THE PERFORMANCE OF STABILIZED PEBBLE BASES UNDER LIGHT URBAN TRAFFIC IN LUANDA, REPUBLIC OF ANGOLA

A. J. L. D. A. Soares[1] and A. T. Visser[2]

[1]Graduate student, University of Pretoria and Africon Engineering International (Pty) Ltd PO Box 905, Pretoria, 0001, South Africa. E-mail: arthurdas@africon.co.za

[2]South African Roads Board Professor of Transport Engineering, University of Pretoria
E-mail: alex.visser@up.ac.za

ABSTRACT

A housing project with 24 km of residential streets, on a site 8 km south of the city of Luanda in Angola, known as the Nova Vida Project was constructed in 2001. A stabilized natural pebble river gravel mixed with red sand was used as the base and subbase layers. Since the performance of this pavement structure was unknown, a study was executed to determine the performance after five years. This paper presents the evaluation of the performance of selected roads in this development. The paper first presents a brief geotechnical and geological overview of the Luso and Muceque/Quelo Red Sands and their road construction properties. Thereafter the as-constructed laboratory test results are presented, and finally the structural performance of the stabilized pebble material is predicted based on the behaviour during the first five years. The dynamic cone penetrometer (DCP) results demonstrated that the structural life of the pavements will exceed the structural design period (design pavement life) of 15 years before reaching a rut depth of 20 mm, if the assumed conditions prevail during the implementation of the proposed maintenance programme. This conclusion was confirmed by the South African mechanistic design analysis.

Keywords: Design pavement life, Performance, Stabilized pebbles, Structural capacity

1. INTRODUCTION

This Project, in the Nova Vida suburb of Luanda, Angola, was started in 2001 and the projected end date for Phase 1 was April 2005, at which stage 3 500 residences and the respective infrastructure and social services facilities were to have been constructed. This residential project is planned to grow, with further phases being added over at least the next 10 years.

The client, the Ministry of Public Works of the Government of Angola (Ministério das Obras Públicas – MINOP) requested that river gravel pebbles (i.e. material belonging to the client) be used in the concrete mixes and in the construction of the roads in the Project in order to save costs on the extraction, crushing and importation of conventional crushed aggregate material. This pebble material originates in and is extracted from the bed of the Cuanza River at a place known as Bom Jesus, which is situated about 60 km south-east of Luanda in the Bengo Province of Angola. The street pavement design and construction method adopted was to use this natural pebble material mixed with red sands as identified by Dr. Soares de Carvalho (1957, p. 2) in the base and subbase layers of the road pavement and to stabilize these layers with cement. The natural pebble material was mixed with a red sandy soil (at 12% to 14% content), known as the Luso and Muceque/Quelo Red Sands (Horta da Silva and Gomes Teixeira, 1973, p. 95). This mix was stabilized with 5 to 6% OPC (PPC Cement) for both the base and subbase layers, and was placed on the in-situ brown sand subgrade.

Since the expected performance of this stabilized material was unknown, a study was executed to monitor the performance of these pavements. The aim of the paper is to present the results after about five years of service.

Firstly the geotechnical and geological properties of the Luso and Muceque Red Sands and their road construction properties are discussed. Thereafter the results of the as-constructed laboratory tests are presented. Finally the structural performance of the stabilized pebble pavement structure is evaluated, and the remaining life is predicted.

2. CONSTRUCTION MATERIALS

2.1 Geology of Luanda and Nova Vida Site

The area of Nova Vida in Luanda consists predominantly of Pleistocene-Pliocene (up to 12 million years old) sands, laterites (pedogenic iron- and alumina-rich deposits) and argillite (clays and silts) which are underlain by older (between 12 and 36 million years) Miocene-Oligocene sediments consisting of arenites (windblown sandy deposits), argillites, sands and conglomerates

At Nova Vida the red and orange to yellow-grey sandy deposits are encountered principally at higher elevations on slopes and along hilltops, but also at river level, particularly near the eastern boundaries of the site. These sandy deposits form part of the Muceque or Quelo Formation which was deposited in the Pliocene period. The sands tend to be generally pin-holed and cemented by limonite-rich cement. The soils underlain by the Luanda Formation are clayey and, in a desiccated state, often micro-shattered. Layers of sand and of hard rock shelly limestone are also encountered.

2.2 Muceque/Quelo Formation

According to the Unified Soil Classification System (ASTM), the Muceque Sands can be considered as sandy-clayey or sandy-silty soils, symbols SC and SM, or mixed SM-SC.

