CSIR
RESEARCH REPORT

324 Collapse settlement in compacted soils

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National Institute for Transport and Road Research

Bulletin 13

CSIR Research Report 324, NITRR Bulletin 13, pp. 1-34 . UDC.625.731.2:624.131.542] (68), Pretoria, South Africa, 1977

ISBN 0 7988 0496

CSIR Research Report No. 324
UDC 625.731.2:624.131.542](68)
Published in 1977 by the South African
Council for Scientific and Industrial Research
P.O. Box 395
Pretoria
0001.

Printed in the
Republic of South Africa
by the Graphic Arts Division
CSIR, Pretoria

SYNOPSIS

Research into collapse settlement in compacted soils is described, with special reference to recent cases in southern Africa where collapse settlement occurred in road embankments following wetting of the soil.

The laboratory work described consisted largely of oedometer tests over a range of saturations, on soils from four of these road embankments, compacted in a mould especially designed for this purpose. The influence of variations in initial dry density, compaction moisture content, applied pressure and particle size distribution, is discussed.

Examination of these four soils and two naturally-occurring collapsing soils with the electron microscope showed fundamental differences in the structure of natural and recompacted soils.

The Bulletin concludes with a brief review of the principles of soil compaction.

SINOPSIS

Navorsing oor swigting by verdigte grond word beskryf en daar word verwys na gevalle wat onlangs in Suider-Afrika voorgekom het waar padopvullings geswig het nadat die grond nat geword het.

Die laboratoriumwerk wat beskryf word, het hoofsaaklik bestaan uit konsolidometertoetse by 'n reeks versadigingspunte op grond wat uit vier van hierdie padopvullings verkry is en in 'n spesiaalontwerpte vorm verdig is. Die uitwerking van die wisseling van die aanvanklike droë digtheid, verdigtingsvoginhoud, aangewende druk en partikelgrootteverspreiding word bespreek.

Ondersoek van die vier grondsoorte en twee soorte natuurlike swiggrond met die elektronmikroskoop het fundamentele verskille in die struktuur van natuurlike en herverdigte grond aan die lig gebring.

Die Bulletin word afgesluit met 'n kort oorsig van die beginsels van verdigting.

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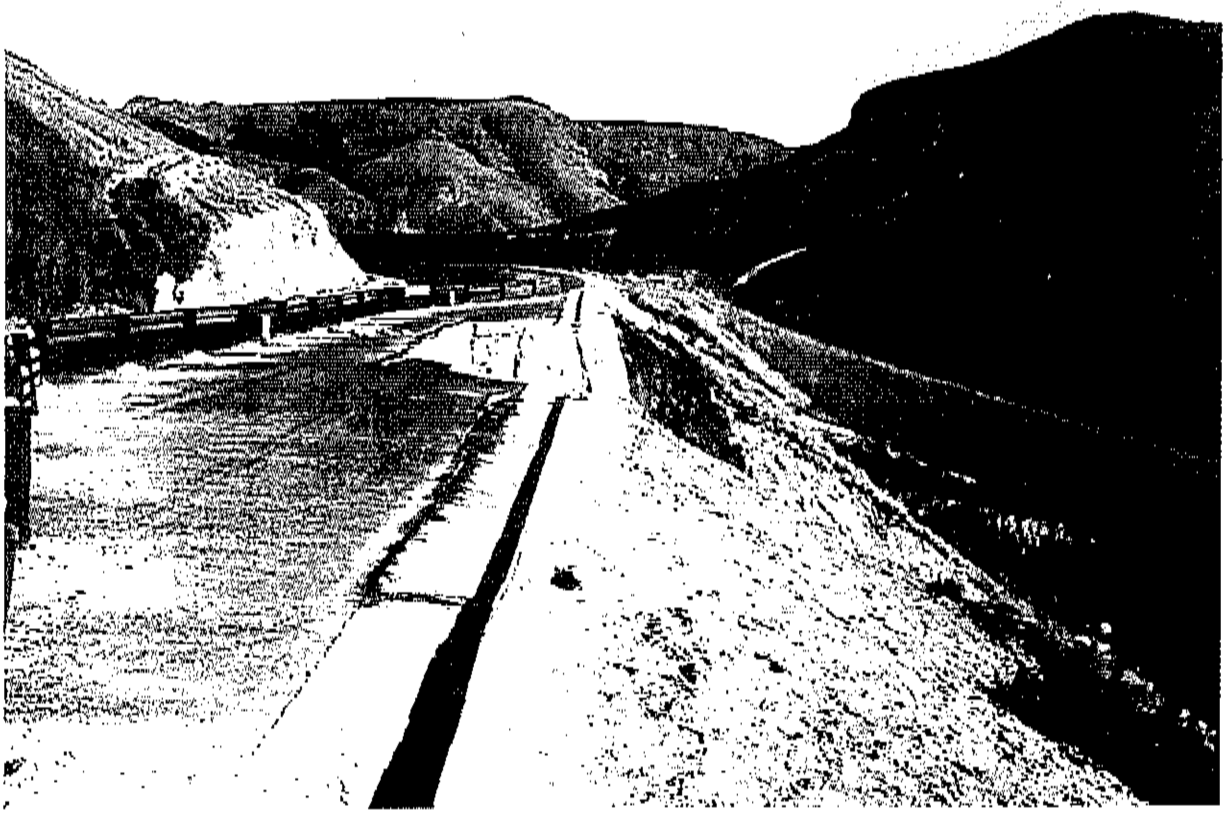

PLATE 1

Road 548, Tzaneen to Haenertsburg.
Slip failure at mileage 17,4 in 1969.

1. INTRODUCTION

Failure in a road embankment may be considered to have occurred when there has been sufficient movement to render the pavement unserviceable. Such movement may take the form of excessive settlement or sliding due to inadequate shear strength. Both types of failure can be minimized by applying the known principles of compaction (reviewed later in Chapter 5). Erosional damage may also take place but, since this is seldom severe enough to render the pavement unusable, it cannot be classified as failure and is not considered here.

In southern Africa failures such as those mentioned above have occurred on a number of occasions in recent years, and many of them have been investigated by the National Institute for Transport and Road Research. Concurrently with these investigations, a great deal of basic research into the factors influencing collapse settlement has been done at the Institute. In this Bulletin all the data obtained, including those already published elsewhere, are gathered together to demonstrate the fundamental importance of adequate compaction.

It is not usual for excessive settlement or shear failure to occur under dry conditions, and it should be understood from the outset that all the failures described have occurred during or immediately following periods of heavy rainfall. It is practically impossible to keep water out of embankments altogether, but the infiltration of water can certainly be reduced.

1.1 FAILURES CAUSED BY SETTLEMENT

Settlement may take place in the embankment, or in the undisturbed soil beneath it. Consolidation of the subsoil can be predicted and treated in exactly the same way as that beneath any other type of foundation and, as it is independent of the compaction conditions in the embankment itself, it will not be considered here.

Settlement within the embankment may take several forms:
Immediate settlement is clearly a minor problem, since it is complete before the road pavement is laid. *Primary consolidation* is only significant if it is numerically large and occurs over a long period: the magnitude of the consolidation is very much a function of the initial state of compaction of the soil, but slow rates of consolidation only occur in very fine-grained soils which are seldom used for road embankments in southern Africa. *Secondary consolidation* is usually so small that it can be ignored.

Collapse settlement, which is similar to collapse in naturally occurring soils, is another matter. It takes place in compacted earthworks under certain conditions (Jennings and Knight[21]). It is one of the major causes of settlement failure in road embankments in southern Africa, and may occur in inadequately compacted earthworks following periods of prolonged rain. This has been the subject of an extensive programme of research which is summarized in this report.

1.2 SHEAR FAILURES

Shear failure can occur along a surface wholly within the embankment, or along one that is partly in the subsoil - the latter is usually attributable to weaknesses in the subsoil as, for example, was the case in the failure of the Rickivy embankment at Pietermaritzburg (Maurenbrecher[29]) and other failures in Natal (Maurenbrecher and Booth[30]). In such cases no amount of compaction of the embankment will improve stability.

A comprehensive investigation into the influence of compaction conditions on the shear strength of fill material has yet to be carried out. Initial work was carried out (Pells et al [33,34]) to determine the variations in effective stress parameters due to different methods of measurement. Typical shear strength parameters for southern African soils have also been published (Pells and Booth[32]). Subsequently testing has proceeded on an *ad hoc* basis, and the information obtained to date is briefly described in Section 3.6 of this Bulletin.

2. SOME EXAMPLES OF FAILURE IN ROAD EMBANKMENTS

Only a few of the failures that have occurred in road embankments in southern Africa in recent years have been the subject of investigation, the thoroughness of which was in each case limited by the time and resources then available. The cases considered here are therefore only examples, chosen because they were fairly well documented and because each failure was largely due to inadequate compaction.

2.1 CASE 1 : HAENERTSBURG

Road 548 from Tzaneen to Haenertsburg was built in 1964 as an alternative to the very steep route via Magoebaskloof. Near the Haenertsburg end there are numerous large embankments, some of them over forty metres high measured from toe to crest. As most of these are built on steep side slopes, the maximum thickness of the fill material in any embankment is probably no more than twenty metres.

The early months of 1969 were the wettest since the construction of this road, and fourteen embankments near Haenertsburg were damaged (Williams and Clauss[42]). In most cases the damage consisted of erosion and a little settlement, but at Mileage 17,4 the sudden slip failure shown in Plate 1 (Frontispiece)

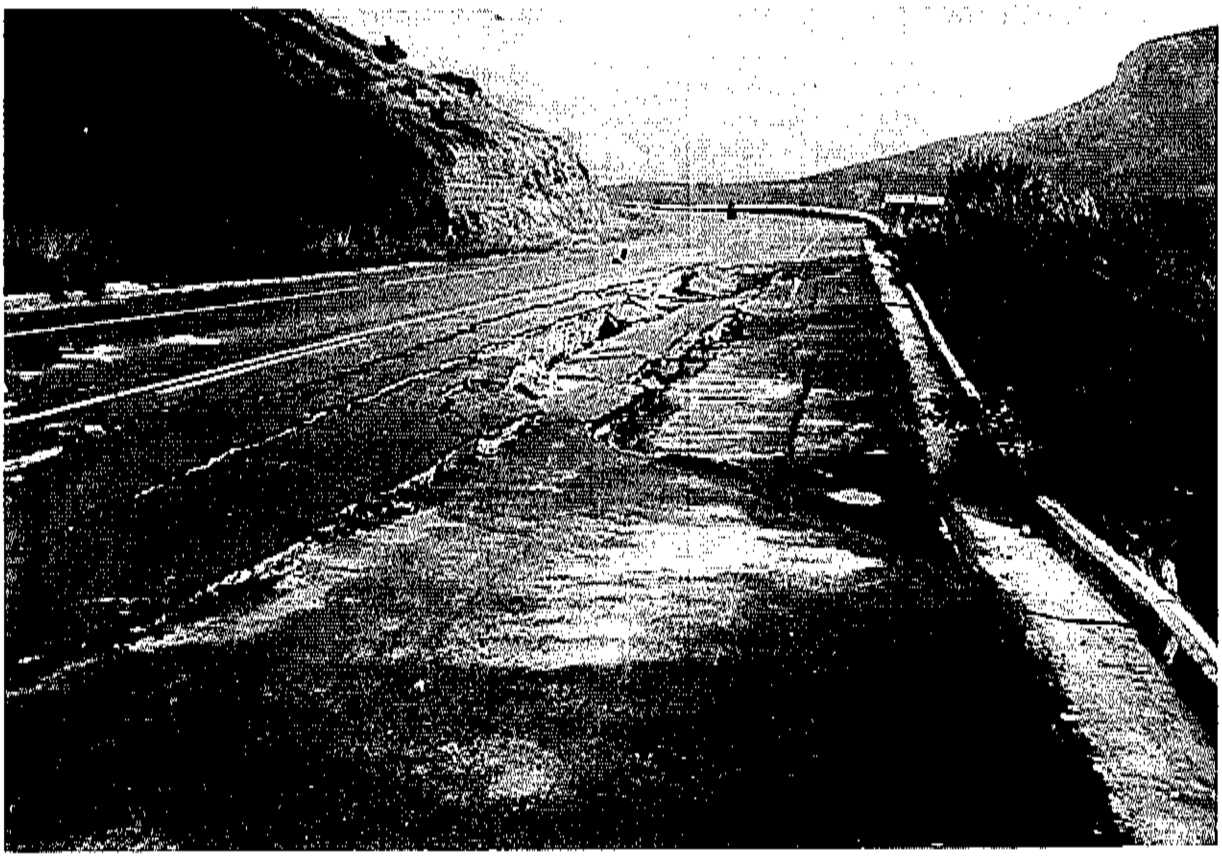

PLATE 2

Road 548, Tzaneen to Haenertsburg.
Probable settlement at mileage 17,4 in 1972.

occurred. An investigation was carried out into this failure (Maurenbrecher[26]), and meanwhile the damage was repaired.

Further and more extensive damage began during heavy rains in January 1972, and continued until the following April. January 1972 was the wettest month at Haenertsburg since 1958, and the period January/February 1972 was the wettest in the area since 1923. An analysis of the records showed that seasonal rainfall similar to that experienced in 1971/72 could be expected about once in twenty years, and rain of the intensity of that in January 1972 as frequently as every ten years (Booth[5]).

The damage caused was so extensive that an eight-kilometre length of road had to be closed to traffic. Deformation recurred at most of the places affected in 1969 - Plate 2 shows what happened at Mileage 17,4. At first it appeared that slip failure was again taking place, but since movement soon slowed despite continued heavy rain the deformation was later attributed to collapse settlement.

The most widespread damage was caused by erosion and collapse settlement, though there were a few slope failures such as that shown in Plate 3. The erosional damage took various forms, including sheet erosion which could be regarded as a form of shear failure. The influence of compaction conditions on the erodibility of soil requires a separate major investigation, and is not considered in this Bulletin. Moreover, much of the erosion could not properly be classified as failure, since the road continued to serve its purpose, although failure was imminent unless urgent repairs were undertaken immediately.

Failures thought to be due to settlement were widespread, and a further example is shown in Plate 4 where vertical deformation eventually reached about half a metre. Such failures were clearly caused by the water, which even ponded uphill of the embankments in some places, and are therefore regarded as collapse settlement. As there was apparently little natural soil beneath the fill material, it must be concluded that collapse took place in the embankments themselves.

The in-situ density of the embankments was subsequently found to be of the order of 80 per cent mod. AASHO. The fill material consisted of decomposed granite which, although it showed some variations in

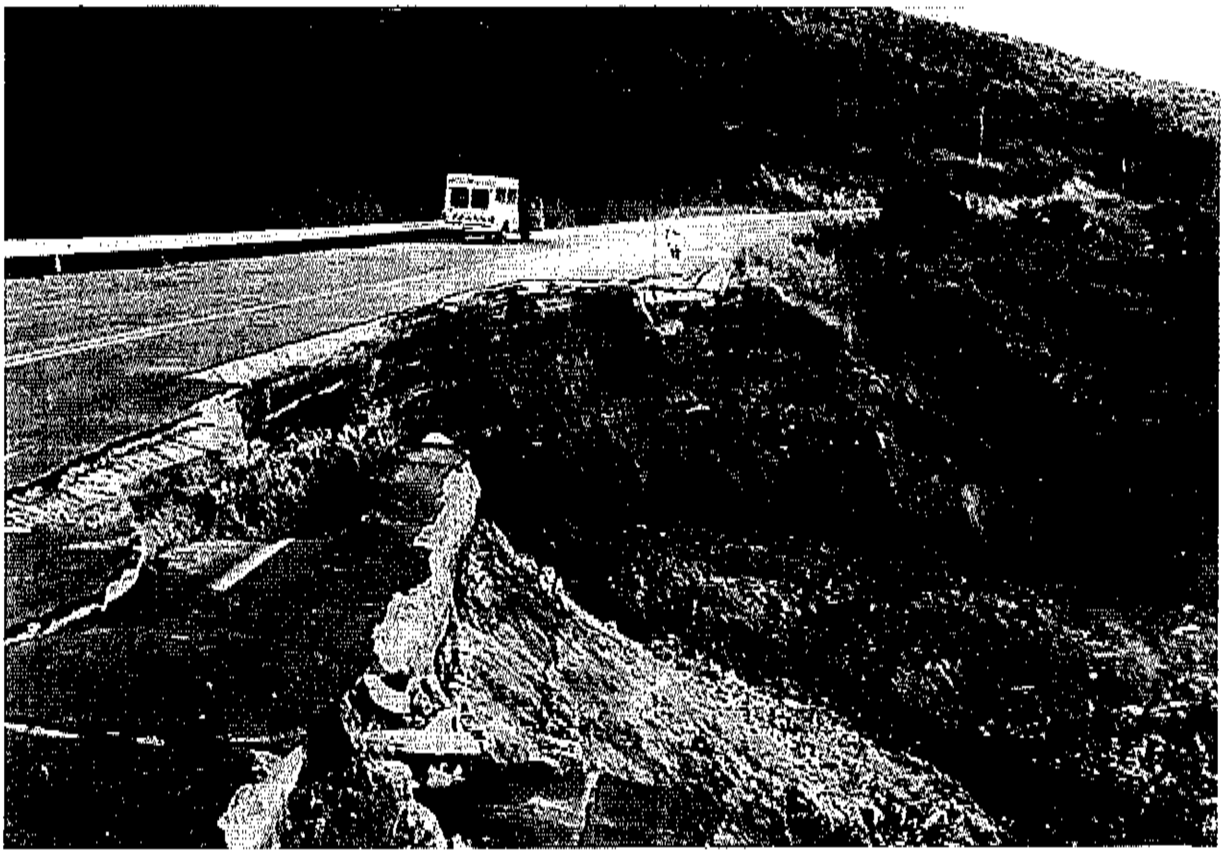

PLATE 3

Road 548 Tzaneen to Haenertsburg.
Slip failure at mileage 12,6 in 1972.

properties, could be identified as of two basic types. They were either red or yellowish-white in colour, the latter containing less fines and being less plastic. This soil has been designated Soil A and was used for a considerable number of laboratory tests. Its properties are given in Section 3.1. Oedometer tests were carried out on undisturbed block samples taken from within each embankment. It was found, as demonstrated in Figure 1, that considerable collapse could occur at low saturations. No collapse took place when the soil was tested at initial saturations of the order of seventy per cent.

