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INVESTIGATION INTO PREMATURE PAVEMENT DISTRESS AT FOUR SITES IN THE CAPE PROVINCE.

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Several instances of premature pavement distress have been reported throughout the Cape Province of South Africa. Extensive investigation of four of the sites showed quite conclusively that the causes of distress emanated from the degradation of the unstabilized basecourse. Samples were obtained from both distressed and undistressed areas of the sites and compared. The results showed that the plasticity index and passing 0,075 mm fraction had increased to unacceptable limits in the distressed areas of three of the sites. At these three sites, it was shown that the degradation was associated with the presence of excess moisture and as a result of the degradation, the basecourse materials in the distressed areas were less capable of providing adequate support. The continued presence of excess moisture further lowered the bearing strength and led to the observed distress. At the fourth site, the distress was due to the low dry crushing strength of the aggregate. The action of the traffic and/or construction processes caused the weak sandstone gravel aggregate to disintegrate and form a layer of loose fines just below the surface seal, resulting in the observed distress.

At the time of construction no tests or associated limits were included in specifications to quantify the durability of the materials (i.e. their ability to resist degradation, generally in the form of disintegration). Therefore, the wet and dry 10 % Fines Aggregate Crushing Test (10 % FACT) and the Texas Ball Mill (TBM) test were investigated as tests likely to be suitable for the early identification of suspect materials. From this work climate-related limits are proposed for the 10 % FACT test and major modifications are proposed for the TBM test, along with tentative limits. These limits are based on the actual performance of the few aggregates tested in the investigation.

1. INTRODUCTION

The majority of pavements in the Cape Province are constructed of unstabilized granular aggregates covered by a thin surface seal. Although most of the road network constructed in this way has performed well, several instances of premature pavement distress have recently been reported.

This paper reports the results of the investigation into the causes of the distress and recommends tests most suitable for identifying problem materials in the future. Limits and modifications to the tests are tentatively proposed.

2. BACKGROUND

An unpublished initial investigation of eleven sites throughout the Cape Province carried out by the Provincial Roads Department, concluded that the distress in the form of severe rutting and crocodile cracking, emanated from the degradation of the base-course. In the Cape this is normally a G2 (draft TRH14, 1980) material, i.e. the addition of an approved soil binder to the crushed rock is permitted. Comparison of the results from the completion data with the results from the material subsequently obtained from distressed areas showed that the percentage fines passing 0,075 mm (P075) and the plasticity index (PI) had both increased. It was also reported that the basecourses in the distressed areas were wet, especially near the top.

The initial investigation also showed that the specification at the time of construction did not include limits for identifying material susceptible to breakdown or degradation.

Four of the eleven sites: TR9/8 (Hanover to Richmond), TR17/2 (Hanover to De Aar), TR45/3 (Port Alfred to Lover's Twist) and TR2/2 (Bot River to Krige Station), were investigated further. At each site a distressed and a sound area was selected so that any variation in material properties could be identified. Dynamic Cone Penetrations (DCPs) and air permeability tests of the surfacing were carried out along with extensive sampling of the basecourse in each section. The laboratory testing included moisture contents (m/c), Atterberg Limits, gradings, maximum dry density (MDD)/optimum moisture content (OMC) relationships, and California bearing ratios (CBRs) at different moisture contents.

3. IDENTIFICATION OF SOURCE OF FAILURE

The source of failure was identified by DCP results, changes in material properties, differences in moisture contents and their effects on bearing strength.

3.1 D C P results

Layer strength diagrams from the DCP tests (in accordance with Kleyn et al, 1982) were inconclusive in identifying the problem layer(s). Only one of the four sites (17/2) gave a CBR strength predicted from the DCP (DCP CBR) of less than the specified soaked requirement for any of the layers. At this site, the DCP CBR in the distressed basecourse, for the full depth, was 80, whilst the worst 50 mm between 30 mm and 80 mm, gave a CBR as low as 50 (Table 1) at the in-situ moisture content at the time of investigation.