According to the geological description, a fundamental characteristic of the Muceque Formation is its unstable or collapsible structure, which causes serious differential settlement when it is subjected to saturation under loads.

Once this soil is saturated, even under low loads, it settles dramatically. This settlement is caused by the failure of the inter-granular bonds (Horta da Silva, 1971 and Horta da Silva and Gomes Teixeira, 1973). When dry these sands can support relatively high loads, up to 8 kg/cm² (785 kPa), without suffering large settlements.

In the construction of shallow foundations and road base layers, up to 1 m in depth, it is safe to apply a stress of 1 kg/cm² (98 kPa). Given the unstable nature of the soil, it should, whenever possible, be drenched and compacted by means of vibration in order to collapse the structure as far as possible.

For deep foundations it is advisable to sink boreholes using percussion drills in order to verify the foundation of the Muceque Formation and the nature of the underlying layers. It may be necessary to do further tests such as SPTs or dynamic penetration or static penetration tests.

The geotechnical condition of the Muceque/Quelo Formation revealed low natural apparent density. The use of this sand (undisturbed) in road construction and foundations needs to consider the wide variations in CBR as a result of the low apparent density.

2.3 Muceque/Quelo formation as a pavement foundation

CBR tests were done in the Luanda area on the Muceque/Quelo formation by Ferreira and Silva (1957, p. 14). The average CBR values, shown in Table 1, were obtained from field and laboratory tests at depths of 0,35, 0,70, 1,0 and 1,5 m.

Table 1: CBR test results – Average values

		Depths (m)			
		0,35	**0,70**	**1,00**	**1,50**
Laboratory CBR (Mod AASHTO)		27	25	23	24
CBR undisturbed samples	**Dry**	6	3	2	3
	Soaked	3	2	1	2
In-situ CBR (DCP)		6	2	3	3

This soil formation is composed of two completely distinct fractions, one being a coarse silt and the other being fine and fundamentally constituted of iron oxide and aluminium. In relation to road foundations, this soil presents (apart from the extremely low resistance in its natural state, owing to its low apparent density) a wide variation in CBR as shown in Table 1. The CBR value to be used in the pavement calculations must consider the available compaction methods.

Ripping and recompaction is used to remove collapse potential of soils, and with efficient compaction, any settlement and cracking is reduced to a minimum. Serious cracking may also result from settlement under a pavement with cemented layers (TRH 13, p. 55).

2.4 River Pebbles

River pebbles are lithified sedimentary rocks that consist of rounded fragments larger than 2 mm (0.08 in.) in diameter; it is frequently contrasted with breccia. Breccia commonly results from processes such as landslides or geological faulting, in which rocks are fractured and angular. Water would cause rounding and smoothing of the pebbles.

The Cuanza River pebbles were used for the construction of the base and subbase layers, and constitute 82% – 86% of the layer material. These coarse sedimentary pebbles consist of round and nearly round fragments larger than 2 mm.

This natural pebble material was mixed with a red sandy soil (content from 12% to 14%). The pebble material was classified as a G5 material, and has a maximum size of 63 mm that is half of the compacted layer thickness. The sieve analysis of the pebble material showed that it has a grading similar to a G4 material.

2.5 PAVEMENT DESIGN

The final design for ES0.3 design category was achieved from the mix designs, trial and error and experience obtained during the project. Because excessive cracking was experienced in the layers mixed with 6% cement, the design mixture was reduced to 4,5% OPC for the base layer and 2,5% OPC for the subbase layer, which still gave the layers high strength. The pavement structure for lightly trafficked streets is shown in Table 2.

Table 2: Final pavement configuration implemented

Surfacing:	25 mm continuously graded (fine) asphalt (AC)
Tack coat:	30% stable grade Cat 60, 50/50 diluted with water @ 0,55 l/m²
Prime:	MC70 @ 0,7 l/m², diluted with 15% paraffin (a penetrating prime is not normally used on cemented layers, unless the cement has gained much of its strength)
Base:	125 mm (150 mm for roads 7 m wide) (G5) river pebble with 12% – 14% red soil mixture @ 4,5% cement and compacted to 98% of Modified AASHTO density. UCS between 750 kPa & 1 500 kPa (C4)
Subbase:	125 mm (150 mm for roads 6 and 7 m wide) (G5) river pebble with 12% – 14% red soil mixture @ 2,5% cement and compacted to 95% of Modified AASHTO density. UCS between 750 kPa & 1 500 kPa (C4)
Subgrade:	150 mm in-situ rework and compacted to 93% of Modified AASHTO density. CBR > 7

Atterberg limits and sieve analysis tests were done on the mixture, consisting of the in-situ brown sand and the rounded stone pebbles with 14% of reddish silty sand, both before and after stabilization, as shown in Table 3.

