POOR PERFORMANCE OF COUNTERMEASURES ON A ROAD ON AN EXPANSIVE CLAY ROADBED

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ABSTRACT

An investigation into the cause of the severe longitudinal cracking on a 1 km section of a road in the North West Province of South Africa showed that this was due to the highly expansive clay roadbed. Although reasonable countermeasures had been taken which had worked elsewhere, they proved insufficient. However, the damage would probably have been worse had they not been taken. It is not usually economic ─ especially for a low volume road ─ to totally eliminate the negative effects of an expansive clay roadbed. The most that is usually attempted is to reduce them to an acceptable degree and extent. In this case the attempt was unsuccessful. The reasons of the unexpectedly poor performance appear to have included an adverse combination of factors including ponding due to rain during construction, poor lateral drainage, a thick, highly expansive and unusual clay, and an unavoidable delay in sealing the initial cracking. It was recommended that the cracks be resealed on a regular basis and the drainage improved and maintained. Nothing further could be done except at considerable expense. However, cracking continued and after 18 years the section was in a poor condition. The investigation also showed up deficiencies in the laboratory test methods used for such clays.

1. INTRODUCTION

An investigation into the cause(s) of the severe longitudinal cracking of parts of the new bitumenised road P65/1 between Swartdamstad and Dikebu (Ranteberg) and road D623 to Makapanstad at the Swartdamstad intersection in the North West Province of South Africa is described. The new road P65/1 was constructed over the alignment of the existing unsealed road and the section in question completed in July/August 2006. Longitudinal cracking was first observed in February 2008 at which time some of the cracks were 30 mm wide and at least 600 mm deep. A short section of the existing bitumenised road D623 at its intersection with P65/1 was reconstructed to improve its horizontal alignment and this also suffered similar cracking.

Although the cracks were sealed and the widest filled and sealed with a geotextile and rubber-bitumen, additional cracks about 10 mm wide appeared which were also sealed. In July 2009 new but only narrower cracks were observed.

The whole road from Swartdamstad to about 2 km short of Dikebu is underlain by mudstone, sandstone, marl, and shale of the Irrigasie Formation of the Beaufort Group of the Karoo Supergroup (Geological Survey, 1978). Although weathered Beaufort mudrocks can have plasticity indices (PIs) of up to about 36 (Venter, 1983), in the Pretoria area such rocks do not usually give rise to expansive clay soils except in poorly drained areas.

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The land types present indicate the general soil cover to be mostly red, apedal, freely drained soils of low plasticity, but with localized areas of melanic (PI ≤ 32) Bonheim and vertic (PI > 32) Arcadia clay soils (Soil and Irrigation Research Institute, 1978).

2. PAVEMENT DESIGN

The roads lie in the Moderate macroclimatic region for the purpose of pavement design (COLTO, 1996) and receive a mean annual rainfall of 650 mm, mostly in summer.

2.1 Road P65/1 (Swartdamstad – Dikebu)

The Transvaal (1995) Class III pavement design intended for 0,3 – 1,0 MESA required a double seal (6,7/13,2 mm) on a cement-stabilized C3 (COLTO: 1998) quality base over a C4 subbase, and untreated G7 and G8 selected layers, all 150 mm in thickness. A fill – typically of G8 or better quality – was provided in order to ensure a height of 750–850 mm from natural ground level to the shoulder breakpoints.

The road cross-section comprised two 4,1 m wide lanes with shoulders 1,2 m wide, stabilized to a distance of 300 mm outside the seal (Figure 1).