TABLE 1: BASECOURSE CBR VALUES FROM THE DCP LAYER STRENGTH DIAGRAMS

Site	Average DCP CBR (%) for full depth of basecourse		DCP CBR (%) for the worst 50 mm depth of basecourse	
	Distressed	Sound	Distressed	Sound
9/8	250	280	150	180
17/2	80	125	50	80
45/3	170	185	120	150
2/2	>250	>250	185	>250

All other pavement layers gave DCP CBRs well in excess of the soaked specification requirements. However, Table 1 shows that in every case, the CBRs for the worst 50 mm of basecourse in the distressed areas are all lower than those for the corresponding sound areas. The results therefore show some indication that the problem layer could be the basecourse.

It is important to realise that the DCP only measures the strength of each layer of the pavement for the moisture content at that moment in time. If the basecourse moisture content has been higher at some time in the past, giving a lower bearing strength, the DCP would not have detected this. Elsewhere in the paper it is suggested that this was probably the case at sites 9/8 and 45/3, which are shown to be "acceptable" by the DCP results.

3.2 Material Properties

The basecourse specification at the time of construction required, inter alia, a PI not greater than 6 and the P075 fraction to be between 4 and 12 percent. Table 2 shows that at three of the distressed sites, the PIs were unacceptably high, while the sound areas were still acceptable. At two of the distressed sites, the P075 was also unacceptable. The results from the completion data at the sites showed the materials to be within specification and in close agreement with the results of the sound areas.

At three sites (9/8, 17/2 and 45/3) the gradings of the distressed materials were always finer than those of the sound materials, although, in general, they still remained within the specification envelope. It was only on the finer sieves, and especially the 0,075 mm, that the specified limit was exceeded.

The results from Site 2/2 suggest that there is no difference between the distressed and sound areas and that both are satisfactory. However, the results are slightly misleading as a thin layer of loose fines was observed at the interface of the basecourse and the surface seal in both distressed and sound areas. As this layer of fines was a relatively small proportion of the whole grading, it is not reflected in the P075 results in Table 2.

TABLE 2: COMPARISON OF MEAN PI AND P075 FOR THE DISTRESSED AND UNDISTRESSED AREAS

Site	No of Results	PI		P075 (%)		Material Type
		Distressed	Sound	Distressed	Sound	
9/8*	4	8	5	17	9	Beaufort Sandstone
17/2	4	10	6	15	8	Dolerite
45/3	4	8	4	8	7	Tillite
2/2	4	NP	NP	5	5	Sandstone gravel

* The results are for the sandstone used in the top 100 mm of the basecourse.

3.3 Moisture contents

The basecourse moisture contents were generally higher in the distressed areas than in the sound areas (Table 3).

TABLE 3: COMPARISON OF FIELD MOISTURE CONTENTS (FMC) WITH OPTIMUM MOISTURE CONTENT (OMC)

	Average FMC of basecourse (%) (2 results)		OMC (%)*		FMC/OMC	
Site	Distressed	Sound	Distressed	Sound	Distressed	Sound
17/2	7,2	5,0	8,0	8,0	0,90	0,62
9/8	5,2	4,5	7,3	7,2	0,71	0,62
45/3	5,5	4,3	6,5	6,9	0,85	0,62
2/2	2,7	2,8	7,0	8,2	0,39	0,22

* All OMCs were taken at MOD AASHTO compaction

Three of the sound sites have FMC/OMC ratios of 0,62 whereas the corresponding distressed site ratios are between 0,71 and 0,90. For the basecourse under normal conditions, one would expect an FMC/OMC ratio of about 0,6 in all climatic areas of southern Africa (Emery, 1984 - Table VI). The higher FMC/OMC ratios of the distressed sites are indicative of excess moisture and/or lower grade material (m/c being highly correlated with material type). Reports and observations suggest that excess moisture was the more likely cause of the distress.