Table 3: Sieve analysis, Atterberg Limits and Grading Modulus (GM) of in-situ brown sand, red sand and pebbles before and after stabilization (mixed material)

In-situ brown sand, rounded stone with 14% of reddish silty sand	**GM**	**Sieve analysis (% passing)**										**Atterberg Limits**		
		53,0 (mm)	**37.0 (mm)**	**26.5 (mm)**	**19,0 (mm)**	**13,2 (mm)**	**9,5 (mm)**	**4.,5 (mm)**	**2,0 (mm)**	**0,425 (mm)**	**0,075 (mm)**	**LL**	**PI**	**LS**
Before stabilization	2,25	100	88	75	61	52	46	39	33	27	15	32	**13**	6,5
Stabilized with 5% cement	2,17	100	86	73	58	51	46	41	36	30	17	30	**8**	3,5

No ICL (Initial Consumption of Lime) testing was done. The relatively high cement content required to achieve the design UCS and durability would suggest that the ICL was relatively high (TRH 13, p. 23). Furthermore, the small decrease in PI from 13 to 8 shown in Table 1 supports this contention.

3. VISUAL ASSESSMENTS

The visual assessments were done in August 2005 in accordance with TMH 9 (1992). Table 4 summarizes the functional criteria and recommended treatments and priorities. These results show that after about 5 years the pavement surfaces showed little aggregate loss, and the structural assessment confirmed some block, longitudinal and transverse cracks, and minor rutting (Soares 2007, p. 4-16). These minor defects would require routine maintenance but there is no need to plan rehabilitation in the short to medium term.

Table 4: Functional Criteria and Recommended Treatments and Priorities (TMH9)

Description	Condition	
Riding quality	G- Good	Functional Criteria
Skid resistance	G – Good	
Surface drainage	A – Adequate	
Road verge	S – Safe	
Sidewalks	VG – Very good	
Paved shoulder / Embankments.	VG – Very good	
Overall pavement condition.	VG – Very good	
Recommended treatments and priorities.	C – Maintenance must be programmed, such as sealing of minor cracks	Summary

The consequences of deferment are not serious, however, as the rate of deterioration is slow or traffic volumes are low (TMH 9: 1992, p. 48). Visual evaluations should be done on a regular basis, together with measurement of rut depths at periodic intervals. The objective of the recommended treatment, routine maintenance and rehabilitation programme is to ensure that the Nova Vida pavements last at least for the 15-year design period.

4. STRUCTURAL CAPACITY

The structural capacity was evaluated by performing Dynamic Cone Penetrometer (DCP) tests and relating the as-constructed structural capacity to pavement life. A mechanistic analysis was also performed and a resultant life was determined.

4.1 Dynamic Cone Penetrometer (DCP) Method

The DCP method used was based on the results for two approaches, the lightly stabilized sub-layers and granular sub-layers approach (Jordaan, 1994) since it was not clear in which condition the layers were.

4.1.1 Lightly stabilized sub-layers

The structural capacity of pavements containing lightly cement-treated bases, is dependent on the properties of the cement-treated layers. For these pavements, the structural capacity is a function of the DCP rate of penetration through the top 50 mm (excluding the asphalt overlay) of the pavement structure (DN_{50}) and the number of DCP blows required to penetrate to a depth of 200 mm (DSN_{200}).

Table 5 indicates the rate of deformation (R_L), for pavements with lightly cement-treated layers as given Jordaan (1994, p. 4-26). The structural capacity is a function of the DCP rate of penetration through the top 50 mm of the pavement structure (DN_{50}) and the number of DCP blows required to penetrate a depth of 200 mm (DSN_{200}). The rate of deformation for the pavements reveals a high predicted number of E80s that can be carried (except Roads 17 and 35 whose data is doubtful). Traffic counts executed in August 2005 showed that the E80 to be carried in the next 11 years is about 0,31 million E80s. This means that the roads have an expected life much greater than the 15 year design life.

