THE DEVELOPMENT OF A STRUCTURAL DESIGN CATALOGUE FOR LOW VOLUME ROADS

H. WOLFE, G.D. VAN ZYL*, P. PAIGE-GREEN*, S.J. EMERY**

Consultant, Division of Roads and Transport Technology, P O Box 395, Pretoria, 0001.

*Division of Roads and Transport Technology, P O Box 395, Pretoria, 0001.

**Professor, Department of Civil Engineering, University of Stellenbosch, Stellenbosch, 7600.

SUMMARY

The procedure used for the development of a pavement design catalogue for low volume roads (LVRs) comprising granular bases and subbases, is discussed in this paper. A data base on the composition and performance of a number of existing LVRs was used to select the most appropriate design procedure for use in the development of the design catalogue. The Factor of Safety, DCP and S-N methods currently used for the mechanistic design of granular pavement layers, were evaluated. The S-N method was found to predict carrying capacities that were relatively close to the carrying capacities estimated from field measurements and this method was consequently used for the development of the catalogue. The catalogue was developed by stepwise addition of 150 mm thick pavement layers and/or stepwise increase in material strength to optimise the pavement structure for the specific traffic category. The catalogue caters for traffic carrying capacities between 5 000 and 800 000 E80's.

OPSOMMING

Die prosedure wat gebruik is vir die ontwikkeling van 'n plaveisel ontwerpkatalogus vir lae volume paaie bestaande uit granulêre kroon- en stutlae, word in die referaat bespreek. 'n Databasis vir die samestelling en gedrag van 'n aantal bestaande lae volume paaie is gebruik om die mees toepaslike ontwerpmetode vir gebruik in die ontwikkeling van die ontwerpkatalogus te bepaal. Die Veiligheidsfaktor, DKP en S-N metodes wat huidiglik vir die meganistiese ontwerp van granulêre plaveiselmateriale gebruik word, is geëvalueer. Daar is gevind dat die S-N ontwerpmetode verkeersdravermoëns voorspel wat relatief naby is aan waardes uit veldmetings verkry. Die katalogus is ontwikkel deur stapsgewyse byvoeging van 150 mm dik plaveisellae en/of stapsgewyse verbetering van materiaalsterkte ten einde die plaveisel te optimiseer vir die spesifieke verkeerskategorie. Die katalogus maak voorsiening vir verkeersdravermoëns van 5 000 E80's tot 800 000 E80's.

INTRODUCTION

The current pavement design catalogue for paved roads given in TRH 4 (CSRA, 1985a) caters for a lowest traffic class of less than 200 000 E80's per lane (E0). The implication of this is that the recommended designs for E0 traffic are conservative for traffic less than 50 000 or 100 000 E80's.

The optimum traffic volume for gravel roads determined from economic considerations falls noticeably short of the optimum traffic volume of the present standard of paved road (TRH 4 catalogue) when determined on the same basis. For example, paved roads with relatively low traffic volumes do not derive the total road user benefits which they ought for that standard of facility, and on the other hand, gravel roads with high traffic volumes incur excess road user costs higher than those which would have accrued on a road of a higher standard. This results in a marked break in the spectrum of road standards between unpaved roads and the lowest standard of paved roads. It has been clearly illustrated (SABITA, 1992) that the upgrading of many unpaved roads to a paved standard can be economically justified at low traffic volumes (as low as 20 vehicles per day in cases where a surfacing is put on an existing gravel road with minor reshaping of the gravel road required). The minimum TRH 4 (CSRA, 1985a) standard is, however, considered to be excessively conservative for these roads and although the risk of failure is low, the ever increasing cost of construction usually prohibits their use.

This paper describes the procedure which was used to develop a design catalogue for low volume roads (LVRs) with design traffic as low as 5 000 E80's in which granular materials as specified in TRH 14 (CSRA, 1985b) are utilised for the bases and subbases.

A data base on the composition and performance of a number of existing LVRs was used to select the most appropriate design procedure for use in the development of the design catalogue. The TRH 4 (CSRA, 1985a) E0 traffic category was subdivided into 4 categories and E1 into 2. Once a suitable design procedure had been selected, the catalogue was developed by stepwise addition of 150 mm thick pavement layers and/or stepwise increase in material strength to optimise the pavement structure for the specific traffic category.

