Effective Moduli and Stress Dependence of Pavement Materials as Measured in Some Heavy-Vehicle Simulator Tests

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The purpose of this paper is to describe a method for determining field-effective moduli of pavement materials and to demonstrate stress dependence, especially of unbound materials. The effective moduli were determined from resilient deflections measured with a multidepth deflectometer at different depths within pavement structures. The results of measurements on four structures are reported. These structures cover light, unbound pavements to stronger inverted structures. The heavy-vehicle simulator was used to determine the moduli at different wheel loads and at various stages of trafficking. The ELSYM5 linear elastic-layered program together with the measured depth deflections were used in an iteration technique to determine effective moduli. The stress-stiffening behavior of granular materials and the stress-softening behavior of subgrade materials were clearly demonstrated. The constants in the models normally used to describe the behavior of these materials could be determined from the field results at various stages of trafficking and under changing moisture conditions. Laboratory and field values of stress-dependent moduli for granular materials were compared and a shift factor was determined. The effect of subgrade support on the effective moduli was also illustrated.

Linear elastic-layered mechanistic models for analyzing pavement structures are currently used to predict pavement behavior under assumed conditions of traffic loading and environment (1-3). In these models, each layer is characterized by a resilient modulus (E_R) and Poisson's ratio (ν_R). The accuracy of the input values influences the reliability of the answers, and considerable research has been done on material characterization (4). The resilient properties (E_R, ν_R) of pavement materials may be determined in the laboratory by various tests in which field stress and environmental conditions are simulated. The resilient moduli may also be determined from measured field deflections. For the latter technique, input moduli are selected so that the calculated deflection corresponds to the measured deflection (5,6). Such moduli are termed "effective" moduli.

In recent years, it has become evident that it is difficult or even impossible to predict the behavior of road pavements solely from laboratory test data. Accordingly, there has been considerable interest worldwide in the development of methods for the accelerated testing of full-scale road pavements. Equipment for accomplishing this has been developed in such diverse countries as the United States, Denmark, Australia, Germany, Japan, and South Africa. The South African contribution, the heavy-vehicle simulator (HVS) (7) developed by the National Institute for Transport and Road Research (NITRR) of the Council for Scientific and Industrial Research (CSIR), is perhaps the most versatile of the various test rigs developed to date because it is the only equipment that is mobile and that can be used on normally constructed road pavements. Material properties to be used as inputs to mechanistic models can be determined from actual field measurements. The purpose of this paper is to describe effective field moduli and the stress dependence of some pavement materials (granular base layers and typical subgrades) as measured in HVS tests. It is beyond the scope of this paper to describe the performance of the pavement structures tested. It will be shown that stress dependence could be important when pavement structures are analyzed under varying wheel loads.

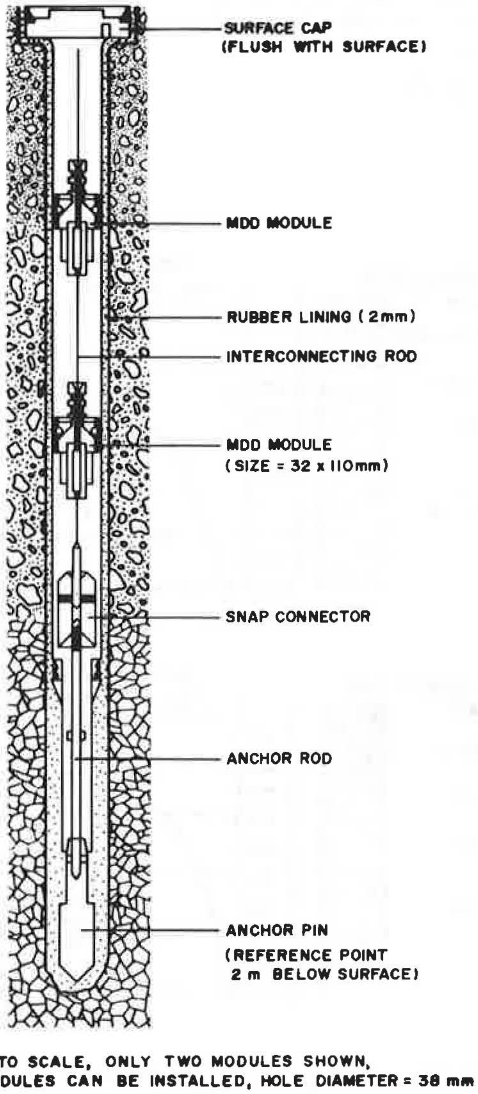
DEPTH DEFLECTION MEASUREMENTS

The resilient deflection on the road surface has been used as an input parameter for many evaluation models (8). The shape of the deflection bowl together with the peak deflection can be used to determine the properties of the pavement layers. At NITRR, a technique has been developed for determining layer properties by measuring the resilient deflections at various depths within the pavement structure. The device developed for this purpose is called the multidepth deflectometer (MDD) (9). Three MDDs are normally installed in an existing pavement structure to be tested by the HVS. A special technique has been developed to drill a hole 38 mm in diameter to a depth of 2 m (9) without major disturbances to pavement layers. The hole is lined with a thin rubber lining, which does not influence deflection measurements but prevents moisture and loose material from damaging the transducers. At the end of an HVS test, the structure is opened and the MDD holes are inspected for any abnormalities. Measurements of resilient deflections with depth are taken throughout the test under various wheel loads and these yield a very good record of the change in structural response of the pavement. The MDD uses a reference point at a depth of 2 m, and up to six modules can be installed in one hole (Figure 1). The MDD modules are normally placed at the interfaces of the layers. Figure 2 shows typical depth-deflection curves measured by an MDD. The surface deflection is usually also independently measured with a deflection beam and correlates very well with the deflection measured by the MDD module just below the surface cap. The deflection beam has also been used to verify that the MDD hole does not significantly influence the resilient deflection of the pavement structure.