Table 5: Rate of increase in rut depth based lightly stabilized sub-layers approach

Road description	15	15	17	35	39
Chainage (m)	10	125	100	100	0
DSN_{200} (number of DCP blows)	219	533	597	Refusal	995
Cement Stabilizer Content (%)	6	6	4,5	6	5
DN_{50} (mm/blow)	1,25	1,25	2,5	Refusal	0,15
R_L (Predicted rate of increase in rut depth (mm) per million E80s)	3,0	7,4	65,7	-	2,2
Existing rut depth (mm)	4	4	5	4	5

Although the laboratory design aimed at a C4 material, the DSN_{50} and DSN_{200} values are extremely high. This suggests that the material was much stronger than a lightly stabilized layer, and the assumption of analysis for a lightly stabilized pavement may not be appropriate.

4.1.2 Granular sub-layers

The lightly stabilized layers of these pavements may act like granular layers. The moisture regime in which the pavement is operating, is indicated by the total number of blows required to penetrate the pavement layers to a depth of 800 mm (DSN_{800}) (Jordaan, 1994, p. 4-25). The structural capacity is the number of standard 80 kN axles the pavement is able to carry before a rut depth of 20 mm develops as calculated in Table 6.

The structural capacity is much higher than is normally reported, as any prediction greater than 10 million E80s is beyond the model inference space. The values are however reported as calculated to demonstrate the unrealistically high predictions and inappropriateness of the granular layer model.

Table 6: Structural capacity values based on the granular sub-layers approach

Road	Chainage (m)	DSN_{800}	DSN_{800} (80^{th} percentile)	Structural capacity (10^6 x E80s)	Rut depth (mm)
Road 15	10	1 320	1 056	1 148	4
Road 15	125	1 422	1 138	1 492	4
Road 17	100	772	618	176	5
Road 35	100	Refusal	Refusal	High structural capacity	4
Road 39	0	1 801	1 441	3 408	5

4.1.3 Remaining life of pavement sections

Based on the above results, the predicted structural capacity in E80s to reach a rut depth of 20 mm is high. A traffic analysis done in August 2005 revealed that in the remaining 11 years of the design life the roads will carry a further 0,31 million E80s. This exceeds the original pavement design of 0,26 million E80s (ES 0,3).

The structural life of the pavements will therefore exceed the structural design period (design pavement life) of 15 years before reaching a rut depth of 20 mm, if the assumed conditions prevail and with implementation of the proposed maintenance programme. Note that in these analyses it was assumed that the rutting takes place in the structure, but indications (DCP blows) are that the stabilized layers had not rutted, but only the asphalt. The results may therefore not be completely applicable.

4.2 South African Mechanistic Design Analysis

The predicted life was obtained from an analysis based on the South African Mechanistic Pavement Rehabilitation Design Method (De Beer et al, 1994). Two packages were used, namely the ELSYM5 program, developed by the University of California and the mePADS, developed by the CSIR. The ELSYM5 calculations show an ultimate life of 4,5 x 10^6 E80s and the automated mePADS program calculations show 3,5 x 10^6 E80s for each pavement.

This is a pavement structure with a cemented base and subbase and the first phase will last up to the point where the subbase reaches the end of its predicted effective fatigue life (Theyse *et al.*, 1996, p. 34). The effective fatigue life phase and the equivalent granular phase (Phase 2) of the cemented material, are used to calculate the layer life for the cemented layer, as shown in Figure 1.

In Phase 2 (equivalent granular subbase) and Phase 3 (both equivalent granular subbase and base) the cemented material is in an equivalent granular condition; the values are included in Tables 7 and 8.

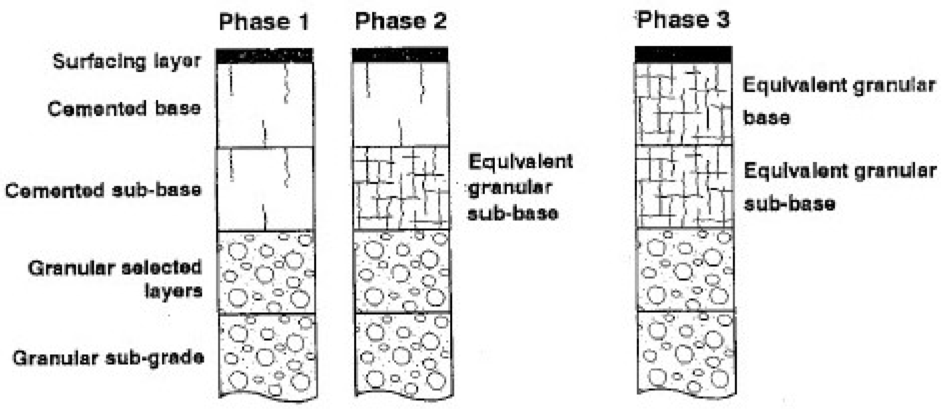

Figure 1: Nova Vida pavement structure with cemented base and subbase

The South African Mechanistic Pavement Rehabilitation Design Method calculations were done by hand, according to the RR 91/242 Manual (De Beer *et al.,* 1994) and simulations were done using the mePADS program on each road, with two different sets of Elastic Modulus values, as shown in Table 6, named iterations 1 and 2 to show sensitivity.