At Site 9/8 the drainage was reportedly poor before the distress with standing water in the side drains. An investigation by consultants in 1982, prior to the improvement of the drainage, showed that the basecourse moisture content was 5,9 % and higher than that shown in Table 3 for the distressed area. The FMC/OMC ratio would then have been 0,82 and more consistent with the FMC/OMC ratios of the other distressed sites. It appears that the improvement of the drainage has reduced the FMC to a more acceptable value and, at this moisture content, the material is capable of giving the acceptable strengths shown by the DCP results mentioned earlier.

At site 17/2 there was a different problem; here the moisture was reportedly trapped into the pavement. ~~It appears that the PI of the~~ original dolerite used became unacceptably high. In an attempt to reduce the PI the basecourse was slushed with water over a period of approximately three weeks. The pavement was then surfaced only two days after the slushing operation, and excess moisture was almost certainly trapped in the pavement. This does not represent normal practice in the Cape Province, which currently require granular bases to be at 0,5 of OMC prior to sealing.

Site 45/3 had a drainage problem caused by a "boxed-in" type of construction. During construction, it was reported that the basecourse was wet and rutting under site traffic. Remedial herringbone, no-fines concrete drains were built into the basecourse so that water could be drained through the relatively impermeable unsurfaced shoulder. However, the drains became silted and the outlets partially blocked. The result was that water was trapped in the pavement in the outer wheel track, where the worst distress occurred. An unpublished investigation by the Cape Provincial Roads department in 1982 reported that free running water was flowing between the basecourse and the surfacing. This was not observed at the time of this investigation. However, this shows that at some stage in the past, the basecourse was much wetter. This would therefore explain the acceptable DCP CBRs obtained from this investigation at a lower moisture content.

Site 2/2 was significantly drier and excess moisture was not thought to be the cause of failure.

3.4 Effects of moisture on the CBR

Once it was established that excess moisture was present in three of the basecourses at some stage in their service lives, the effects of excess moisture on the bearing strength of the materials were investigated.

In an effort to simulate the likely site compaction and moisture history of the materials, all samples were compacted at their respective Mod. AASHTO OMCs using the three compactive efforts specified by the National Institute for Transport and Road Research (NITRR, 1979 - Method A8), and the samples tested at OMC, 1/2 OMC and 1/4 OMC. The 1/2 OMC and 1/4 OMC moisture contents were obtained by drying back the samples for each compactive effort at 50 °C and equilibrating in sealed plastic bags for 48 hours. After testing, the 2,54 mm CBRs were obtained for 98 percent compaction (Table 4) and plotted against moisture content (Figure 1).

Figure 1 shows that the sound materials, which have moisture contents at the expected equilibrium (i.e. 0,62 of OMC - Table 3), give CBR values well in excess of the 80 specified at the soaked moisture contents. However, the field moisture contents at the distressed sites all give lower CBRs, with those at Sites 17/2 and 45/3 being very close to the 80 limit. The improvement in the drainage at 9/8 has reduced the moisture content and this has improved the bearing strength of the material.

It is also shown that an increase in moisture content in the distressed areas may reduce the bearing strengths to below the soaked CBR specified limit of 80. Table 4 shows that the soaked CBRs, on material from the distressed areas, are all now below 80. The completion data after construction showed that the materials gave acceptable CBRs at 98 percent Mod. AASHTO compaction, although

TABLE 4 : RESULTS OF THE EFFECTS OF MOISTURE CONTENT ON CBR

Site	Condition	Approximate moisture content	98 % CBR	Average m/c top 30 mm**
9/8	Distressed	Soaked	74*	-
		OMC	86	7,3
		1/2 OMC	180	3,8
		1/4 OMC	370	1,9
	Sound	Soaked	91*	-
		OMC	112	7,4
		1/2 OMC	223	3,2
		1/4 OMC	309	1,6
17/2	Distressed	Soaked	60*	-
		OMC	72	7,6
		1/2 OMC	193	3,9
		1/4 OMC	358	2,0
	Sound	Soaked	107*	-
		OMC	143	6,2
		1/2 OMC	410	3,3
		1/4 OMC	>218	1,9
45/3	Distressed	Soaked	62*	-
		OMC	104	5,1
		1/2 OMC	319	2,2
		1/4 OMC	380	1,6
	Sound	Soaked	89*	-
		OMC		6,2
		1/2 OMC	192	3,0
		1/4 OMC	305	2,1

