FOAM BITUMEN STABILISED SAND AS AN ALTERNATIVE TO GRAVEL BASES FOR LOW VOLUME ROADS

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ABSTRACT

The stabilisation of Kalahari type aeolion sands, using the foam bitumen process, has been experimented with on several occasions. A large scale field study is reported on in this paper and the long-term performance is discussed. Visual condition is related to structural parameters and guidelines in design are suggested. The use of the Dynamic Core Penetrometre, (DCP), vane shear tests and material grading play an important role in quality control and the conclusion is made that the mechanistic design and evaluation process can be used with confidence in predicting performance.

INTRODUCTION

Most of South West Africa's pavement materials consist of Kalahari sand and calcrete. Calcrete can make a good base and subbase material but tends to have a high soluble salt content with dispersion characteristics and therefore it may be necessary to chemically modify this material. When calcrete is stabilised with cement or lime, reflection cracks and carbonation usually result which may cause a loss strength.

The above-mentioned problems a lack of good quality gravel, as well as a sharp increase in the construction, and maintenance cost of roads, necessitated an investigation into alternative pavement materials. In order to use sand in road construction Marais and Freeme have previously suggested different types of bitumen stabilisation (1) and a mixture of Kalahari sand and calcrete has been used in a hot mix asphalt (2). Foam bitumen has also been experimented with in South Africa but only on a very small scale (3). In Australia and the USA however, foam bitumen is being used widely with a great deal of success.

Two experimental test sections were built where the foam bitumen process was used to establish whether Kalahari sands could be used more effectively. The experimental sections under discussion in this paper were built towards the end of 1984 and were made up of a 1,5 km section on the Oshakati to Okatana road and a 3,6 km section on the Ondangwa-Onandjokue road. The purpose of the experiment was to evaluate the material under local conditions and to establish standards to be used for construction control. At the same time it was attempted to develop a design method whereby binder content, mix properties and thickness could be used to predict structural performance.

BACKGROUND

The basis of the foam bitumen process is to add cold water to a hot penetration grade bitumen to produce foam. This foam is then sprayed onto the aggregate (sand or gravel) and mixed in. Foaming dramatically increases the surface area of the binder so that a relatively small quantity of binder can be mixed with aggregate with a high surface area (fine sand).

Normally the sand or gravel is required to have between 5 and 15 percent material passing the 75 micron sieve. The material should be well graded without plasticity and generally no restriction is placed on the maximum size although a fine material is preferred. Normal compaction and mixing equipment is used for layer work but for the foam bitumen process an additional mixer is required. This mixer, figure 1, normally is a rotation mixer [Pulvimixer] modified with a spray bar inside the mixing chamber where the water and bitumen is mixed. The final binder content in the material is dependent on the thickness of the layer of material being stabilised, the flow rate of the bitumen and the speed of the mixer.

As the quality of the product cannot be effectively determined by using normal methods such as California Bearing Ratios (CBR), Unconfined compressive strengths (UCS) and Marshall, other methods must be used. The Hveem method is generally used in the USA, and the Dynamic Cone Penetration (DCP) and vane shear test have been used locally (4), but it is believed that the dynamic triaxial test may be preferable. This test is however not a routine test in South Africa. To avoid a purely empirical approach in design, the experiment also attempted to develop ways and means to use mechanistic design procedures.