DATA BASE

The data base was developed by Paige-Green (1991a to g, 1992a to b) who determined the pavement composition of 23 LVRs in the Orange Free State and Transvaal from test pits, in-situ density measurements, laboratory tests and DCP soundings. The tests were conducted in the outer and inner wheel paths and the centre of the road. The data thus generated were used to classify the pavement materials according to TRH 14 (CSRA, 1985b) standards. The resultant pavement structures were analyzed with various mechanistic procedures.

Paige-Green also measured ruts in the outer and inner wheel paths at the test positions and estimated the traffic volumes. The traffic was estimated from traffic counts conducted by the road authority at nearby positions. Results of these traffic counts are normally presented as average daily traffic (ADT) and percentage heavy vehicles. The following conversion factors as used by the Transvaal Provincial Administration (TPA) prior to 1992, were used to convert the traffic to E80's:

- Number of axles per heavy vehicle = 2,3.
- E80 per axle = 0,25.

The cumulative number of E80's over the pavement to the date of testing were then determined from the following equation:

$$E80's = (ADT/2 \times \%\ \text{Heavy vehicles} \times 2{,}3 \times 0{,}25) * 365 * \text{Age in years}$$

The ADT was divided by 2 to get the traffic per lane (or per direction).

The data are summarized in Table 1. The number of E80's necessary to form a 20 mm rut was calculated from Paige-Green's data by linear extrapolation of the measured rut and traffic data and are also given in Table 1. This will give a conservative indication of the carrying capacity of the pavement because the rate of permanent strain (rut) development with load applications (traffic) is not a constant (linear), but continuously diminishing until it eventually approaches a constant value. The number of load repetitions where the rate of permanent strain approaches a constant value, depends inter alia on the stress associated with the applied load, material type and initial compaction of the pavement layer (Wolff, 1992).

DESIGN PROCEDURE SELECTION

The design procedure used for the compilation of the LVR catalogue of designs, was selected by comparing the performance of as-built LVR pavements with granular bases and subbases as determined form field measurements, to performance predicted with various design methods. Performance was predicted using the Factor of Safety (FOS) method proposed by Maree (1978), the Dynamic Cone Penetrometer (DCP) method proposed by Kleyn et al (1984, 1987) and the S-N method proposed by Wolff (1992). The South African design methods (FOS and DCP) calculate pavement carrying capacities in terms of 80 kN axles. The traffic on the as-built LVRs was also converted to E80's. Therefore, in order to be able to compare the carrying capacities calculated with different design methods to one another and to the field obtained values, the carrying capacities calculated with the S-N method also had to be in terms of E80's.

The FOS method is currently used in South Africa for the mechanistic design of granular layers in pavement structures. It is described by Maree in various publications (Maree, 1978 and 1982). The elastic material parameters for granular materials proposed by Freeme (1984) were used in the

TABLE 1: Summary of data from field measurements on LVR's (Paige-Green, 1991 a-g; 1992a,b).

