SUSTAINABLE APPROACH TO UPGRADE LOW VOLUME ROADS IN PETRUSVILLE

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Abstract

Research undertaken in Petrusville and Philipstown was to illustrate how non-conventional materials performed as a sub-base. Roads investigated in this research paper were classified as light or low volume roads where the daily traffic equalled to 100 vehicles per day. Main objectives was to achieve a reduction in the overall life cycle cost, vehicle operating cost and the depletion of non-renewable resources for the roads. The outcome the road authorities and the different stakeholders wanted to achieve was that non-conventional materials can replace good quality road construction materials if the approach is cost effective and sustainable in the long term.

Keywords: CBR, DCP, Low volume roads, non-conventional materials

1. INTRODUCTION

Unsealed roads make up more than 75 per cent of the total road network in South Africa and makes up 95 per cent of the low volume road network (Paige-Green, 2007). Unsealed roads in South Africa consists either of earth or gravel standards. These roads provide access for rural communities to schools, health facilities and goods to markets. Due to adverse climatic conditions it makes access impossible on these roads. Tests sections identified for this paper is located in Philipstown and Petrusville area which forms part of the Renosterberg Municipality in the Northern Cape. Due to the nature of the environment dolerite was used as a sub-base and a 60mm thick interlock pavers for the surfacing. Detailed discussions will be looked at in the methodology and results sections.

There were also numerous projects undertaken in a number of countries in Africa showing favorable results when using non-conventional materials to upgrade unsealed roads to sealed standards. There is still a concern that LVRs constructed from non-conventional materials presents a high risk. The Malawi approach which will be discussed later will clearly outline that LVRs constructed from non-conventional materials does not present a much higher risk than roads constructed from good quality road materials. Main purpose of the paper will be to outline and highlight the developments that took place over the past 20 years that led to cost-effective and sustainable paved LVRs (Pinard et al 2015).

It is evident that usage of non-conventional materials in upgrading LVRs attracts a lot of research attention. Therefore, this paper investigates such an approach in Petrusville and Philipstown, Northern Cape.

2. LITERATURE REVIEW

Main problem identified by the Renosterberg Municipality was that most of the roads caused accessibility problems during the rainy seasons for the communities in Petrusville and Philipstown. The inclement weather caused driving discomfort on the gravel roads where drivers needed to divert from the wheel path. The deterioration of the roads was due to a lack of maintenance, increase in traffic, ineffective drainage and inadequate cross-fall. Funding was a major challenge for the Municipality to maintain and upgrade gravel roads. The approach adopted was to create a sustainable approach where the maintenance and vehicle operating costs can reduced by using non-conventional materials and construction techniques to upgrade the gravel roads to paved standard.

The streets investigated for upgrading during phase 2 for Renosterberg Municipality was funded by the Department of Cooperative Governance, Human Settlements and Traditional Affairs. Due to the nature of the area suitable base course materials was not found in the vicinity. The only suitable crushing plant located was in De Aar which is approximately 110 kilometers from Petrusville and 55 kilometers from Philipstown. This would have caused major financial implications on the budget for the project. Research already pointed out that earthwork is about 70% of the total budget. A Borrow Pit was located 2 kilometers from the area which was mainly used during the 80s and 90s there was no records found. During phase 1 which started in 2011 the borrow-pit and subgrade was tested to discern whether the material would be suitable and if an import layer was needed if the sugbrade was weak. The findings and discussion section which will be looked at later will provide suitable evidence on phase 1 and 2 of the project and whether the roads have failed structurally and functionally. Table 1 below illustrates some of the roads that were investigated in this research paper for Philipstown and Petrusville.

