tensile strength of a rock is proportional to its crushing strength, which in turn is related to its aggregate crushing strength, the latter can be specified and is easier to determine. To summarise, it is suggested that the aggregate strength required in a stabilized base is a simple function of the rigidity and flexural strength desired of the layer, decreasing with increasing rigidity to a limiting value equal to the strength of the rigid base.

A fifth requirement, that of durability of the compacted, stabilized base is less often specified and does not yet appear to have been applied in this country. Durability testing (wetting and drying and/or freezing and thawing) is, however, regarded as the 'proof' test by which all other tests are judged. It should therefore be applied whenever unknown materials are to be employed to supplement the other short cut tests such as CBR or unconfined compressive strength. Strength loss due to fatigue and shrinkage cracking are not the only ways in which stabilized bases can lose their strength. Some materials (such as some calcretes) contain sufficient soluble salts or gypsum to either inhibit the cementitious reaction, to react with the reaction products to form deleterious minerals, or simply to disintegrate the stabilization bonding by alternate solution and crystallisation and/or hydration and dehydration. This has been dealt with in the previous two chapters and in Appendix D. Wetting and drying tests are widely used in the United States as a general durability test and have also been found useful in the case of a salty (with NaCl) cement stabilized base in Australia by Smith (1962). Regarding the limits applied to this test, the U.S. Federal Aviation

Agency/ ...

Agency (1964), Item p. 301) for example specify that the weight losses when subjected to the AASHO T 135 wet-dry test shall not be more than 14% for granular soils, 10% for the more plastic granular and silty soils and 7% for clay soils. The Agency omits the AASHO T 136 freeze-thaw test in areas not subject to frost penetration, but the same limits are applied to both tests. The importance of soluble salts should not be underestimated, and until more experience is gained it is recommended that both conductivity and wetting and drying tests should be carried out on stabilized calcretes, particularly those with relatively high conductivities.

A comprehensive treatment of all types of stabilized bases (including specifications) has been given by Woods (1960).

9.9.5 Specifications for calcretes

In view of the findings regarding the requirements of natural calcrete bases and the foregoing discussion, the following specifications for lime and cement stabilized calcrete bases appear reasonable, though no experience is available on the long term performance of calcrete bases to such a specification (Table 9.12).

The specifications are not unusual (except for allowing the high soil constants in Table 9.7) and call for little comment. Grading and soil constants were derived from Table 9.7 and Figure 9.9. Conductivity limits are derived in Appendix D and the gypsum limits in Netterberg (1969). Aggregate strength is merely a guess and assumes that aggregate degradation during compaction (particularly the finishing of the final base) can be controlled. If the aggregate is very weak and would yield plastic fines on degradation, sufficient lime or cement should be used to ensure that this extra source of clay can be dealt with during service. This can be

TABLE/ ...

TABLE 9.12 SUGGESTED INTERIM SPECIFICATIONS FOR LIME OR CEMENT STABILIZED CALCRETE BASES AND SUBBASES

PROPERTY	REQUIREMENT
Max. particle size	2" or less
Min. apparent grading modulus	< 1.5?
Max. apparent % passing 0.0164"	> 55?
Max. soil constants	Not to exceed the appropriate natural base requirement (Table 9.7) after stabilization, except that the liquid limit requirement can probably be waived for pavements not subject to frost heave.
Max. soluble salts	Conductivity limits as in Table 9.7 before stabilization, plus gypsum and anhydrite limited to 0.25% SO_3 if the apparent PI or the minus two micron content of the material 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 9.7?
Stabilized strength, CBR swell and relative compaction	As required (normal requirements of the authority concerned).
Durability	Compacted specimens of cement stabilized calcrete and lime stabilized calcrete yielding high early strengths shall not suffer a weight loss of more than 14% for granular soils, 10% for the more plastic granular and silty soils and 7% for clay soils after 12 cycles of the AASHO T135 wet-dry test? Max. volume change after 12 cycles 2%?
Delay between mixing and compaction	As for cement-stabilized base in the case of cement-stabilized calcretes and lime stabilized calcretes yielding high early strengths. As for lime stabilized base for lime stabilized calcretes not yielding high early strengths.

done either by arbitrarily reducing the PI and LS requirements below that required for natural bases in Table 9.7 or, possibly better, by determining the amount of additive required on the whole sample after it has been tumbled in the Los Angeles machine for 500 revolutions, preferably without the steel shot. The duability requirement is that of the FAA (1964) and is believed to be a common United States specification. It can probably be dispensed with once the behaviour of the calcretes in a certain area are well known.