ROAD NO.	PAVEMENT STRUCTURE	AGE (years)	N*	BASE PARENT MATERIAL	CUMM. TRAFFIC (E80's)	RUT (mm)	ESTIMATED TRAFFIC TO 20 mm RUT (E80's)
D514	125 G4**/200 G6/G8	6	2	Granite	$5,4x10^4$	11	$9,8x10^4$
D736	200 G6/100 G6/G7	6	2	Shale	$1,3x10^4$	4	$6,5x10^4$
D466-E	200 G5/200 G6/150 G6/G6	7	2	Laterite	$7,2x10^4$	8	$1,8x10^5$
D466-W	185 G5/100 G6/G8	7	2	Laterite	$9,2x10^4$	6	$3,1x10^5$
D390	175 G6/100 G6/G8	8	2	Laterite	$3,5x10^4$	16	$4,4x10^4$
D2485	200 G6/200 G6/G5	8	4	Shale	$8,8x10^3$	6	$2,9x10^4$
D804-8	200 G6/200 G6/G7	4	5	Andesite	$8,8x10^3$	11	$1,6x10^4$
D804-11	200 G5/200 G5/G5	4	5	Andesite	$8,8x10^3$	12	$1,5x10^4$
D804-19	200 G6/200 G6/G6	4	5	Calcrete	$7,3x10^3$	2	$7,3x10^4$
S191	150 G4/150 G5/150 G6/G6	9	3	Dolerite	$3,7x10^4$	14	$5,3x10^4$
S65-6	175 G4/175 G5/G7	11	3	Dolerite	$3,6x10^4$	8	$9,0x10^4$
S65-57	120 G4/200 G6/G6	11	3	Dolerite	$2,8x10^4$	6	$9,3x10^4$
S63	150 G4/200 G5/G5	10	3	Dolerite	$1,6x10^4$	7	$4,6x10^4$
P13/2	125 G4/150 G5/100 G6/G7	30	3	Dolerite	$1,1x10^5$	12	$1,8x10^5$
D467	150 G5/200 G6/G6	7	2,8	Norite	$3,8x10^4$	8	$9,5x10^4$
D540	200 G6/200 G6/G6	5	2,8	Laterite	$1,3x10^4$	10	$2,6x10^4$
D410-5	200 G5/200 G5/G5	3	5	Chert wad	$4,4x10^3$	8	$1,1x10^4$
D410-1	200 G5/200 G6/G5	7	5	Chert wad	$2,3x10^4$	5	$9,2x10^4$
P172-2	150 G4/200 G5/G6	7	5	Shale	$3,6x10^4$	6	$1,2x10^5$
D132	200 G5/150 G5/G7	4	5	Shale	$8,7x10^3$	7	$2,5x10^4$
D804	200 G4/200 G4/G5	7	5	Andesite	$2,1x10^4$	5	$8,4x10^4$
D404	140 G4/150 G6/G8	8	5	Shale	$5,8x10^3$	6	$1,9x10^4$
D421	125 G4/150 G5/200 G6/G7	4	1,8	Shale	$1,3x10^4$	12	$2,2x10^4$

* Weinert climate factor N (Weinert, 1964)

** Notation - 125 mm layer of G4 quality material. Bottom layer indicates in-situ material.

analysis of the as-built LVRs. The MECDE computer code (CICTRAN, 1988) was used for the calculation of the horizontal and vertical stresses in the centre of the granular layers.

The DCP method was developed by Kleyn (Kleyn, 1984; Kleyn and Van Zyl, 1987). Kleyn uses the penetration depth (mm) per blow of the dynamic cone penetrometer (DN value) to classify material according to the TRH 14 (CSRA, 1985b) classification. A narrow range of DN values is given for each material type from G1 to G10. The average of the range was used as the DN value for a specific material type in the calculation of the carrying capacities of the as-built LVR's.

The S-N design method is described by Wolff (1992). The G2 to G6 materials were considered to be non-linear elastic. Non-linear elastic properties proposed by Wolff (1992) were used in the analysis. The G7 to G10 materials were considered to be linear elastic. Resilient moduli for G7 to G10 material were again obtained from Freeme (1984). The resilient moduli proposed for dry material in the paper were used in the analysis. The MICHPAVE computer code (Harichandran and Yeh, 1988) was used for the calculation of stresses and strains in the granular and subgrade layers.

The FOS and DCP design methods are well-known to the South African pavement engineering fraternity and are therefore not discussed in more detail. The S-N method, on the other hand, was introduced fairly recently. The application of the method in the determination of the carrying capacities of the as-built LVR's is therefore discussed in more detail in the following paragraphs.

The carrying capacity of each pavement structure was determined with the S-N method as follows:

> The sum of the principal stresses theta (Θ) was calculated in the centre of the base and subbase layers (normally G4 to G6) in the pavement structure for the standard 40 kN double wheel load. The vertical elastic strain was calculated on top of the first subgrade layer (G7 to G10) for the same load. A failure criterion of 20 mm permanent deformation (rut) for the total structure was considered. This rut was divided equally between the base, subbase and subgrade layers to provide a starting point. The number of repetitions needed to accumulate the allocated rut in each of the base, subbase and subgrade layers were then determined by executing the following steps:
>
> For base and subbase (G4 to G6 material):
>
> 1. Enter the S-N graph for the specific material type (e.g. Figure 1) on the vertical axis at the calculated Θ;
> 2. Move horizontally to the allocated strain (allocated rut divided by layer thickness);
> 3. Move vertically to read off the number of load repetitions on the horizontal axis.