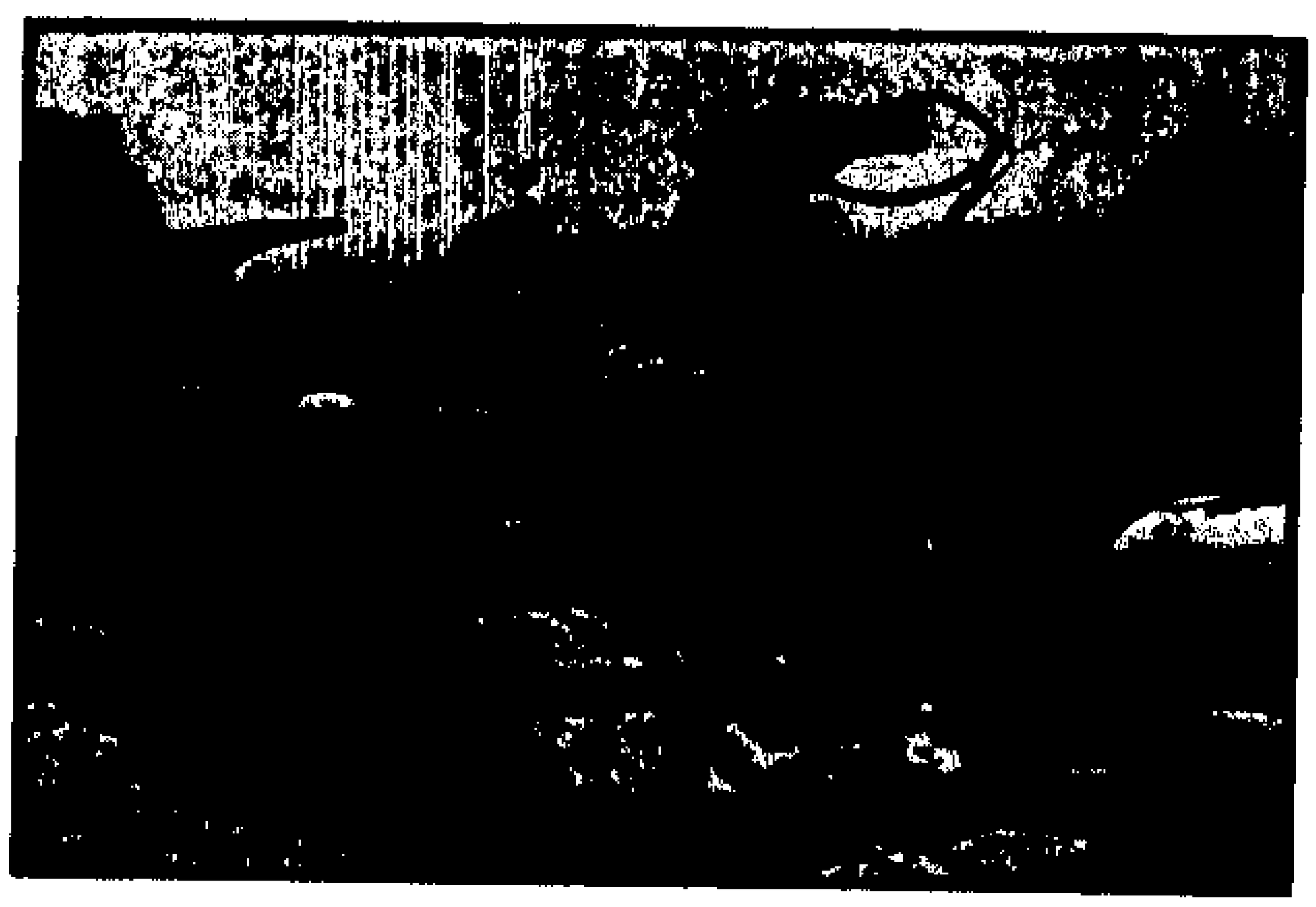

Figure 1

Tractor left, bitumen tanker top middle, water-cart right and mixing plant in the middle of the photo

PLANNING AND EXECUTION

The experimental sections were built during a roads contract where the standard design consisted of a 150 mm calcrete gravel base course followed by a 150 mm calcrete subbase and a 150 mm selected layer on top of sand or calcrete fill.

A control section was selected from a typical design and is hereafter referred to as Section A. For the purpose of the experiment

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the conventional base and subbase courses were replaced by foam bitumen layers of different thicknesses and different bitumen contents. Three sections were built at Okatana, using 125 m to 175 m thick bases stabilised with between 3 and 4 % foam bitumen. The layer below this stabilised layer was selected to ensure a CBR of at least 45, at 97 percent Mod AASHTO density. The seal in all cases consisted of a bituminous seal.

MATERIAL CHARACTERISTICS

Material properties were measured during the experiment, and after the pavement had been completed various tests were done from time to time on the foam bitumen layer. The average field characteristics as measured during construction of the pavement are shown in Table 1. It is clear from this table that the stiffness of the foamed base course layer, as depicted by CBR, varies greatly from that of the conventional design, i.e. Section A.

TABLE I

Average characteristics of pavement materials at the time of construction

	A	B	D	E
Base course thickness (mm)	150	125	175	125
Through 0,425 mm sieve (%)	38	72	70	86
Through 0,075 mm sieve (%)	12	16	16	12
PI	6	2	2	1
CBR* before foam	134	46	44	92
after foam	-	8	9	14
% bitumen	-	3,7	3,1	4,1
Vane shear (kPa)	-	321	270	299
DCP mm/blow for top 120 mm	-	9,2	8,0	10,9
Subbase layer (mm)	150	150	150	150
CBR	50	40	60	50
PLASTICITY Index (PI)	4	3	2	7
Selected layer (mm)	150	-	-	-
CBR	20	-	-	-

* CBR at field density

Note: For Section A the unstabilised or conventional section, a 19 mm chip seal with a "koffiemoer" ie. crusher dust mixed with 7 % bitumen, and for all the foam stabilised sections a 6,5 mm seal was used.