Bitumen or tar stabilized calcretes

No information has been obtained on specifications for bitumen or tar stabilized calcretes, though it should be available from the New Mexico and Texas State Highway Departments. A limitation on water absorption (porosity) will probably be desirable.

9.9.6 Effect of lime type

The effect of calcium and magnesium in limes has been studied for example by Fossberg (1965), who found no advantage between a high calcium lime, a magnesian lime and a dolomitic lime in reducing the plasticity index and linear shrinkage. The magnesian lime was the most effective in increasing the unconfined compression strength. None of the five soils tested was a calcrete. The clay minerals present included calcium montmorillonite, kaolinite, chlorite (?) and ferruginous soil chlorite. The Cape Roads Department (1965), however, in a study of two calcretes from the Kimberley - Griquatown area, found that a high calcium lime was the most effective of the four limes tried in reducing the plasticity. The one dolomitic lime was the least effective. It is a pity that the clay minerals present in the two calcretes were not determined. In the absence of this information the above results should not be extrapolated to calcretes in general.

9.10/ ...

9.10 SELF-STABILIZATION

Tests for self-stabilization have been considered in Section 7.10, and in Section 8.13 it was shown that the CBR may be doubled or even trebled by laboratory wetting and dying cycles. The possible causes of self-stabilization were considered in Section 8.13.3. How to achieve self stabilization in practice will depend on the operative mechanism. All six mechanisms considered will probably require at least one wetting and drying cycle. The last two mechanisms (reaction between $CaCO_3$ and clay minerals or opaline silica) are probably long term ones and little can probably be done to accellerate them beyond merely placing a blacktop surfacing. If further laboratory work shows that the CBR is also increased using 60° - 70° C drying cycles, it is suggested that the following procedure would take advantage of the other mechanisms:

1) Ascertain whether the material is one that improves in soaked strength after one wetting and drying cycle. Use material which has not been dried below its natural moisture content. Compact at optimum moisture content, dry at 60 - 70° C and determine the 4 day soaked CBR. Both 0.1" and 0.2" CBR should be considered. If this is substantially greater than the ordinary 4 day soaked CBR the material will benefit from one drying cycle during construction. The petrifaction degree test may prove to be just as reliable, but it is suggested that in the first instance the CBR (or both) be used.

2) During excavation and construction, the material should be kept at or above its natural moisture content until after compaction, when it should be allowed to thoroughly dry out.

3) If no improvement in laboratory strength is evident after one cycle, determine the soaked CBR after at least 5 cycles, each consisting of one day soaking and one day

drying/ ...

drying at 105°c, (or possibly at a lower temperature if further work shows this to also be indicative). If a substantial improvement is recorded, the material is probably one that will benefit from several wetting and drying cycles during construction. Most calcretes will probably fall into this latter category. The wetting and drying cycles during construction should be of sufficient duration to thoroughly wet and dry the material. It may possibly be advantageous to continue to roll the material while it is drying out, particularly if it is not very clayey (drying has itself been used for compaction in the past). The number of cycles required under field temperature conditions is unknown, but might well be more than that required in the laboratory at 105° C.

4) If the laboratory CBR decreases with cycling the material should possibly not be used as base course.

5) Any field improvement gained should be assessed by some means other than density measurements. A curvature meter, Benkelman beam, in situ CBR, plate bearing or hand operated dynamic cone penetrometer should be suitable, though the latter device may not record an improvement in the upper inch of base.

While it must be admitted that this section is largely speculative, the doubling and trebling of the CBR by means of laboratory wetting and drying cycles is sufficiently substantial to recommend that a small road experiment should be carried out using material shown to self-stabilize in the laboratory. Examples of such materials are shown in Table 8.13. The locations of these particular samples are described in Appendix B.

Increasing the solubility of carbonate by the addition of salt, hydrochloric acid, organic substances, or carbon dioxide to the compaction water will probably

hasten/ ...

hasten the self-stabilization, though the first two substances should not be used if the road is to be surfaced.

9.11 REASONS FOR UNUSUAL PERFORMANCE OF CALCRETE ROADS

The possible reasons for the unusually good performance of calcrete roads in general can be summarised as follows:

1) Calcretes only occur in relatively dry climates. The rainfall is usually less than 22 inches, the potential free water surface evaporation more than 75 inches and the temperature normally well above freezing throughout the year. Frost heave is non-existent. The water table is generally deep (more than 40 ft) and subgrades are generally reasonably favourable (the soils being somewhat skeletal - Chapter 3).