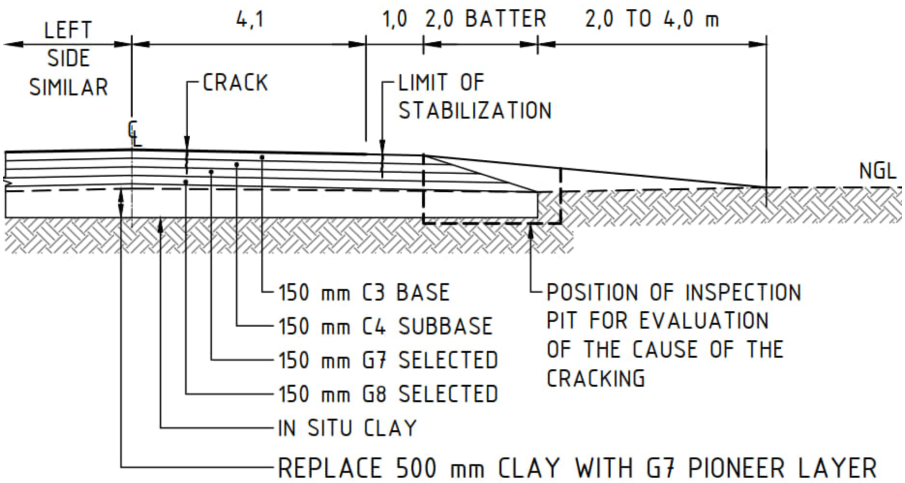

Figure 1: Typical half cross-section of P65/1

Over a 951 m section between km 9 and 10 the clay was replaced to a depth of about 300 - 500 mm below the natural ground level over the full road cross-section to the toes of the fills, i.e. a width of about 14,6 m. This clay was used to flatten the side slopes to a slope of about 1 in 6. The bottom of the excavation received normal roadbed preparation and was smooth and level, without undulations or remnants of clay. The backfill was of least G7 quality and constructed as a primer layer according to COLTO:1998 Clause 3307(c).

Precautions taken at the culverts included vee-design box culverts with vinyl-taped, waterproof joints and the provision of building-type water stops in the floor slabs.

2.2 Road D623 (Swartdamstad – Makapanstad)

The short, newly constructed section of road D623 did not wholly follow the alignment of the old road. The new section was of similar design to that of P65/1 except that no clay was replaced and therefore the side slopes were not flattened with clay.

Cracking occurred on this section both where the new alignment deviated from the old and where the formation of the new road was still located on parts of the old road.

3. SITE WORK

A visual assessment of the roads was carried out in April 2009 using TMH9:1992 and the positions of inspection pits decided upon in order to determine the cause(s) of the cracking and to check the as-built cross-section. The profiling and sampling were carried out by Matrolab in July 2009 under the supervision of the second author.

3.1 Road P65/1 (Swartdamstad – Dikebu)

Although longitudinal cracking was also seen elsewhere on this road, very severe longitudinal cracking of Degree and Extent 5 typical of roads on expansive clay roadbeds only occurred over a total length of about 700 m between about km 9 and 10 (Figure 2). (The leaning telephone poles are also an indication of severely expansive clay.) This cracking was up to about 30 mm in width and at least 600 mm deep in places before sealing. Some cracks had reopened and were spalling, and strips of road between the cracks had sunk to a depth of about 10 mm in places.

Figure 2: Longitudinal cracking on road P65/1 near km 9 after three years in April 2009

In contrast to road D623, no cracking was seen on the shoulders. Block cracking of Degree 2 and Extent 3 about 1-2 mm in width on a spacing of about 1,5 m was also present.

The terrain was almost flat with a very slight slope from right (east) to left (west). The extent of the cracking was less in the left lane and was less severe where the cover was greater.

Black clay was observed in the side drain inverts and clay was said to extend to a depth of at least 5 m in places. Although clay had been reported in only one hole of the centreline survey (which was carried out at intervals of 200 m to at least 600 mm below the surface of the existing unsealed road), the necessity for and extent of countermeasures was identified by further testing during construction.

The construction had been caught by rain during an exceptionally wet season, resulting in water ponding along a substantial length of the east side. However, as it was in a slightly lower area, the ponding was probably greatest where the cracking was now worst. The undercut for the backfill was also caught by rain in places and mitre drains were cut for drainage.