Table 1: Streets upgraded from unpaved to paved

Street Name	Length (m)	Width (m)	Town
Koppie Street	312.87	5.40	Petrusville
Kombro Street	230.92	5.40	Petrusville
Street 1	555.00	5.40	Philipstown
Street 2	296.04	5.40	Philipstown

2.1 Climate

The roads upgraded in Philipstown and Petrusville falls under the dry climatic region as seen in figure 1 with a Weinert N-Value of >5. The weather data obtained in 2014 from the weather bureau in 2014 also indicated that the annual rainfall was 386 mm falling in a semi-arid area (Weather Bureau, 2015). The material obtained from the borrow pit was weathered dolerite which can be found more abundantly in South Africa (Kleyn and Bergh, 2008). Weinert N-value was also developed when Karoo dolerite was used in different parts of the country. When the Karoo dolerite was used in the western parts of Bloemfontein (climatic region is dry) the roads performed satisfactorily for more than 20 years. In the eastern parts of the country (climatic region is wet) the roads did not give the desirable results and were subjected to cracking and loss of shape. The roads in the eastern parts of the country was investigated to determine why the results where unfavorable. The conclusion reached was that the base material deteriorated from the borrow material from where it was extracted (Weinert, 1980). It is evident from the above results that dry environment materials tend to give much higher CBR values than in a wet environment. Materials in a wet environment are more susceptible to moisture changes in the underlying pavement materials than in a dry environment. Weinert N-value was created to illustrate the influence the climatic conditions will have on natural road building materials as well as on performance of the road.

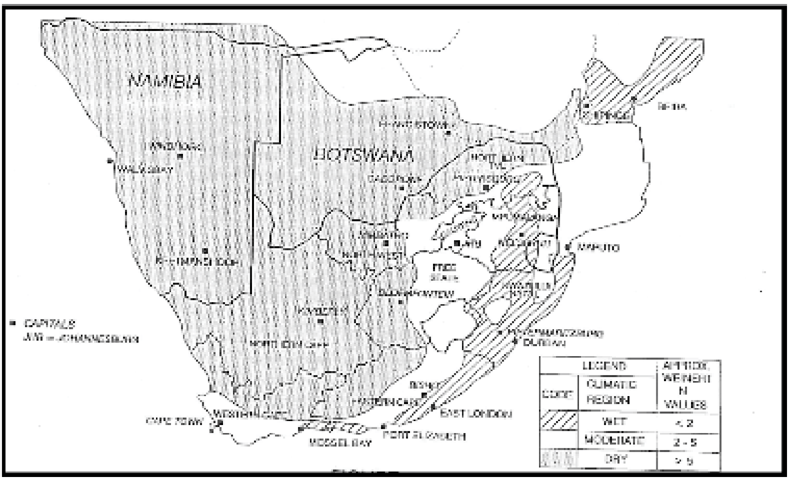

Fig. 1: Weinert N-Value for South Africa Africa, Namibia and part of Botswana (Weinert, 1980)

2.2 Traffic

More than fifty years ago a 'rule of thumb' was established that stated roads can be upgraded from earth to gravel if the daily traffic exceeds 50 vehicles per day (vpd). The rule also stated that gravel roads can upgraded to sealed standard if the daily traffic equals to 200 vehicles per day (Gourley et al 2002). In developed countries like the USA, low volume roads are defined by roads carrying 400 vehicles per day. In African countries it is defined by a road that carries approximately 300 vehicles per day which is about 1 million equivalent standard axles. Yet the above definition of low volume roads are very rarely used (Pinard et al, 2015). As seen in figure 2 it is clearly evident that pavement deterioration for high volume roads (>0.5 million equivalent standard axles) is caused by vehicle loads rather than the environment. For roads carrying less than 0.5 million equivalent standard axles environmental factors needs to be considered when designing low volume roads. Main environmental factors affecting low volume roads are the moisture changes in the pavement, fill and subgrade (Pinard, 2011). That is why the surfacing option should be carefully considered to eliminate the effects the environment (temperature and rainfall) will have on these roads.