FOLLOW UP DATA

Pavement performance was monitored continuously. For this purpose a program was implemented whereby DCP, vane shear stiffness of the top layer, deflection, radius of curvature and the visual condition of the road were monitored from time to time. It was found that cracking developed after about two years of service but the crack pattern indicated little relationship with structural parameters. However, during 1987, about 3 years after construction, longitudinal cracks also started to develop in some isolated areas.

Table II contains a summary of some typical characteristics that were measured at different dates (December 1984, September 1985, June 1986 and October 1987).

EVALUATION OF RESULTS

Data obtained during construction and during the life of the pavement can be analysed statistically. A structural analysis of the pavement is also possible if the measured properties are used.

Statistical analysis

Statistical analysis in the form of a correlation matrix can give a good idea of the relative correlation between different variables. Table III summarises the more important correlations between the different variables using data from all sections. A value of 1,0 indicates perfect correlation whereas 0 shows that no relationship exists.

Looking at Table III the following deductions can be made:

- A high binder content correlates well with initial shear strength; newly constructed material with a high binder content shows a low shear strength but with age shear strength increases more in the material with a higher binder content.
- Dynamic cone penetration (DCP) stiffness is not influenced to a great extent by the percentage binder or the thickness of the stabilised layer but there is a tendency for a higher binder content to increase the DCP value which implies a

TABLE II

Change in material properties with time

Section	Dynamic Cone (mm/blow)			Vane Shear (kPa)			Deflection (mm)			Curvature (m)		
	12/84*	6/86	10/87	12/84	6/86	10/87	9/85	6/86	10/87	9/85	6/86	10/87
A	2,84	2,73	3,20	-	650	620	0,15	0,20	0,17	156	158	153
B	9,2	4,74	4,62	321	498	579	0,27	0,29	0,28	137	149	160
D	10,9	5,61	4,70	270	446	457	0,32	0,27	0,31	130	159	164
E	8,0	3,80	-	299	289	385	0,30	0,22	0,21	149	-	204

Average coefficient of variation of the data above is 20 %
Note: * Date of measurement

TABLE III

Correlation matrix of measured variables

VARIABLE	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
1. % binder at top	1,0															
2. % binder at bottom	0,8	1,0														
3. Vane shear	-0,6	-0,7	1,0													
4. % Fines	-0,7	-0,6	0,6	1,0												
5. CBR swell	-0,7	-0,5	-0,5	-0,9	1,0											
6. CBR at field density	0,8	0,7	-0,6	-0,8	-0,8	1,0										
7. DCP	+0,3	+0,2	-0,2	0,6	0,4	-0,5	1,0									
8. Deflection: 1 year	0,1	0	-0,3	-0,3	-0,1	0,2	+0,3	1,0								
9. Deflection: 3 year	-0,1	-0,3	0	0	0	0,1	+0,3	0,9	1,0							
10. Longitudinal cracks: 1 year	-0,7	-0,5	0,6	0,9	0,5	-0,7	0,3	0	0	1,0						
11. Longitudinal cracks: 3 years	-0,4	-0,4	0,4	0,6	0,5	-0,4	0	0,4	0,5	1,8	1,0					
12. Other cracks: 1 year	-0,5	-0,4	0,5	0,8	0,6	-0,6	0,3	0	0	0,9	0,7	1,0				
13. Other cracks: 3 years	-0,3	-0,2	0,4	0,6	0,5	-0,5	0,1	0	0	0,8	0,7	0,9	1,0			
14. Rutting	0	0	-0,2	0,2	0,4	-0,2	+0,2	0,6	0,6	0,2	0,4	0	0	1,0		
15. Thickness	0,1	0	-0,3	0	0,1	0	0,3	0,1	0,2	-0,3	-0,2	-0,4	-0,6	0,6	1,0	
16. Compact moisture	-0,6	-0,4	0,1	0,5	0,7	-0,5	0,1	0,1	0,2	0,5	0,3	0,1	0,3	0,4	0,3	1,0

lower stiffness of the layer. It is also interesting to note that an increased fines content (percentage through the 75 micron sieve) leads to a higher DCP value.