There appears to be considerable evidence that specifications for surfaced roads in more arid areas can be less severe than in more humid areas. This principle will probably be incorporated in the new Department of Transport specifications in the form of a regional design factor. As usable calcretes only occur in areas receiving less than about 22 inches of rainfall (equivalent to a Thornthwaite Moisture Index of -20) the factor will probably be 0.5 or less in calcrete areas. Climate is taken into account in road design by 35 states of the U.S.A. (Yoder, 1959). Wyoming for example varies the cover thickness according to rainfall, water table, frost action and traffic. If the first three of these are favourable, it could add up to a halving of the cover required. The Kansas saturation coefficient in calcrete areas would be 0.7 or less.

There is further evidence that not only may cover thickness be reduced, but that the soil constants of possibly all materials in the arid and possibly semiarid environment can be increased. For example, the National

Association/ ...

Association of Australian State Road Officials (1968) states that in arid areas (receiving less than 15 inches of rainfall per annum) the maximum PI of base and surface courses may be increased from 6 to 10 and 9 to 15 respectively, the LS from 2 to 4 and 3 to 6 respectively, the PI X % minus 0.42 mm from 180 to 200 for base courses, and the CBR relaxed from the desirable 80 to as low as 30 for base courses (NAASRA, 1968). The data from frost affected areas (eg. the WASHO road test) is not valid in other areas (Yoder, 1959). A similar conclusion was reached by Jewell (1968), working in Australia. He stated that the classification systems for base materials evolved from those for subgrade soils and that "The present specification limits for pavement materials appear to be those corresponding to classification groups A-1 and A-3, which were quoted as not having a tendency to heave under frost action. That feature, which is so desirable for North American conditions, seems to have led to the subsequent adoption of those limits in the AASHO specifications for base and subbase materials, from which the Australian specifications were derived". Jewell's study also showed that bases with PI's of 17 and 18 are giving satisfactory service under favourable conditions in the Australian environment and suggested a relaxation of the Australian limits. The South African soil constant requirements were in all probability also derived from the AASHO limits. The usually accepted Australian specifications have also recently been criticised by Smith (1969) as not having been developed for Australian conditions. Smith recommended that a whole new set of criteria and specifications should be found to suit Australian conditions, but that in the interim the Texas specifications appeared to be the most suitable for Australian conditions. These specifications permit for example PI's of 10 to 15, depending on the class of base. It is submitted that the Texas specifications are very probably also suitable for the semiarid

and/ ...

and arid zone of Southern Africa.

2) Recent work by the Texas Highway Department (McDowell, 1966) even casts doubt upon the reliability of the liquid limit and PI as indicators of the performance of bases under Texan conditions. After an investigation of 42 Texan base course samples (including several caliches) McDowell (1966) concluded "That the data for the roads sampled do not show any correlation between the soil constants (L.L. and P.I.) and pavement performance..... That evidence obtained in this report does not substantiate the idea that Texas should use a maximum L.L. of 25 and P.I. of 6 in specifications for base materials" and recommended "That AASHO consider revision of M-147 "Standard Specification for Materials for Soil-Aggregate Subbase, Base and surface course so as to permit the use of materials having higher liquid limits and plasticity indexes when the wet method of preparation is used. Maximum limits of 35 for L.L. and 12 for P.I. are suggested for top 6 to 8 inches of base and 40 and 20 respectively for subbases". McDowell did not specifically give climate as a reason for allowing higher soil constants in Texas and rather seemed to imply that this was because of the Texas wet soil preparation method. In mentioning McDowell's work Jewell (1968) implied that climate was the reason, and in all probability it is a combination of both. Both McDowell and Jewell considered that more weight should be attached to strength tests (such as the Texas Triaxial) rather than merely relying chiefly on soil constants and grading to provide an estimate of compacted stability as is done in the AASHO and most Australian specifications. It is of interest to note that Hveem (1944; in Wooltorton, 1954) has pointed out that many of the most satisfactory pavement mixtures possess considerably less than maximum density. McDowell also recommended that traffic weight and volume and surfacing thickness should also be taken into account.