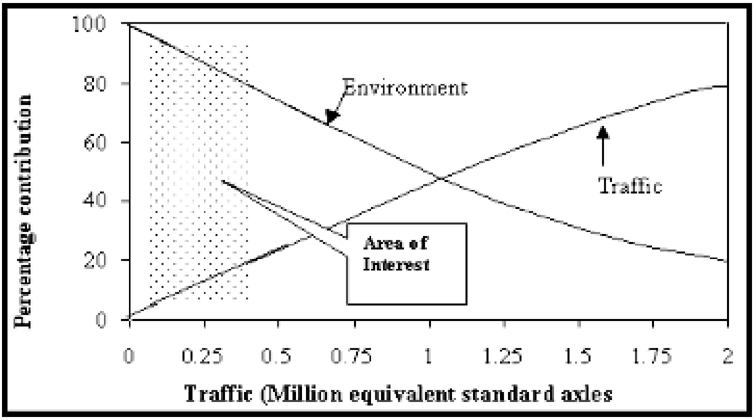

Fig. 2: Effect vehicle loads and the environment has on the pavement's performance (Pinard, 2011).

2.3 Low Volume Roads in Other Countries

2.3.1 Malawi:

The Malawi project carried less than 200 vehicles per day and the material used for the base and subbase was tested using the Overseas Road Note 31. Nine sections were investigated where marginal quality materials were used as base and sub base.

These roads consisted of reduced pavement thickness (less than 150mm), five meter wide carriageways, use of non-conventional materials (laterite, quartzite and weathered granite) without stabilization and a thin surfacing (bitumen). Laterite is a pedogenic material that uses its own self-stabilizing properties where compacted strength requirements can be relaxed. According to Weinert's soil classification system laterite, quartzite and weathered granite was found to be a good-fair road construction material. These roads compared to roads designed from the traditional standards have performed reasonably well despite the non-standard nature of it. Main objective was to ensure that the roads are cost effective and sustainable and that it reduces the life cycle costs compared to roads constructed from traditional methods. (Pinard, 2011). The design traffic load for the low volume roads investigated was 0.28 million ESA which equals to 200 vehicles per day. Conventional approach would require a crushed stone base with a CBR (California Bearing Ratio) value >80, PI (Plasticity Index) <6 and a narrow grading envelope. Construction process involved was that the embankment should be raised and the material be compacted “to refusal” (compacting with the heaviest plant available without breaking down the material).

Table 2: Atterberg Test Results (Pinard 2011)

Road section	**Base Type**	***Pavement layer***	**Soil Parameter**			
			MAX LL		***MAX PI***	
			Value	***Specification***	***Value***	***Specification***
Ntchisi (School)	Laterite	Base	36	-	16	6
		SB/SG	38	35-55	19	6-20
Ntchisi (Hospital)	Laterite	Base	33	-	16	6
		SB/SG	42	35-55	19	6-20
Ntchisi (Standard)	Laterite	Base	NP	-	NP	6
		Subbase	30	35-55	14	6-20
		Subgrade	35	-	18	-
Dowa	Weathere d Granite	Base	43	-	17	6
		SB/SG	34	35-55	13	6-20
Rumphi	Weathere d granite	Base	32	-	18	6
		SB/SG	NP	35-55	NP	6-20
Cape Maclear	Quartzitic gravel	Base	24	-	14	6
		SB/SG	35	35-55	16	6-20

Results obtained on these roads were 95-97% BS (British Standard) compaction instead of the conventional heavy compaction of 93-95 BS (British Standard). Table 2 clearly demonstrates under the British standards that the LL (Liquid Limit) for all the roads investigated fully complies. All the materials tested for the subbase and subgrade were compliant with the minimum PI values. The PI was extremely high for the base courses after monitoring the roads for 8 years after their construction they have performed exceptionally well. LL and PI is also a good indicator of properties and the behavior of the materials when used as a road pavement (Pinard 2011). The reason why these roads performed beyond their design life was that proper drainage was constructed and quality construction techniques used. The findings are well supported by similar projects that were undertaken in Malawi and other countries (Pinard 2011).

2.3.2 SADC region:

In the Southern African Development Community region different types of sands are evident, but the most common type of sand found is the Kalahari sands. Kalahari Sands can also be found in arid and semi-arid areas of the world (in all seven continents). A third of the earth surface is covered with sands and sandy materials over a large portion of their surface. One of the sites where bases were constructed from red sand was in the Free State Province in South Africa along the Hoopstad-Bultfontein road in 1962 (Netterberg and Elsmere 2014). Sands used in this project were only suitable as fill and selected subgrade yet it was used as base course on these roads. Some sections were stabilized with cement, bitumen/ road tar and mechanical stabilization. One section was investigated where it was found that no stabilization occurred. All the sections performed reasonable well especially where 4-5% Portland cement was used (Netterberg and Elsmere 2014).