* Average of two results from different laboratories.
** No soaked moisture contents were recorded, i.e. at 98 % MAASHTO density

no results were available for the sandstone used at 9/8.

However, Figure 1 only serves to estimate the moisture effects on the CBR at the minimum specified compaction requirement (i.e. 98 percent of MDD). If the site densities are higher, then the in-situ CBRs are likely to be higher. Site densities at the time of this investigation ranged between 101 percent and 106 percent of Mod. AASHTO compaction and for the distressed areas were, in fact,

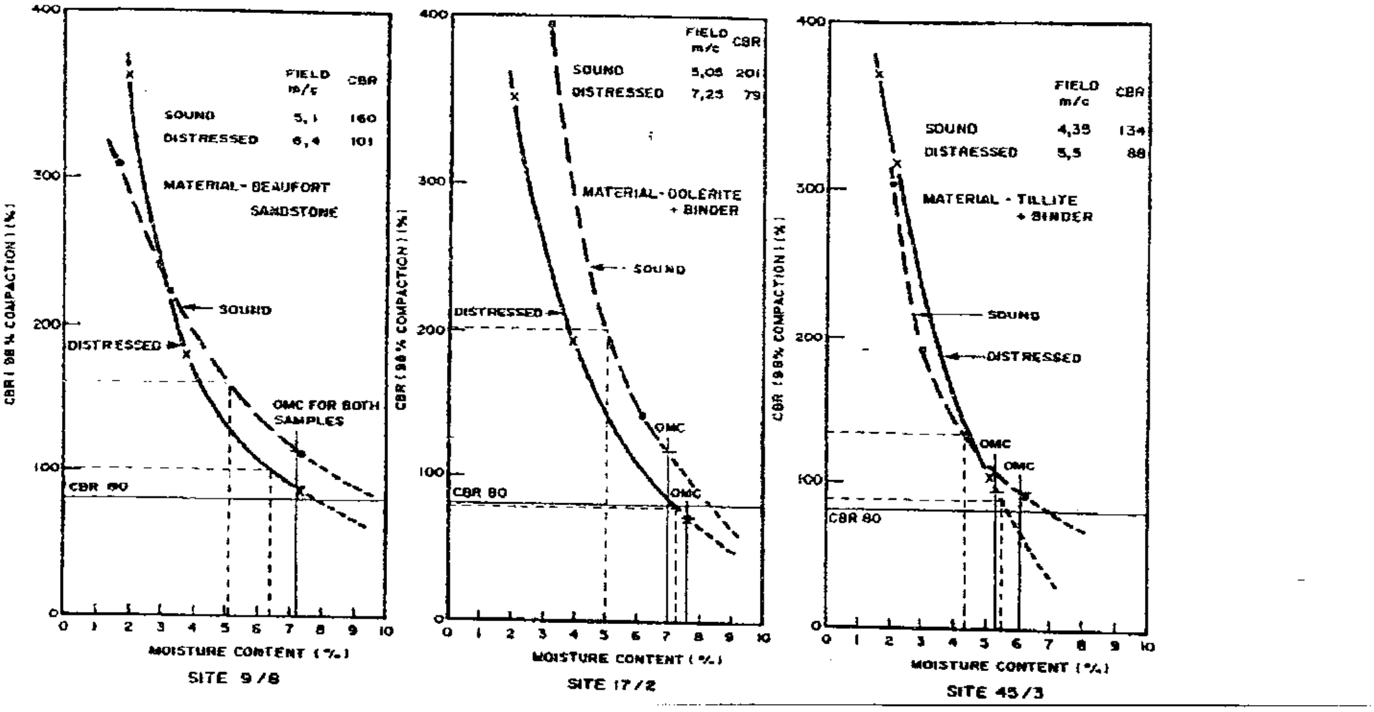

FIGURE 1 - *98% CBR VERSUS MOISTURE CONTENT RELATIONSHIP*
(all samples compacted at MOD. AASHO OMC)