A short trench (Figure 1) was cut across the right-hand shoulder 4,4 km north of Norokie for profiling and sampling in the area of worst cracking. The purpose of this work was threefold: to check that the design – particularly the full-width partial replacement of the clay – had been correctly constructed and to test the backfill for its grading and the roadbed soil to confirm its apparently active nature.

3.2 Road D623 (Swartdamstad – Makapanstad)

This short, reconstructed section exhibited two to three longitudinal cracks of Degree and Extent 5 in the right-hand lane only and a crack 20 - 30 mm wide on the right-hand shoulder along the edge of the stabilization. The total length affected was only about 50 m.

The lane cracks were 20 - 30 mm in width before sealing and evidence of previous longitudinal cracking in the resealed portion of the old sealed road was also present. The roadbed in the side drains was black clay and houses in the area were said to be cracked.

The drainage appeared to be adequate towards the west.

It was not clear whether or not there had been partial replacement of the clay, but it appeared that the right-hand side (which exhibited the worst cracks) was on new construction and the left-hand side was over the old road. A short trench was therefore dug on the right shoulder across a 30 mm wide crack at the edge of the stabilization.

4. RESULTS

The trenches started at about the shoulder breakpoint, extended for at least 1,5 m towards the toe of the fill, and were about 1,0 m deep or just into the in-situ soil at the shoulder breakpoint. A hand-auger was then used from the bottom of the trench to obtain disturbed samples from the clay roadbed.

The backfill on P65/1 was found to extend the full length of the trench and was not found in a hole about 3 m from the shoulder breakpoint. The clay in this hole was intact.

A summary of the most relevant laboratory soil indicator test results together with their classifications and the results of repeat testing is shown in Tables 1 and 2.

The test methods used were as follows:

- soil fines (P425) preparation for index tests (LL, PI and LS) and hydrometer analysis: TMH 1 Method A-1(a) (1986) – this involves boiling and drying at 105 - 110°C;
- index tests : TMH 1 Methods A2, A3 and A4;
- grading : ASTM D 422-63 (reapproved 1990), with the following deviations:
  - oven-dried soil fines used instead of the air-dried fraction passing 2,00 mm;

- optional paddle dispersion device used instead of the preferred air dispersion device. However, freshly prepared sodium hexametaphosphate used as dispersant and slurry soaked overnight as prescribed.

Table 1: Summary of soil mechanics test results on P 65/1 site (Test Pit 1)

Depth	**Classification**		WPI	**P425**	**P002**	**Potential expansiveness**			**Pi**	**GM**	**Water content**	**Remarks**
	COL-TO	**Unified**	[3]	[4]	[4]	**K-B**	**VDM**	**Wilson**				
m	[1]	[2]	%	%	%	[5]	[6]	[7]	%		%	[8]
0,0 - 0,08	G10	(MH)	-	-	-	Yes?	Yes?	Yes?	-	-	-	Cladding
0,08 - 0,60	G5	GC-GM	1	23	3	No	Low	Low	5	2,26	6	Fill
0,60 - 1,10	G7	(GW)-GC	1	12	2	No	Low	Low	11	2,61	5	Backfill
1,10 - 1,60	G10	(MH)*	27	91	9	Yes	Low	High	30	0,36	28	In-situ
1,60 - 2,00	G10	(MH)	34	91	9	Yes	Low	V. high	37	0,32	28	In-situ
2,00 - 2,20	G10	(MH)	19	86	9	Yes	Low	Med.	22	0,49	25	In-situ
0,08 - 0,60	G5	SC	2	26	3	No	Low	Low	8	2,13	-	Repeat [9]
0,60 - 1,10	G6	GW-GM	2	17	2	No	Low	Low	10	2,50	-	Repeat [10]

Notes:
[1] **Apparent**, based on indicators only; G10 only in TRH4:1996 and TRH14: 1985
[2] ASTM D 2487-06. Bracketed tentative due to incomplete data. * Borderline CH
[3] Weighted PI = PIxP425/100
[4] P425 = % passing 425 µm; P002 = % passing 2µm
[5] Kantey & Brink (1952) for buildings
[6] Van der Merwe (1976) for buildings
[7] Wilson (1964, 1976) for buildings
[8] In-situ described as moist, black, becoming light grey with depth, stiff, uniform, intact, sandy silt
[9] Also oven-dried [10] Air-dried [11] Laboratory testing by Matrolab, Pretoria