- The increased stiffness of the stabilised material with time is of great importance. Increased stiffness may be due to the oxidation of the binder and an increase in density caused by the kneading effect of the traffic. However, the drying out of the base course can also have an influence if it is borne in mind that moisture, which is kept at an optimum during compaction, will eventually be lost due to environmental influences.

- The deflection correlates fairly well with the stiffness of the base course layer as measured with the DCP.

- A better correlation is found with design parameters if early cracking is considered. This implies that more binder, less fines (minus 75 micron material), lower CBR swell and less compaction moisture may result in fewer cracks at an early stage.

- Additional hairline cracks which form after two years do not correlate well with the above variables but there is a correlation between longitudinal cracks that developed after 2 years and deflections. It therefore seems as if longitudinal cracks which develop after two years may be structurally associated, i.e. they were probably caused by traffic. This may be true if it is considered that a high correlation exist between initial hairline cracks and longitudinal cracks after 3 years i.e. cracks initiated within the layer and was later aggrevated by traffic.

- As can be expected rutting is greater where thicker stabilised layers with lower stiffness have been used. In general, it seems that the thicker base courses have less cracking.

- Higher binder content resulted in lower initial stiffness (in terms of vane shear and DCP values), but the stiffness increased with time. This was especially true for material within the wheelpath as also indicated by DCP values eg. the average DCP in the left wheelpath is 6,48, in the right wheelpath is 7,08 and 7,38 mm/blow between wheelpaths after three years.

The development of hairline cracks can, therefore directly be attributed to material characteristics and specifically the characteristics of the fines. A higher percentage of fines (less than 0,75 mm material) seemed to increase CBR swell and also required a higher moisture content for compaction, thus leading to subsequent shrinkage, especially in the top layer of the stabilised base course.

Structural analysis

In order to investigate the structural strength of the pavement, a mechanistic model was compiled. This was done by relating field measured pavement characteristics to layer stiffness values and simulation of deflections and radii of curvature with the help of a multi layer model. Through a process of iteration the stiffness values of the layers were changed until the calculated deflections and radii of curvature were similar to the measured values.

BACKCALCULAT.

The characteristics of the final model are shown in Table IV. The average stiffness values for the different layers of the different sections are almost the same except for the values of the base course. In this case the section with the highest bitumen content had the lowest stiffness value for the base. It is interesting to note that the stiffness of the stabilised base course of Section D increased from 120 in September 1985 to 160 in October 1987 and in Section B from 95 to 150. In the control section however, there is no indication of a gain in strength with time.

The stiffness values of the top layer are low and can be an indication of the reason why stiffness increased with time under traffic ie. the layer was still being kneaded around by the traffic and thus being compacted. The development of longitudinal cracking in the wheeltracks can thus be related to a low bearing capacity of the base in the early days.

In order to use the collected information to compile design standards certain graphs were plotted as shown in Figures 2 to 5. Figure 2 shows the relationship between the average shear strength and average DCP value of different sections, from which it is obvious that the field DCP may also be used with confidence. When DCP values and its deduced stiffness values from the mathematical model are plotted as in Figure 3 and compared with the same type of data for shear strength versus stiffness modulus, (Figure 4), it is clear that DCP's also give accurate information in this respect. When the pavement performance is taken into account it seems as if a minimum stiffness modulus of 120 MPa must be reached a year after construction. From the graphs, gain in strength with time (Figure 5), as well as the relationships between DCP or vane shear with stiffness moduli, it seems that the minimum shear strength of 300 kPa and a maximum DCP value of 8 mm blow is required directly after construction. When mechanistic analyses are used to determine the terminal life and specifically rutting, it is found that an average rut depth of 10 mm after 15 years can be expected. This is based on traffic counts that have revealed a use of about one hundred E80 axles per day at present.

TABLE IV

Layer characteristics based on measurements October 1987

	OKATANA		
	A	B	D
Base course layer			
Thickness (mm)	150	125	175
Modulus* E_{87}	360	150	160
E_{86}	370	90	120
(MPa) E_{85}	390	95	120
Subbase layer			
Thickness (mm)	150	150	150
Modulus (MPa)	250	280	280
Lower layers (combined)			
Modulus (MPa)	800	800	800

* Stiffness values at years 1987, 1986 and 1985

On the other hand, if the present average rate of 0,7 mm rutting per 8 months period and 1,8 mm in 34 months is extrapolated logarithmically, then an average of 7 mm rutting will be achieved within 15 years. It is obvious that the rate of deformation declines with time, and it can be assumed that the stiffness (due to density and hardening) of the base course has now stabilised.