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The reason for poorer quality bases performing better in drier climates than in wetter climates is presumably mainly simply due to there being less moisture around (thcugh even in dry areas road bases can possess fairly high moisture contents). Another reason is that soils from dry areas tend to possess cementation or chemical structure and aggregated clays (Wooltorton, 1954).

3) The actual liquid limit and PI tests may possibly be somewhat unreliable on calcretes (Sections 7.3 and 7.5). It is suggested that the Department of Transport (1958) liquid limit procedure will tend to yield results on the high side.

4) Mechanical analyses are probably inaccurate and representation by weight may be misleading due to an apparent variation of bulk particle specific gravity with size fraction, flocculation and probably wrong specific gravities used in sedimentation analyses, variable particle strength, and the presence of needle-like palygorskite and sepiolite clay particles instead of the assumed spherical particles (Section 7.7, 8.7.2 and 8.8.3).

5) The usual soil constant specifications applied are plasticity index and liquid limit, (though bar linear shrinkage is often specified in South Africa in addition) whereas calcretes are prone to yield higher PI s and liquid limits for the same probably more reliable bar linear shrinkage than many other materials (Sections 8.4.7, 8.4.9 and 8.4.10). These high liquid limits and PI s are probably largely caused by particle porosity and other causes and do not necessarily indicate the presence of deleterious components under South African conditions.

6) The unusually low and even negative shrinkage indices (Section 8.4.2) and unusually low linear shrinkages in comparison with their PI s (Section 8.4.10) indicate

little/ ...

little or no shrinkage below the plastic limit. As the contracting soil approaches its maximum shear resistance at the shrinkage limit, the smaller the shrinkage index the more stable the compacted soil, a particularly important feature under desiccated conditions (Wooltorton, 1954). The most important object in road design is volume change control, "since plasticity is but another aspect of volume change control" (Wooltorton, 1954; and Sections 7.5, 7.6 and 8.4.10). Volume change obviously means deformation, change in density and change in strength.

7) The unusually high toughness indices indicate that many calcretes are unusually strong near the plastic limit.

8) Calcretes appear to be unusually porous (Section 8.15) and can therefore tolerate more expansive fines (a higher LS, PI and LS or PI X percent passing 40 mesh) than less porous materials before the compacted mass undergoes any volume change. Much interesting discussion of the porosity control of PI X % minus 40 mesh is contained in Wooltorton (1947 and 1954).

9) The high carbonate content of calcretes probably causes favourable cementation (Section 8.13.2), and aggregation and flocculation (Section 7.7) of all clay minerals except palygorskite and sepiolite. Other cementation mechanisms may also be operative.

10) Palygorskite and sepiolite are common clay minerals in calcretes (Section 3.4.5). Palygorskite has the highest critical moisture content of all the common clays (Section 7.3), in other words it is stronger than any other clay at the same moisture content. The critical moisture content is about the plastic limit for cohesive soils and palygorskite posessess the highest plastic limit of all the crystalline clays (Section 8.4). The

presence/ ...

presence of palygorskite and/or sepiolite is thus quite possibly the reason for the unusually high toughness indices of calcretes. Palygorskite and sepiolite possess a number of other unusual properties which are reflected to some extent in the calcretes containing them. Palygorskite has for example a PI equal to montmorillonite, but is non-expansive.

11) The Federal Aviation Agency (Section 9.4.1) considers that the LAA and the sulphate tests are unreliable on United States caliches and that some caliches which have failed the tests have made satisfactory bases.

12) Calcretes may contain small quantities of diatoms, which influence their Atterberg limits and cause them to exhibit unusually high optimum moisture contents and to react with lime to yield exceptionally high and rapid strengths.

13) Calcretes usually occur in relatively undeveloped areas where traffic is light, though the proportion of heavy vehicles may be high.

14) Lastly, the specifications of the U.S.A. (such as the AASHO) and Britain are based on the average or leached podsolic soils of those countries containing illite as the cominant clay mineral. They are conservative and it has been shown by a number of observers that mixtures not complying with these specifications have given satisfactory performance (Wooltorton, 1954). In connection with their specification for aggregate and soil aggregate subbase, base and surface courses, AASHO M147-65 states "The requirements are intended to cover only materials having normal or average specific gravity, absorption and gradation characteristics. Where other materials are to be used, appropriate limits to their

use/ ...

use must be specified". This point is also made by Hewes and Oglesby (1954) who mentioned that appropriate limits must be set for certain local base materials such as caliche, gypsum, cinders and limerock. In the same connection Wooltorton (1954) went on to mention five other states of the U.S.A. which allowed higher liquid limits than the AASHO specifications (North Dakota: 35 and PI of 15, South Dakota: 35, Ohio: 35, Minesota: 35 and Idaho: 40.