Table 2: Summary of soil mechanics test results on D623 Site (Test Pit 2)

Depth	**Classification**		**WPI**	**P425**	**P002**	**Potential expansiveness**			**PI**	**GM**	**Water content**	**Remarks**
	COL-TO	**Unified**	[3]	[4]	[4]	**K-B**	**VDM**	**Wilson**				
m	[1]	[2]	%	%	%	[5]	[6]	[7]	%		%	[8]
0 - 0,30	G4	GC-GM	1	27	3	No	Low	Low	5	2,19	6	Layer work
0,30 - 0,90	G5	SM-SC	2	34	3	No	Low	Low	7	1,98	8	Layer work
0,90 - 1,30	G10	SC	8	38	13	Yes	Low	Low	23	1,86	12	Fill
1,30 - 1,70	G10	(MH)	22	82	7	Yes	Low	Med	27	0,51	22	In-situ
1,70 - 2,20	G10	(MH)*	33	80	8	Yes	Low	High	41	0,65	20	In-situ
2,20 - 2,90	G10	(MH)	25	89	11	Yes	Low	High	28	0,41	25	In-situ
2,90 - 3,10	G10	(MH)*	26	88	8	Yes	Low	High	29	0,46	24	In-situ
1,70 - 2,20	G10	SC*	27	80	12	Yes	Med.	High	34	0,80	-	Repeat [9]
0,0 - 0,30	G4	SM-SC	2	28	5	No	Low	Low	6	2,13	-	Repeat [10]
0,30 - 0,90	G5	SM-SC	7	35	3	No	Low	Low	7	1,96	-	Repeat [10]
0,90 - 1,30	G10	SC	8	39	13	Yes	Low	Low	21	1,78	-	Repeat [10]

Notes:
[1] **Apparent,** based on indicators only; G10 only in TRH4:1996 and TRH 14: 1985
[2] ASTM D 2487-06. Bracketed tentative due to incomplete data. * Borderline CH
[3] Weighted PI = PIxP425/100
[4] P425 = % passing 425 µm; P002 = % passing 2µm
[5] Kantey & Brink (1952) for buildings
[6] Van der Merwe (1976) for buildings
[7] Wilson (1964, 1976) for buildings
[8] In-situ described as moist, black, becoming grey then olive with depth, stiff, uniform, intact, sandy silt
[9] Also oven-dried
[10] Air-dried
[11] Laboratory testing by Matrolab, Pretoria

A summary of the results of mineralogical and chemical analyses carried out on an oven-dried sample of the roadbed soil are shown in Table 3.

Table 3: Summary of soil mineralogical and chemical test results

Fraction [1]	Quartz %	Felspar %	Calcite %	Bassanite %	Smectite %	Extractable cations (cmol(+)/kg) Na	K	Ca	Mg	CEC [2]
Whole	92	-	3	3	2	4,0	1,5	87	5,8	33,7
Passing 2 µm [3]	72	16	-	-	12	-		-		-
Passing 2 µm [4]	52	-	-	-	32	-		-		-

Notes

[1] Sample from 1,1 – 1,6 m in Test Pit 1: LL= 66, PI = 30, LS = 14,0, P425=91, PO75 = 74, P002 = 9; pH (H_2O) = 7,47; pH (KCl) = 6,69

[2] CEC = cation exchange capacity, Amm.acetate method, pH 7

[3] Using Calgon dispersant

[4] Repeat without Calgon : 9 % goethite and 7 % kaolinite also reported, but no felspar

[5] Mineralogy and chemistry by Institute for Soil, Climate and Water, Pretoria according to the methods of the Non-Affiliated Soil Analysis Work Committee (1990)

5. DISCUSSION

5.1 Geology and Soils

Only a possible localised problem would have been predicted from a study of the geological and land type maps and the centreline survey. The field inspection indicated the roadbed over the cracked section to be expansive clay soil and that precautions were indeed warranted.