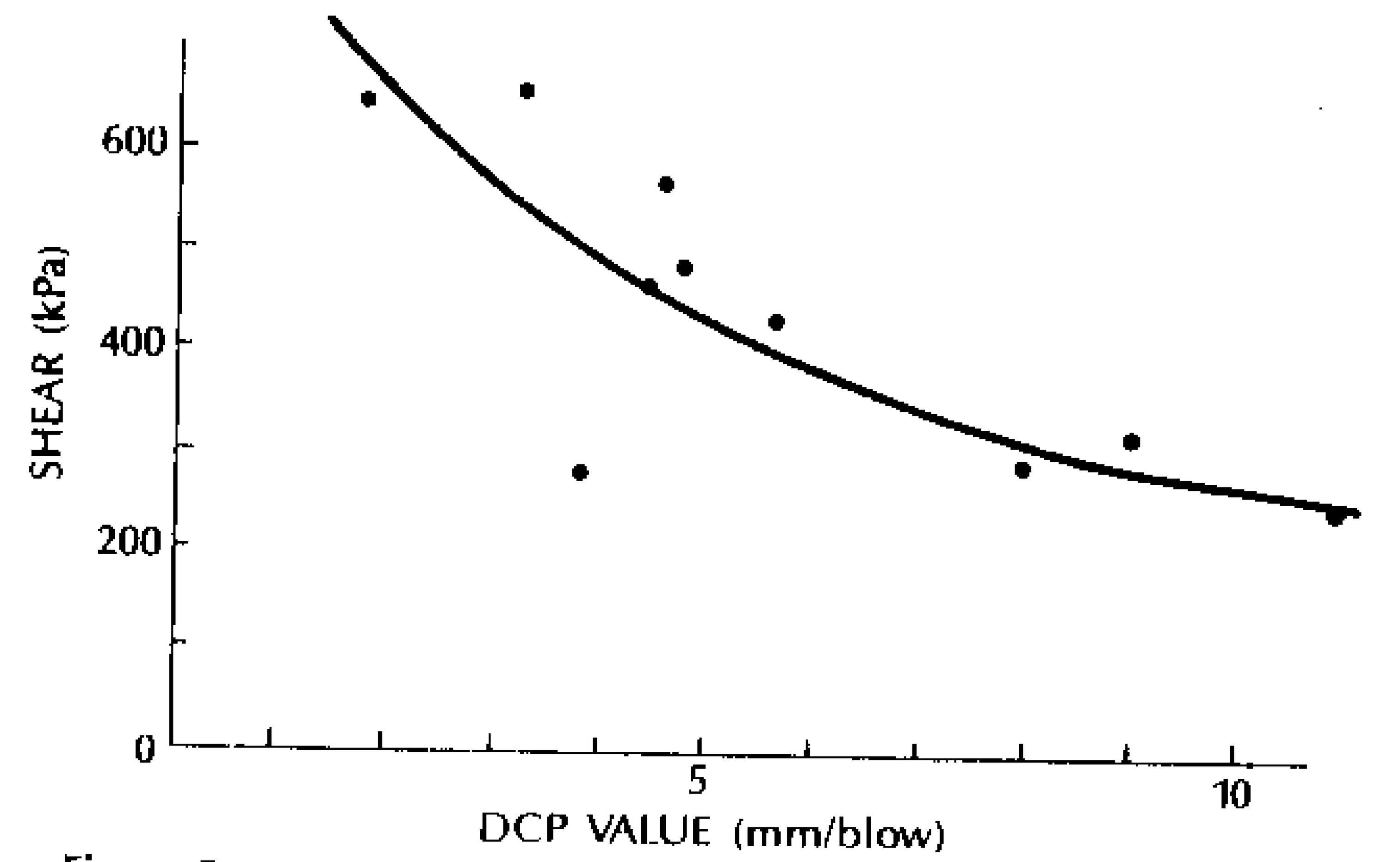

Figure 2
Relationship between average value of DCP and shear strength

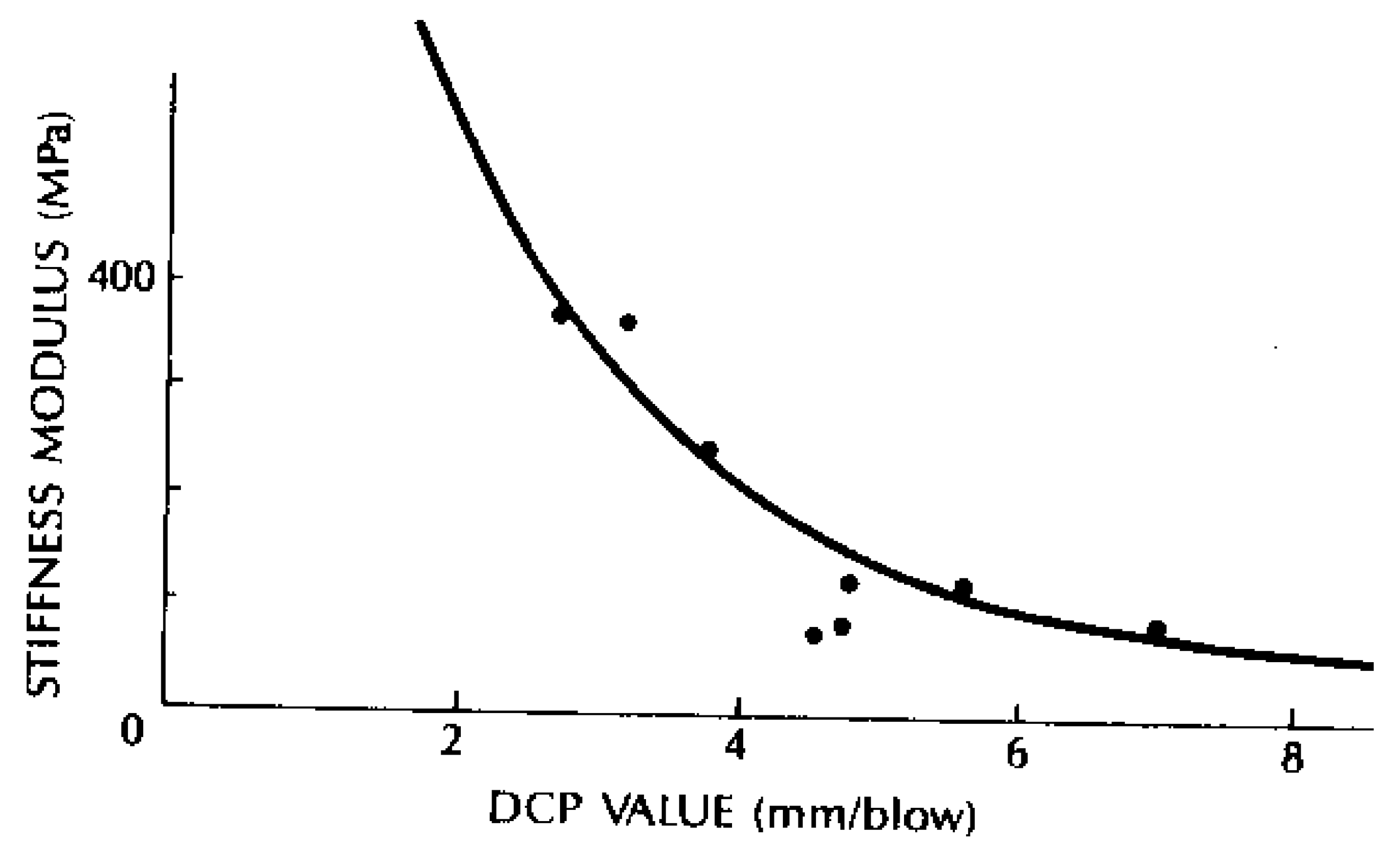

Figure 3
Stiffness modulus as a function of DCP value

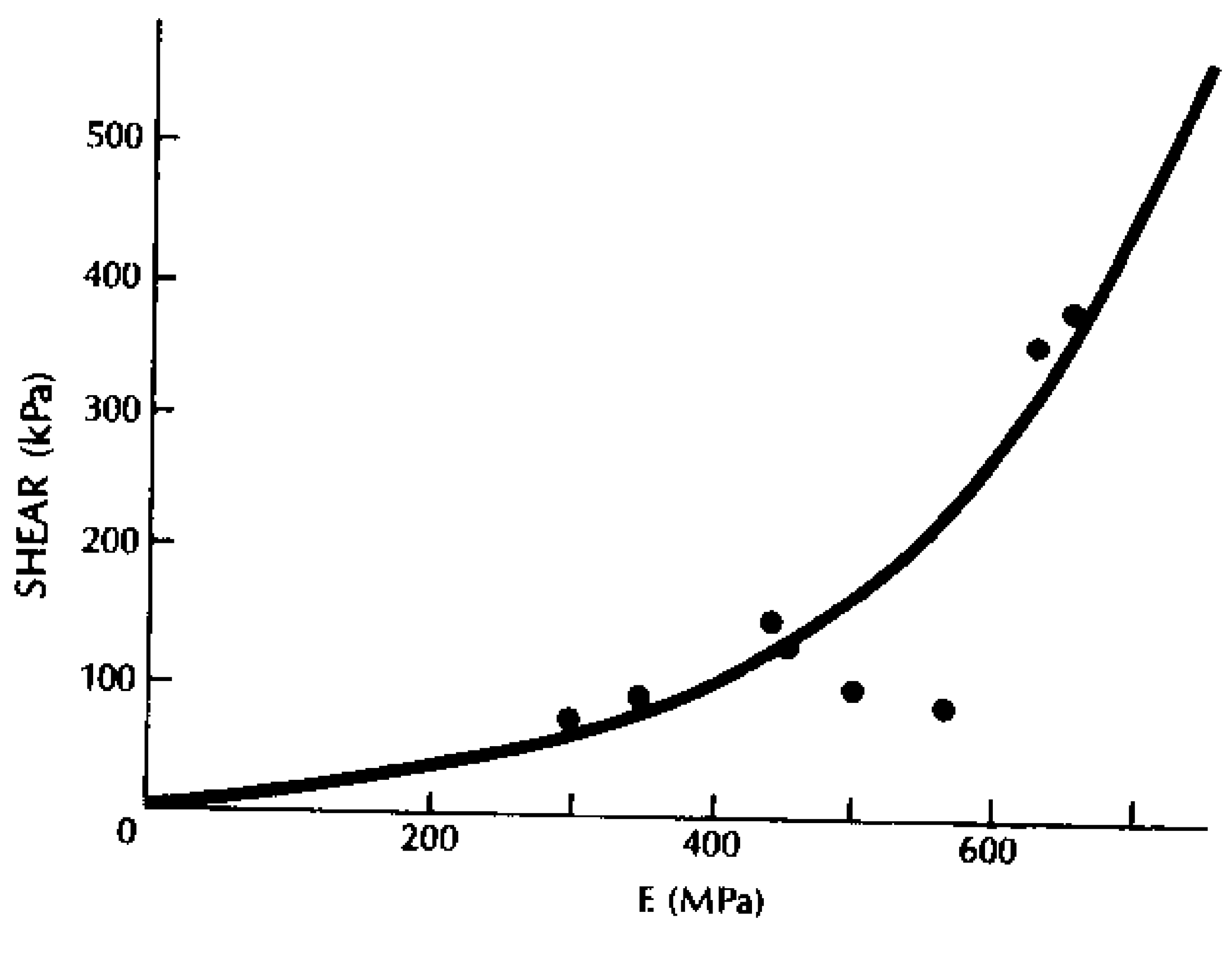

Figure 4
Stiffness modulus as a function of shear strength

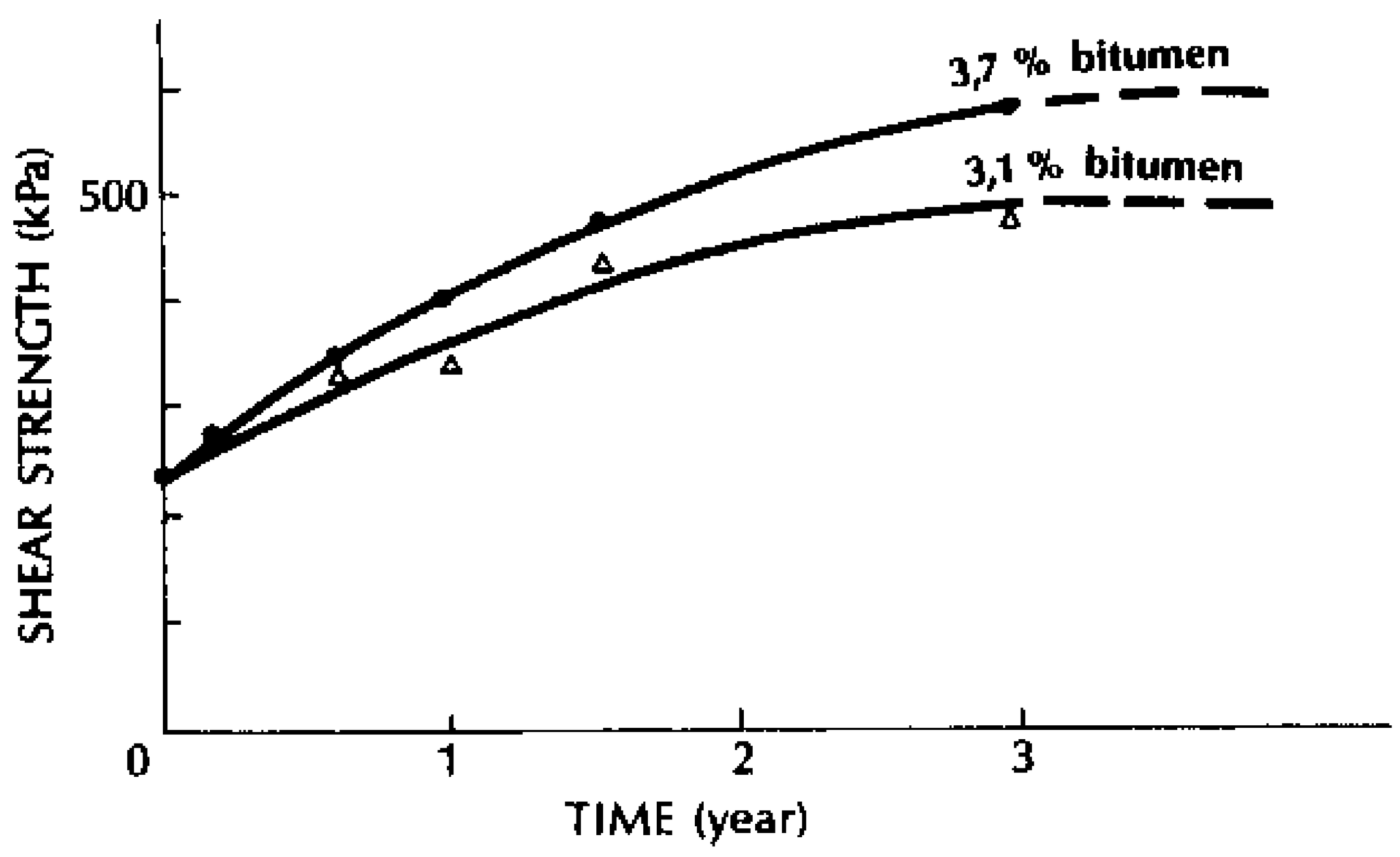

Figure 5
Increase in stiffness with time

CONCLUSIONS

Due to the fact that performance of the test sections has been monitored for almost four years, certain concluding remarks can be made with a fair amount of confidence:

1) The experiment provided useful results and can be considered worthwhile.