9.12 CONCRETE AGGREGATE

The only known use of calcrete as coarse aggregate in concrete in South Africa took place in 1964 in the building of a bridge over the Omuramba Ovambo on Trunk Route 1/10, Operet-Andoni. It is believed that calcrete was in fact used unintentionally, but in 1968 the bridge showed no signs of failure of any kind.

The work by NBRI on the four calcretes already described (Section 8.12) has shown that adequate strengths can be developed when calcrete aggregates are used. Calcrete of a Mohs hardness of 3.5 (10% F.A.C.T. of 11 tons) yielded concrete with a compressive strength of 5,000 psi and a transverse strength of 700 psi (Figures 8.29-8.32), and the harder calcretes yielded higher strengths. Their durability too, was generally satisfactory. Only Sample C1 (1724) was somewhat unsatisfactory in this respect.

Calcretes will often be potentially alkali-silica reactive and they may be potentially alkali-carbonate reactive. Where they are to be used as aggregate with a cement having a total alkali content of 0.6% (expressed as Na_2O), their reactivity should be checked by a method such as ASTM C227. ASTM 289 is probably unreliable on calcretes.

One/ ...

One argument that can probably be levelled against the use of crushed calcretes as coarse concrete aggregate is the question of water absorption. Most calcretes would fail the SABS 718-1962 limit of 1% . Many would meet the minimum 10% F.A.C.T. requirement of 12 tons for concrete subject to surface abrasion and 8 tons for other concrete. The water absorption method given in SABS 718 probably yields lower results than the ASTM C127 used in this study.

In general, there would appear to be no argument against the use of calcrete having a Mohs' hardness of more than 3.0 for coarse aggregate for concrete for structural or pavement (perhaps more than 3.5) work. Depending on the strengths required it may be necessary to use strong mixes with the weaker calcretes. A big disadvantage of calcrete deposits is their often small size and great variability. It will normally only be economic to work a calcrete deposit for concrete aggregate when nothing else is available. The use of calcrete as concrete aggregate offers considerable savings in the construction costs of buildings in Ovamboland for example. At the present time all concrete aggregate is carted by road a distance of about 120 miles at considerable cost. There are possibly adequate quantities of suitable hardpan calcrete and hard nodular calcrete in the Ondangua area for use as coarse concrete aggregate. It may be advisable to check the soluble salt contents of some calcrete aggregates.

As far as limestone concrete aggregates in general are concerned, Valton (1947) has shown that there is no valid objection to their use in any form of concrete. Limestone aggregates are particularly useful for certain applications.

9.13/ ...

9.13 CONCLUSIONS

1) Calcrete is the second most widely used class of road material in the Republic. In South West Africa and in Southern Africa in general it is probably the most important road material of all.

2) Calcretes are used wherever they are found (normal annual rainfall less than about 550 mm or N>5 in most cases).

3) Calcretes have already been successfully used for all classes of road material, including unstabilized base courses under bitumen, under light and medium traffic. They have not yet been used as bituminous surfacing aggregate in Southern Africa.

4) If the 20 cycle weathering test of the Department of Transport (1958) is accepted as excessively severe for the climatic conditions prevalent in Southern Africa, the very best calcretes can meet all the requirements for surface dressing aggregates. The better ones can meet all requirements for use in premixed surfacings and penetration macadams.

   Calcretes have been successfully used as bituminous surfacing aggregate in South Australia and New Mexico.

5) It is shown that calcretes which would fail the normal South African base course specifications of grading and soil constants have been successfully used as natural base and subbase courses in five roads in the Cape Province (on the Cape Flats and at Port Elizabeth) under surfacings varying from surface treatments to 2 inches of penetration macadam and traffic counts varying from about 200 to 3,000 vpd.

6)/ .

6) It would appear that plasticity and grading specifications for calcrete base courses can be relaxed somewhat. Up to 55% passing 0.42 mm and PIs varying from 15 to 8 (linear shrinkages from 6 to 3) appear satisfactory depending on the traffic count. No relaxation of CBR appears justified at this stage, though a 0.2" value of 80 appears satisfactory under light traffic. These soil constants and grading requirements are similar to those successfully applied to caliches in Texas.