The X-ray diffraction analyses showed the presence of only about 2% expansive clay mineral (smectite) in the whole sample of in-situ roadbed soil from the pit on P65/1 and only about 12% in the fraction passing 2 µm. This is far less than would be expected from a soil with a PI of 30. However, the high cation exchange capacity supported the high PI and a repeat analysis without the use of Calgon (which can coat and mask the clay particles) reported a more reasonable amount of 32% smectite.

Bassanite ($CaSO_4 \cdot H_2O$, i.e. plaster of Paris) is unknown in such soils and was probably formed by the partial dehydration of gypsum ($CaSO_4 \cdot 2H_2O$) – known in such soils (Fey, 2010) – during the inadvertent oven-drying of the sample. This would have caused aggregation of the fines used in the index and sedimentation analyses, resulting in misleadingly low results being obtained.

5.2 Trench and Profile Observations

At both sites the trench and auger holes showed the presence of at least 1,0 m of cover over the in-situ "clay" soil and the latter to be over 1 m in thickness.

5.2.1 Road P65/1

In the case of P65/1, from the trench and pit it was evident that the pioneer backfill did extend to the toe of the fill as specified. The profiling also showed it to be within the thickness of 450 - 550 mm specified and the presence of the specified clay side-cladding was also confirmed.

The profile below this comprised about 500 mm of a gravel shoulder with a PI of 5 and a GM of 2,3, classifying as a potential G5 material. Below this was the clayey gravel backfill with a PI of 11, a LS of 6 and a GM of 2,6, just failing the PI limit of 10 for a G5 and the LS limit of 5 for a G5 and a G6, and thus classifying as a potential G7.

The in-situ roadbed soil from 1,1 to 2,2 m was described as a black, sandy **silt** which was confirmed by laboratory testing as a highly plastic silt (MH) with one borderline CH (highly plastic clay),with PIs of 22 - 37 typical of expansive clays. The WPIs and water (moisture) contents of this soil were about 30 and 28 % respectively in the upper 0,9 m, decreasing to about 20 and 25 % respectively in the last 0,2 m. All of this in-situ soil would probably classify as a G10 material.

5.2.2 Road D623

The 30 mm wide longitudinal crack at the edge of the stabilization was followed down into the lower fill where it ended. The roadbed at this point was also wet.

The upper 300 mm of this profile comprised a well-graded gravel shoulder with a PI of 5 and a GM of 2,2, classifying as a potential G4 material. Below this was 600 mm of gravelly sand with a PI of 7 and a GM of 2,0 classifying as a potential G5 material. The 400 mm of fill below this had a PI of 23 and a GM of 1,9, probably classifying as a G10 material.

The in-situ roadbed soil was described as a sandy silt and confirmed by the laboratory testing as MH or borderline CH with PI s of 27– 41 typical of expansive clays. The WPIs and moisture contents of this soil varied between 22 and 33 and 22 and 25%, respectively.

5.3 <u>Laboratory Soil Mechanics Test Results</u>

5.3.1 Identification of Potentially Expansive Clays

Expansive - also called active - clay soils expand when they wet up and shrink when they dry out. They are usually identified on the basis of their visual appearance and from the so-called ‘foundation indicator’ tests, i.e. their liquid limit (LL), plasticity index (PI), linear shrinkage (LS) and their grading, particularly the fraction passing 425μm (P425) and 2 μm (P002 – the so-called ‘clay fraction’). The soil constants are usually weighted by multiplying by the fraction passing 425 μm to give a weighted LL (WLL), WPI and WLS to represent the whole sample. The ‘clay fraction’ is always reported on a whole sample basis.