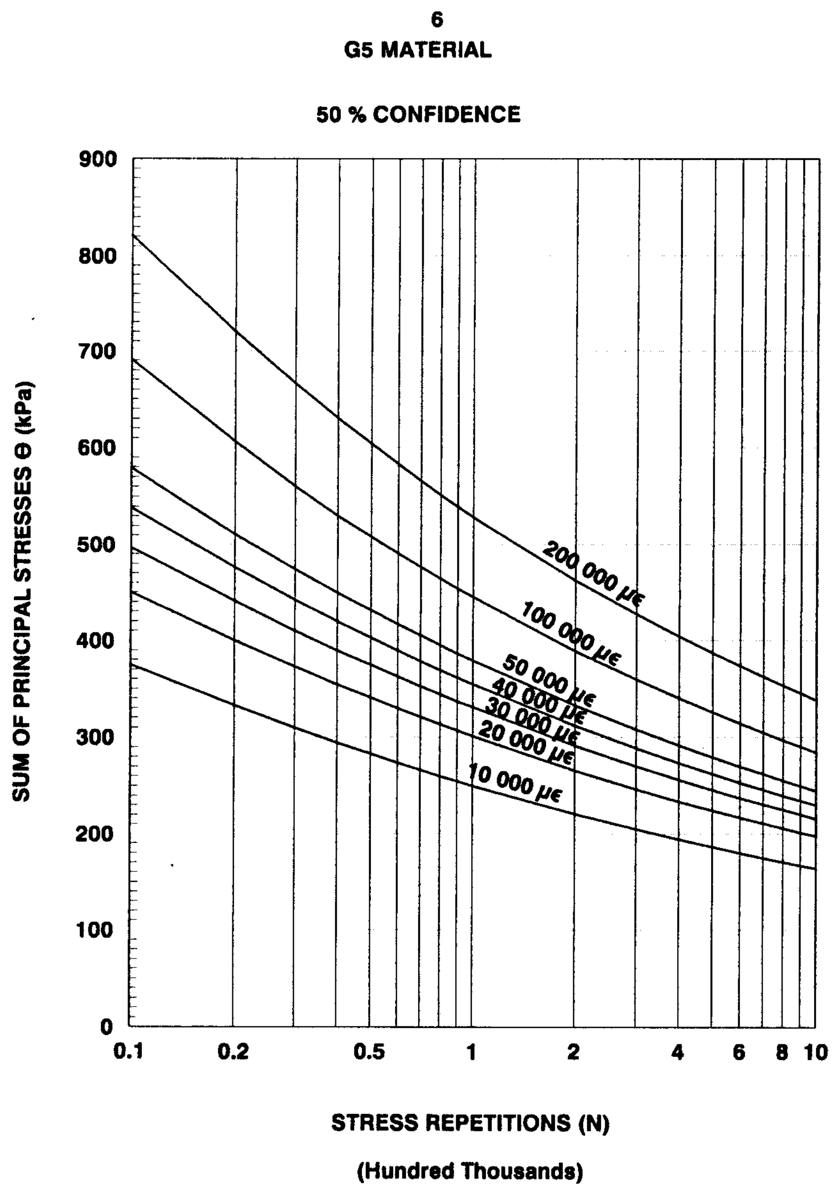

FIGURE 1: S-N GRAPH FOR G5 MATERIAL AT A 50 PER CENT CONFIDENCE LEVEL.

For subgrade (G7 to G10 material):

4. Enter the subgrade strain criteria graph (Figure 2) on the vertical axis at the calculated vertical elastic strain on top of the subgrade;
5. Move horizontally to the allocated rut (Category A, B and C refers to 8, 12 and 18 mm rutting respectively (Paterson, 1978));
6. Move vertically to read off the number of load repetitions on the horizontal axis.