2.2 CASE 2 : MELSETTER

Road improvements near Melsetter in Eastern Rhodesia during the late 1950's led to the construction of a large number of embankments. In 1963, towards the end of a summer that was wetter than usual, much settlement took place in these embankments, and this has been experienced at intervals ever since.

The fill material consists of a red silty soil, which is more fully described in Section 3.1 as Soil D. Block samples were obtained from the embankment shown in Figure 2, and shearbox tests were carried out to determine the effective strength parameters of the soil. Meanwhile standpipe piezometers were installed as shown in Figure 2, but even at the end of the rainy season in April 1972 no water was reported. Stability analyses were carried out using the SLOP2 program for circular slip (Szendrei and Pells[38]) and a factor of safety of 1,75 was determined for zero pore pressure. An average pore-pressure ratio of 0,44 would have been necessary for failure, and it was concluded that this was highly improbable for the critical failure circle shown in Figure 2.

Double oedometer tests were also carried out on undisturbed samples of the fill material, the results of which are shown in Figure 3. The dry density of the soil varied, but at 80 per cent mod. AASHO considerable collapse settlement was shown to be possible. Since shear failure was highly improbable, it seemed likely that the deformation in this particular embankment was due to collapse settlement.

2.3 CASE 3 : SOUTH WEST AFRICA

Approximately twenty kilometres of road in Khomas-hoogland were reconstructed between 1971 and 1973.

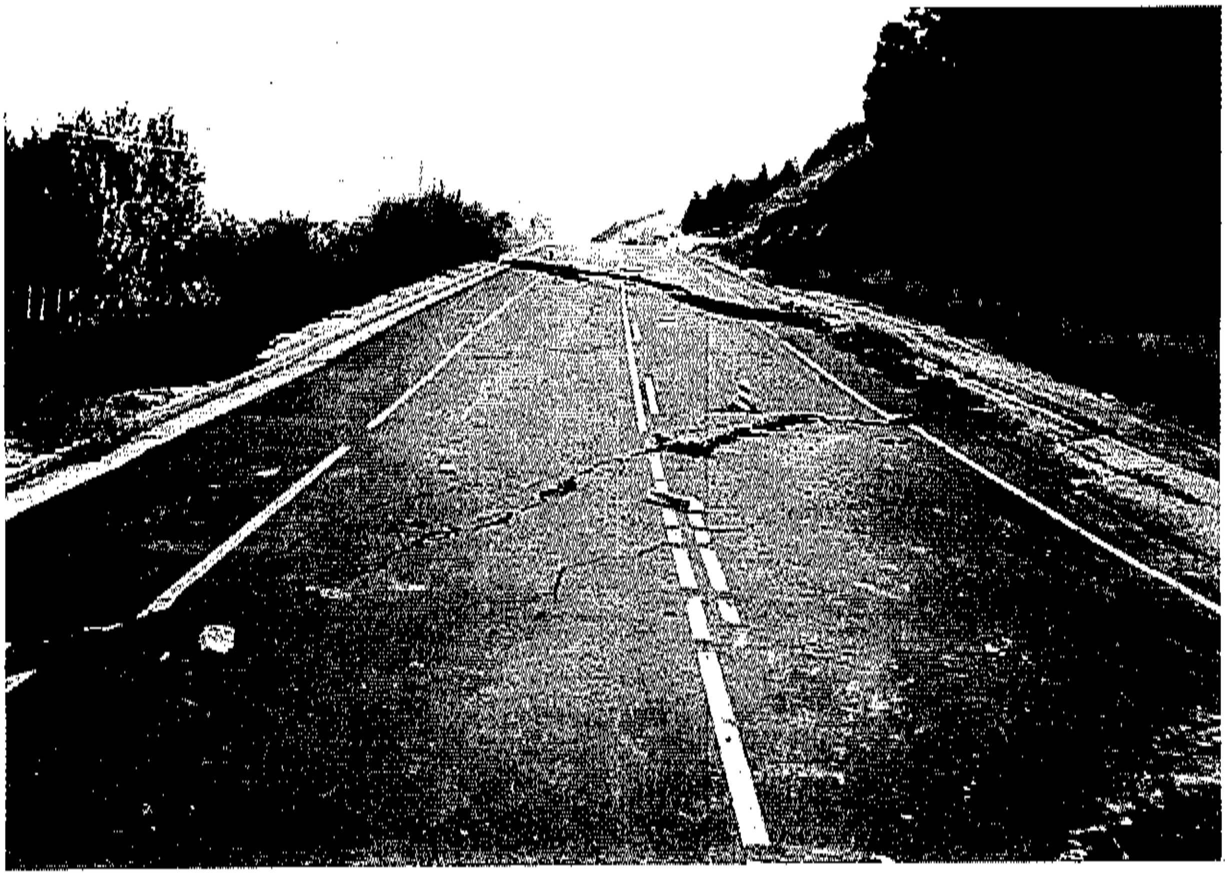

PLATE 4

Road 548, Tzaneen to Haenertsburg.
Settlement at mileage 15,8 in 1972.

During the winter of 1972 it was found that cracks such as those shown in Plate 5 had occurred on about twenty embankments. Typically the cracks ran parallel and close to the road shoulder, often in groups of three or more. Similar problems recurred in April 1973 after a period of moderate rainfall. Investigations led to the conclusion that these cracks were caused by movements of the slope due to shear failure at shallow depth. There was also one case of collapse settlement when a culvert became blocked and water was impounded by the embankment. Collapse may have contributed to other failures.

All the embankments were built of almost identical soil, a decomposed mica schist, the properties of which are given in Section 3.1, under the heading Soil C. It was shown that in this soil, which is substantially cohesionless, the angle of friction is dependent on the density. Instability occurred close to the slope because of the low density of the soil there. Though stable under dry conditions, when there was probably a significant soil suction as well, the factor of safety fell to unity when seepage forces were set up by percolating rainwater.

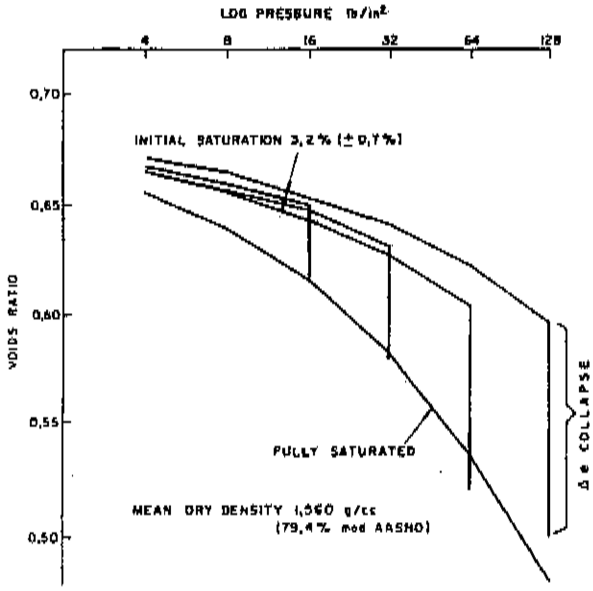

FIGURE 1

Soil A - e versus log P curves for undisturbed samples from embankment at mileage 14,0 road 548 Haenertsburg to Tzaneen.

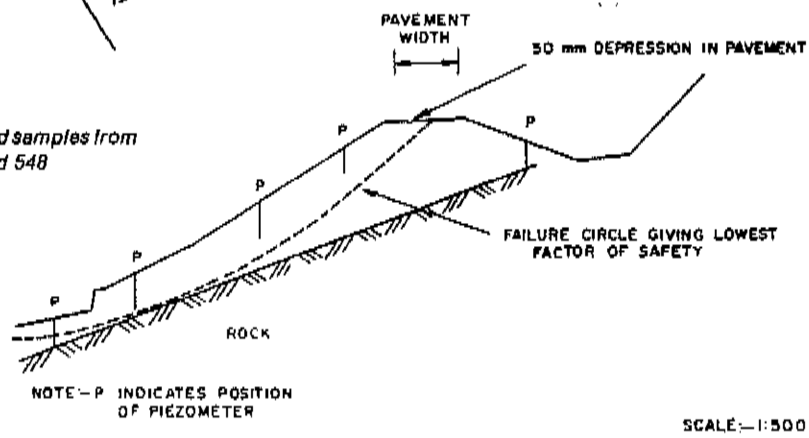

FIGURE 2

Cross-section of embankment at mileage 36,2 Melsetter-Skyline road.

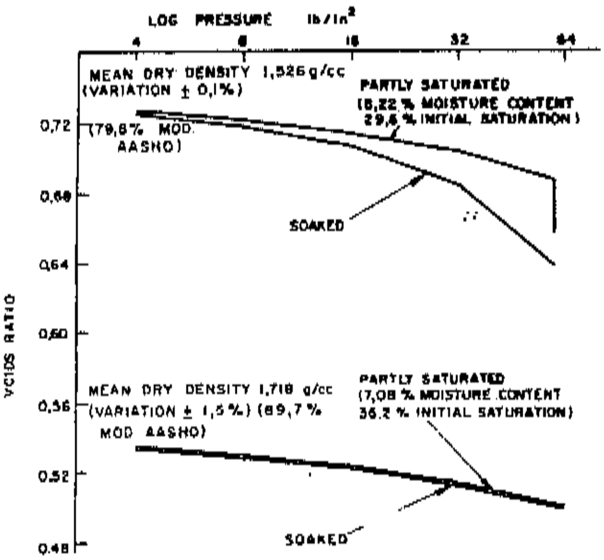

FIGURE 3

Soil D - e versus Log P curves for undisturbed samples of different densities from embankment at Melsetter.

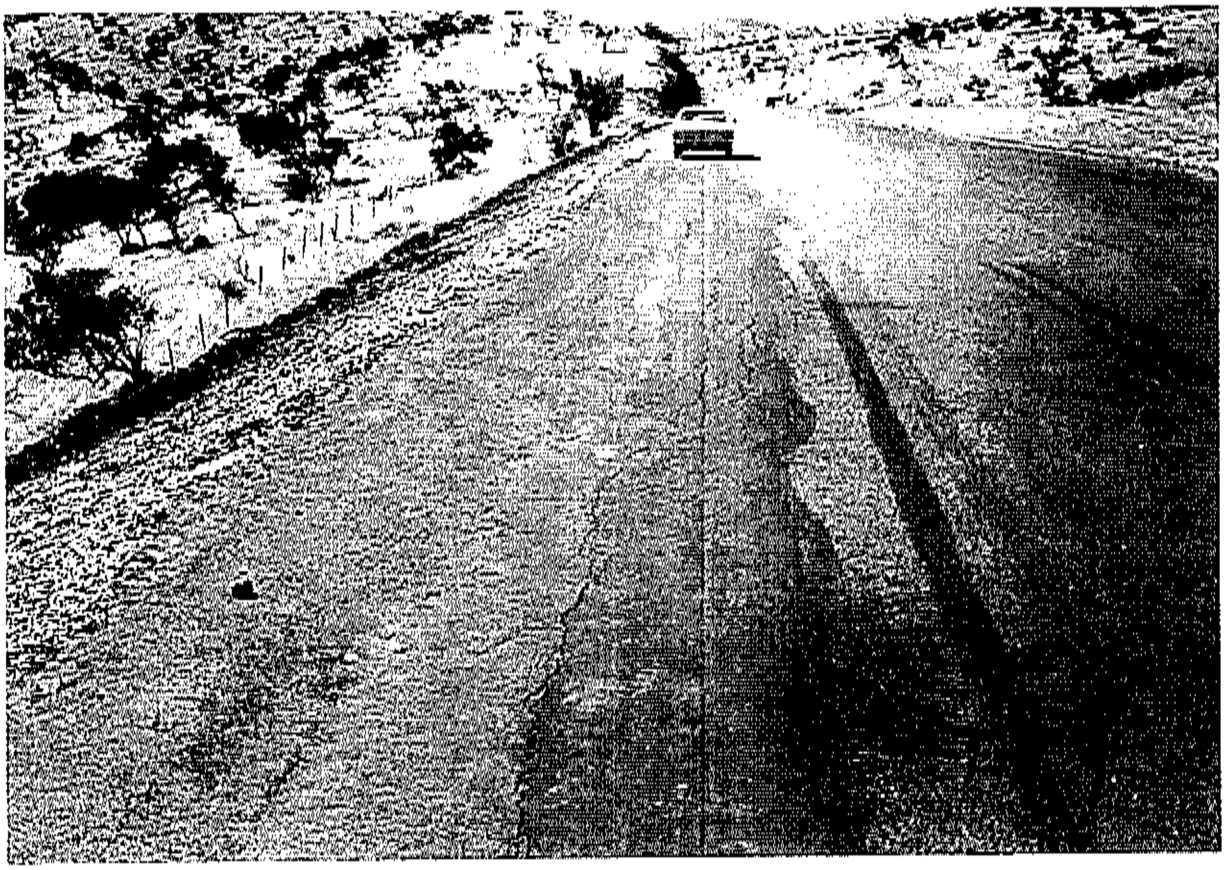

PLATE 5
South West Africa
Several open longitudinal cracks caused by slope instability in 1972.

3. DESCRIPTION OF SOILS AND TESTING PROCEDURES

A comprehensive series of oedometer tests was carried out to examine the factors influencing collapse settlement, the results of which are considered in Chapter 4. A few tests on undisturbed samples from various embankments were considered in Chapter 2. All other tests, however, were carried out on soil recompacted in the laboratory. This was necessary since the main purpose of the investigation was to assess the influence of initial conditions on the behaviour of the soil. Therefore, several different soils were studied in considerable detail to examine all the major variables. Standard preparation and testing procedures were adopted throughout to eliminate as many other variables as possible.

3.1 PROPERTIES OF OILS TESTED

Tests were carried out on six soils, four of which were actually sampled from road embankments and may be regarded as typical, at least for the areas from which they were obtained. (Soils A, C and D have already been introduced in Chapter 2.) The fifth and sixth soils, Soils E and F, were introduced later because undisturbed samples of natural collapsing soils were required for the investigation reported in Section 4.7. The location, geological origin and mineralogy of these soils are given in Table 1.

The typical fill materials, which were all residual, came from different geological series. Quartz is predominant in all the soils, and they all contain some kaolinite. All these soils could fairly be described as silty sands though, as shown in Figure 4, there are considerable differences in the particle size distributions. (It should be noted that gravel, cobbles and boulders were removed from the materials used in the laboratory tests - with the exception of Soils D, E and F which contained negligible amounts - and that the gradings used apply to the material finer than 2 mm.) Standard compaction tests were carried out on each soil (on material finer than 19 mm) using both the Proctor and mod. AASHO tests, and the results are summarized in Table 2. The

TABLE 1

Location and origin of soils.

Soil No.	A	B	C	D	E	F
Location Latitude (S) Longitude (E)	Haenertsburg 20° 59' 23° 59'	Durban 29° 51' 30° 56'	South West Africa 22° 41' 16° 48'	Melsetter 19° 52' 32° 41'	Pretoria 25° 45' 28° 16'	Durban 29° 52' 30° 59'
Parent rock Geological Series Geological Age	Granite Basement Archean	Sandstone Table Mountain Early Palaeozoic	Schist Khomas Proterozoic	Quartzite Umkondo Pre-Cambrian	Drift	Drift
Minerals > 20%	Quartz, Kaolinite	Quartz	Quartz, Mica	Quartz, Kaolinite	Quartz	Quartz
Minerals 5% - 20%	Felspar	Kaolinite, Felspar	Felspar			
Minerals < 5%	Illite	Illite, Mica	Kaolinite, Chlorite	Felspar, Mica	Kaolinite, Illite	Kaolinite

results of some of the standard index tests are also given in Table 2, and it can be seen that the soils all have low plasticity.

Soil A was separated into particle sizes and then remixed to give the four USPR classes shown in Figure 5. Compaction tests were carried out on each of these classes, and the results shown in Figure 6 were obtained. Subsequently the particles coarser than 2 mm were removed, and consolidation tests were carried out on the remainder as discussed in Section 4.5.