DETERMINATION OF EFFECTIVE RESILIENT MODULI

The surface and depth deflections discussed in the previous section can be used to determine the effective elastic moduli of the various pavement layers by using linear elastic theory (5,6) and (according to G. Ahlborn of the University of California at Berkeley) a linear elastic-layered program such as ELSYM5. An iteration technique is normally followed. The initial values of the various moduli are estimated and depth deflection curves similar to those given in Figure 2 are calculated by the program. The calculated values are compared with the measured ones and the moduli are adjusted until measured and calculated values agree. With experience, this iteration technique converges within a few iterations. Initially, a modulus is assigned to the subgrade so that the calculated deflection agrees with the measured deflection on the subgrade. Thereafter, the moduli of the other pavement layers are determined. The slope of the depth deflection curve at any point is an indicator of the modulus of the material at that depth. When the measured slope is steeper than the calculated one, the modulus of the material has to be increased, and vice versa.

Figure 1. MDD installed in pavement structure.

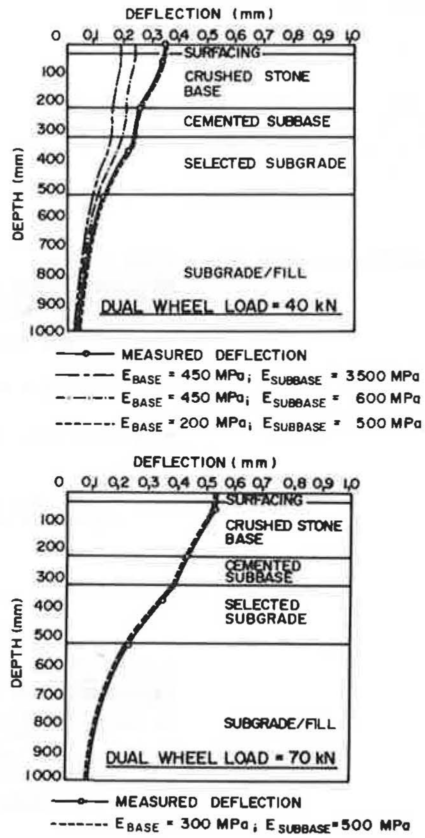


(The depth below the surface will also influence the value of the modulus.) The elastic modulus calculated in this way is called the effective modulus (5).

Figure 2 shows typical deflection curves fitted through the mean MDD deflections under a 40-kN and a 70-kN dual wheel. The dual wheel of the HVS was approximated by two circular loaded areas with contact stress equal to the measured contact stress in the field. This approximation is reasonable at low wheel loads but less accurate at high loads, because the actual contact area is of a long rectangular shape at high loads.

By determining the effective moduli of the different layers at various stages of trafficking, the behavior of the different layers can be monitored throughout the test. The stress dependence of the various pavement materials can also be established by measuring deflections at different wheel loads. These measurements can be used to determine the nonlinear effective moduli. In HVS tests, the wheel load normally varies between 20 and 100 kN. The deflections are therefore measured under real or overloaded road conditions (equivalent axle loads of 40-200 kN).

Figure 2. Measured and calculated depth deflections (Road P157/1).



The moduli determined from the depth deflections can be used as input values for further detailed mechanistic analyses of the structures tested. The confidence in the calculated stresses and strains will be higher because the models have been calibrated according to actual measured depth deflections. It is also possible to draw meaningful conclusions from the values of the determined moduli. For example, a cement-stabilized subbase layer may exhibit an initial effective modulus higher than 3000 MPa (3). However, in the postcracked phase, the modulus may drop below 500 MPa. The state of the layer can therefore be determined from the measured resilient depth deflections.

STRUCTURES TESTED BY HVS AND TYPICAL DEPTH DEFLECTION CURVES

Four typical pavement structures will be considered. Two structures represent the light unbound pavements that were used for two-lane rural roads in the 1950s. The other two structures have crushed-stone bases with cemented subbases (so-called inverted designs) and are currently used for the more important routes and for some freeways in the Transvaal Province. Figures 3 and 4 show the structures as well as their typical depth deflection curves. The material qualities are indicated by the material codes currently used in national pavement design documents (10,11). The key to these codes is given in Figure 5. As expected, the light pavements (Figure 3) show much greater deflections than the heavy pavements (Figure 4).

Only the initial depth deflections (at the start of the HVS test) could be measured on the light pavement structures, because the MDD holes collapsed during the tests. On the heavy structures, the deflections could be monitored throughout.

FIELD EFFECTIVE MODULI

The effective elastic moduli determined from the measured resilient depth deflections are shown in Tables 1 and 2. The material codes for the different pavement layers are also given.

Granular materials, showing stress-stiffening behavior, are most often described by a model that gives the modulus as a function of the first stress invariant:

$$M_R = K_1 \theta^{K_2} \quad (1)$$

Figure 3. Light pavements and their depth deflection curves.