The number of load repetitions obtained for each of the base, subbase and subgrade layers will most probably not be the same. The aim is to get the number of load repetitions the same for each of the base, subbase and subgrade layers. This number of load repetitions will then constitute the carrying capacity of the pavement structure. In order to get the number of load repetitions the same for each of the base, subbase and subgrade layers, less permanent deformation (rut) is then allocated to the layers with high numbers of load repetitions and more to layers with low numbers of load repetitions. The total rut must still not exceed 20 mm. Steps 1 to 6 are then repeated. The number of load repetitions obtained for each of the base, subbase and subgrade layers should now be closer to one another. Another re-allocation of permanent deformation (rut) and repetition of steps 1 to 6 may be necessary before the number of load repetitions determined for each of the base, subbase and subgrade layers are approximately the same. This number then is the carrying capacity of the pavement structure.

Using graphs like Figures 1 and 2 is a tedious process. However, the process can be speeded up considerably if an electronic spreadsheet and equations for the different curves are used.

Use of the subgrade strain criteria graph (Figure 2) for designing against permanent deformation in Category A, B and C roads, implies that almost all rutting in pavements takes place in the subgrade and very little in the structural layers. For example, the Category A function in Figure 2 gives the number of vertical strain repetitions induced by a certain wheel load (normally 40 kN) that will cause 8 mm permanent deformation in the subgrade. This is near to the 10 mm permanent deformation that is allowed in Category A roads. Similarly the Category B and C functions give the number of strain repetitions that will cause 12 and 18 mm permanent deformation respectively in the subgrade. These values again are near to the 15 and 20 mm permanent deformation allowed for Category B and C roads respectively. The S-N method allows for rutting in the granular structural layers and also in the subgrade as shown in the preceding paragraphs. Figure 2 is used to assess the rutting in the subgrade and the Category A, B and C lables for the different functions on the graph, should actually be replaced with 8, 12 and 18 mm permanent deformation. It is possible for a low-volume road, depending on the pavement composition, to have 12 mm permanent deformation occurring in the base and subbase layers cummulatively and only 8 mm permanent deformation occuring in the subgrade. In such cases, the Category A function in Figure 2 would be used to determine the number of vertical strain repetitions.

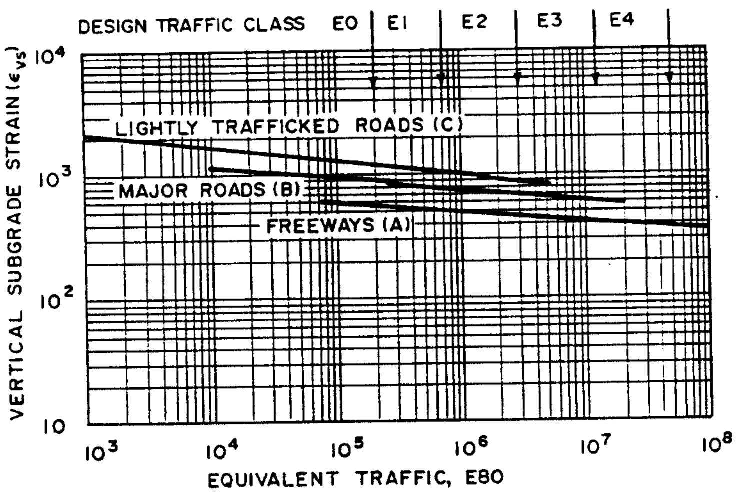

FIGURE 2: RECOMMENDED VERTICAL SUBGRADE STRAIN CRITERIA FOR DIFFERENT ROAD CATEGORIES.

The 50 per cent confidence level curves are used for the design of granular layers in LVR pavements. It was found that curves based on a 80 per cent confidence level led to conservative designs when compared to information from the data base. A 50 per cent design confidence level for LVRs is also recommended by Hudson et al (1987).

The results of the calculations are shown in Figure 3. The S-N method predicts carrying capacities that are relatively close to the carrying capacities estimated from field measurements for the different pavement structures. The DCP method predicts higher carrying capacities than estimated from the field measurements and predicted with the S-N method. The FOS method gives the least consistent prediction of carrying capacity. It also has the poorest correlation with the carrying capacities estimated from field measurements when compared with the other two prediction methods.