3.2 PREPARATION OF SOIL FOR COMPACTION

The preparation and storage of the six soils was the same in each case. Soil for the standard compaction tests was oven-dried overnight at 110 °C and then sieved through a 19 mm screen. Gravel retained on the screen was discarded and the compaction tests were carried out on the remainder.

Soil intended for oedometer and shearbox tests was also oven-dried, but was sieved dry through a 2 mm screen. The gravel retained on this screen usually had a considerable amount of fine soil adhering to it which was gently washed off the gravel and through the screen. All the material which had passed the screen was again oven-dried and thoroughly mixed before being stored in galvanized iron bins.

3.3 COMPACTION OF OEDOMETER AND SHEARBOX SPECIMENS

It was decided to compact the oedometer specimens directly into the ring using static pressure. (A similar method was later used to compact soil into the shearbox cutting rings for subsequent extrusion into the shearbox itself.) The decision to use this approach has been justified elsewhere (Booth[10]), based on a review of published findings such as those by Gau and Olson[16] and Shackel[36]. It has been shown that static compaction gives the most uniform specimens, especially where the height to diameter ratios of the specimens are one third or less. No attempt was made to verify this during the present investigation.

It was also found that the method chosen gave a high degree of control over the average dry density of a specimen. Of 544 oedometer specimens made, it was shown (Booth[10]) that less than eight per cent had an average dry density which deviated more than $\pm$ one per cent from the mean for the series of which it formed part, whereas of the 66 natural samples tested, as many as 58 per cent fell outside these limits. It should also be borne in mind that these dry densities were determined after the soil had been subjected to wetting or drying which might have caused small changes in volume.

Compaction was carried out in a specially made brass mould, shown schematically in Figure 7. It consists of a 3 inch (76,2 mm) diameter cylinder split along both the horizontal and vertical axes. Into this the standard oedometer ring fits in such a way that the inner faces are flush. Flanged pistons slide into each end of the cylinder, and are arranged so that when the flanges stop against the body of the mould the faces of the pistons are exactly at the ends of the oedometer ring. Plate 6 shows the mould partly assembled, demonstrating the position of the oedometer ring. Later, a second mould, which works on the same principle, was made to compact the 60 mm-square x 20 mm-thick shearbox specimens.

The testing programme for each soil required sets of specimens, each with a particular compaction moisture content and initial dry density. Dry soil from the storage bin was therefore mixed with water to the specified moisture content. The mould, fully assembled with an oedometer ring, was placed on the bench with the bottom piston in position. Then the exact

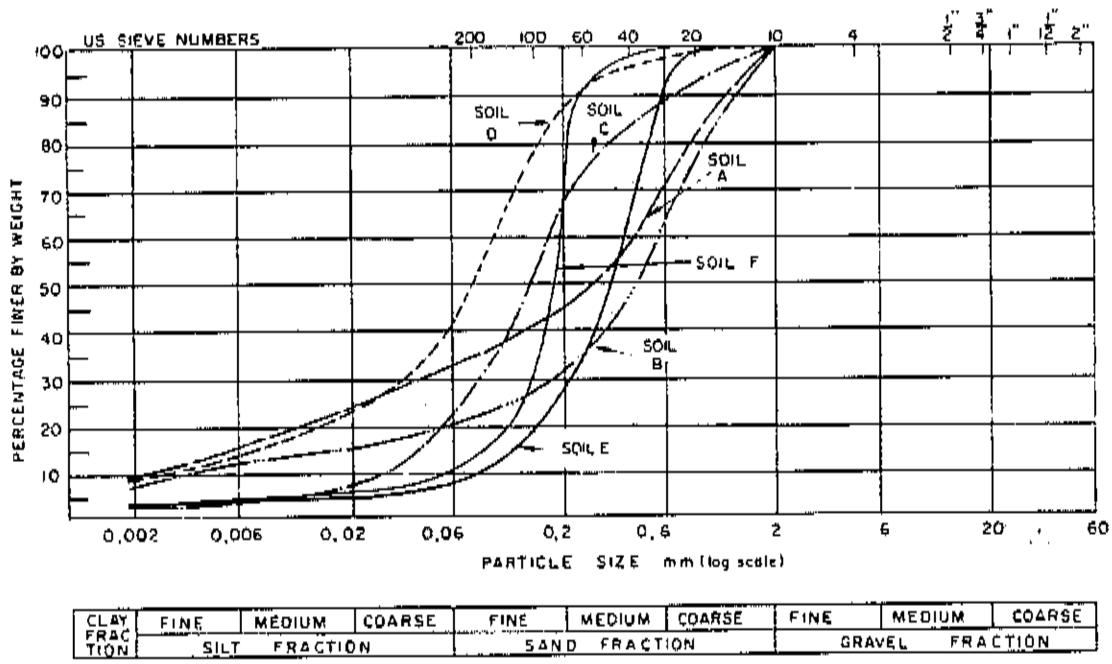

FIGURE 4

Particle size distribution of soils tested in oedometer.

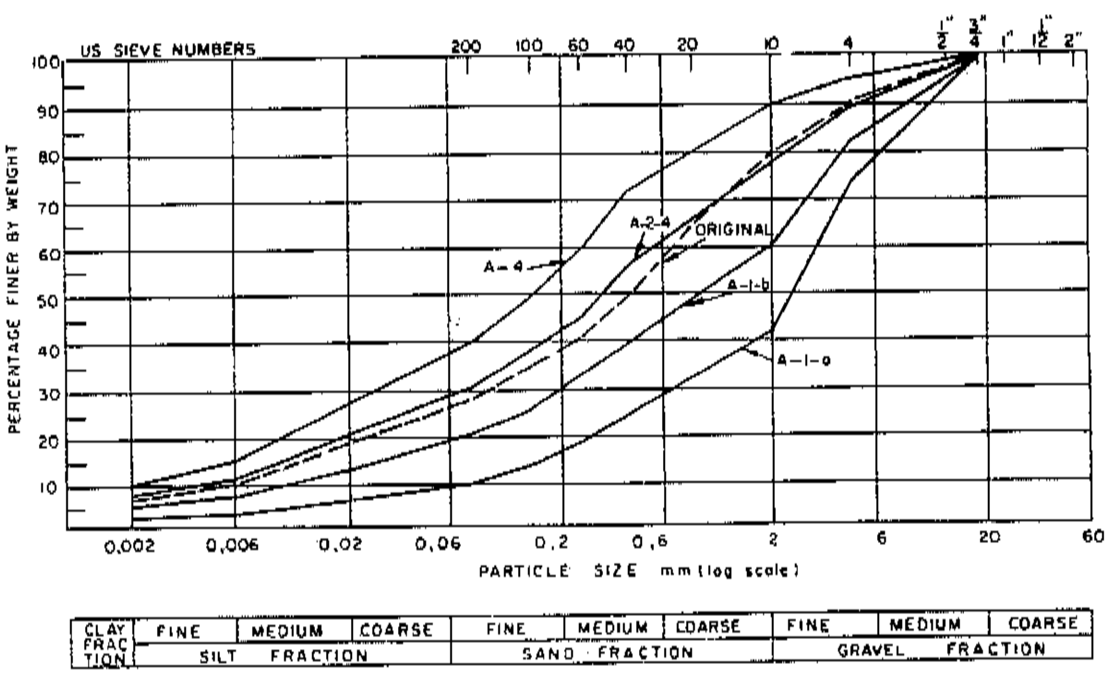

FIGURE 5

Soil A - particle size distribution of USPR classes used for compaction tests.

TABLE 2

Summary of index properties of soils.

SOIL		A	B	C	D	E	F
Sand fraction	%	68	80	75	56	92	88
Silt fraction	%	23	12	22	34	5	9
Clay fraction	%	9	8	3	8	3	3
Mod. AASHO test							
γd	kg/m^3	1966	2140	2007	1915	2060	2097
o.m.c.	%	10,5	7,0	8,5	12,0	6,2	10,8
Proctor test							
γd	kg/m^3	1846	2055	1942	1750	1985	1975
o.m.c.	%	13,0	9,8	9,5	13,0	8,0	11,8
Liquid Limit	%	30,1	24,1	27,0	23,9	17,0	non
Plastic Limit	%	28,8	21,3	27,0	21,4	15,2	plastic
Plasticity Index	%	1,3	3,1	0	2,5	1,8	
Linear Shrinkage	%	1,7	2,7	0	1,3	0	0
Activity		0,1	0,4	0	0,3	0,6	0
Specific gravity		2,625	2,614	2,716	2,661	2,644	2,851

amount of soil required to give the specified dry density was weighed out, put carefully into the mould and spread as evenly as possible. The top piston was then inserted and pressed home under a static pressure applied by hand, by a dead load or by a hydraulic press, as necessary. The oedometer ring, now containing the compacted soil, was removed from the mould and placed on a porous disc to obviate damage during handling.

The specimens compacted in this way appeared, on visual examination, to be of uniform consistency. As already noted it was not considered part of this work to make quantitative measurements to justify this. There was no evidence of either deformation of the oedometer rings or failure of the soils when the rings had been removed from the mould. It should be stressed that the densities used were relatively low, so much so that there was some risk of the compacted specimens slipping from the rings if not very carefully handled.

3.4 CONDITIONING OF OEDOMETER SPECIMENS FOR TESTING

The testing programme required that each set of initially identical specimens should be tested over a range of moisture contents. Therefore, the possible range of initial saturation, which was the parameter used for control in the laboratory, was zero to 100 per cent. As the latter specimens were fully saturated, they could be tested under water in the normal way. All other specimens were partly saturated, and were tested sealed in polythene as for the partly saturated part of the double oedometer test (Jennings and Knight[21]). It was assumed that there was no loss of moisture during each test.

In any one set, all specimens were initially compacted at the same moisture content, and normally it was necessary either to increase or decrease the saturation to obtain the required range. To increase the saturation the specimen was placed in a humid room until it

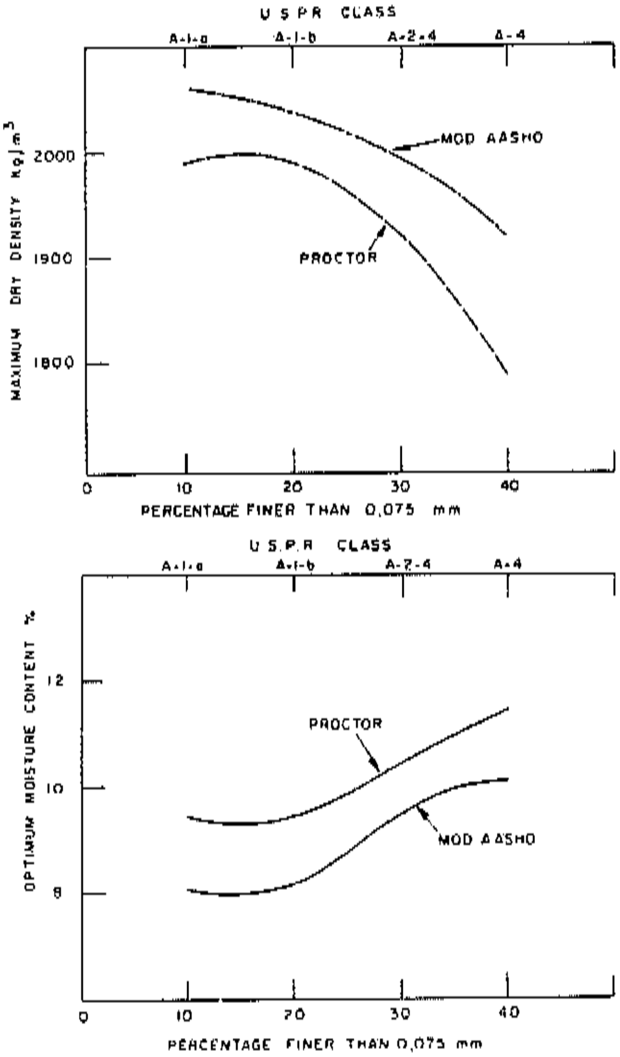

FIGURE 6 (left)

Soil A - summary of moisture content versus density results for various particle size distributions.

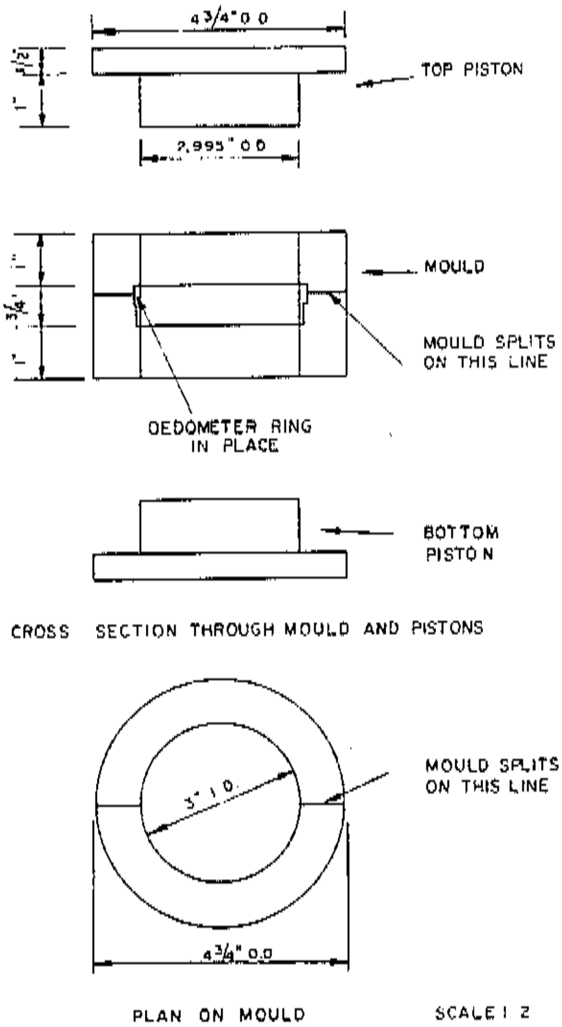

FIGURE 7 (right)

Arrangement of mould for compaction of 3" dia. x 3/4" thick oedometer samples.

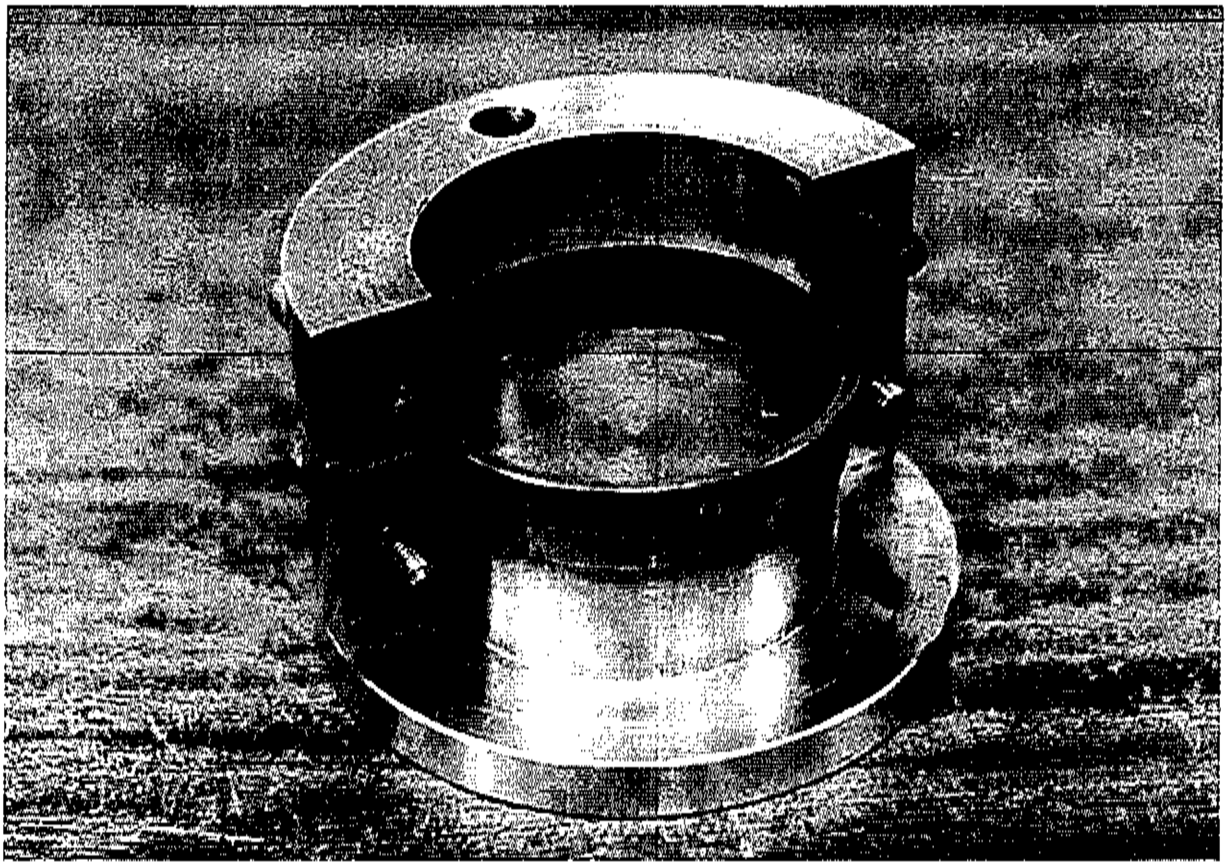

PLATE 6

Mould for compaction of oedometer samples.
Mould partly assembled showing position of ring.

was a little wetter than required. It was then dried to the specified saturation in the same way as those specimens which had been compacted wetter than they were to be tested. The drying was achieved by placing the specimen, ring and porous disc in the open laboratory, and allowing moisture to evaporate slowly. By determining the total mass at intervals, the saturation could be estimated quite accurately, since the mass of the ring and porous disc was known, and that of the dry soil could be assumed. Once the required saturation was reached the specimen was placed in a double polythene bag and stored in the humid room ready for testing. The exact thickness and mass of each specimen were determined immediately prior to testing, but volume changes were found to be minimal.