Table 3: Laboratorty Test Results For Hoopstad-Bultfontein Road In 2013

Section		**A**	**B**	**C**	**K**
150mm Base	**Units**	**Neat Sand**	**Sand+ 3% OPC**	**Sand+ 5 OPC**	**Crusher-Run**
In-Situ CBR					
Unsoaked	%	(n=1)	(n=2)		(n=18)
Max.	%	122	>333		>140
Mean (x)	%	81.0	>153	(>150)	109.0
Min.	%	58	153		66
SD (s)	%	22.7			26.2

Soaked	%	(n=6)	(n=3)		
Max.	%	36	145		
Mean (x)	%	29.2	121.7		
Min.	%	26	105		
SD (s)	%	4.3	20.8		
Mean UCS					
Dried	kPa	180	2200	1800	
Cured	kPa	20	1400	2000	
Mean ITS					
Dried	kPa		8	50	
Cured	kPa		24	33	
Mean Deflection	mm	0.20	0.15	0.25	0.20
Mean Deflection	mm	0.25			0.30
Mean ROC	mm	110			180

Notes: Figures bracketed estimated by authors.

Table 4: Grading Of Neat Sand Base

Particle Size (mm)	**Percentage Passing**
1,18	100
0,841	100
0,600	99
0,420	(97)
0,250	87
0,150	46
0,074	(9)
0,060	7
0,020	6
0,006	5
0,002	3
GM	0.94
Uniformity coefficient (Cu)	2.5
Coefficient of curvature (Cc)	1.6
Classification:	
AASHTO	A3/borderline A - 2 - 4 (0)
Unified	SP–SM (poorly graded sand with silt)
Colto	potential G7 at best (no CBR)

Base course was compacted with a 50 ton pneumatic roller followed by steel wheel roller (Netterberg and Elsmere 2014). Investigation conducted up until November 1963 on all sections has shown little distress during this time. Further investigations also revealed that if the conditions are dry the un-stabilized section would have a mean unsoaked CBR of 80 as tabled table III. Findings also shows relative compaction of the neat sand section which was not reported. Yet the section that was stabilized with 3 and 5% ordinary Portland cement (OPC) showed results of 92 and 90 % MAASHO (Modified Association of State Highway Officials). Neat sand section was in a good condition with un-soaked and soaked California Bearing Ratio conforming to the specifications. The neat sand base was non-plastic with a borderline of A-2-4 (0) classification which is poorly graded sand with silt. Neat sand base also had a small Unconfined Compressive Strength (UCS) after drying compared to sand section stabilized with 3% ordinary portland cement (Netterberg and Elsmere 2014). Triaxial test also indicated that the sand used was not the usual cohesionless types and that it showed some form of cohesion when tested. Saturation should be avoided when the testing was conducted it showed a low mean soaked in-situ CBR of 29. In June 1963 the maximum settlement equaled to 13mm. Visual observations were continued up to 1974 yet no records could be found of such assessment that took place (Netterberg and Elsmere 2014). Netterberg and Elsmere 2014 concluded that the visual observations were measured up to 1974, but no records could be found. Surveyor accountable for this results recorded that no distress occurred up until 1974.

2.4. Responsibility of Roads and Transport within a District

Roads and Transport is not only the responsibility of national and provincial government alone but local and district municipalities have a key role to play for the planning and regulation of roads and transport. District Municipalities (DMs) also needs to play a key role to plan and coordinate roads and transportation planning within their boundaries to promote economic and social development. According to the Municipal Structures Act the following roles the District Municipality needs to play:

- 83 (3) (b) promoting bulk infrastructural development and services for the district as a whole;
- 84 (1) (f) Municipal roads which form an integral part of a road transport system for the area of the district municipality as a whole.
- (g) Regulation of passenger transport services.
- (h) Municipal airports serving the area of the district municipality as a whole.