Comparison of field performance of LVRs with performance predicted with various design methods, thus indicated that the S-N method compares well with field measured performance. The S-N method was thus selected for compiling the LVR design catalogue. The S-N curves are ideally suited for designing pavements comprising lower quality gravels as bases and subbases against permanent deformation. The curves were developed for the materials at their equilibrium moisture content and predictions of permanent deformation can be expected to be reasonable where pavement layers are kept dry through maintenance action or the application of thick bituminous seals (e.g. Cape seal) or asphalt surfacings in wet regions or regions where maintenance action is expected to be infrequent (SABITA, 1992).

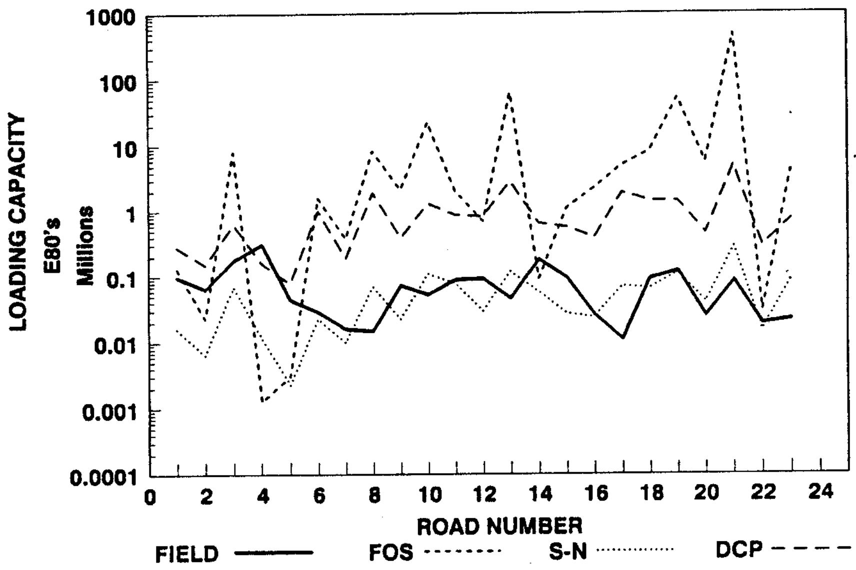

FIGURE 3: COMPARISON BETWEEN LOADING CAPACITIES OF PAVEMENTS STRUCTURES DETERMINED FROM FIELD MEASUREMENTS AND WITH VARIOUS DESIGN METHODS.

CATALOGUE DEVELOPMENT

The proposed design catalogue for pavements with granular bases and subbases is given in Table 2. The S-N method was used to compile a design catalogue for LVRs with granular bases and subbases for moderate to dry and wet regions. An 80 kN axle was used in the calculations (40 kN double wheel load; 520 kPa tyre pressure). A failure criterion of 20 mm permanent deformation was used in the compilation of the catalogue. However, any rut depth can be considered as failure criterion.

Elastic deflections on the surface of the proposed structures play an important role in the decision on the type of surfacing seal to be used. The design of the surfacing seals should therefore receive careful attention as elastic deflections may be higher than normal (SABITA, 1992).

A layer thickness of 150 mm was used for G4 to G6 materials because this material is normally obtained from borrowpits and the difference in cost between a 100 and 150 mm layer is therefore not excessive. Where the cost of the material is significant, such as crushed stone, the use of a thinner layer was considered. However, layers less than 125 mm in thickness were deemed impractical from a construction viewpoint.

TABLE 2: Proposed design catalogue for LVR's with granular bases and subbases compiled with the S-N design method.

TRAFFIC CLASS	TRAFFIC (80's)	PAVEMENT STRUCTURES		
		MODERATE TO DRY REGIONS	WET REGIONS	LOW MAINTENANCE
E0-1	< 5 000	# 150 G6* 150 G8 150 G9 G10**	150 G5 150 G7 150 G9 G10	25 A+ 150 G6 G10
E0-2	5 000 - 30 000	150 G5 150 G7 150 G9 G10	150 G4 150 G6 150 G8 G10	25 A 150 G6 150 G7 G10
E0-3	30 000 - 100 000	150 G4 150 G6 150 G8 G10	150 G4 150 G5 150 G6 150 G7 G10	25 A 150 G5 150 G9 G10
E0-4	100 000 - 200 000	150 G4 150 G5 150 G8 G10	150 G3 150 G6 150 G9 G10	25 A 150 G4 150 G9 G10
E1-1	200 000 - 400 000	150 G4 150 G5 150 G7 150 G9 G10	150 G3 150 G6 150 G8 G10	25 A 150 G4 150 G8 G10
E1-2	400 000 - 800 000	125 G2 150 G6 150 G9 G10	125 G2 150 G5 150 G9 G10	25 A 150 G4 150 G5 150 G8 G10