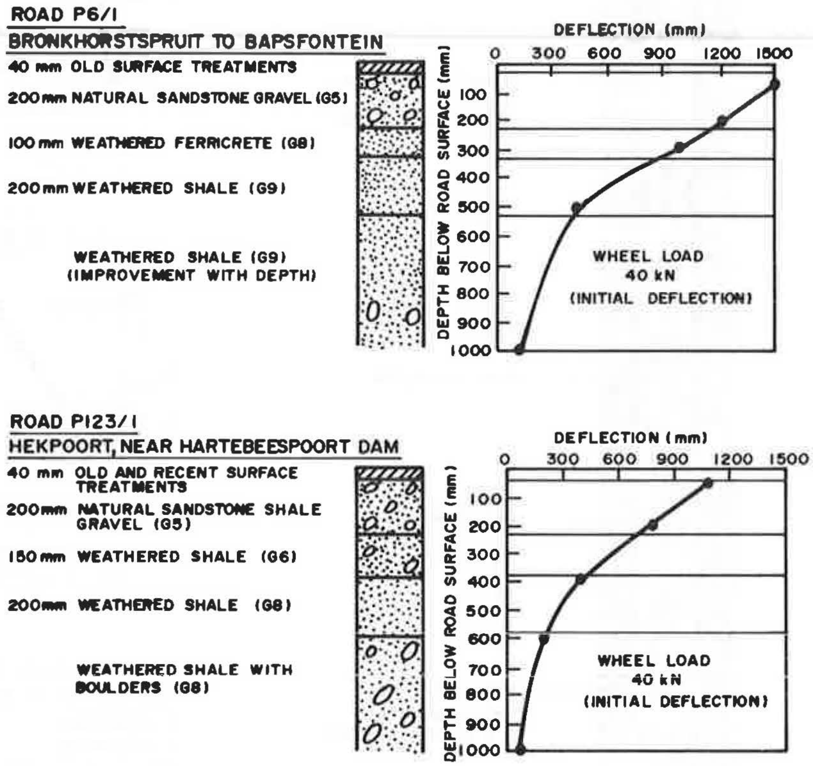


Figure 4. Heavy pavements and their depth deflection curves.

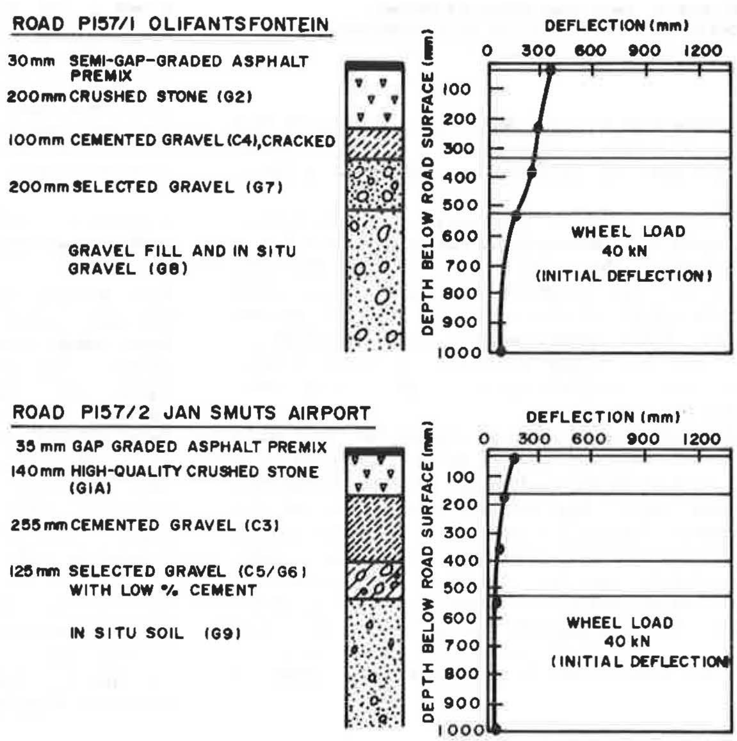


Figure 5. Key to material codes and symbols.

SYMBOL	CODE	MATERIAL	ABBREVIATED SPECIFICATIONS
	G1	Graded crushed stone	Dense - graded unweathered crushed stone Max size 37,5 mm G1A: 86–88 % of apparent density G1B: 98 % mod. AASHTO
	G2	Graded crushed stone	Dense - graded stone and soil binder, Max size 37,5 mm, min. 98% mod. AASHTO
	G4	Natural gravel	CBR ≮ 80 : PI ≯ 6
	G5	Natural gravel	CBR ≮ 45 : PI ≯ 10 to 15 depending on grading ; Max size 63 mm
	G6	Natural gravel	CBR ≮ 25; Max size ≯ 2/3 layer thickness
	G7	Gravel - soil	CBR ≮ 15; Max size ≯ 2/3 layer thickness
	G8	Gravel - soil	CBR ≮ 10 at in situ density
	G9	Gravel - soil	CBR ≮ 7 at in situ density
	C3	Cemented natural gravel	UCS 1,5 - 3,0 MPa at 100 % mod AASHTO; Max size 63 mm
	C4	Cemented natural gravel	UCS 0,75 - 1,5 MPa at 100 % mod AASHTO; Max size 63 mm
	C5	Treated natural gravel	Modified mainly for Atterberg limits

Table 1. Effective elastic moduli and stress states: Roads P6/1 and P123/1 (light structures).