Table 4 shows a summary of some identification criteria based upon these tests. Of these, the criteria most commonly used in South Africa are those of Van der Merwe (1964, 1976). These criteria are those developed to identify soils having the **potential** to damage single-storey **buildings**. The criteria for roads are less well-defined although those for buildings can be used as a first approximation.

These criteria are of course only for the soil itself. If it is potentially expansive it is necessary to calculate the total and preferably also the differential heave of the whole soil profile for a more reliable estimate of the potential damage. One suggestion for roads is that no special precautions are required when the potential total heave is less than 30 mm (TRH9:1982).

The effects of differences in test methods other than due to the LL device are unknown, but mean that the Zimbabwe and AASHTO limits should be much lower for use with results obtained by TMH 1 (1986) Methods A1(a), A2, A3 and A4. This may render them not much greater than those for buildings. The method of soil preparation used to obtain the test results from which the Kantey-Brink, Van der Merwe and Wilson criteria were derived differed from that of TMH1 Method A1(a) (Netterberg, 2019), implying that they should also be lowered when Method A1(a) is used.

Table 4: Some Identification criteria for potentially active clay soils

Potential activity	**LL**	**PI**	**LS**	**WLL**	**WPI**	**WLS**	**P002**	**Notes**	**Refs.**
	%	**%**	**%**	**%**	**%**	**%**	**%**		**[6]**
Buildings									
Yes	>30	>12	>8,0	-	-	-	-	[1]	1
Yes	-	-	-	-	>12	-	>12-70	[1,2]	2
Medium	-	-	-	-	12-23	-	-		3
High	-	-	-	-	23-32	-	-		3
Very High	-	-	-	-	>32	-	-		3
Roads									
Yes	-	-	-	>50	≥28	-	>20	[1,3]	4,5
Marginal	50-60	25-35	-	-	-		-	[1,4]	6
High	>60	>35	-	-	-	-	-	[1,4]	6

Notes:

[1] **All** the criteria must be satisfied

[2] A Williams activity diagram plot of WPI vs P002 (Van der Merwe, 1976) is used to classify the results into low, medium, high and very high degrees of potential expansiveness

[3] **Partial** criteria for roads in Zimbabwe combined from both sources and adjusted by subtraction of 4,0 units from LL and PI for difference in LL device **only**

[4] **Partial** criteria for roads in the United States

[5] References: 1. Kantey & Brink (1952). 2. Van der Merwe (1976). 3.Wilson (1964, 1976). 4. Mitchell (1982),. 5. Van der Merwe (1983). 6. AASHTO T 258-81 (1998)

5.3.2 *Results*

According to the apparent COLTO/TRH4/TRH14 classification the layerwork materials were satisfactory except in the case of the 0,90 - 1,30 m layer of fill on D 623 which, because of a PI of 23, would probably only classify as a G10 in spite of its good GM of 1,9. However, its estimated CBR using Kleyn's method and VKE average factors (in Netterberg, 1994) at 93% would be at least 10, which would classify it as a G8 material if it were not for its excessive PI. Moreover, its WPI – which is what also matters in this case – was only 9 (Table 2), classifying it as non-expansive.

In all cases the in-situ material was worse than a COLTO G9, and would classify as a TRH4:1996 and TRH14:1985 G10.

In the Unified soil classification system ASTM D2487-06 the in-situ material all classified as MH, i.e. a sandy, **elastic silt** of high plasticity although some were borderline CH (highly plastic clay). Such soils plot below the Casagrande 'A' – line, which is unusual for South African expansive soils (Kantey & Brink, 1952). The one repeat classified as an SC, i.e. a clayey sand, which is more usual and likely. Although they may actually be OH (highly plastic organic silts), the necessary test was not carried out.

According to the most commonly used Van der Merwe (1976) activity chart, the in-situ soils would all classify as of low potential expansiveness. However, according to the Kantey-Brink criteria for buildings they would all be regarded as potentially expansive, to the Wilson criteria for buildings as of medium to very high potential, and to the partial Zimbabwe and AASHTO criteria for roads as mostly expansive.