all higher than those shown in the completion data, which, for the three sites, ranged between 98,0 percent to 99,6 percent.

The Mod. AASHTO MDDs of the material now present in the distressed basecourses were also all slightly higher (by up to 70 kg/m³) than the MDDs at completion (see footnote Table 5). Nevertheless, the present densities suggest that densification of the distressed base course layers has taken place. This may therefore explain the high DCP CBRs shown in Table 1 compared with those shown in Figure 1 at 98 percent compaction.

Figure 2 shows the CBR/moisture relationship at the field densities and Table 5 gives a comparison of the CBR results at the field moisture contents for 98 percent compaction, field compaction and from the DCP.

Table 5 shows that only two areas, 17/2 distressed and 45/3 sound give agreeable results between the laboratory CBR at in-situ compaction and moisture content, and the DCP CBR. In the other areas the general trend is for the DCP CBR to be higher.

TABLE 5 : COMPARISON OF LABORATORY AND DCP CBRs AT SITE MOISTURE CONTENT

	Distressed			Sound		
Site	CBR at 98 % Compaction %	CBR at In situ Compaction %	In situ DCP CBR %	CBR at 98 % Compaction %	CBR at in situ Compaction %	In situ DCP CBR %
9/8	100	130	250	160	200	280
17/2	80	85	80	200	270	125
45/3	90	110	170	130	180	180

Notes: Site densities and % compactions were as follows compared with nearest available data at completion:

		Field Density Investigation	(kg/m^3) Completion	MDD (kg/m^3) Investigation	Completion	Compaction Investigation	% Completion
9/8	distressed	2291	2108	2166	2151	105,8	98,0
	sound	2221	2300	2166	2137	102,5	107,6
17/2	distressed	2303	2210	2239	2220	102,9	99,6
	sound	2301	2300	2267	2220	101,4	103,6
45/3	distressed	2302	2170	2274	2205	101,2	98,4
	sound	2301	2170	2267	2205	101,5	98,4

However, the CBR versus moisture content relationships shown in Figure 2 relate well with the actual observed performance of the materials. It is shown that at two of the sites (17/2 and 45/3) only a slight increase in moisture is required to reduce the CBR of the distressed area to unacceptable limits, even at the higher densities.

The results at 9/8 suggest that the material is acceptable, but this is probably a reflection of the densification of the layer, presumably as a result of traffic on the degraded materials. It is likely that the increase in density in the wheeltrack has caused the observed distress of rutting and crocodile cracking, especially prior to improvement of the drainage when the moisture content was higher.

The DCP CBRs taken in the wheeltracks did not compare well with performance when explaining the observed distress. The investigation showed the DCP to be a very useful tool in comparing the relative strengths of the distressed and sound areas, and it was capable of detecting relative weaknesses within a particular layer. However, this investigation suggests that further in-situ DCP-CBR correlations are required on materials with CBRs in excess of 80 before the DCP can be confidently used to predict in-situ CBRs of basecourse (G1 and G2) materials. After all, the relationship devised by Kleyn (1975) and shown in the layer strength diagrams of Kleyn et al (1982), was based on laboratory results in the CBR range of 1 to 80.

