Performance of a Full-Scale Pavement Design Experiment in Jamaica

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The design, construction, and first 4 years' performance of seven experimental sections of road built on the May-Pen bypass in Jamaica are described. The road sections comprise varying pavement thicknesses of cement-stabilized and unstabilized marly limestones on a relatively weak, imported, clay subgrade having an in situ California bearing ratio (CBR) of 6 percent overlying a granular drainage blanket. One section has a full-depth pavement of locally available river shingle mechanically stabilized with limestone fines. The sections have a common surfacing of 50 mm of asphalt concrete. Traffic to date has been 3.0 million standard axles in one direction and 0.6 million in the other, with an average unidirectional flow of 1,300 vehicles/day. To date the primary mode of failure for the sections with unstabilized limestone bases has been deformation in the wheel tracks (rutting) with no associated cracking. The degree of rutting was revealed to be well correlated with early-life deflection, indicating a traditional mode of failure dependent on vertical stresses and compressive strains throughout the pavement. A rut depth progression model was developed that predicts rutting in terms of deflection and cumulative traffic. The model agrees closely with deflection criteria curves developed in the United Kingdom for roads having naturally cementing roadbases. After 4 years, about 10 percent of the road pavement (unstabilized bases only) reached a failure condition corresponding to a rut depth of 15 mm. From layer thicknesses, strength distributions, and the deflection performance model, it was possible to predict the performance of the thicker sections of road pavement. At the end of the study, the road was performing well in comparison with expectations from appropriate design charts but the predictions indicated that the relationship between overall thickness and traffic-carrying capacity was somewhat steeper than those of the design charts and therefore the thicker sections of road might not perform as well, relatively, as the thinner sections. The sections with cement-stabilized limestone and mechanically stabilized river shingle all performed better (showing no significant deformation) than the unstabilized limestone bases.

In Jamaica, shortages of high-quality stone have meant that marly limestones have been used for roadbases. These materials are variable in quality and are usually chemically stabilized with cement. However, in view of the traffic loadings experienced by the roads it was believed that by suitable thickness design and material selection some of the limestones ought to make satisfactory roadbases without being stabilized. To demonstrate this ability, the Ministry of Construction and the Overseas Unit TRRL agreed to cooperate on the construction and evaluation of a full-scale pavement design experiment to compare the performance of various stabilized and unstabilized marly limestones in roadbases.

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There are some areas of Jamaica where marly limestone is not readily available. River shingle is potentially an alternative material for use in roadbases provided it can be mechanically stabilized by adding the correct quantity of finer-graded material. Crushed marl ranging in size from 5 mm down to dust is suitable for this purpose, and therefore a base comprising mechanically stabilized river shingle was included in the experiment.

The properties of the materials, the design of the experiment, and an evaluation of the test sections after 4 years' traffic, during which the traffic reached 0.6 million standard axles in one direction and 2.9 million in the other, are described herein.

DESIGN OF THE EXPERIMENT

The experimental sections were constructed as part of the May-Pen bypass situated about 60 km west of Kingston. The original experiment consisted of eight test sections, each 91 m (300 ft) long, separated by nonexperimental transitions of 30 m (100 ft) between sections. The first section was subsequently abandoned because of the unusual traffic patterns arising from the proximity of a junction.

The primary purpose of the experiment was to compare the performance of different types and thicknesses of roadbase. The design of the experiment allowed the following aspects of performance to be studied.

1. The relative performance of different thicknesses of good-quality marly limestone roadbase.
2. A comparison between a roadbase of poor-quality marly limestone stabilized with cement and a good-quality unstabilized limestone of similar thickness.
3. A comparison between a full-depth pavement of river shingle mechanically stabilized with fine-graded limestone and the same total thickness of conventional pavement comprising a base of good-quality limestone and a subbase of lower quality.
4. Comparisons between conventional designs using different qualities of roadbase and subbase materials.

A cross section of the experiment showing the thicknesses of layers and material types is shown in Figure 1.

MATERIALS

Classification and strength tests on all the materials taken from the road after compaction are presented in Table 1; the mean particle size distribution of the materials is shown in Figure 2.

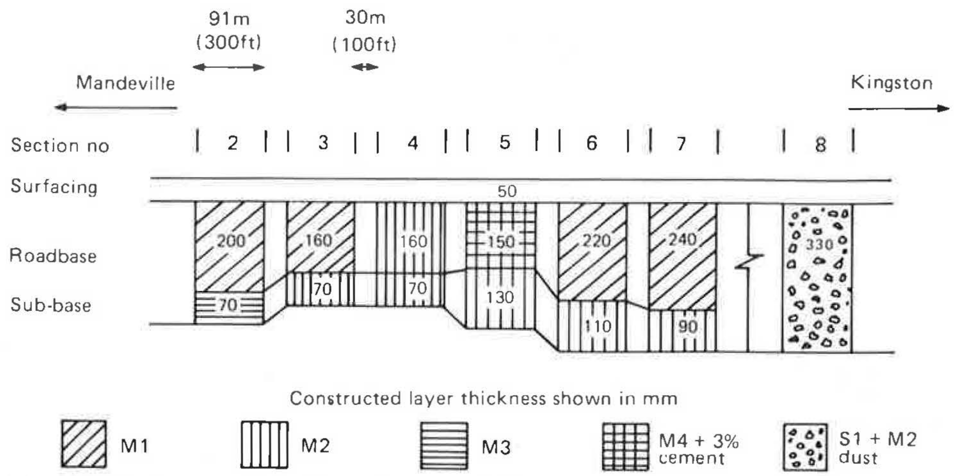


FIGURE 1 Cross section of experiment as built.