It should be stressed that this conditioning of the specimens duplicates fairly closely what happens in an embankment. The major difference is that in the field the soil may already be under load when it undergoes a change in moisture content. Moreover the soil in the field does not necessarily reach a higher moisture content and then dry. In each case there was no practical laboratory method of reproducing these changes more closely.

Shearbox specimens were tested as compacted, and therefore required no special conditioning.

3.5 TESTING OF OEDOMETER SPECIMENS

The six oedometers used for this testing are calibrated in Imperial units. The work was therefore carried out using pressures measured in pounds per square inch, and is reported in those units here. Some later work used two oedometers calibrated in tons per square foot.

In general, oedometer specimens were loaded in six logarithmic increments starting with a load of 2 lb/in^2 (13,8 kN/m^2) and finishing with one of 64 lb/in^2 (441,3 kN/m^2), at which pressure the partly saturated specimens were soaked. This procedure was varied in the tests reported in Section 4.4, where the effect of different maximum applied pressures was being specifically studied, and in which final pressures were varied between 16 lb/in^2 (110,3 kN/m^2) and 256 lb/in^2 (1 765,1 kN/m^2). In some of the tests described the six logarithmic increments were increased from 0,125 T/ft^2 (13,4 kN/m^2) to 4 T/ft^2 (429,0 kN/m^2).

In some of the earliest oedometer tests each load was applied as soon as primary consolidation under the previous load was complete. For most of the work, however, loads were applied at the beginning and end of each working day, so that no load was applied for much less than eight hours. The compression under each load increment was measured 30 seconds after the application of the load, and thereafter at logarithmically-increasing time intervals. An identical procedure was used when the specimens were inundated.

3.6 SHEAR STRENGTH OF COMPACTED SOIL

Most, if not all, road embankments in southern Africa are seldom fully saturated, and in many cases (such as those mentioned in Section 2.3) the degree of saturation is low. In these circumstances the total stress methods of stability analysis are not recommended (Bishop and Bjerrum[8]), and effective stress methods should be used. The shear strength is therefore determined in saturated specimens by means of either drained tests or consolidated undrained tests with pore pressure measurement.

Recent work (Pells et al[33,34]) has shown that in practice there is little difference between the effective strength parameters whether measured in the triaxial or the direct shear apparatus, and because of its greater simplicity the direct shear (shearbox) test has been used here. Drained tests were necessary as it is impracticable to measure pore pressure using this method.

The shearbox tests were carried out using a standard procedure throughout. Specimens in any set were saturated and consolidated under different normal pressures in the range 18,5 kN/m² (the lowest that could be conveniently applied) to about 500 kN/m². The consolidation versus time curve was measured on a continous recording device, and the maximum rate of shearing was then determined by the US Corps of Engineers method (Bowles[11]). The rate actually used was always several times slower than the maximum, but, because the soils were all very permeable, little time was lost by testing more slowly. In one instance where a check was carried out by testing at a rate 25 times slower than usual, the same effective stress parameters were obtained.

It should be emphasized that this work was not carried out as an extension of the study of consolidation which has already been discussed, as tests could only be done when there was sufficient soil and when there was adequate time and apparatus available. Although this testing was not, therefore, as comprehensive as it might have been, such data as have been obtained are presented here.

Drained shearbox tests were carried out on specimens of Soils A, B and D which had been compacted at dry densities of 75, 80, 85 and 90 per cent of mod. AASHO maximum. In each case three different moisture contents were used for compacting specimens, and the cohesions and angles of friction measured in these tests are given in Table 3. Specimens of Soil C were also tested with initial dry densities ranging from 80 per cent to 95 per cent mod. AASHO, and the results showed that between these densities the cohesion increases from about 2 kN/m² to 14 kN/m² and the angle of friction from about 36 ° to 43 °.

Two major difficulties arise in interpreting these results. In the first place the plots of peak shear stress versus normal stress are invariably curved, as in the examples shown in Figure 8. The parameters given in Table 3 have therefore been determined from the parts of these curves above about 100 kN/m² normal stress where they approximate to straight lines.

The second problem is that the dry density, which is initially very low, increases during the consolidation stage of each test - these increases were in many cases considerable. Moreover, the amount of increase differs for each normal load, so that the eight or ten tests used to determine the cohesion and angle of friction all had to be carried out on specimens sheared at different dry densities.

Reference to Figure 9, which is discussed in Section 4.2, shows that specimens with initial dry densities in the range 75 to 85 per cent mod. AASHO consolidate to similar densities under a load of 64 lb/in² (441,3 kN/m²). The lack of significant changes in the shear strengths of these particular specimens (see Table 3) is therefore not surprising, nor is the slight rise in cohesion at higher initial dry densities. The results given for Soil C also show the same trend.

There is a very slight tendency for the cohesion to rise with increasing compaction moisture content, but the angle of friction is unaffected for all practical purposes. This rise in cohesion is most noticeable in Soil B (particularly at high initial dry density) and virtually non-existent in Soil D. It was noted during the testing that specimens of Soil B were much less friable when compacted at high rather than low moisture contents.

Observations reported later in Section 4.7 show that the clay particles were more evenly distributed among the sand grains when higher compaction moisture contents were used. Presumably this caused greater cohesive bonding, as well as the lower compressibility noted in Section 4.3.

4. COLLAPSE SETTLEMENT IN COMPACTED SOIL

The soil used as fill material for road embankments in southern Africa is typically sandy. As such it has a high

TABLE 3

Effective shear strength of soil compacted under different conditions.

Compaction moisture content (%)		Dry density (per cent mod. AASHO) 75	80	85	90
	SOIL A				
	Dry density (g/cc)	1,475	1,573	1,671	1,769
4	Cohesion (kN/m²)	22	26	24	32
	Angle of friction (deg.)	35,3	34,8	34,8	34,3
9	Cohesion (kN/m²)	23	27	33	34
	Angle of friction (deg.)	35,2	34,8	33,3	33,2
13	Cohesion (kN/m²)	22	25	29	31
	Angle of friction (deg.)	36,3	34,7	33,3	33,4
	SOIL B				
	Dry density (g/cc)	1,605	1,712	1,819	1,926
3	Cohesion (kN/m²)	27	28	27	31
	Angle of friction (deg.)	34,8	35,2	35,0	34,8
6	Cohesion (kN/m²)	31	32	28	43
	Angle of friction (deg.)	34,0	34,1	33,3	35,2
10	Cohesion (kN/m²)	35	32	32	47
	Angle of friction (deg.)	33,6	35,0	32,1	35,0
	SOIL D				
	Dry density (g/cc)	1,436	1,532	1,628	1,724
4	Cohesion (kN/m²)	27	26	29	24
	Angle of friction (deg.)	33,7	33,7	33,2	34,6
9	Cohesion (kN/m²)	23	24	22	23
	Angle of friction (deg.)	34,5	34,0	34,3	34,2
13	Cohesion (kN/m²)	24	26	23	26
	Angle of friction (deg.)	33,8	33,3	33,2	32,8

permeability and, as already remarked, primary consolidation settlement usually takes place in a relatively short space of time. It is therefore suggested that collapse settlement within the fill material due to subsequent wetting is of more significance.

The collapse settlement depends on a number of variables, and each has been investigated in turn whilst the others have been kept constant. For each set of specimens the results have been plotted in terms of the degree of saturation (or moisture content) which is already known to be an important variable in itself (Jennings and Knight[21]).

The oedometer testing programme, which is summarized in Table 4, was carried out on Soils A, B, C and D in turn. As Soil C was considered not to be a typical soil, testing was curtailed when sufficient results from the road in South West Africa were available for the purpose of the investigation. The full set of tests shown in Table 4 was therefore only carried out on Soils A, B and D. Oedometer tests were also performed on Soils E and F, but only for the specific purposes discussed in Section 4.7.

4.1 COLLAPSE SETTLEMENT IN GENERAL

Collapse settlement may be defined as the settlement that occurs in a partly saturated soil solely because of an increase in the degree of saturation. The phenomenon has been recognized for some time as occurring in undisturbed soils (Terzaghi and Peck[38], for example). The term 'collapse' was first used by Jennings and Knight[20] who conducted the first comprehensive studies on collapse settlement on the

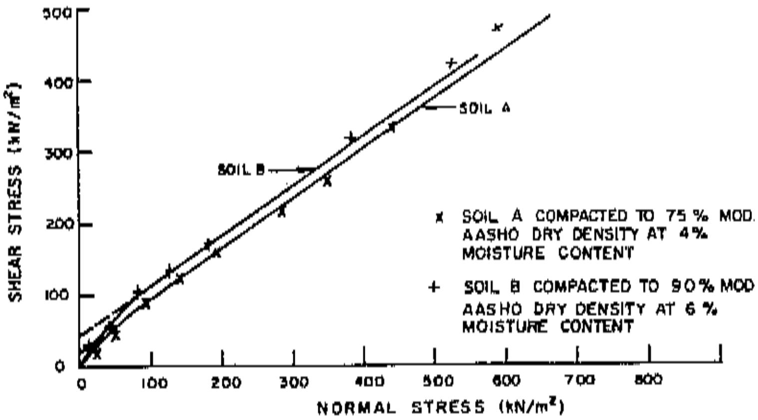

FIGURE 8

Typical plots of shear stress against normal stress.

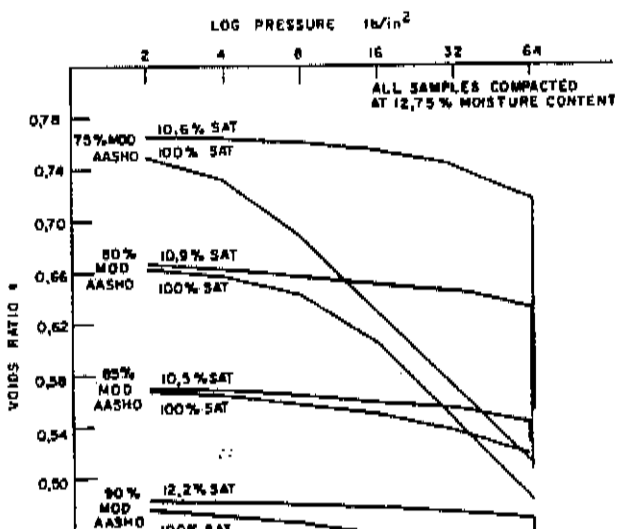

FIGURE 9

Soil A - e versus log P curves for various initial dry densities.

TABLE 4

Summary of oedometer testing programme.

Variable investigated*	Section of Bulletin	Soils used	Conditions applied during tests				
			Initial dry density (% mod. AASHO)	Compaction moisture content(% saturation at 80 per cent mod. AASHO)	Pressure at wetting (lb/in²)	Soil Grading	Soil structure
Initial dry density	4.2	A B C D	70 75 80 85 90	50	64	Natural	Recompacted
Compaction moisture content	4.3	A B D	80	16,7 33,3 50,0 66,7	64	Natural	Recompacted
Pressure at wetting	4.4	A B D	80	50	16 32 64 128 256	Natural	Recompacted
Soil grading	4.5	A B	80 (for grading used)	50	64	A B D	Recompacted
Soil structure	4.7	E F	As for undisturbed	33 and 40	64	Natural	Undisturbed, recompacted, after wetting/ drying cycles

windblown sands of southern Africa (Jennings and Knight[20,21]; Knight[24]). This work was soon extended to residual granite soils (Brink and Kantey[13]; Knight[25]) and cases have now been reported in various soils from many parts of the world (Dudley[15]).

As a first attempt to quantify the collapse, Jennings and Knight[20 21] used the double oedometer test in which one specimen was tested partly saturated and the other fully saturated. At any specified pressure the partly saturated specimen was inundated, so that the collapse settlement at that particular pressure could be determined. The same technique has been extended to a number of partly saturated specimens in this present work. Knight[24] found that the collapse reduced as the degree of saturation increased, and that there was a critical saturation above which collapse did not occur.

Other factors are known to influence collapse. One of the most important of these is the dry density (or initial voids ratio). It appears that only soils with an initially low dry density are vulnerable to collapse. It was suggested (Williams[41]; Jennings[18]) that significant collapse only occurs when the dry density is below 100 lb/ft^3 (1 600 kg/m^3). Collapse is also thought to increase with the applied pressure until the pressure reaches a maximum value of the order of 5 kg/cm^2 (490 kN/m^2) (Aitchison[1]), above which collapse reduces.

That collapse can occur in recompacted soils was never seriously studied, although the possibility had been suggested from time to time (Holtz[17]; Knight[24]). However, in recent years Wagener[40], Barden et al[5] and Barden[3] have shown that recompacted soils do collapse under certain circumstances. Although Barden[3] and Barden *et al*[4] state that clays did not collapse when they were wetter than the Proctor optimum moisture content at the time of compaction, they do not make it clear whether or not they were tested at the compaction moisture content.

The mechanism of collapse was postulated by Knight[24] in terms of the observed structure of the soils. The natural collapsing soils studied consisted predominantly of sand grains with a small proportion of silt and clay particles. The latter were found to concentrate at the points of contact between the sand grains, forming 'clay bridges'. Although the hypothesis suggests that most of the interparticle stresses are transmitted through these clay bridges, the structure remains stable under overburden pressure, even if inundated from time to time. If additional stresses are imposed the clay bridges may still be capable of transmitting them with a minimal change of volume, provided the saturation of the soil is below the critical value. An increase in saturation to above the critical value causes the clay bridges to fail, and collapse settlement then occurs.

The collapse mechanism has subsequently been considered from other angles. For instance it has been suggested that before a soil can collapse it must have a basically unstable structure, a high imposed stress and high suction. If the latter is lost due to wetting, collapse occurs (Barden et al[5]). The removal of the suction from the clay bridges constitutes a reduction in the normal stress on the clay, and it is thought that the consequent reduction in shear strength causes failure (Burland[14]). It is generally accepted that the principle of effective stress cannot be applied to collapse on the macro scale (Jennings and Burland[19]; Blight[7]) but that

TABLE 5

Average dry densities achieved in recompacted oedometer samples.

Conditions specified		Conditions achieved	
Percentage mod. AASHO	Compaction moisture content (%)	Percentage mod. AASHO	Mean dry density (g/cc)
SOIL A*			
70	12,75	70,6	1,386
75	12,75	75,6	1,486
80	4,25	79,8	1,569
80	8,50	80,1	1,575
80	12,75	80,0	1,572
80	17,50	80,2	1,578
85	12,75	84,9	1,669
90	12,75	89,4	1,757
80(A-1-a)	12,75	80,1	1,576
80(A-1-b)	12,75	80,5	1,584
80(A-2-4)	12,75	79,7	1,567
80(A-4)	12,75	78,6	1,546
80(B)	10,08	78,7	1,684
80(D)	13,86	78,5	1,503
SOIL B*			
70	10,08	70,2	1,502
75	10,08	75,0	1,605
80	3,36	78,9	1,690
80	6,72	79,2	1,695
80	10,08	79,8	1,705
80	13,44	79,9	1,710
85	10,08	84,9	1,817
90	10,08	89,3	1,910
80(A)	12,75	80,0	1,573
80(D)	13,86	78,7	1,508
SOIL C			
70	12,72	68,8	1,380
75	12,72	73,9	1,480
80	12,72	78,4	1,574
85	12,72	82,6	1,657
90	12,72	86,8	1,742
SOIL D			
70	13,86	69,8	1,337
75	13,86	74,6	1,429
80	4,62	78,8	1,510
80	9,24	78,9	1,511
80	13,86	79,1	1,514
80	18,48	79,1	1,515
85	13,86	84,0	1,609
90	13,86	88,7	1,698
SOIL E†			
Undisturbed	—	78,4	1,616
77,7 (0)	7,50	77,8	1,602
77,7 (7)	7,50	77,4	1,595
77,7 (17)	7,50	77,0	1,586
77,7 (56)	7,50	76,8	1,582
SOIL F†			
Undisturbed	—		
76,3 (0)	10,00		
76,3 (56)	10,00		

* The figures in brackets refer to the regrading of the soil - see Section 4.5.

† The figures in brackets refer to the number of wetting and drying cycles - see Section 4.7.

this principle may still be valid when considering microshear (Aitchison[1]).

4.2 INFLUENCE OF INITIAL DRY DENSITY

It has already been mentioned that the intitial dry density of a soil is known to affect its potential collapse. Although some arbitrary criteria have been proposed to obtain some idea of this collapse (Jennings and Knight [22]; Anan'ev and Gil'man[2], for example, there is virtually no quantitative information available. This is presumably due to the difficulty and improbability of finding undisturbed samples of identical soil having a sufficiently wide range of densities. In this present work specimens were used which had been compacted in the laboratory, and because of this it was possible to obtain any density desired.