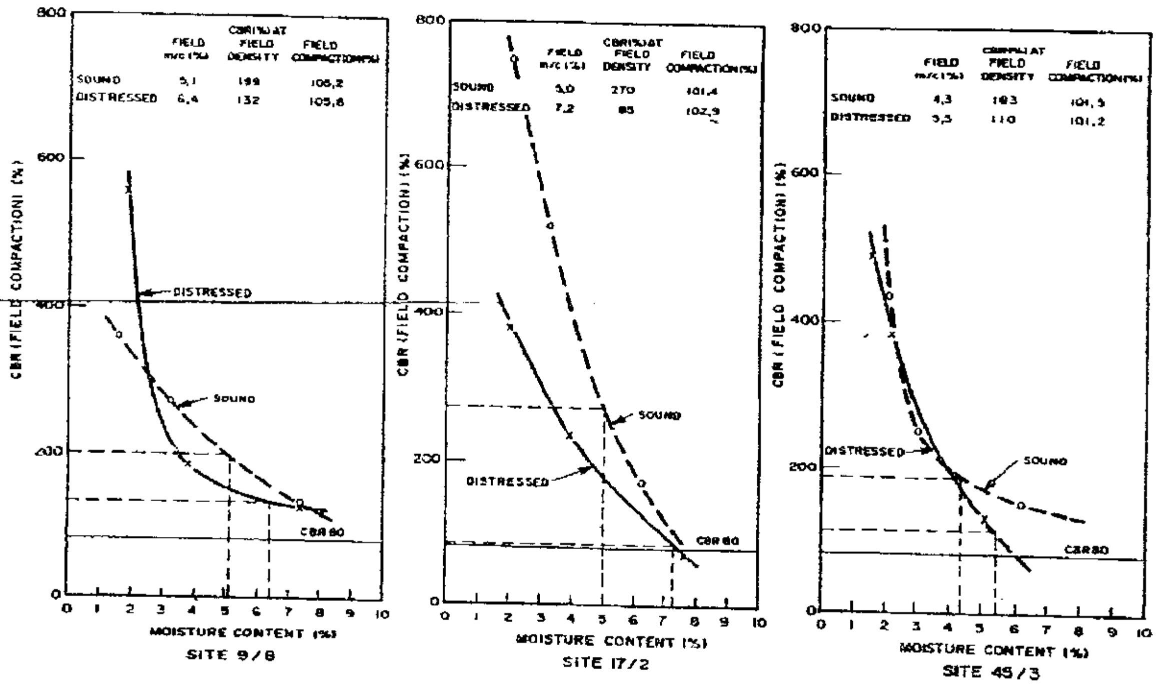

FIGURE 2 - *CBR AT FIELD COMPACTION VERSUS MOISTURE CONTENT RELATIONSHIP*

4. DEGRADATION

From the field and laboratory investigations, it was concluded that at three of the sites (9/8, 17/2 and 45/3) the cause of failure was moisture-associated in that moisture had either entered or been trapped in the basecourse. The presence of excess moisture had caused degradation of the original material shown by the change in material properties. It was also shown that the degraded material in the presence of excess moisture might have given unacceptably low bearing strength (Table 5).

The results from the sound areas show that, provided that water is kept out of the pavement and the expected equilibrium moisture content is maintained, the bearing strengths of the basecourses should remain in excess of requirements.

It was therefore concluded that the presence of excess moisture would first cause the material to degrade to unacceptable limits. The degraded material would then have a lower bearing strength under normal moisture conditions. In the presence of excess moisture there was a further reduction in bearing strength leading to the distress observed.

At Site 2/2 water does not appear to have been the cause of failure. Instead there is a thin layer of loose fines just below the surface seal at the top of the basecourse. The layer of fines appears to have been formed as a result of disintegration of the relatively weak sandstone gravel. According to Grant and Netterberg (1984), such a layer of fines could lead to excessive flexing of the surface seal leading to the observed distress.

At all four sites there was evidence that the materials in the distressed areas had degraded. However, the specification at the time of construction did not include limits for identifying degradable materials. Appropriate tests were therefore investigated.