Dual-Wheel Load (kN)	Effective Elastic Moduli				Calculated Stress States	
	E_{Base} (MPa)	E_{SB} (MPa)	E_{SL} (MPa)	E_{SG} (MPa)	θ_{Base} (kPa)	$(\sigma_1 - \sigma_3)_{SG}$ (kPa)
Road P6/1[a]						
20	60	25	30	70	87	22
40	60	20	25	65	148	45
60	70	20	20	60	183	58
80	85	45	15	55	272	71
100	110	60	15	50	321	80
Material code	G5	G8	G9	G9		
Road P123/1[b]						
20	40	50	60	110	131	23
40	50	50	50	100	238	44
60	55	65	80	90	364	61
80	70	60	60	70	434	75
100	80	60	70	70	519	91
Material code	G5	G6	G8	G8		

Note: SB = subbase; SL = selected layer; SG = subgrade.

[a] Surfacing (multiple old brittle surface treatments) was assumed to have a modulus of 1000 MPa for all wheel loads.
[b] Surfacing (multiple old and recent flexible surface treatments) was assumed to have a modulus of 500 MPa for all wheel loads.

where

$\theta = \sigma_1 + \sigma_2 + \sigma_3$,
$\sigma_1, \sigma_2, \sigma_3$ = principal stresses, and
K_1, K_2 = laboratory-derived regression constants.

By calculating the stress states in the crushed-stone layer, the average sum of the principal stresses can be determined. This average can be compared with the effective elastic modulus in order to determine the constants in the above equation. The principal stresses were calculated at three depths within the crushed-stone layer, which corresponded to the top, middle, and lower parts of the layer. These stress states were determined at the center of the dual-wheel load and under one of the wheels of the dual-wheel load. In Tables 1 and 2, the average calculated stress state (θ) in the base has been included for each wheel load. The actual value of θ is dependent on the wheel load, the pavement structure, and the way in which θ is calculated. The support provided by the subbase and subgrade clearly influences the value of θ (compare Table 1, top, with Table 2, bottom). HVS testing and experience have shown that, of the four structures tested, the structure of Road P157/2 (Figure 4, bottom) can carry the largest number of standard axles. This structure, which has a rather thick cemented subbase, shows the highest values of θ.

Cohesive materials showing stress-softening behavior can be described by a model that gives the modulus as a function of the deviator or vertical stress:

$$M_R = K_1 + K_3(K_2 - \sigma_d) \quad \text{if } K_2 > \sigma_d \tag{2}$$

or

$$M_R = K_1 + K_4(-K_2 + \sigma_d) \quad \text{if } K_2 < \sigma_d \tag{3}$$

where σ_d is the difference in principal stresses ($\sigma_1 - \sigma_3$) and K_1, K_2, K_3, and K_4 are material constants (K_3 and K_4 describe the rate of change of M_R with σ_d). The maximum calculated deviator stress (under the center of the dual-wheel load) has been included in Tables 1 and 2.

The effective moduli of the subbases and selected subgrade layers have also been determined; they,

Table 2. Effective elastic moduli and stress states: Roads P157/1 and P157/2.

Repetitions of 70- and 100-kN Wheel Load	Dual-Wheel Load (kN)	Effective Elastic Moduli				Calculated Stress States	
		E_{Base} (MPa)	E_{SB} (MPa)	E_{SL} (MPa)	E_{SG} (MPa)	θ_{Base} (kPa)	$(\sigma_1 - \sigma_3)_{SG}$ (kPa)
Road P157/1[a]							
10	40	200	500	80	160	264	29
	70	300	500	80	160	403	46
1×10^6	40	162	230	47	119	266	39
	70	290	350	50	115	366	56
1.75×10^6	40	178	365	37	105	247	33
	70	225	460	45	105	411	55
1.94×10^6	40	195	180	51	109	210	27
	70	235	235	57	103	353	48
	100	263	260	71	110	490	61
Material code		G2	C4	G7	G8		
Road P157/2[b]							
10	40	335	1100	750	230	370	32
	70	520	1400	600	140	618	40
	100	725	1650	500	90	879	41
0.48×10^6	40	250	900	500	150	370	30
	70	420	1400	350	115	625	39
	100	600	1300	500	87	848	44
1.42×10^6	40	260	1100	750	115	379	23
	70	380	950	400	75	592	34
	100	425	1100	500	72	846	43
1.70×10^6	40	190	900	95	90	371	24
	70	230	1250	65	70	630	31
	100	275	1500	45	55	884	33
Material code		G1A	C3	C5/G6	G9		

[a]Semi-gap-graded asphalt premix (30 mm) was assumed to have a modulus of 4000 MPa.
[b]Gap-graded asphalt premix (35 mm) was assumed to have a modulus of 3000 MPa.

too, show stress-dependent behavior. It is of special interest to note that the cemented subbase in Road P157/1 (Table 2, top) has very low effective moduli and that the values are eventually similar to the granular base modulus. At 1.94×10^6 repetitions, the cemented subbase acted like a stress-dependent granular layer. At the end of the test when the subbase was recovered, it was found to be fully cracked into small pieces, which confirmed the moduli measured. The cemented subbase on Road P157/2 (Table 2, bottom) also showed stress-stiffening behavior, although its effective modulus was much higher. At the end of the test this layer was not fully cracked.