For district municipalities to perform this function it needs to be well resourced but currently district municipalities does not have the capacity or the funds. Local Municipalities are also trying their utmost best to perform this function the DMs need to carry out.

2.5 Surfacing Option

The following surfacing options were used for Philipstown and Petrusville:

- The surfacing option used for Philipstown and Petrusville was a double zig zag 225 x 112.5 x 60 mm thick paving block. The compressive strength for the paving blocks was 25 MPa.
- Precast semi-mountable kerb, type 7 and 8b and tranisition kerb between types 8b and 7 semi-mountable kerb at the start and end of the bellmouth. This was used for drainage purposes.
- G 4/5 material was required for the project's subbase which is commonly used as a subbase material. Results will clearly highlight the performance of the material after construction.
- Bedding or jointing sand was tested in terms of its grading (SANS 3001-GR1) and Plasticity (presence of clay) (SANS 3001-GR10).
- The following tests were done on the concrete paving blocks: Strength tests: Tensile splitting test, Abrasion resistance and Water adsorption test according to SANS 1058 (Kilsaran, 2015).

2.6 Cost Comparison

Table 5: Cost Comparison of a Class I road and the Low Volume roads in Petrusville and Philipstown

Road Design Standard	Earthworks Amount (R)	Wearing course (R)	Tender Amount (R)	Total km of roads upgraded
LVSR (Petrusville and Philipstown)	2,867,000	7,593,000	14,508,000	22
Class I	11,995,705.40	26,726,833.20	221,966,154.63	49.5

The above costing comparison is evidence why low volume roads hold significant cost savings in terms of earthworks and also wearing course. For Philipstown and Petrusville almost half the amount of roads were upgraded compared to the Class I road and still a massive saving were achieved on the earthworks. If roads can be designed cost effective and sustainable for roads belonging to the province and district municipalities. Money will be available to carry out routine and periodic maintenance. If such savings cannot be achieved the paved roads will deteriorate to a point beyond repair.

3. RESEARCH METHODOLOGY

3.1 Design Method

Design method used for the Low Volume Roads in Petrusville was the TRH4 (NITRR, 1996), Malawi design manual for low volume sealed roads using the DCP design method, 2013.

3.2 Field Investigation

The field investigation was undertaken after the wet and dry seasons to obtain more accurate results. The following tests and assessments were done on the road base after 4 years in service:

- The Determination of the In-Place Density and Moisture Content of Soils and Gravels by Nuclear Methods- TMH 1: 1986, Method A10 (b).
- Visual Assessment on the cross-section and road drainage.
- Dynamic Cone Penetrometer Test (TMH 6: 1984, Method ST6).
- Bulk Sampling was also done for the laboratory testing which is highlighted in the next section.

3.3 Laboratory testing

Field work and Laboratory testing was conducted to investigate the performance and to determine the properties and characteristics of the non-conventional materials. Samples were extracted on the subgrade in 2011 and the subbase in 2015 and the following tests formed part of the investigation:

- Sieve Analysis, Percentage of Material Passing 0.075 mm Sieve (TMH 1: 1986, Method A1 (a), A5)-% Passing Sieves.
- Atterberg Limits Analysis (TMH 1: 1986, Method A2, A3 & A4).
- Maximum Dry Density and Optimum Moisture Content (OMC), California Bearing Ratio (CBR) Analysis (TMH 1:1986, Method A7 & A8).
- Uncofined Compressive Strength & Indirect Tensile Strength of Stabilized material (TMH 1: 1986, Method A14 & A16T).

4. FINDINGS AND DISCUSSION

The traffic for 2000 and 2004 was available for the Petrusville and Philipstown area, but it was assumed that the traffic date for 2004 was highly unreliable. 2010 a traffic count was conducted manually before construction started in 2011. The total Annual Average Daily Traffic of 100 vehicles per day with 1-2 heavy vehicles was obtained from the data received. The design traffic load was calculated to be in the vicinity of 0.10 million equivalent standard axles. The design life of 15 years was also included in the calculations (TRH4, 1996).