Specimens of Soils A, B, C and D were tested at dry densities of 70, 75, 80, 85 and 90 per cent of mod. AASHO maximum. Specimens of each soil were compacted at identical moisture contents, equivalent to 50 per cent saturation at 80 per cent mod. AASHO dry density, which is normally close to the Proctor optimum. Because of differences in specific gravity and maximum dry density, the compaction moisture content varies from soil to soil. The actual mean dry density achieved for each series of specimens was not always exactly that nominally required. The latter (expressed as a percentage of mod. AASHO maximum) are, for the most part, used in the following discussion, but the actual values can be obtained from Table 5.

For each oedometer test a plot of voids ratio versus the logarithm of pressure can be drawn in the conventional way. Typical plots are given in Figure 9. This

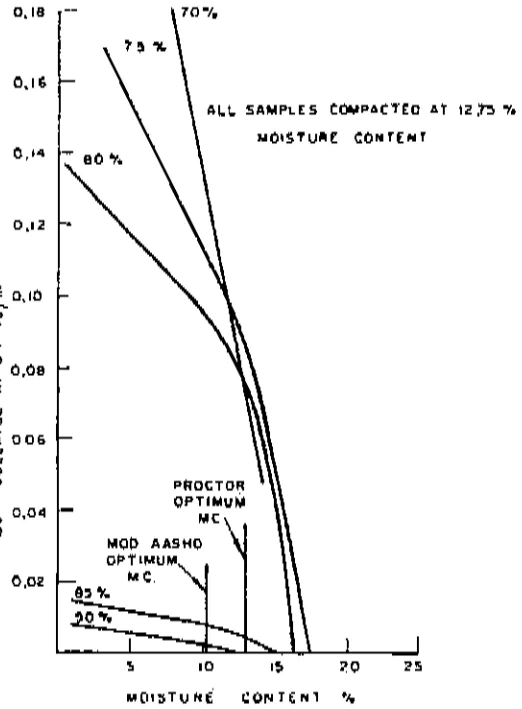

FIGURE 10

Soil A - collapse versus moisture content for various densities.

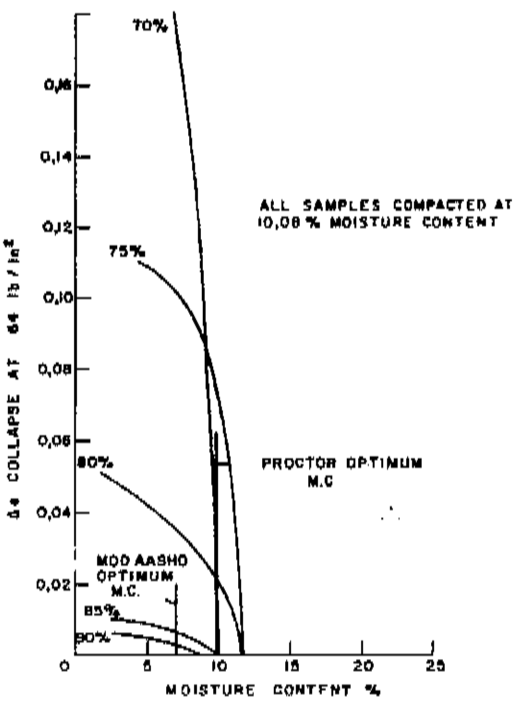

FIGURE 11

Soil B - collapse versus moisture content for various densities.

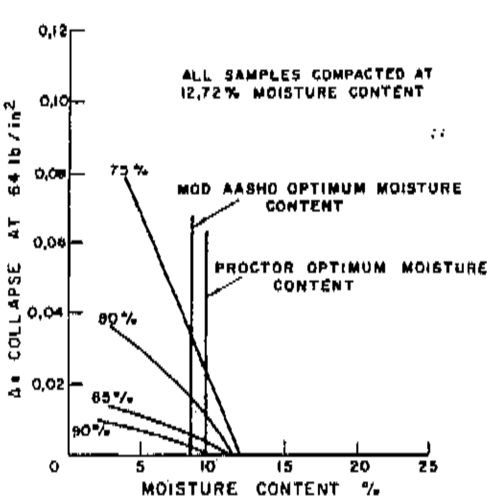

FIGURE 12

Soil C - collapse versus moisture content for various densities.

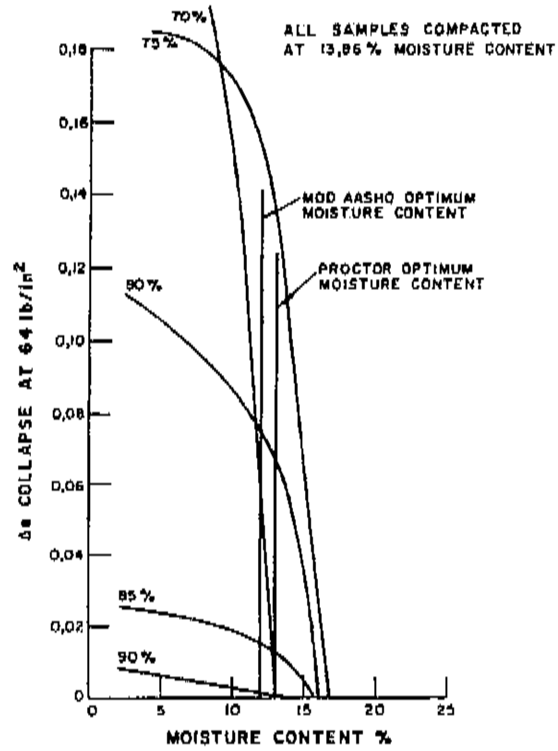

FIGURE 13

Soil D - collapse versus moisture content for various densities.

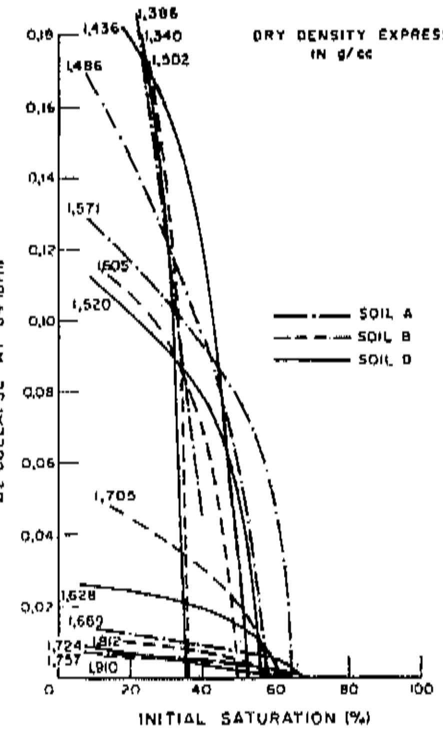

FIGURE 14

Collapse versus saturation curves for three soils compacted to various densities.

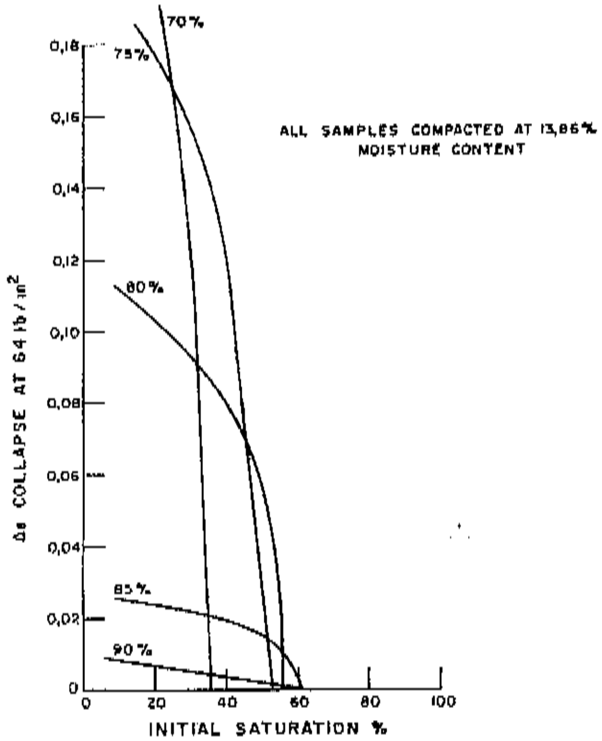

FIGURE 15

Soil D - collapse versus initial saturation for various densities.

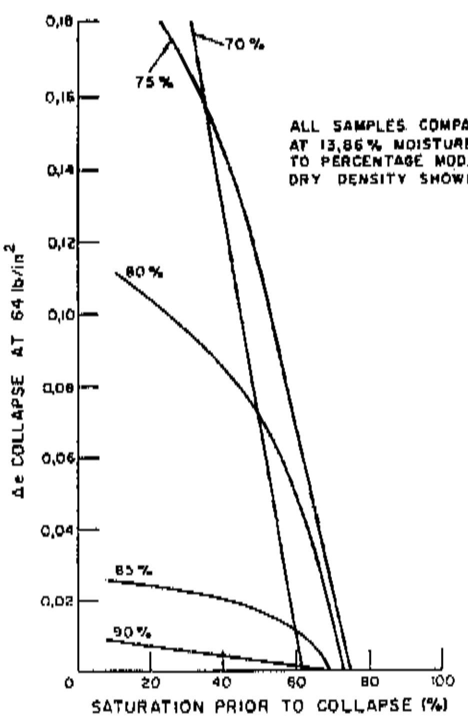

FIGURE 16

Soil D - collapse versus saturation prior to collapse for various dry densities.

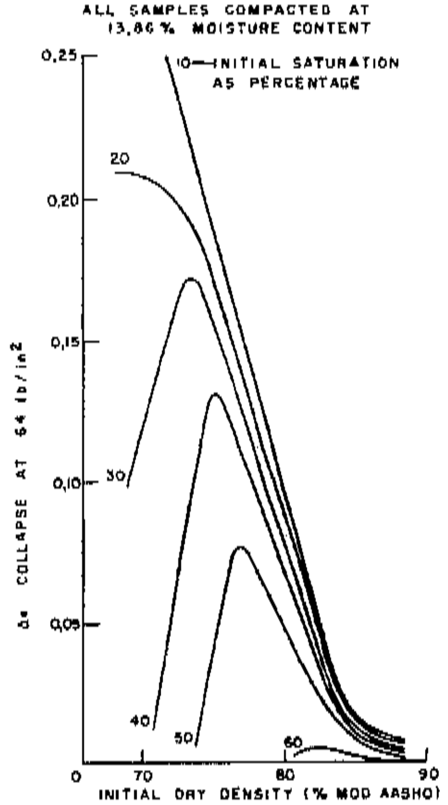

FIGURE 17

Soil D - collapse versus dry density curves for various initial saturations.

does, in fact, demonstrate that specimens compacted to initial dry densities of between 75 and 85 per cent mod. AASHO all consolidate to similar final densities under a pressure of 64 lb/in² (441,3kN/m²). It also shows that specimens that were tested fully saturated (soaked) actually compress rather more than initially identical specimens which were tested partly saturated and then inundated. However, in this type of presentation only selected results can be used (in this case 8 out of the 58 available) and, since inevitably there is some scatter of results, other more general methods must be used to summarize the data obtained.

For each soil and initial dry density a curve was drawn to relate collapse (expressed as change in voids ratio) at 64 lb/in² (441,3 kN/m²) to moisture content. These are grouped together in Figures 10 to 13 for Soils A to D respectively. (For the 70 per cent mod. AASHO specimens it was not always possible to draw a satisfactory curve, and this has been omitted altogether from Figure 12.) As these figures are very similar for all four soils, some general conclusions can be drawn. Both the mod. AASHO and Proctor optimum moisture content (though this has, in fact, been compacted at a lower moisture content). Secondly, it is evident that the critical moisture content increases as the dry density reduces, and reaches a maximum value for densities of the order of 75 per cent mod. AASHO, after which the value begins to fall again.

It is obvious that the amount of collapse rises rapidly at lower initial dry densities. It is not, however, very easy to determine the dry density below which this rapid rise commences. Figures 10 and 11 would suggest that it is below 85 per cent mod. ASSHO, but for Soil D in Figure 13 it would fall between 85 per cent and 90 per cent mod. AASHO. (Soil C is ignored in the following more detailed discussion, since the effect of the high mica content is not propery understood.) Since Soil D has the lowest maximum dry density, this leads to the conclusion that density must not be considered only in terms of the mod. AASHO test. For this reason Figure 14 was drawn to show collapse versus initial saturation for Soils A, B and D in terms of the absolute densities. The curves separate into two groups: those for initial dry densities above about 1 650 kg/m³ and those for dry densities below this value. Collapse in the latter was several times that of the denser soil. The influence of the relative density can still be seen, and some dual standard would be required to prescribe an initial dry density which would produce only a small collapse. This dry density might have to exceed both 85 per cent mod. AASHO and 1 650 kg/m³.

In Figures 10 to 13 collapse was related to moisture content. Similar curves can be obtained by plotting collapse against either the initial saturation (before the commencement of loading) or the saturation prior to collapse. These are shown, for example, for Soil D in Figures 15 and 16 respectively.

The data already presented can be replotted in several different ways. This has been done for Soils A, B and D, but only Soil D is considered here by way of example.

In Figure 17 the collapse in Soil D has been plotted against the initial dry density for various initial saturations. This shows very clearly how collapse becomes minimal in soil above a particular dry density, in this case about 85 per cent mod. AASHO. It is also interesting to note that for any saturation there is a density at which collapse is a maximum, and that this density rises with increasing saturation.

Another way of considering the same data is shown in Figure 18, where the percentage recuction in volume in Soil D during collapse has been plotted in terms of the initial dry density and the moisture content during the test. This demonstrates the previous comment that the critical moisture content has a maximum value for an initial density of between 75 and 80 per cent mod. AASHO.

Initially, all specimens in any set had virtually the same voids ratio. The reduction in voids ratio in those inundation) there was a considerable variation in the voids ratio. The reduction in voids ratio in those specimens tested fully saturated, was invariably about ten per cent greater than that of the samples tested partly saturated and then soaked.

It was also found that the final voids ratio was not the same for all initial densities, but had a maximum value for soil with an initial dry density of the order of 80 per cent mod. AASHO. This is shown in Figure 19, where voids ratio. The reduction in voids ratio in those against initial dry density. The voids ratios immediately prior to collapse have also been included in this figure, expressed as a function of the moisture content.

4.3 INFLUENCE OF COMPACTION MOISTURE CONTENT

Almost all of the published work on collapse settlement has been in relation to natural soils, where the question of compaction moisture content does not arise. Some of the few references (Barden *et al*[5]; Barden *et al*[4]) to laboratory tests on compacted soils suggest that collapse does not occur when the soil is compacted wetter than the Proctor optimum moisture content. It is not, however, clear whether these specimens were also loaded at the compaction moisture content. The results of the tests discussed in Section 4.2 show that collapse can occur in some soils when the moisture contents are a little above the Proctor optimum, but only within a certain range of densities.

Specimens of Soils A, B and D were compacted to a density of 80 per cent mod. AASHO at moisture con-

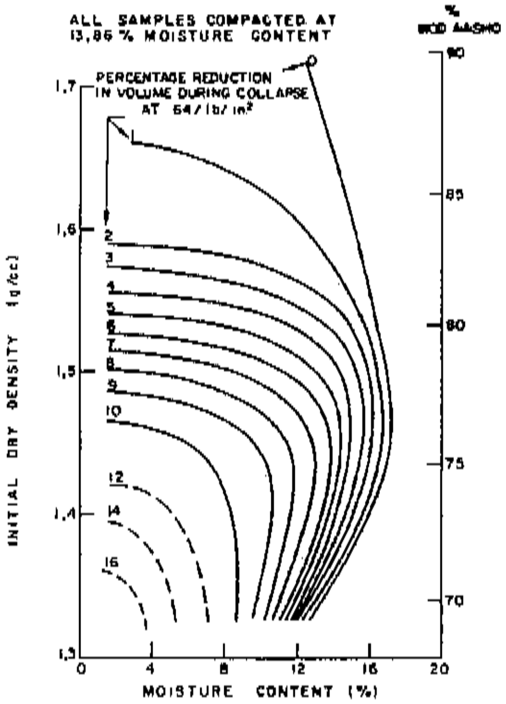

FIGURE 18

Soil D - percentage collapse at different dry densities and moisture contents.

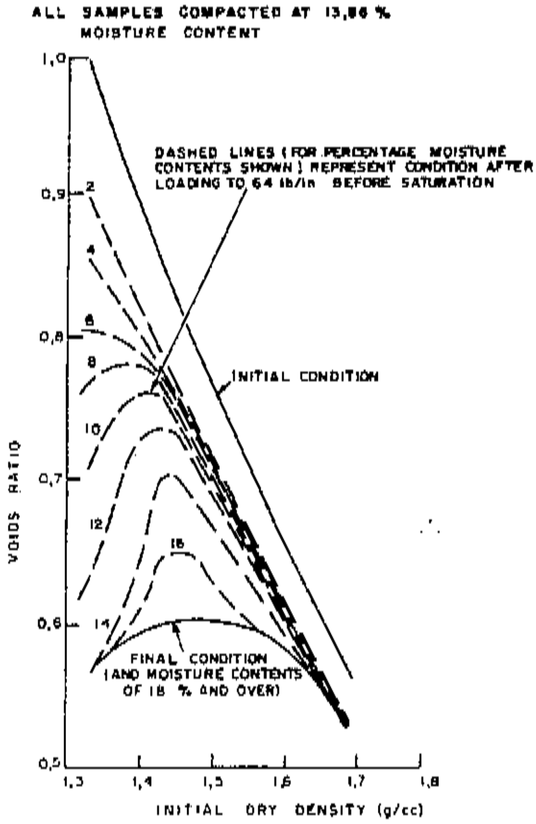

FIGURE 19

Soil D - voids ratio prior to collapse at 64 lb/in² for various moisture contents.