2) The traditional CBR method of design cannot be used on foam bitumen layers but is still valid for the selected and subgrade layers.

3) The foam bitumen base course does not contribute to structural strength in the first two years after construction as shown by the deflections and rutting with time. This layer must be placed on a firm subbase and rutting can be expected to occur during the first two years.

4) Cracks are more likely to appear if the fines (smaller than 0,075 mm) are more than 12 %. If this type of material has to be used, it is recommended that the moisture content at compaction is kept below the laboratory determined optimum.

5) It is advisable to keep the thickness of the foam bitumen layer less than 150 mm with a bitumen content of more than 3 percent. The layer must be properly compacted. It is advisable to use a single or double seal since the seal can play an important part not only in protecting the base course layer from early day traffic but also to prevent it from drying out with time or even getting soaked during the wet season.

6) Indications are that plastic behaviour of the layer in the early stages may contribute to cracking. To avoid this, it may be beneficial to use a well graded aggregate in order to increase mechanical stability. The vane shear test will give a good indication of an appropriate material and a minimum initial value of 300 kPa or a maximum DCP value of 8 mm/blow of the time of construction is recommended in this regard.

7) It seems as if the mechanistic design approach can be used with confidence in order to extrapolate beyond experimental findings and to predict future performance.

In closing, it is evident that foam bitumen can be used successfully provided suitable materials are selected and sound engineering and construction practices are applied, specifically in mixing and compaction.

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