5. RESULTS

The following tests were conducted according to the TMH 1: 1986, Method A10 (b).The Determination of the In-Place Density and Moisture Content of Soils and Gravels by Nuclear Methods. The average relative compaction for the subbase and subgrade for Street 1 and 2 was 100 % MOD AASHTO. Subbase and subgrade for Kombro and Koppies street were respectively 95 and 93 % MOD AASHTO.

IDD and FMC is not very significant other than calculating the relative compaction.

Table 6:IDD, FMC and % MOD AASHTO

Road Section	Pavement Layer	In-situ Dry Density (IDD) kg/m^3	Field Moisture Content (FMC) %	% MOD AASHTO determined with troxler
Street 1	Subbase Subgrade	2169 1976	11.5 11.9	100.2 and 100.6
Street 2	Subbase Subgrade	1921 1903	10.5 12.9	100.1 and 100.6
Kombro	Subbase Subgrade	2142 2058	11.5 10.9	95.8 and 95.4
Koppies	Subbase Subgrade	2120 2010	8.5 10.0	91.7 and 92.0

Visual Assessment

Test Sections investigated:

The following coordinates are for the Petrusville area where sampling was done for Koppies and Kambro Street:

- Test Pit 1: 25 Y0031614 X3329218
- Test Pit 2: 25 Y0031491 X3329110
- Test Pit 3: 25 Y0031614 X3329218
- Test Pit 4: 25 Y0031678 X3329200

The following coordinates is for the Philipstown area where sampling was done for Street 1 and 2:

- Test Pit 1: 25 Y0050965 X3368221
- Test Pit 2: 25 Y0051171 X3368127
- Test Pit 3: 25 Y0051067 X3368142
- Test Pit 4: 25 Y0051051 X3368129

The laboratory testing will clearly outline what tests were conducted on the samples extracted and what results were achieved on these roads. Testing was done on materials extracted from the pavement materials after more than 4 years in service meaning that the roads were subjected to warm and cold conditions. In 2011 field investigations comprised of sampling and testing of the subgrade at preselected sections exhibiting failure, as well as at sections in a better condition, to get an overall impression of the subgrade conditions.

In 2015 visual inspection along the road indicated that no discernable distress of the pavement occurred in the form of surface and base failures. Results in the next section clearly outlines that the sub-base is still in a sound condition and has not suffered from shear deformation through vehicle loads. This is also an indication that the drainage on all the roads was sufficient were properly constructed.

Fig 3: One of the test pits for Petrusville (Koppie Street)

Fig 4: One of the test pits for Philipstown (Street 2)

Dynamic Cone Penetrometer Test

The dynamic cone penetrometer was conducted according to TMH 6: 1984, Method ST6. Dynamic Cone Penetration (DCP) tests were carried out at all 4 test holes before and after stabilisation. The analyses of the DCP-CBR are tabled in table VII. The DCP_CBR results confirm that the bearing capacity after stabilisation improved significantly for in-situ and soaked strengths. Three of the DCPs for street 1, 2 and Koppies street showed weakend areas which was not recorded for the subgrade. The overall impression from the results tabled was that the high strengths could further improve over the coming years.

Table 7: Results for DCP-CBR

Road Section	Pavement Layer	Type of material	Maximum in-situ strength		Maximum soaked strength	
			DCP-CBR	Stabilized DCP-CBR (2015)	DCP-CBR	Stabilized DCP-CBR (2015)
Street 1	Subbase	Dark brown sand with dolerite	41	72	10	30
	Subgrade	Dry reddish dark brown dense sand with shale	-	-	-	-
Street 2	Subbase	Dark brown sand with dolerite	72	110	8	28
	Subgrade	Slightly moist reddish brown dense sand with shale	-	-	-	-
Kombro	Subbase	Dark brown sand with dolerite	31	54	6	31
	Subgrade	Oversized dolerite rocks	-	-		
Koppies	Subbase	Dark brown sand with dolerite	95	68	22	29
	Subgrade	Dry olive dense weathered dolerite	-	-		

Bulk Sampling was also done for the laboratory testing which is highlighted in section 4.3.