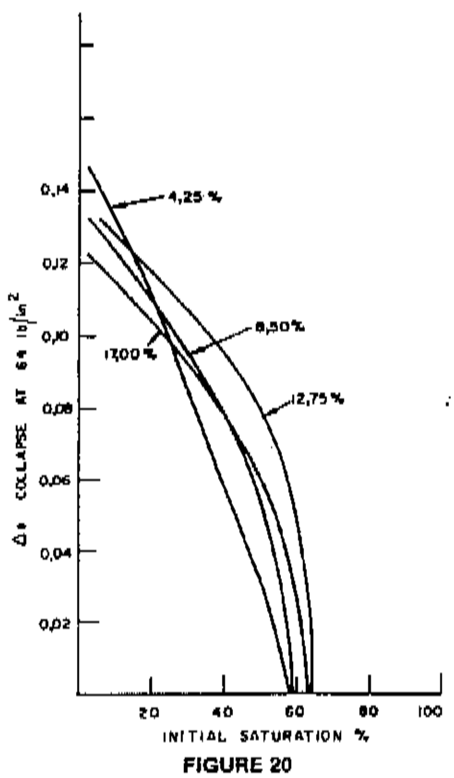

FIGURE 20

Soil A - collapse versus saturation for samples compacted to 80 per cent mod. AASHO dry density at various moisture contents.

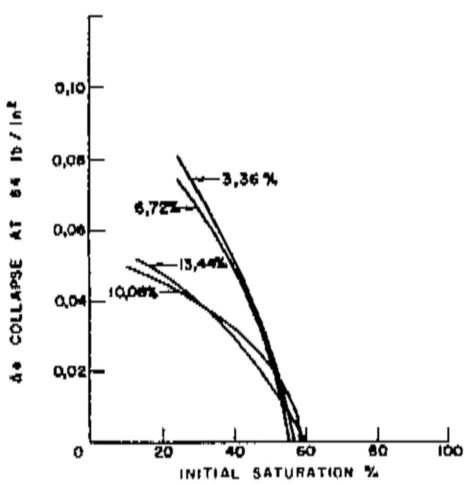

FIGURE 21

Soil B - collapse versus saturation for samples compacted to 80 per cent mod. AASHO dry density at various moisture contents.

tents equivalent to saturations of 16,7; 33,3; 50,0 and 66,7 per cent. Each set of specimens was tested in the oedometer over the same range of saturations as before, and all were inundated at 64 lb/in² (441,3 kN/m²) applied pressure. The curves of collapse versus initial saturation are given in Figures 20, 21 and 22 for Soils A, B and D. These show very little variation in collapse settlement with changes in compaction moisture content. There is, perhaps, a tendency for specimens (particularly of Soil B) compacted at the higher moisture contents to collapse rather less. In practical terms there is no significant change in collapse or in critical saturation with compaction moisture content.

The mean total consolidation of each set of specimens was determined and is shown in Figure 23. Once again those specimens tested fully saturated consolidated rather more than those tested partly saturated and then soaked. It was also apparent that soil compacted below the Proctor optimum moisture content (given in Table 2) consolidated appreciably more than when compacted wetter.

4.4 INFLUENCE OF APPLIED PRESSURE

It is already known (Knight[24]; Aitchison[1]) that the amount of collapse that may occur is dependent on the pressure applied at inundation. The relationship between collapse and pressure is not, however, understood.

Series of tests were carried out on Soils A and D with final pressures of 16, 32, 64, 128 and 256 lb/in² (110,3; 220,6; 441,3; 882,6 and 1 765,1 kN/m²) respectively. Tests were also carried out on Soil B at the three highest pressures but, although the results were entirely consistent with those for the other soils, there were not sufficient of them from which to draw conclusions and they are not presented here. In all cases the specimens were compacted to a dry density of 80 per cent mod. AASHO at a saturation of 50 per cent.

The curves of collapse versus initial saturation are given in Figures 24 and 25 for Soils A and D respectively. (These curves would be identical if plotted in terms of moisture content, since the initial dry densities were all the same.) Two curves do not fit the general pattern, that for a load of 64 lb/in² (441,3 kN/m²) in Figure 24 and, to a lesser extent, that for a load of 32 lb/in² (220,6 kN/m²) in Figure 25. These two sets of data are anomalous and correct curves can be conjectured. Discounting the anomalies, Figures 24 and 25 show that the critical initial saturation falls with increasing pressure. At initial saturations close to the critical values the amount of collapse is actually less at higher pressures, but at low saturations the position is reversed. However, the curve for a pressure of 256 lb/in² (1 765,1 kN/m²) does not cross that for a pressure of 128 lb/in² (882,6 kN/m²) in Figure 24. There is therefore some ground for believing that (for these soils and density) collapse reaches a maximum at pressures of this order.

The same data can be replotted in terms of collapse against pressure (to a logarithmic scale) for various moisture contents, as in Figure 26. This shows clearly how at any moisture content there is a maximum amount of collapse, and how this occurs at higher pressure for drier soils. Selected plots of voids ratio versus logarithm of pressure illustrate the same point. In Figure 27 the specimen with an initial saturation of 57,3 per cent would have collapsed at any pressure below 128 lb/in² (882,6 kN/m²).

4.5 INFLUENCE OF SOIL GRADING AND TYPE

The tests discussed so far were carried out on Soils A, B, C and D at the gradings sampled in the field after the removal of particles over 2 mm in size. Variations in behaviour have been noted which can only be attributed to differences between the soils. These could be due to the soil types, exemplified by the mineralogical differences summarized in Table 1, or the variations in particle-size distribution shown in Figure 4. Tests were therefore carried out to assess the significance of these factors.

Initially Soil A was divided into USPR classes, as already mentioned in Section 3.1, and shown in Figure 5. Specimens of each grading were compacted to the same dry density, 80 per cent mod. ASSHO for the original grading, at the moisture content equivalent to 50 per cent saturation for the original grading, and tested as before. The collapse versus initial saturation curves obtained are shown in Figure 28, and lead to the conclusion that these relatively small changes in grading did not significantly alter the behaviour of the soil.

More radical differences in grading were made to Soils A and B, the former being reconstituted to Gradings B and D (as shown in Figure 4) and the latter to Gradings A and D. (Grading D, for instance, means the original grading of Soil D was finer than 2 mm.) Oedometer tests were carried out on specimens compacted to 80 per cent mod. AASHO at the moisture content giving 50 per cent saturation, in both cases for the soil whose grading was being used. The results can be compared in terms of either the soil type (Figure 29 for example) or grading (Figure 30). These results, of which only examples are shown here, suggest that the amount of collapse is a complex function of soil type, grading and dry density. It appears from Figure 29 that the initial dry density is the dominant factor. It is, indeed, reported (Barden *et al*[5]; Brackley[12]; and Barden[3]) that any soil may collapse under the right conditions.

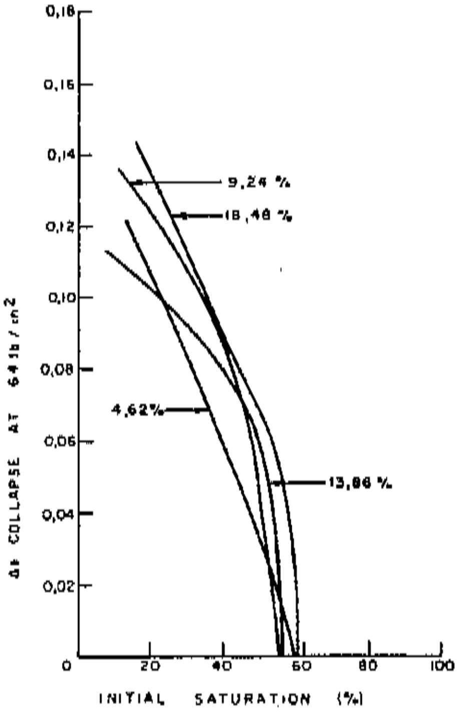

FIGURE 22

Soil D - collapse versus initial saturation for samples compacted to 80 per cent mod. AASHO dry density at various moisture contents.

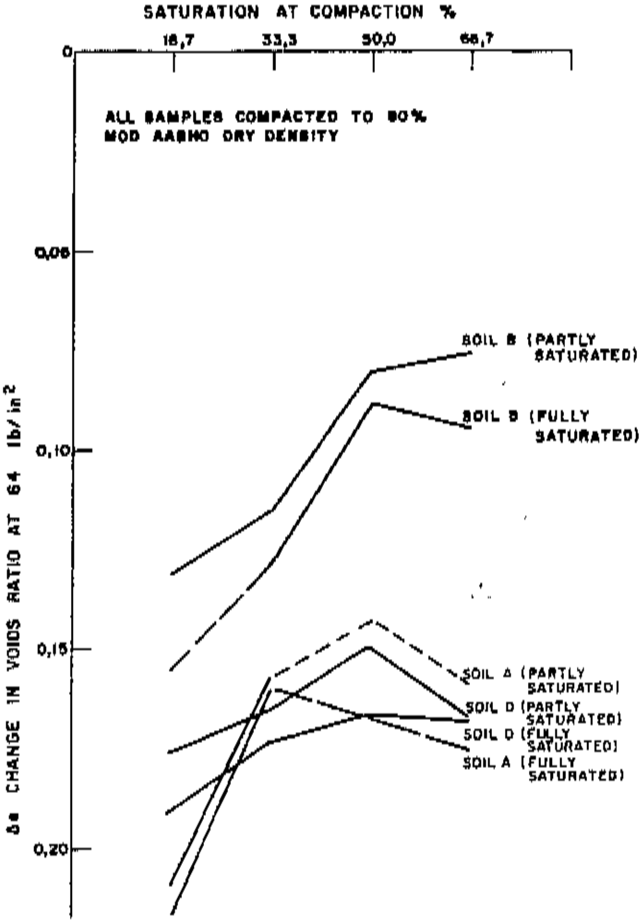

FIGURE 23

Total consolidation (both partly and fully saturated) in samples of three soils compacted at various saturations.

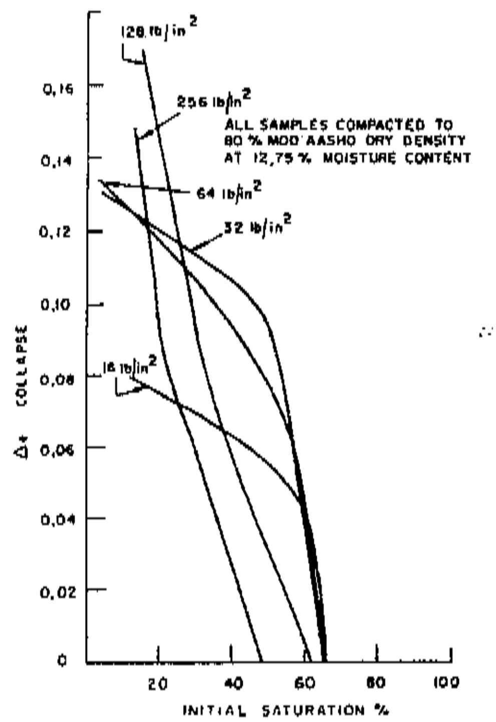

FIGURE 24

Soil A — collapse at various loads versus saturation.

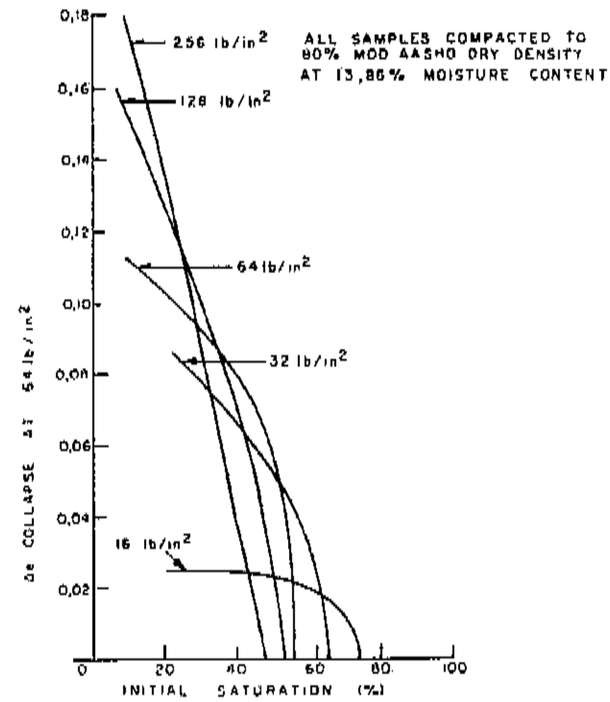

FIGURE 25

Soil D — collapse at various loads versus initial saturation.

CSIR Research Report 324, NITRR Bulletin 13, Pretoria, South Africa 1977

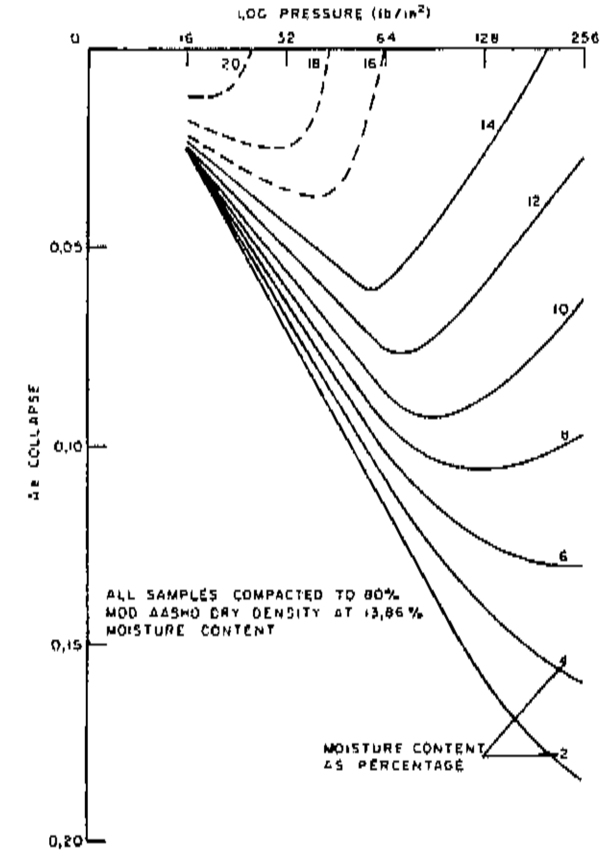

FIGURE 26

Soil D — collapse versus log pressure curves for various moisture contents.

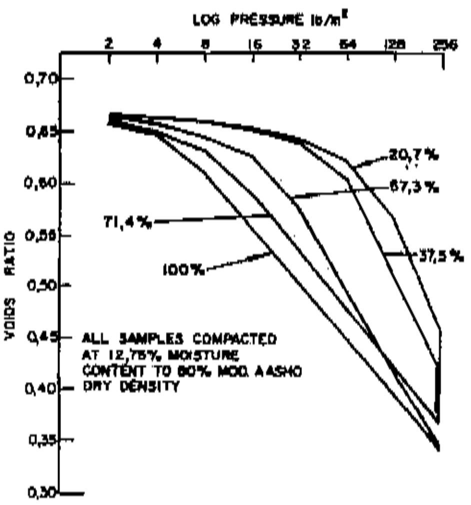

FIGURE 27

Soil A — e versus log P curves for various degrees of saturation loaded to pressures of 256 lb/in².

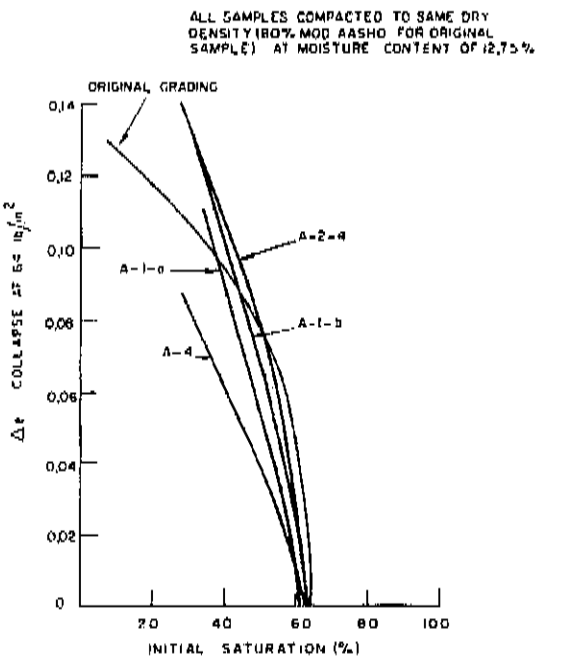

FIGURE 28

Soil A — collapse versus initial saturation for samples with different particle size distributions.