This anomaly is due to the very low P002 fractions reported. This may be due to the effect of oven-drying on the smectite, and/or gypsum (i.e. aggregation by bassanite) and/or simply due to most of the smectite being naturally coarser than 2 μm.

The one repeat analysis of the in-situ soil yielded a much lower LL (60 vs 77), PI (34 vs 41) and LS (12 vs 14) and the same P425 (80), but the P075 was lower (36 vs 60) and the P002 higher (12 vs 8). This higher P002 was just sufficient to classify the sample as borderline low/medium expansive by the Van der Merwe method. It also plotted above the Casagrande A-line as expected for such soils

It is therefore **concluded** that the in-situ soil was indeed highly expansive at both sites and, according to the test methods used, was a silt and not a clay.

It is also **concluded** that the Van der Merwe method alone should not be used as a means of classifying such soils and that Wilson's (1964, 1976) and Donaldson's (1972) and Williams and Donaldson's (1980) preference for the WPI alone is supported.

Although the moisture contents of 20 - 28% were fairly high, as they were all less than the plastic limit further expansion is possible. Such soils would also shrink on drying, causing settlement and cracking.

<u>5.4 Precautions Taken</u>

The longitudinal cracking seen on both roads was typical of roads built over expansive clay roadbeds with no or inadequate precautions.

However, at least in the case of road P65/1 all reasonable precautions as recommended by TRH9:1982 and Netterberg (1988, 1992) had been taken. In particular, these included the following:

- building over the previous alignment (under which the moisture regime is expected to have largely stabilized itself);
- removal of the old gravel and replacement of the uppermost approximately 500 mm of clay (which has the greatest effect);
- flattening the sideslopes with this clay in order to keep the seasonal moisture changes and any side drains as far as possible from the carriageway; and
- waterproofing the culverts.

In the case of the short section of road D623 only the first precaution was taken as far as was practicable.

In the case of road P65/1 all the above four precautions had been carried out, and yet within two years the road had become badly cracked.

If the profile in TPI on P65/1 is assumed to continue to a depth of 5,0 m, using only the WPI criteria for classification as suggested by Wilson (1964, 1976, Table 4) and Van der Merwe's (1964, 1976) method endorsed by (TRH9:1982), it is estimated that the potential maximum ground heave was about 75 mm. This should have been reduced to about 55 mm by the 500 mm replacement and to about 50 mm by the 500 mm of cover. A cover of 1,0 m should have reduced the potential heave to about 40 mm, which still exceeds the suggested limit of 25 (Weston 1980) to 30 mm (TRH9:1982) above which precautions are necessary.

The total cover of about 1,3 m on P65/1 and 1,1 m on D623 was more than the 750 mm specified. Roads with cover of 2 - 3 m in this province ─ such as the P51/1 (R556) to Sun City ─ and elsewhere have also experienced similar or even worse damage.

An inspection after 18 years in February 2024 showed that the 1 km section on P65/1 had deteriorated significantly since 2009 and exhibited extensive and severe longitudinal cracking of Extent and Degree 5, varying in width up to about 20 mm with associated incipient potholing and extensive patching and potholing in both lanes, over the worst part (Figure 3). However, the riding quality was fair and the culverts had not apparently heaved, indicating that at least these precautions were adequate.

Figure 3: Condition of P65/1 near km 9 in February 2024 (this culvert had been reinstated due to settlement during construction)

5.5 Reasons for Poor Performance

The reasons for the unexpectedly poor performance of this section of P65/1 appear to be the following:

- The design, although successful in Zimbabwe with the replacement of 600 mm of clay – and their standard precaution (Mitchell, 1982; Netterberg, 1988) – and on the Roodekuil experiment (up to at least six years) (Netterberg and Bam, 1984) and, apparently under similar conditions in the Bon Accord area, may be less suited to the present situation with a greater thickness of “clay” combined with lateral drainage problems.
- This “clay” is unusual in that, according to the test methods used, it mostly classifies as a highly plastic silt in spite of its high plasticity, and mostly plots below the Casagrande A-line.
- The unfortunate ponding during construction over this section probably aggravated the problem – cracking attributed to the same cause was experienced on the Roodekuil experiment.