This analysis method does not take into account the stabilized pebble material actually used for construction, but rather a similar G5 material stabilized to a C4 quality. More advanced mechanistic analysis methods incorporating advanced materials are not currently available, as confirmed by De Beer et al. (1999, p. 29).

Table 6: Elastic Modulus values used for Roads 15, 17, 35 and 39

Elastic Modulus (MPa)					
1st Iteration			2nd Iteration		
Base/Subbase			Base/Subbase		
Phase 1	Phase 2	Phase 3	Phase 1	Phase 2	Phase 3
1 500	1 000	750	1 500	800	300

Table 7: ELSYM5 predictions of pavement life

Road	Calculated E80s			
	1st Iteration			
	Phase 1	Phase 2	Phase 3	Total
15 and 39	4,5 x 10^6	2,2 x 10^5	3,1 x 10^4	4,75 x 10^6
17 and 35	4,3 x 10^6	1,1 x 10^5	1,1 x 10^5	4,52 x 10^6
	2nd Iteration			
15 and 39	4,5 x 10^6	1,9 x 10^5	3,2 x 10^4	4,72 x 10^6
17 and 35	4,3 x 10^6	1,1 x 10^5	3,7 x 10^4	4,45 x 10^6

Table 8: mePADS phase results for two iterations

Road	mePADS E80s			
	1st Iteration			
	Phase 1	Phase 2	Phase 3	Total
15 and 39	1,6 x 10^6	1,3 x 10^6	9,6 x 10^5	3,00 x 10^6
17 and 35	1,6 x 10^6	1,3 x 10^6	10 x 10^5	3,00 x 10^6
	2nd Iteration			
15 and 39	1,6 x 10^6	1,1 x 10^6	1,2 x 10^5	2,71 x 10^6
17 and 35	2,1 x 10^6	1,1 x 10^6	1,2 x 10^5	3,32 x 10^6

By July 2005 the pavements were showing rut depths between 4 and 5 mm, as a result of the asphalt deformation, with high structural capacities from DCP test values. This was due to the hardness of the pavement layers, which had still probably not reached the full pre-cracked condition, confirming that the total fatigue life (Phase 1) of the pavements had not yet elapsed (and that they had not yet reached post-cracked or an equivalent granular state, i.e. Phases 2 or 3).

The structural life of the pavements will therefore exceed the structural design period (design pavement life of 0,26 million E80s, i.e. ES 0.3) of 15 years before reaching a rut depth of 20 mm, if the assumed conditions prevail with the implementation of the rehabilitation and maintenance programme.

5. CONCLUSIONS AND RECOMMENDATIONS

It was found that stabilization of the mix of river gravel pebbles and Muceque Red Sands with OPC was a feasible solution. Initially 6% cement was used, but excessive shrinkage cracking resulted in the cement content being reduced to 4.5%. After about 5 years of service the roads had rutted to between 4 and 5 mm. It was found that this rutting occurred primarily in the 25 mm continuously graded asphalt surfacing. This level of rutting is still considered satisfactory.

The DCP required unusually large numbers of blows to reach a penetration of 800 mm. Because of the high stabilizer content in the base and subbase the DCP test results showed that the pavement life is very high. The DCP analysis procedures assume that rutting takes place throughout the pavement, which is not the case as most of the rutting occurred in the asphalt surfacing. This approach therefore has limited applicability.

The mechanistic analysis suggests that the life of the pavements would be in excess of 3.5 million E80s, which is much greater than the design life of 0.3 million E80s. This is due to the high stabilizer content in the base and subbase. Although the stabilizer content could have been reduced, the testing regime showed that durability could have been compromised. It is therefore important to note that in any performance analysis not only the structural capacity but also durability must be considered.

Deflection testing, such as with the Falling Weight Deflectometer (FWD) or the Benkelman Beam, should be done to yield a further structural evaluation of these pavements to confirm their structural capacity and life. Maintenance of these streets must be carried out to ensure that the design life, and potentially a much longer life, can be achieved.

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