4.6 RATE OF CONSOLIDATION AND COLLAPSE

Throughout the testing reported so far in Chapter 4, measurements of consolidation were taken at intervals of time from 30 seconds upwards. This was done both for primary consolidation (in fully and partly saturated specimens) and for collapse. The average times for 90 per cent consolidation (determined by the root time method) in sets of fully saturated specimens are given in Table 6 and it can be seen that these times are very short, and do not differ much between the variables investigated. It has also been found that the partly saturated specimens consolidated up to three times more rapidly than those tested fully saturated.

It was observed that when the specimens were inundated there was a distinct 'reaction time' before collapse commenced. Time curves obtained from continuous readings are shown in Figure 31 to illustrate this.

Because there is no immediate settlement it is possible to determine the actual time for 90 per cent collapse from the plotted curves, assuming the movement in 24 hours to be 100 per cent collapse. The average times for each set of specimens are given in Table 7. These are very variable, principally because at saturations just below the critical (where the amount of collapse was small) the rate of collapse was normally very slow. The averages have been based on the mean of the square roots, but this has not eliminated the variations. The conclusion drawn from Table 7 confirms this observation, namely that where the amount of collapse was small, the rate was very slow. At high dry densities it was so slow that a realistic time for 90 per cent collapse could not be estimated. It is also interesting to note that the rate of collapse was a good deal slower than the rate of consolidation in the same specimen, presumably because some time is required before clay bridges are softened to the point of failure.

The fact remains, however, that where significant collapse occurs it takes place quickly, and the rate of collapse is therefore of minor concern. It does not quite confirm the statement (Aitchison[1]) that the rate of collapse is unimportant because it is instantaneous or of short duration.

4.7 STRUCTURE OF COLLAPSING SOIL

In order to learn more about the reasons for soil behaviour it has become fairly common to examine the soil structure microscopically, and this approach is equally valid with collapsing soils. Early studies were made using optical microscopes (Knight[23, 24]) but more recently the development of the scanning electron microscope has made more satisfactory observations possible (Smart[37]; Barden[3] and Barden *et al*[4]). Specimens of the compacted soils used in this work have been examined by means of this technique and photographs have been taken to illustrate the findings.

It was found that in compacted soil the finer silt and clay particles were distributed all over the coarser sand grains, as shown in Plate 7 for instance. No particular difference was observed between dense and less dense soils, except for the obviously greater interparticle voids in the latter. It was, however, noted that in soils compacted at a high moisture content the distribution of fine particles tended to be uniform, whereas in soil compacted very dry these aggregated in clumps. These two conditions are illustrated in Plates 8 and 9 respectively, and probably account for the differences in behaviour discussed in Section 4.3.

It is significant that the clay bridges (Knight[24]) were not found in the compacted soils. By way of comparison, undisturbed specimens of Soils E and F, which are naturally-occurring collapsing sands, were examined. Clay bridges were found, such as the one shown in Plate 10, but when the same soils were recompacted to the same dry density the clay bridges had disappeared (Plate 11). There is clearly a fundamental difference in structure between natural and recompacted collapsing soil. (See Plates 8 to 12.)

It was suggested by Knight[24, 25] that the clay bridges are formed during the seasonal cycles of wetting and drying that occur in soils in the climate of southern Africa. An attempt was therefore made to model this in the laboratory. Soils E and F were compacted in the oedometer rings in the usual way to the same dry density as the undisturbed samples. Wetting was achieved by standing the specimen, secured between two porous discs, in a bowl and allowing water to rise slowly to saturate it. The drying part of the cycle consisted of allowing the water level to fall again slowly, after which the specimen was placed for at least twelve hours in an oven at 50 to 60 °C. Sets of specimens were subjected to different numbers of successive cycles, and incipient clay bridges were observed after 56 cycles (Plate 12). Sets of oedometer tests were also carried out, giving the results in Figures 32 and 33. Surprisingly it was found that, particularly for Soil E, collapse was greater in the soil after recompaction than in the undisturbed state. Figures 32 and 33 show how the behaviour of the recompacted specimens tends to change towards the natural soil as the number of successive wetting and drying cycles increases. These effects are much more marked for Soil E than for Soil F.

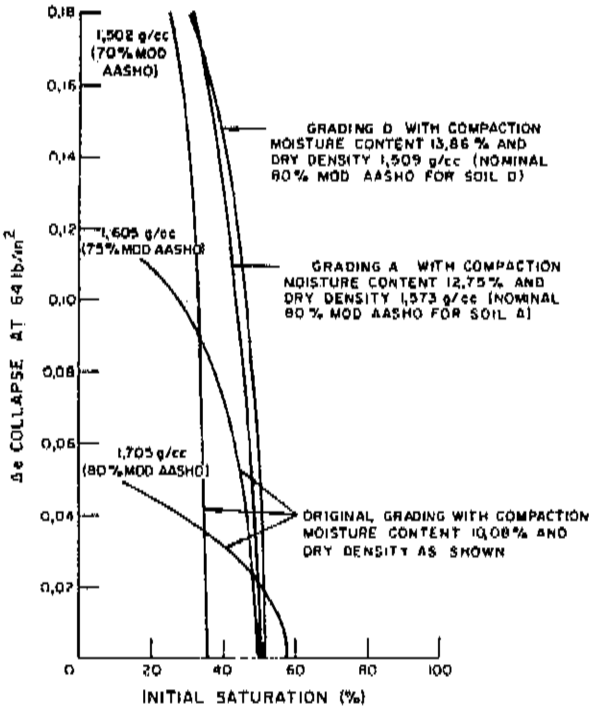

FIGURE 29

Soil B — collapse versus saturation for regraded soil.

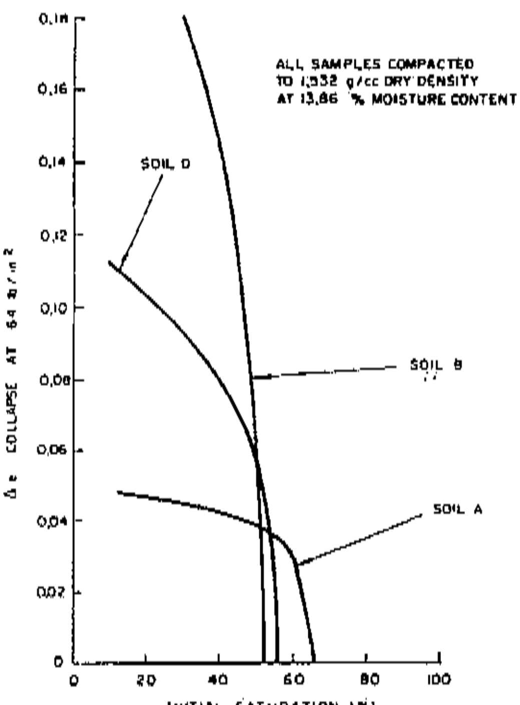

FIGURE 30

Grading D — collapse versus saturation for three soils at same dry density and compaction moisture content.

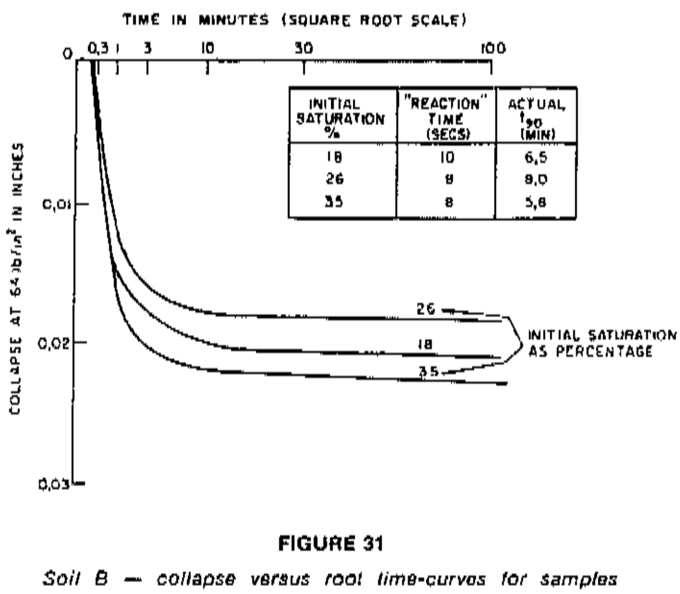

FIGURE 31

Soil B — collapse versus root time-curves for samples compacted to 80 per cent mod. AASHO dry density at 10,08 per cent moisture content — plotted from continuous readings.

TABLE 6

Average time (in minutes) for 90 per cent consolidation in fully saturated samples.

Variable	Soil		
	A	B	D
Initial dry density (relative to mod. AASHO maximum)			
70 %	1,5	1,5	1,2
75 %	1,7	1,0	1,4
80 %	1,7	1,6	1,7
85 %	0,9	0,9	1,1
90 %	1,0	1,6	1,5
Compaction saturation (percentage at 80 per cent mod. AASHO)			
16,7 %	1,0	1,0	1,5
33,3 %	1,7	1,3	2,9
50,0 %	1,7	1,6	1,7
66,7 %	2,3	1,0	2,5
Applied pressure			
16 lb/in²	1,5	1,2	1,6
32 lb/in²	1,3	1,1	1,6
64 lb/in²	1,7	1,6	1,7
128 lb/in²	0,9	1,0	1,2
256 lb/in²	1,0	1,0	1,0

TABLE 7

Average times (in minutes) for 90 per cent collapse.

Variable	Soil		
	A	B	D
Initial dry density (relative to mod. AASHO maximum)			
70 %	6,0	8,2	3.3
75 %	5,7	11.4	4,5
80 %	7,4	15,9	7.4
85 %	>120	-	52.2
90 %	-	-	-
Compaction saturation (percentage at 80 per cent mod. AASHO)			
16,7 %	8,5	24.5	4,7
33.3 %	11,9	14.6	5.3
50,0 %	7.4	15.9	7,4
66.7 %	43,0	22.6	8,3
Applied pressure at collapse			
16 lb/in²	20.2	-	18,5
32 lb/in²	13.7	-	6,4
64 lb/in²	7,4	15.9	7.4
128 lb/in²	18,3	16.4	9,7
256 lb/in²	16,2	-	7.3

NOTE These times are based on mean of square roots

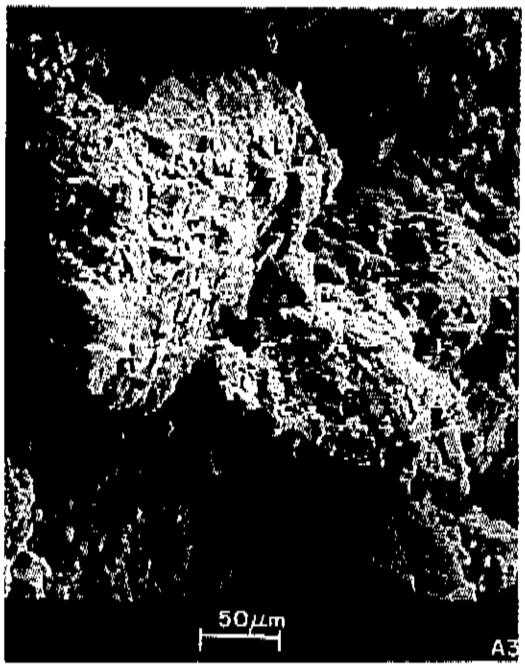

PLATE 7

Soil A compacted to 80 per cent mod. AASHO dry density at 12,75 per cent moisture content.

x250 enlargement showing clay particles adhering to sand grains.

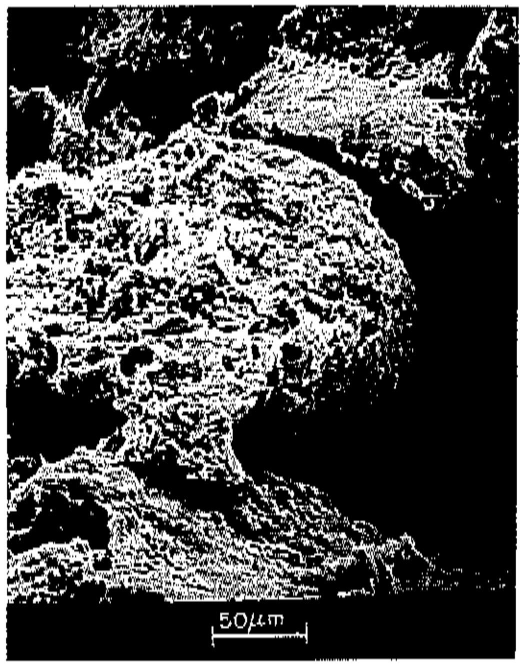

PLATE 8

Soil B compacted to 80 per cent mod. AASHO dry density at 13,44 per cent moisture content.

x300 enlargement showing clay particles adhering to sand grains.

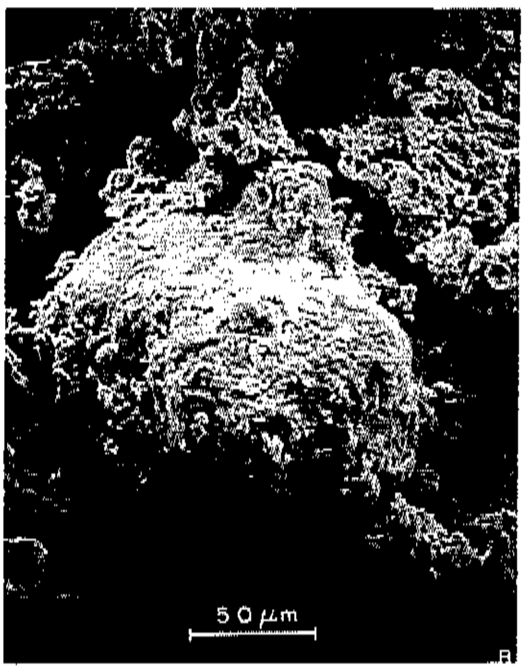

PLATE 9

Soil B compacted to 80 per cent mod. AASHO dry density at 3,36 per cent moisture content.

x400 enlargement showing clay particles tending to aggregate in clumps.

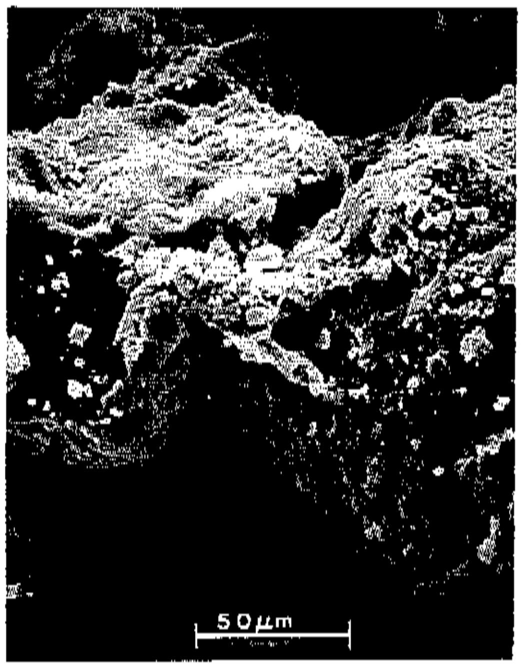

PLATE 10

Undisturbed sample of Soil E.

x 500 enlargement showing clay bridge between sand grains.

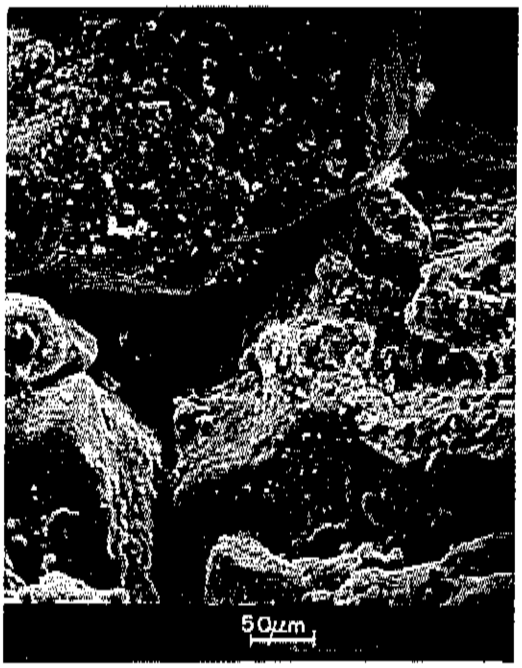

PLATE 11

Soil E recompacted to 1,6 g/cc dry density at 7,5 per cent moisture content.

x 200 enlargement showing distribution of fine particles on sand grains.

PLATE 12

Soil E recompacted to 1,6 g/cc dry density at 7,5 per cent moisture content and subjected to 56 cycles of wetting and drying.

x200 enlargement showing possible movement of fine particles into voids between sand grains.

5. PRINCIPLES OF COMPACTION OF SOIL

Compaction may be defined as the application of mechanical energy to the soil as a result of which the particles are rearranged in such a way that the density is increased. The principles of compaction are not always properly understood, and a brief review of them will therefore be given. These principles have been known for many years, but are seriously neglected in the standard soil mechanics text-books.