4.1 10 % FACT wet/dry ratio

Of the durability tests at present available, only the ratio of wet/dry 10 % Fines Aggregate Crushing Test (10 % FACT) value suggested by Weinert (1980) and recommended by NITRR (1980 - draft TRH14) is in common use for testing South African aggregates. However, whilst this is a good measure of the likely reduction in strength of the aggregate, and hence the degree of disintegration likely to be expected when wet, it gives no indication of any weathering products which may be released to the detriment of the pavement.

For the four sites investigated, samples were taken from each of the quarries or borrow pits used. Wet and dry 10 % FACT tests were carried out for comparison with TRH14 (NITRR, 1980) requirements (Table 6).

TABLE 6 : 10 % FACT VALUES COMPARED WITH TRH14 LIMITS

Site	Sample No	Climate data N Value[2]	Im[3]	MAR[4]	Material type	10 % FACT Dry (kN)	Wet/dry ratio	TRH14 (1980) limits (kN/%	Remarks
9/8	10317	>10	-35	350	Sandstone	173	52	140/75[1]	Reject
9/8	10318	>10	-35	350	Dolerite	199	58	110/75	Reject
17/2	10308	>10	-35	350	Dolerite	239	87	110/75	Accept
17/2	11164	>10	-35	350	Dolerite	258	37	110/75	Reject
45/3	10321	> 5	-20	700	Tillite	228	47	160/70	Reject
2/2	10328	5-10	+20	800	Sandstone gravel	94	66	140/75	Reject

1. 140/75 = minimum dry strength of 140 kN and minimum ratio of 75 %
2. N value = Weinert's climatic index
3. Im = Thornthwaite's Moisture Index
4. MAR = Mean Average Rainfall

The results show that all the materials, except one, would have been excluded by the TRH14 limits. However, similar materials under normal moisture conditions, and in dry areas, have performed well. Adoption of the existing TRH14 limits would have led to the rejection of these materials. It is therefore proposed that the 10 % FACT limits be climate-related.

Results from 25 other quarry samples of basecourse material being used successfully throughout the Cape Province show that, of the 6 samples obtained from the drier areas, within the Weinert (1980) climatic areas N > 10, none of the aggregates achieved the minimum 10 % wet/dry ratio requirement of 75 %. It was only in the climatic areas N = < 5 that the existing TRH14 limits did not exclude materials of proven performance. Although the moisture sensitivity of the material is still a problem in the drier areas, it is likely to be less of a problem there than it is in areas of higher rainfall. Thus consideration should be given to the relaxation of the wet/dry limits in drier areas to avoid unnecessary rejection of material.

From the results presently available the following climate related limits are suggested for consideration:

Climatic N-value	10 % FACT wet dry ratio
< 5	> 75 %
5 - 10	≮ 70 %
> 10	≮ 55 %

It is however, still important that excess disintegration does not take place in the dry state. Therefore the retention of the minimum dry limits as shown in Appendix I of TRH14 (NITRR, 1980) are still recommended.

4.2 Texas Ball Mill Test

The test that seemed most capable of simulating the degradation processes was the Texas Ball Mill (TBM) test specified by the Texas Highway Department Materials and Test Division (1978). The test requires a representative sample of the whole aggregate grading to be rotated in a watertight cylinder in the saturated condition for 600 revolutions with an abrasive charge of six steel sphere included. The abrasive charge simulates the traffic action and the likely disintegration during construction, whilst the presence of excess water will simulate any weakening of the aggregate and release of detrimental weathering products. It is highly unlikely that any decomposition will take place in the ten minutes required to perform the test.

For this investigation modifications to the standard testing procedure were made. Four subsamples of the same material were used so that the varying degrees of the degradation processes could be simulated better (Table 7). After each treatment the whole sample was retained for grading and Atterberg Limit Tests for comparison with the original results.