The selected layers of Roads P6/1 and P157/2 showed stress-softening behavior. No clear trends were established for the selected layer of Road P123/1, and the selected layer of Road P157/1 showed stress-stiffening behavior.

SUMMARY OF STRESS-STIFFENING BEHAVIOR OF GRANULAR BASE LAYERS

Figures 6-8 show the effective resilient moduli of the granular base layers as functions of either the dual-wheel load or the sum of the calculated principal stresses. These figures show the pronounced stress dependence of granular base layers.

The values of the effective moduli of the low-quality bases for the light structures are very low. For both pavements, there seems to be a lower limit of 40-60 MPa and a marked increase in modulus for wheel loads above 60 kN. It is interesting to note that the actual value of the modulus of the base may be lower than that of the subgrade (refer to Table 1). The ratio of the base modulus to the underlying subbase modulus ranges from 3.5 to 1.8 for Road P6/1 and from 1.3 to 0.8 for Road P123/1. This ratio therefore changes from structure to structure and also changes with wheel load.

The base and subbase layers in the light structures eventually showed spectacular shear failures in the HVS tests, which indicated that the shear stresses and strains in these layers were high and that near-failure conditions existed at the time of the measurements. The low moduli can therefore also be attributed to the unfavorable stress states.

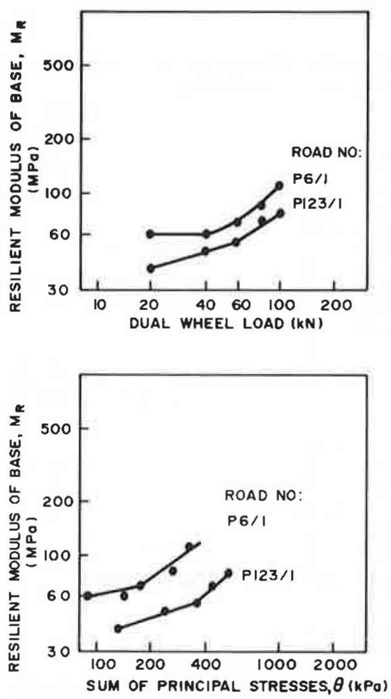
The heavy structures, with better-quality bases (Roads P157/1 and P157/2), show typical stress-dependent behavior (Figures 7 and 8). Very neat straight-line relationships on log-log scales were determined. In these tests the influence of repetitions and environmental changes was also investigated. The value of the modulus decreased throughout the test, although no apparent physical changes (e.g., degradation) took place. A gradual decrease in soil suction due to the repeated loading of the HVS could explain this behavior. In the final stages, when the bases became wet, the modulus showed a further decrease and also became less stress dependent (the slope of the line decreased). In the test on Road P157/2 (Figure 8) the moisture caused a greater change in the modulus than the repetitions of the wheel load.

Figures 7 and 8 also show the moduli as measured in repeated-load triaxial tests done at constant confining pressures. The laboratory tests completely overestimated the actual value of the modulus. The slope of the line (K_2) determined in the laboratory was reasonably close to the field-determined slope, but the position of the line (abscissa K_1) was much lower in the field. The difference between laboratory and field moduli could be attributed to a difference in stress states, a Poisson's-ratio effect, or a difference in soil suction. The laboratory constants were determined in constant confining pressure triaxial tests, which do not accurately simulate true field conditions. The way in which θ was calculated (see previous section) may overestimate the value of θ, which could also lead to the observed difference. A constant value of 0.35 was assumed for Poisson's ratio; this ratio is actually also stress dependent and could also change during the test. Laboratory tests have shown (4) that the soil suction in the unsoaked condition primarily influences K_1. The soil suction in the laboratory may have been higher than the suction in the field during the dry part of the HVS test, which led to the overestimation of the modulus in the laboratory.

Tests with the road rater (8) that used low dynamic loads (2-7 kN) have yielded different results in that the laboratory moduli appeared to be too low. According to this study (8), the field modulus would decrease with increase in shear strain. It appears that at higher wheel loads (high shear strain) the laboratory measurements overestimate field moduli, although no clear trends could be established from Figures 7 and 8. Table 3 shows the field-determined constants in $M_R = K_1\theta^{K_2}$. It can be concluded that the trends measured in the laboratory will probably hold in the field but that a shift factor of 0.3-0.5 is necessary to predict field moduli. The laboratory modulus should be multiplied by this shift factor. In the dry condition, K_2 is of the order of 0.9-1.0 for both pavements and K_1 of the order of 0.5-2.0. In the wet state, K_2 dropped to a value of 0.3-0.4 and K_1 increased to a value of 15.0-27.0.

The ratio of the base modulus to the subbase modulus ranges from 0.4 to 1.1 for Road P157/1 and from 0.2 to 0.5 for Road P157/2. Apparently, the ratio is always less than 1.0 for structures with cemented subbases unless the subbase is completely cracked. For Road P157/2 the ratio is always less than 0.5.

Figure 6. Stress dependence of granular-base resilient modulus as measured in HVS test on Roads P6/1 and P123/1.



SUMMARY OF STRESS-SOFTENING BEHAVIOR OF SUBGRADE MATERIAL

Figures 9-11 show the effective resilient moduli of the subgrades as functions of either the dual-wheel load or the calculated deviator stress on the subgrade. In most cases there is a decrease in the modulus as the wheel load or the deviator stress on the subgrade increases.