- The possibly high permeability of the backfill may have allowed water to penetrate the roadbed. Although such a coarse backfill has apparently been successfully used to stabilize the moisture regime by creating a sump effect (and also was said to have worked well in the Bon Accord area), more reliable results might be obtained with an impermeable (but non-expansive), well-compacted backfill, and only a pioneer layer on top of the clay for a prewetting technique should be coarse and permeable.
- The lateral drainage from right to left probably aggravated the problem by providing a source of water to the backfill.
- The excavation for the clay roadbed may have dried out or become excessively wet before it was backfilled. In this case the excavation both on this and the successful Bon Accord case (not described) had accidentally become flooded and had to be specially drained.
- An unavoidable delay between the initial cracking and the crack sealing probably allowed water to penetrate the backfill and the underlying clay.
- The replacement of only 500 mm of clay (as in Figure 1) may have been insufficient in this case.
- Widening a road on active clay tends to render it likely to crack at the interface – this was probably the cause of the shoulder cracks on D623.

In the case of Road D623 the reasons may include the relative position of the old alignment combined with the wide shoulder cracks which would have fed the runoff down into the lower fill and underlying clay, combined with marginal drainage.

In the design of roads over active clay roadbeds it is accepted that it is not normally economic to provide a perfect solution, especially for a low volume road – which often receives no special precautions at all. The best that can normally be done is to mitigate the effects to an acceptable level. However, in the case of P65/1, the precautions used were not successful, whereas in the Bon Accord case apparently similar precautions used in similar circumstances worked well. The reasons for this are not clear.

6. REMEDIAL MEASURES

The most effective remedial measure would have required the installation of a vertical plastic membrane to a depth of about 2,0 m along the worst section, at least on the right-hand side and preferably on both sides, as used on one previous occasion in South Africa and most extensively in Australia and Texas (Evans & Mc Manus 1999; Steinberg, 1998).

It was recommended that consideration should be given to improving the drainage as far as possible to prevent water ponding near the road and especially along the right-hand side, that any drains should be wide rather than deep and lead the water far away into the veld and/or under the culverts far away to the west, cracks should be resealed before each rainy season, and the culverts maintained regularly in order to maintain their water tightness, flow, and the prevention of ponding, especially along the east side.

7. CONCLUSIONS

1. It is not usually economic – especially for a low volume road – totally to eliminate the negative effects of an active clay roadbed. The most that is usually attempted is to reduce them to an acceptable degree and extent.
2. In the case of P65/1 all reasonable precautions that could be economically carried out were taken.
3. The damage would probably have been worse without them.

4. The reasons for these precautions not having been more successful in the case of P65/1 appear to be the unfortunate ponding experienced during construction, probably exacerbated by unusually severe fluctuations in moisture content due to the drainage situation, the probably permeable backfill, the 500 mm of “clay” roadbed replaced being insufficient, and a probably substantial depth of an apparently unusual clay.
5. In the case of D623 the reason appears to be due to the only precaution having been to build over the old alignment where possible, possibly aggravated by widening and marginal drainage. However, in view of the poor performance of P65/1 with the “full-house” precautions the performance of D623 may have been no better with them.
6. Although the general behaviour and principles for the design of countermeasures on active clay are understood, all the factors affecting their performance have been inadequately verified and are therefore less well understood.
7. Oven-drying of the soils before testing resulted in misleadingly low indicator test results being obtained, resulting in them classifying as of low potential expansiveness in the Van der Merwe method and as silts and not clays.
8. Clay roadbed soils should be tested using only air-dried material and according to foundation indicator and not road indicator methods.
9. The Van der Merwe method alone should not be relied upon as a means of identifying expansive clay soils and the weighted PI alone may be more reliable.
10. A study of the performance of this road in relation to the as-built record supplemented by further, deeper sampling and testing of the roadbed should be carried out.

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