TABLE 7 : MODIFICATIONS TO THE TEXAS BALL MILL TEST

Sub-sample No.	Treatement			Degradation Process
	Wet	Dry	Abrasive	
1	No	No	No	Original sample for comparison
2	Yes	No	Yes	Disintegration and release of existing weathering products (standard test procedure)
3	No	Yes	Yes	Disintegration only
4	Yes	No	No	Mainly release of existing weathering products with minimal disintegration.

The results of the TBM Tests with the modifications shown in Table 7 are given in Table 8. The suitability of the material shown in Table 8 is based on the following proposed limits which are partly based on the Texas Highway Department Specification (1972):-

1. The PI should still not exceed 6 after any of the TBM treatments.
2. The amount of fines passing the 0,425 mm sieve shall not exceed 40 % and shall not increase by more than 20 percentage units from the original passing 0,425 mm fraction.

Experience with arenaceous rocks of the type used at site 2/2 and 9/8 suggest that for these rock types the maximum percentage passing 0,425 mm should be reduced to 35 % with a maximum percentage unit increase of 18.

service. For example, at Sit 9/8 the modified TBM results confirm that the sandstone is weak and likely to degrade to give unacceptably high fines in the presence of water. Water is also shown to release plastic fines causing an increace in PI. However, this would not be shown in the standard TBM test and illustrates the need for modifications. In the standard test, the large disintegration producing a high percentage of non-plastic fines, would have masked the PI increase. Similar comparisons may be made for the other sites.

TABLE 8 RESULTS OF THE MODIFIED TBM TESTS ON THE QUARRY SAMPLES

Site	Sample No	Material	Treatment	P425	PI	Remarks
9/8	10317	Sandstone	Original sample	11	2	Rejects as PA
			After normal TBM	34	2	exceeds 6 and
			~~After dry TBM~~	~~16~~	~~4~~	~~P425 increase~~
			After TBM (no shot)	12	7	by 23 units
9/8	10318	Dolerite	Original sample	18	3	Acceptable
			After normal TBM	27	3	
			After dry TBM	20	3	
			After TBM (no shot)	13	3	
17/2	11164	Dolerite (quarry km 3,0)	Original sample	8	11	Reject on high PIs
			After normal TBM	22	9	
			After dry TBM	15	9	
			After TBM (no shot)	15	13	
17/2	10308	Dolerite (quarry km 13,1)	Original sample	19	3	Acceptable
			After normal TBM	25	3	
			After dry TBM	26	2	
			After TBM (no shot)	20	3	
45/3	10321	Tillite	Original sample	4	6	Marginal. If 10% FACV results are acceptable, then accept.
			After normal TBM	13	6	
			After dry TBM	7	5	
			After TBM (no shot)	8	6	
2/2	10328	Sandstone gravel	Original sample	7	NP	Reject on increase of P425
			After normal TBM	28	NP	
			After dry TBM	27	NP	
			After TBM (no shot)	N/A	-	

8. CONCLUSIONS

a. The cause of distress at three of the sites (9/8, 17/2 and 45/3) was excess moisture trapped into or allowed to enter the pavement. The excess moisture degraded the basecourse to unacceptable limits. As a result of the degradation, the basecourse materials in the distressed areas were less capable of providing adequate support. The continued presence of excess moisture further lowered the bearing strength and led to the observed distress.

b. The distress at Site 2/2 was due to the low dry strength of the aggregate. The action of the traffic and/or construction processes caused the weak sandstone gravel material to break down or disintegrate and form a layer of loose fines just below the surface seal, which resulted in the observed surface distress.

c. Consideration should be given to the use of the 10 % wet/dry FACT and the modified TBM Test to identify the materials susceptible to degradation. Climate-related limits are proposed for the 10 % FACT test and major modifications are proposed for the TBM Test along with tentative limits, based on the limited number of aggregates tested.

d. If materials susceptible to degradation must be used, good construction control and drainage are essential to prevent excess moisture from being trapped in or infiltrating into the basecourse.

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