The subgrades of the light structures are clearly stress dependent and stress softening, but the actual change in the modulus is not very large (Figure 9). For Road P6/1 a change in wheel load from 20 kN to 100 kN causes a change in subgrade modulus of less than 30 percent. For Road P123/1 the change is less than 40 percent. These changes are smaller than the changes in the base modulus. However, because the subgrade is a much thicker layer, the effect of these changes on the total deflection is probably just as pronounced as the effect of the stress stiffening of the base. The cohesive material model [M_R = f (deviator stress)] has been applied to slightly idealized lines put through the calculated values in Figure 9. The regression constants thus determined by using Equa-

Figure 7. Stress dependence of granular-base resilient modulus as measured in HVS test on Road P157/1.

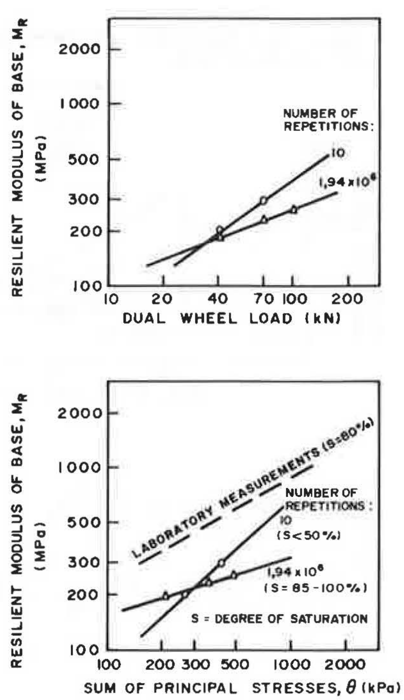
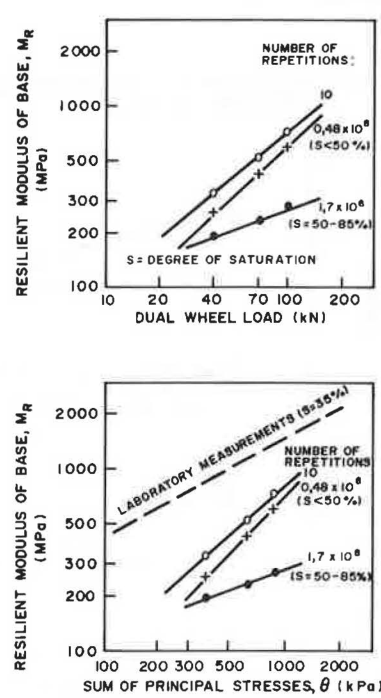


Figure 8. Stress dependence of granular-base resilient modulus as measured in HVS test on Road P157/2.



tion 2 or 3 are shown below (M_R in MPa; σ_d in kPa):

	Regression Constants			
Road	K1 (MPa)	K2 (kPa)	K3	K4
P6/1	65	45	+0.24	-0.43
P123/1	70	75	+0.94	0

The subgrade of Road P6/1 became more stress dependent at deviator stresses above K_2. Normally the slope K_4 is less than the slope K_3, as in the case of Road P123/1. The K_1-values are very similar for the two roads.

The subgrade for Road P157/1 showed very little stress dependence (Figure 10). At high loads this subgrade even exhibited slightly stress-stiffening behavior, but this could be attributed to the granular nature of the subgrade at this site. The value of the modulus changed from 160 MPa to 103 MPa during the test. Table 4 shows the regression constants determined at the end of the test. The effective modulus of the subgrade for Road P157/2 showed typical stress-softening behavior when compared with the dual-wheel load (Figure 11). However, the calculated deviator stress on the subgrade in this complex pavement structure does not necessarily reflect the wheel load, and when the modulus is compared to the calculated deviator stress, a somewhat different relationship emerges (Figure 11). The regression constants are shown in Table 4. As in the case of Road P157/1, the modulus (and K_1) decreased during the test. The value of K_1 dropped from 140 MPa initially to 55 MPa at the end. The moisture conditions play an important role and may override the stress conditions. Toward the end of the test and under wet conditions, the subgrade behaved in the typical stress-softening way. The regression constants determined at 1.42×10^6 repetitions could be typical wet values.

DISCUSSION OF RESULTS

The South African mechanistic design method (3) includes tabulated input moduli of typical road pavement materials. These values correspond to the standard legal dual-wheel load of 40 kN. The previous paragraphs have shown that effective moduli are very dependent on the applied wheel loads. The

Table 3. Field-determined constants in $M_R = K_1\theta^{K_2}$.

Road	No. of Repetitions	Regression Constants[a] K_1	K_2
P157/1	10	1.1	0.93
	1 940 000 (wet)	27.0	0.32
P157/2	10	1.8	0.89
	480 000	0.6	1.02
	1 700 000 (wet)	15.0	0.43

[a] M_R in MPa, θ in kPa.

Figure 9. Stress dependence of subgrade resilient modulus as measured in HVS test on Roads P6/1 and P123/1.