It is also necessary to stress that there are differences between the compaction of soil in an embankment and the compaction of a road pavement. In the latter case selected granular materials are laid in thin layers so that a high degree of control over the compaction is possible. Embankment material, on the other hand, is often anything but ideal, and a good deal of thought has to be given to the best method of compacting it.

5.1 MECHANISM OF COMPACTION

The mechanism of compaction depends on the nature of the soil. In cohensionless soils (sands and gravels) the compactive effort causes a mechanical rearrangement of the soil particles into a denser packing. A limited amount of water assists this process, 'lubricating' the coarse particles (Proctor[35]). This concept is not strictly correct for a clay soil where the particles have a physico-chemical bonding rather than a purely physical structure (Lambe[26]). For most fill materials, which are neither entirely cohesionless nor pure clay, the truth lies somewhere in between. However, it will be more convenient here to consider qualitatively the mechanism for a pure clay.

Without going into details of the physico-chemistry, the compaction process can be explained (Lambe[26]) as follows. At low moisture contents the orientation of the soil particles is initially random, because of flocculation. This occurs because there are insufficient water molecules to neutralize all the electrical charges. In this condition the density is low. As moisture is added, more of these charges are neutralized and the structure becomes less flocculated and the clay particles become partially orientated. As a result, the density increases to a maximum. As yet more water is added the clay particles become fully orientated, but they are forced further and further apart to make room for the additional adsorbed water, thus again reducing the density of the soil. This process is illustrated schematically in Figure 34, in terms of the familiar moisture versus density curve.

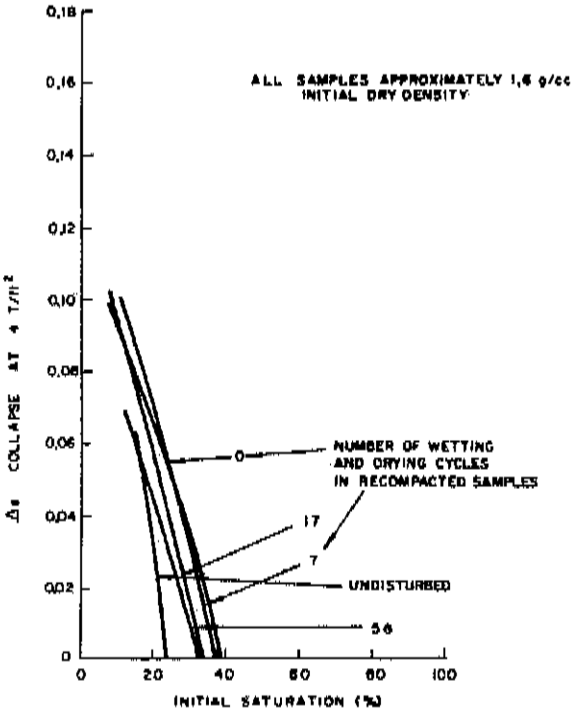

FIGURE 32

Soil E — collapse versus saturation for samples subjected to different numbers of wetting and drying cycles.

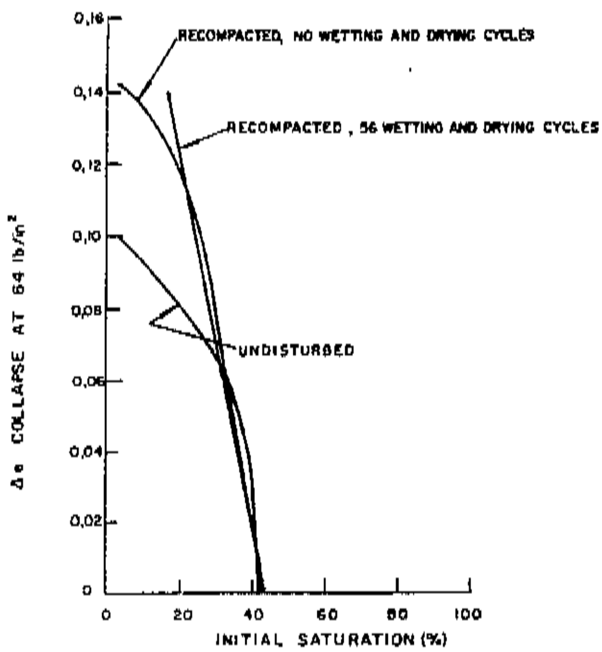

FIGURE 33

Soil F — collapse versus saturation for undisturbed soil, recompacted soil and soil subjected to wetting and drying cycles.

5.2 TYPES OF COMPACTIVE EFFORT

Soil may be compacted either in the laboratory or in the field by applying energy by one of several means. In many cases in practice the energy applied is a combination of two types; for instance in the case of a vibrating roller, the energy is applied as static pressure as well as vibratory energy.

The principal types of compactive effort are:

(a) **Dynamic compaction.** This is used in most standard laboratory compaction tests. The commonest of these tests are the Proctor (or B.S. Light or standard AASHO) test and the modified AASHO (or B.S. Heavy) test. There are a number of other versions of the basic dynamic compaction test. They all involve different amounts of energy applied to a given volume of soil. In the field, dynamic compaction is rarely used because of the movement required to apply sufficient energy to the much larger volume of soil involved. Examples of the plant used in the field for dynamic compaction are the frog hammer (for small volumes of soil), and the impact roller.

(b) **Static compaction.** This is considered the best method for the preparation of uniform laboratory specimens (Gau and Olson[16]). Although in the field this method is seldom used on its own, almost all other types of field compaction include static compaction. An example of pure static compaction would be surcharge loads applied to an embankment to expedite consolidation.

(c) **Cyclic applications of static loads.** This is probably the commonest method used for field compaction. It is the only way energy is applied when using smooth-wheeled rollers, and is also a factor in all other rolling techniques.

(d) **Vibratory compaction.** This type of compaction is usually only applied to soils which are virtually cohesionless, either in the field or in the laboratory. Vibratory methods are as a rule not suited to cohesive soils.

(e) **Kneading compaction.** This type of compaction is the one usually used to compact clay. Sheepsfoot or grid rollers, and to a lesser extent pneumatic-tyred rollers, employ this principle. Kneading is also used in some laboratories for the preparation of clay specimens for testing. The kneading action, used to rework the clay, takes place under static pressure; thus the old bonding system is destroyed and replaced by one that is better orientated. It is not a method recommended for compacting cohesionless soils.

5.3 MOISTURE-DENSITY RELATIONSHIP

The variation in dry density with moisture content in a laboratory compaction test (as already illustrated in Figure 34) is familiar to most engineers. Figure 35 shows the moisture density relationship for Soil A compacted at three different compactive efforts. It comes as no surprise to find that the maximum dry density achieved is increased by applying more energy. It may not be so widely known, however, that the optimum moisture content (the moisture content at which the maximum dry density is achieved) is lower for higher compactive efforts.

The compactive effort applied in the field also varies from one type of plant to another and different compactive efforts are required for layers of different thickness. If the thickness of the soil layer is kept constant, it is possible to plot moisture-density curves for various items of plant, in which case the nature of the plant is the only variable. An example of this is shown in Figure 36, taken from a report (Williams and Maclean[43]) where many such relationships are given. The maximum dry density and optimum moisture content vary in exactly the same way as they did for different compactive efforts in the laboratory test.

5.4 FACTORS AFFECTING FIELD COMPACTION

It is widely believed that if a layer of soil is rolled often enough, the specified density can be achieved. Examination of Figure 36 shows that this will not be achieved if the wrong plant is chosen. It has also been shown (Williams and Maclean[43]) that there comes a point where further rolling of a layer of soil achieves virtually no increase in its density. This is illustrated in Figure 37 in terms of variation in moisture content for one item of plant, and in Figure 38 for various types of plant each of which was working at its own particular optimum moisture content.

It has also been shown (Lewis[27]) in full-scale field trials that the maximum density which can be achieved for each soil, moisture content and item of plant varies with depth. Figure 39 shows an example of this variation in dry density with depth. It is therefore apparent that the thickness of the layer being compacted must not exceed a certain maximum, as beyond this thickness the specified density will not be achieved at the bottom of the layer.

A quarter of a century of study of this subject at the Transport and Road Research Laboratory in Great Britain has resulted in the publication of a standard specification for compaction (Ministry of Transport[31]). This is written in terms of the type of compaction plant, the intensity of static loading and the nature of the soil. The maximum depth of the compacted layer ranges from 3 in to 12 in (75 mm to 300 mm) and the minimum number of passes of the roller varies from 3 to 16.

6. CONCLUSIONS AND RECOMMENDATIONS

The work reported in this Bulletin has shown that the initial compaction of soil has an important bearing on its subsequent behaviour in road embankments.

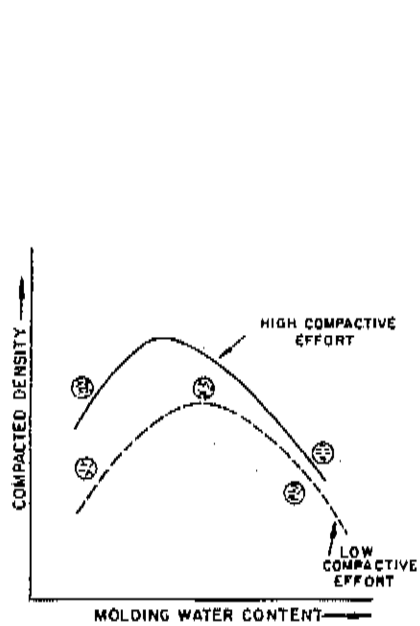

FIGURE 34

Effects of compaction on orientation of clay particles

(After Lambe[26])

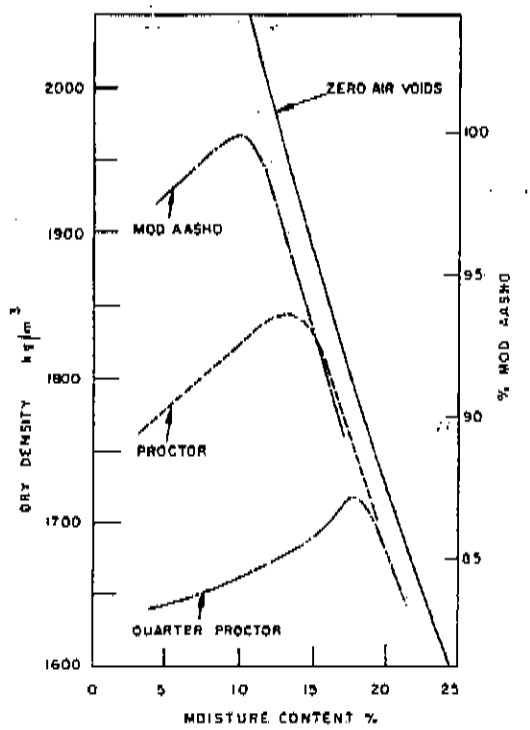

FIGURE 35

Soil A — moisture versus density curves for three compactive efforts.

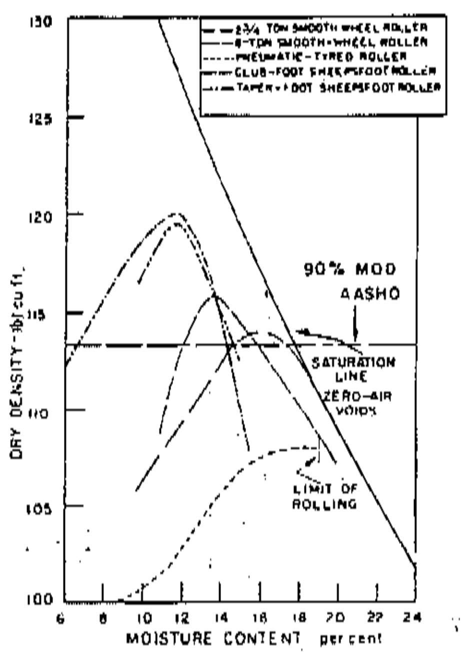

FIGURE 36

Effect of moisture content on dry density of sandy clay compacted by different types of plant

(After Williams and Maclean[43])

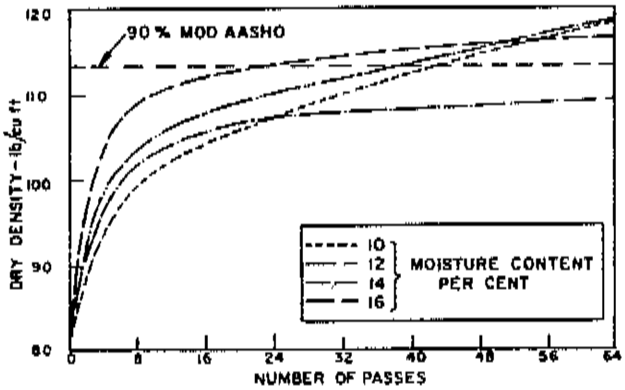

FIGURE 37

Relationship between dry density and number of passes of taper-foot sheepsfoot roller for sandy clay soil.

(After Williams and Maclean[43])

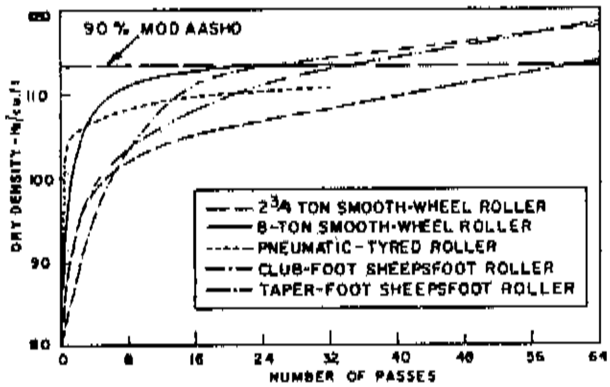

FIGURE 38

Effect of number of passes on dry density of sandy clay compacted at optimum moisture content for the plant.

(After Williams and Maclean[43])

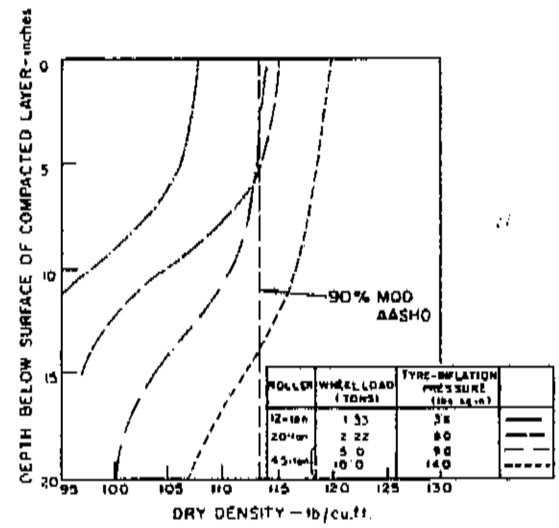

FIGURE 39

Relation between dry density and depth below the surface of sandy clay compacted by pneumatic-tyred rollers.

(After Lewis[27])

Variations in the moisture content of the soil during compaction and in the dry density achieved will determine the structure of the compacted soil mass. This in turn will affect the settlement that is likely to occur under any given set of circumstances, and the maximum shear strength that can be developed.

It is necessary to consider how compaction affects the soil behaviour, what is the ideal post-construction condition to strive for, and what steps can be taken during construction to achieve this end.

The main conclusions reached during the present study which could influence construction practice are:

(a) Collapse settlement can occur in almost any soil that has a relatively low dry density and which has been loaded below a critical moisture content (Section 4.2). Collapse can be minimized providing the soil is initially compacted to a dry density that is greater than both 85 per cent mod. AASHO and 1 650 kg/m³.

(b) The critical moisture content, which is not easy to determine without a good deal of testing, varies from soil to soil and with the initial dry density. For the typical fill materials tested, the critical moisture content appears to be one or two per cent in excess of the Proctor optimum (Section 4.2). This could be used as a rule of thumb criterion particularly when the dry densities exceed 85 per cent mod. AASHO.

(c) There appears to be a tendency for specimens compacted at low moisture contents to experience both greater collapse and greater total settlement than those compacted wetter (Section 4.3). In practical terms this difference is not significant.

(d) The amount of collapse settlement does not necessarily increase with increasing pressure. For any moisture content there is a pressure above which the amount of collapse reduces (Section 4.4). This pressure is higher for soils with lower moisture contents (Figure 26).

(e) The amount of collapse depends to some extent on both the soil grading and mineralogy (Section 4.5). The relative importance of these variables is not clear, but both appear to be much less significant than the initial dry density.

(f) In fully saturated soil, the rate of collapse is several times slower than the rate of consolidation, which itself is slower than that in partly saturated soil (Section 4.6). After wetting and before collapse begins, there is also a brief reaction time which is so short that for practical purposes it can be ignored.

(g) There are differences in structure and behaviour between naturally occurring collapsing soils and the same soils when recompacted (Section 4.7). Indeed, there may be a greater tendency to collapse and higher critical saturations in the recompacted soil.

(h) Throughout the work it was observed that a soil tested fully saturated almost invariably consolidated more than the same soil loaded partly saturated and then wetted to cause collapse. This difference is of the order of ten per cent.

From a practical point of view, therefore, the most satisfactory results will be obtained if soil is compacted to a density exceeding both 85 per cent mod. AASHO and 1 650 kg/m³. The compaction moisture content should not be less than the Proctor optimum, and this should be maintained at all points in an embankment throughout the period when the load is increased.

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