\# Double surface treatments assumed on all pavement structures unless otherwise indicated.
* Notation - 150 mm layer of G6 quality material.
** **Pavement assumed to be supported by in-situ material having a CBR of not less than 3 (G10) and semi-infinite depth. Layers shown in the catalogue with lower strength than the in-situ subgrade may therefore be omitted provided that adequate strength exists for the total pavement depth.**
\+ 25 mm asphalt.

The non-linear elastic material characteristics for G2 to G6 material used in the compilation of the catalogue were obtained from backcalculation of MDD measured deflections (Wolff, 1992). Resilient moduli for G7 to G10 material were taken from the paper by Freeme (1984). The material was considered to be linear elastic. The pavement structures for the moderate to dry regions were determined by using the resilient modulus values proposed by Freeme for dry material. The pavement structures for the wet regions were determined by using the resilient moduli values proposed by Freeme (1984) for wet material. The material properties used in the calculations are given in Table 3.

The pavements are assumed to be supported by in-situ material having a CBR of not less than 3 (G10). Layers shown in the catalogue with lower strength than the in-situ subgrade may therefore be omitted provided that adequate strength exists for the total pavement depth.

The materials in the catalogue are **classified** inter alia by their soaked bearing strength in terms of CBR, although the proposed pavement structures relate to **performance** at equilibrium moisture content. When existing roads are to be upgraded, the pavement materials of the existing road need therefore be classified in terms of the soaked CBR in order to relate to the catalogue. Procedures to accomplish this with the DCP and other test methods, are described by Van Zyl et al (1993).

CONCLUSIONS AND RECOMMENDATIONS

The paper described a comparison of performance calculated with various pavement design methods with performance measured from actual LVRs with granular bases and subbases. From the various pavement design methods evaluated, it was concluded that the S-N method recently developed for the mechanistic analysis of granular materials gave the best correlation with the measured performance. This design method was further used in the development of a design catalogue for LVRs with granular bases and subbases, which is recommended for use in the design of LVRs or the upgrading of gravel roads in South Africa.

ACKNOWLEDGEMENTS

The paper is based on research done by the Division of Roads and Transport Technology (DRTT) of the CSIR for the Department of Transport. The Director General of the Department of Transport and the Director of DRTT are thanked for their permission to publish this paper.

TABLE 3: Material properties used for calculations with MICHPAVE in the mechanistic analysis of granular pavement structures proposed for the design catalogue (in imperial units).

MATERIAL TYPE	K_0*	K_1** (psi)@	K_2#	POISSON RATIO	COHESION (psi)	ϕ+ (degrees)	DENSITY (pcf) (kg/m³)
G2	0,80	17 994	0,35	0,20	0	45	140 2242
G4	0,75	2717	0,44	0,25	0	45	135 2162
G5	0,70	9611	0,22	0,33	0	43	130 2082
G6	0,60	6848	0,36	0,35	0	40	130 2082
MATERIAL TYPE	**K_0**	**MODULUS OF ELASTICITY (psi)**		**POISSON RATIO**			**DENSITY (pcf)**
		DRY	**WET**				
G7	0,50	34 783	17 391	0,35	-	-	110 1762
G8	0,50	26 087	13 043	0,35	-	-	110 1762
G9	0,50	20 290	10 145	0,35	-	-	110 1762
G10	0,50	13 043	6 522	0,35	-	-	110 1762

* Earth pressure coefficient used in MICHPAVE.

** Value of K_1 in the equation $M_r = K_1 \Theta^{K2}$ used in MICHPAVE to describe the non-linearity of granular materials.

@ Program written for imperial units (1 psi = 6,9 kPa).

Value of K_2 in the equation $M_r = K_1 \Theta^{K2}$ used in MICHPAVE to describe the non-linearity of granular materials.

\+ Angle of internal friction from the Mohr-Coulomb failure theory.

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