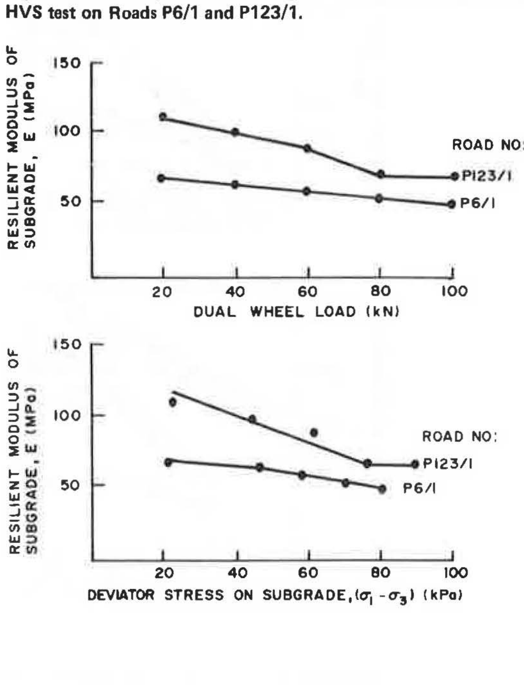


Figure 10. Stress dependence of subgrade resilient modulus as measured in HVS test on Road P157/1.

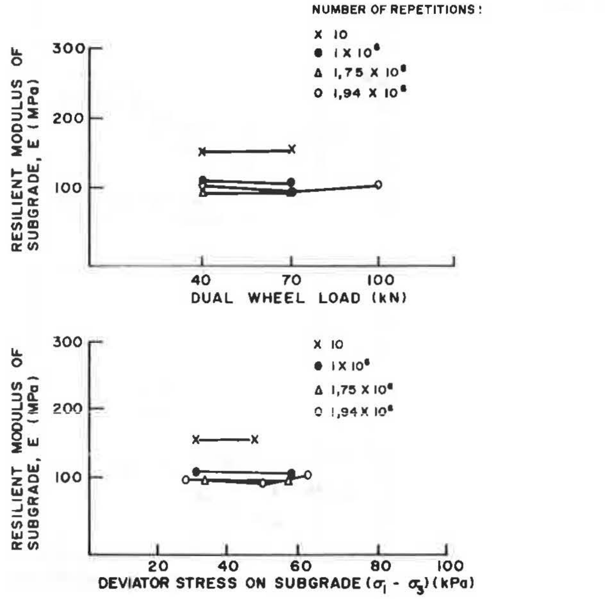


Figure 11. Stress dependence of subgrade resilient modulus as measured in HVS test on Road P157/2.

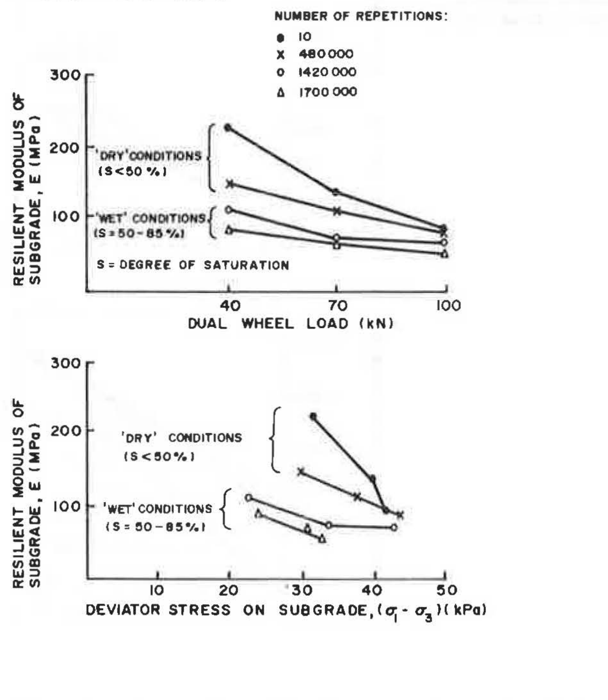


Table 4. Field-determined constants in Equation 2 or 3 for stress-dependent subgrades in Roads P157/1 and P157/2.

Road	Moisture Conditions in Subgrade	No. of Repetitions	Regression Constants[a] K_1 (MPa)	K_2 (kPa)	K_3	K_4
P157/1	Normal	10	160	-	-	-
	In situ	1.94×10^6	103	48	+0.21	+0.60
P157/2	Normal	10	140	40	+11.2	-43.8
	In situ	0.48×10^6	115	39	+4.0	-5.2
	Wet	1.42×10^6	75	34	+3.5	-0.29
		1.7×10^6	55	34	-	-

[a] M_R in MPa; σ_d in kPa.

mechanistic design method (3) is often applied to other situations such as the design of airport pavements or the design of railway track and foundation structures. Enough evidence has been obtained to suggest that the mechanistic method should be applied with care and that the values of the input moduli should be adjusted according to the stress conditions.

The field data of the stress dependence of unbound pavement materials once again confirm the importance of a nonlinear approach to the analysis of pavement structures containing such materials.

SUMMARY AND CONCLUSIONS

A method for determining the effective moduli of different pavement layers has been presented. The method was found to be practical and of great value in interpreting the state of the pavement layers. The multidepth deflectometer functioned well and provided reliable measurements of deflection with depth.

The effective moduli of granular layers, cemented subbase layers, and also subgrade soils have been determined for materials from very light old structures as well as materials from structures currently used by the Transvaal Provincial Administration. On the heavy structures, the effective moduli could be determined throughout an HVS test, and a very good record of the change in structural response of the pavement could be obtained.