7) Less evidence is available for a relaxation of calcrete subbase specifications, but materials with PIs of up to 15 have proved satisfactory in the Port Elizabeth area and values of up to 19 have been successfully used on one road in the Western Transvaal.

8) Calcretes usually make fairly good gravel road wearing coarses which are not prone to corrugate. Specifications based on test results on samples of known performance are suggested. A wide range of soil constants appears permissible depending on the grading of the material. Aggregate strength becomes important when the percent passing 0.42 mm is less than about 50% and a simple field test is used for this.

9) The most economical way of reducing the dust problem on calcrete roads is their careful compaction with water to form a hard surface crust or "blad". A good "blad" is probably as effective as most dust palliatives.

10) The dust problem may also be alleviated by the use of various additives such as sulphite lye, calcium chloride, hydrochloric acid and various other products.

Sulphite/ ...

Sulphite lye has been shown to be remarkably effective on a calcrete road and is the cheapest form of additive available in South Africa, but all additives are expensive in practice.

11) It is suggested that the strength of calcrete road layers can probably be materially increased by subjecting them to a process of wetting and drying cycles.

12) Both lime and cement stabilization of calcretes have been successful in many cases, while bituminous stabilization has also been successfully applied overseas.

Both lime and cement should be tried when a strength increase is desirable since some calcretes yield higher and more rapid strength increases when stabilized with lime than cement.

13) Little information is available on specifications for lime and cement stabilized calcrete bases. Successful bases have been made to the usual South African rural road type of specification.

14) Soluble salts can be a problem in all types of calcretes bases and the specifications suggested incorporate limits on salt content by a simple conductivity test and the durability testing of stabilized calcretes.

15) There are a number of reasons for substandard calcretes yielding unusually good service as road materials, the most important possibly being the climate under which they occur, the probable unreliability of soil constants and mechanical analyses, their low swells, low shrinkages, high porosities and high CBRs in comparison with their indicator results.

16)/ ...

16) Tests such as the soil constants and mechanical analyses are indicator tests and should be regarded as such. More emphasis should be placed on strength testing such as CBR or Texas Triaxial.

17) Lime-reactive calcretes should be treated as cement stabilized materials with respect to the delay allowed between the addition of water and compaction if maximum strengths are to be attained.

18) The use of calcrete and silcrete as coarse concrete aggregate can result in considerable savings in Ovamboland and in the general area covered by the Kalahari Beds.

9.14 SCOPE FOR FURTHER INVESTIGATION

1) A list of specific roads which have used calcrete in their upper layers will be of value in any future large investigation of calcrete roads. Table 9.3. is a start in this direction.

2) Further work on the calcrete-based roads described would be of value. Further information on surfacing thicknesses and the amount of deformation is desirable. The carbonate content of the soil fines and the presence of palygorskite may be important. The assesment of in situ strength is desirable.

3) A survey of existing blacktop calcrete roads should be carried out in order to determine what materials have been successfully used. Test holes should be dug at both failed and unfailed sites and the base and subbase tested for CBR, aggregate strength (dry and soaked), Atterberg limits, linear shrinkage, mechanical analysis and conductivity. The field density should also be compared with the laboratory density and any tests carried out for self-stabilization which may be available (see Section 7.10). Some

form/ ...

form of in situ strength test is highly desirable. Assessment of the performance of the materials should then be made, taking into account the surfacing and subgrade, traffic, climate, drainage, and any failures which may have occurred. Only in such a manner will the best calcrete specifications be arrived at. A similar survey of gravel roads could also be usefully carried out.

4) Test strips should be laid using the calcrete specifications suggested.

5) The investigation of the possibility of failure of calcareous fills and earth dams due to seepage solution may be worth while.

6) Information on aggregate strength requirements for all types of calcrete bases is lacking.

7) Full scale field trials as well as further laboratory experiments should be carried out on self-stabilization.

8) Work should be carried out on the effects of soluble salts in calcrete bases (both stabilized and unstabilized) with a view to appraisal of the existing ideas on limits in the literature. Work should include wetting and drying and perhaps brushing tests on stabilized specimens. Information on the moisture regime in bases could also be usefully applied to soluble salt problems. The effect of cracked surfacings in allowing increased moisture migration appears important.

9) The flocculation factor and colloidal gel test used by Gillette (1934) may be worthy of further investigation.

9.15/ ...

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