Laboratory testing Results: Grading

Sieve Analysis was conducted in terms of the Percentage of Material Passing 0.075 mm Sieve (TMH 1: 1986, Method A1 (a), A5). Material passing the 0.425 mm is mostly used to determine the Atterberg limits. The results in itself are not sufficient to classify the materials used into any G class. Atterberg limits and Grading Modulus (GM) will give us a clear indication of what type of G class material was used as a subbase. Fines passing the 0.075mm are also tested for Atterberg limits giving a clear indication of the moisture sensitivity of the materials. Tabled results in table VIII is the first step towards determining if the material has sufficient fines to achieve the desired density and bearing strength. Most of the materials used were classified as a G5 material, but with different PIs and GM the material were classified as G6 and G7 type of material which is commonly used as a fill material in the road construction industry.

Table 8: Percentage of material passing 0.075 mm sieve

Road Section	Pavement Layer	Percentage passing 0.075 mm sieve	Possible Design Equivalent Material Class
Street 1	Subbase	15	G5/G6
	Subgrade	25	G6/G7
Street 2	Subbase	11	G4/G5
	Subgrade	33	G8
Kombro	Subbase	17	G5/G6
	Subgrade	16	G5/G6
Koppies	Subbase	20	G5/G6
	Subgrade	14	G4/G5

Classification summary

- Values obtained for the PI for the base as seen in Table IX was Slightly Plastic for all 4 roads investigated meaning that linear shrinkage values was between 0.5-1 %.
- Street 2 and Kombro street had subgrade values that were slightly plastic. Streets investigated were difficult to classify based on PI values for the subbase being of a Slightly Plastic nature.
- Subbase for street 1 is classified as a G7 material. Colto requires that for a G7 material the GM should be between 0.75-2.7.
- The subbase material used for street 2 indicates a G6 quality material, based on the findings the linear shrinkage did not exceed 5 % and the PI were slightly plastic. CBR at 95 % MOD AASHTO were also more than the minimum requirement of 25%.
- Subbase material investigated for Kombro street indicates that the material is of a G8 quality where the CBR at 93% MOD AASHTO were more or less equalled to the CBR of 10 specified in the Colto guidelines.
- Koppies street were also of a G7 quality material where the GM of 1.91 was well within the range of 0.75-2.7. CBR at 93 MOD AASHTO was higher than the required CBR of 15 at 93 MOD AASHTO.
- GM is a good tool to use to assess the properties of the soil. Two out of the 4 roads investigated gave GM values higher than 2 indicating that the material used was coarsely graded and a good quality material.

Table 9: Classification of material used for the subgrade and subbase

Road Section	Pavement Layer	Grading Modulus (GM)	Swell at 100 % MOD AASHTO	CBR at 95 % MOD AASHTO	CBR at 93% MOD AASHTO	Liquid Limit (%)	Plasticity Index (PI)
Street 1	Subbase	2.26	0.0	22	18	-	SP
	Subgrade	1.59	-	-	-	34	16
Street 2	Subbase	2.31	0.0	33	20	-	SP
	Subgrade	2.03	-	-	-	-	-
Kombro	Subbase	1.94	0.0	21	11	-	SP
	Subgrade	1.79	-	14	7	-	SP
Koppies	Subbase	1.91	0.0	24	20	-	SP
	Subgrade	1.84	-	-		24	4

5. CONCLUSION AND FURTHER RESEARCH

From the paper the conclusion reached was that the LVR design, specifications and construction techniques used is very different than that of high volume roads. Bigger impacts of the newly adopted approaches are in fact related to social and environmental benefits that occur from upgrading gravel roads to sealed standard. Malawi approach is one of the projects that are in a continued process meaning results currently obtained for non-conventional approach is representative of the average behaviour. The results listed clearly shows that conforming low volume roads to the minimum requirments of Colto or TRH14 are still a challenge. While the dry enviornment of Petrusville and Philipstown showed that non-conventional materials can perform well above the desired result is a major step towards creating a guideline which allows non-conventional materials to be used in any environment.

Probabilities and statistics can be overcome by using a design life of 15 years and using a sensitivity of +/- 5 years. This will accommodate the variability experienced on LVR in practice and not only in guidelines.

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