The stress-dependent behavior of the different pavement materials has been demonstrated very clearly from field data. Granular materials behave in a stress-stiffening way, and most subgrade soils show stress-softening behavior. The stress-dependent models for granular and cohesive materials could be applied to the effective moduli, especially to the moduli of the unbound crushed-stone bases. The regression constants could be determined for different moisture conditions and at various stages of trafficking.

Environmental factors such as moisture have an important influence and can change the basic behavior of the material. The moisture conditions could be as important as the stress states. The regression constants in the stress-dependent models are very dependent on the moisture conditions.

Laboratory constant confining-pressure triaxial tests overestimate the modulus of the crushed-stone bases, and a shift factor of 0.3-0.5 needs to be applied. Although there is a difference in the actual values, the trends apparent in the laboratory were also apparent in the field. The deviator stresses (or the shear strains) in the field could contribute toward the observed differences. The actual wheel load could therefore influence the shift factor.

The modulus of the base depends on the modulus of the underlying subbase, and for the light structures with unbound subbases, ratios of 0.8-3.5 have been determined. This ratio depends on the pavement structure and also on the wheel load. In pavements with cemented subbases, the ratio is always less than 1.0 unless the cemented subbase is completely cracked. In some cases the ratio may be as low as 0.2. The stress states in the base layers of these inverted designs are more favorable (higher values of θ) and the resulting effective moduli are higher.

The stress dependence of the subgrade moduli is generally less than that of the base, but the effect on the total deflection is probably just as pronounced. Not all subgrades exhibit stress-softening behavior.

Where the mechanistic design method is applied to other pavement structures such as airport pavements, the effective resilient moduli should be adjusted to correspond to the stress conditions.

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Sulphlex Pavement Performance Evaluations from Laboratory Tests

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New types of energy-saving materials are being developed for use in test sections within the U.S. road network. One of these materials, Sulphlex, has recently received a great deal of attention and is being developed as an alternative binder to asphalt and portland cement. Mechanical tests were conducted on specimens of asphalt concrete and Sulphlex 233 mixes. The mechanical tests conducted are those necessary to provide the material characterizations for input to the VESYS structural subsystem. The long-term predicted behavior of Sulphlex pavements was compared with that of conventional asphalt pavements for five different structural designs, on two different subgrades, and for three levels of temperature. A set of design criteria was selected to allow an analysis of the performance of each pavement.

In order to reduce the risk of unsatisfactory pavement performance to an acceptable level, engineers must be able to reliably predict pavement behavior over time. Current methods for predicting such behavior for different types of pavements, environments, and traffic loads are limited to what are known today as rational or, more precisely, mechanistic design procedures. The Federal Highway Administration (FHWA) has developed one such procedure called the VESYS structural subsystem, which predicts the pavement's behavior over time based on the mechanical properties of the layer materials, the anticipated traffic loads, and the local environmental conditions. Mechanistic design procedures are so called because they are in fact developed from the laws of mechanics in which the prescribed actions of forces on bodies of material elements are related to the resulting stress, strain, deformation, and failure of the total pavement structure. The properties of the material elements are determined by subjecting specimens of a given material to a series of laboratory load tests and applying the laws of mechanics for the prescribed geometry and environmental conditions. There are both advantages and disadvantages associated with mechanistic-type procedures. One of the most important advantages is that such procedures permit the use of completely new materials and/or new types of pavement structures.

Today, many new types of energy-saving materials are being developed, and test sections are being placed within the U.S. road network. For instance, improved technology and the energy crisis have provided the impetus for new binders, some of which are asphalts modified with sulfur (_1_) and other chemicals to improve their durability and performance. Polymer portland cement concretes (_2_) and concretes that use super water-reducing agents (_3_) are also being produced with properties quite different from those of conventional paving types. Recycled pavements conserve materials and, depending on the types, the quantity and quality of additives can be expected to perform as well as high-quality conventionally designed mixtures (_4_,_5_). Sulphlex (plasticized sulfur) is another new material that might be made to behave similarly to asphalt or portland cement concrete (_6_,_7_).

The purpose of this paper is to evaluate the results of a limited number of laboratory tests conducted by FHWA on Sulphlex and asphalt paving mixtures and to predict the performance of selected structural sections under simulated real-world conditions.

SULPHLEX AS BINDER AND MIX

The primary objectives of an initial FHWA study with Southwest Research Institute were to develop a system to modify sulfur so that it would serve as a binder replacement for asphalt and portland cement and to prepare mixtures of the developed binder with aggregates and measure their properties (_6_). To be used as a pavement binder, sulfur must be modified to exhibit more plastic characteristics. When elemental sulfur is heated to above its transition temperature of about 320°F and rapidly quenched at 68°F, it exhibits a plastic characteristic; hence, it may be called plasticized sulfur. If the material is allowed to return to room temperature, it quickly hardens and brittle sulfur crystals are formed; such a physical change does not lend itself to practical application in the preparation of binders for highway paving. The intent in this study was the conversion of sulfur to a plastic through a chemical reaction, a mechanical change, or a combination of these so that the resulting material at room temperature might have viscosity, penetration, and other characteristics similar to those of asphalt. During the course of the study, more than 450 different formulations were prepared by using combinations of 80 different modifiers. The most promising binders were selected to be evaluated for their behavior when mixed with a select aggregate. The results of laboratory tests conducted on