Limestone M1 is within Jamaican specifications for roadbases but M2 has a higher plasticity and is too fine to comply with the specifications. The strength of the stabilized limestone M4 exceeded the specifications in Road Note 31 *(1)*. The blended river shingle and crushed limestone used in Section 8 is well graded and nonplastic. The CBR value of this material is lower than that of the corresponding pure limestone used as roadbase on other test sections.

To ensure a uniform and relatively weak subgrade condition, a clay was imported and laid to a minimum thickness of 520 mm (20.5 in.) over a 150-mm (6.0-in.) drainage blanket. The material properties of the clay are shown in Figure 3.

CONSTRUCTION

Field compaction trials indicated that a 14-tonne smooth-wheeled roller was adequate for all base materials. During construction, a day density of 100 percent of the maximum dry density obtained in the modified AASHO compaction test was

TABLE 1 MATERIAL PROPERTIES (LABORATORY TESTS)

MATERIAL	MODIFIED AGGREGATE IMPACT VALUE[1]		LL	PI	LABORATORY COMPACTION AND CBR TESTS BS 1377 : 1975 4.5 kg RAMMER					UNCONFINED COMPRESSIVE STRENGTH[2]
	DRY	WET	%	%	MDD Mg/m³ (lb/ft³)	OMC %	CBR @ OMC %	SOAKED CBR(4 day) %	SWELL @ OMC %	N/mm² (lbf/in²)
M1	35	40	Non-plastic		2.01 (125.5)	10.0	125	100	NIL	-
M2	36	36	27	12	1.98 (123.5)	9.5	140	-	-	-
M3	41	47	21	5	1.96 (122.2)	11.2	48	26	NIL	-
M4	24	-	41	27	2.01 (125.8)	9.9	-	15	2.3	-
M4 + 3% cement			-	-	2.01 (125.9)	7.6	300	270	0.11	1.79 (260)
S1 + M2 dust	18	23	Non plastic		2.16 (135.0)	9.1	62	44	NIL	
Jamaican specification (roadbase)			25	6	-	-	-	80	0.3[3]	
Jamaican specification (subbase)			30	15	-	-	-	30	1.0[3]	

1. Tests carried out on aggregate passing 14mm and retained on 10mm
2. Tests on 150mm (6in) cube
3. BS 1377 : 1975 2.5 kg rammer

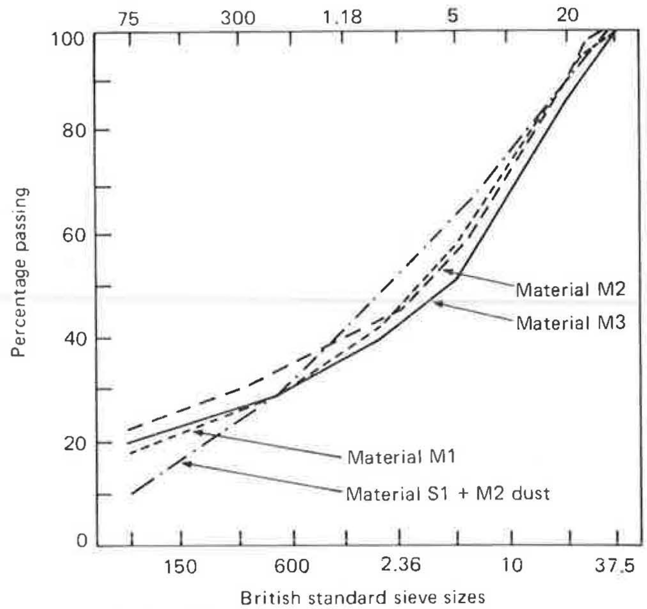
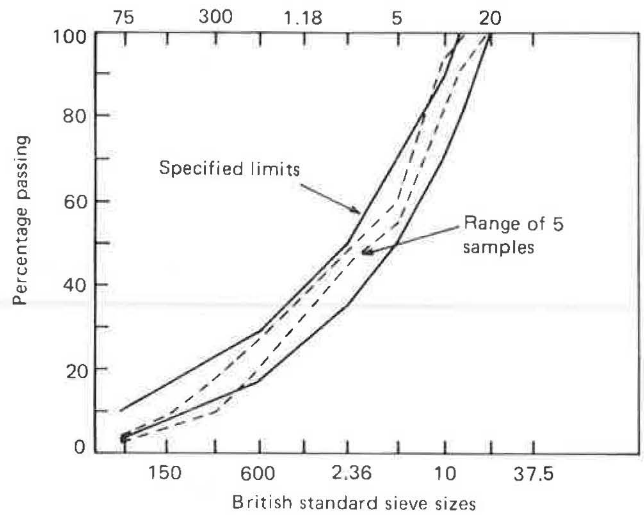


FIGURE 2 Particle size distribution in roadbase materials.

achieved with small standard deviations. The subgrade was compacted to 95 percent of the maximum dry density obtained in the British Standard compaction test (using the 1975 2.5-kg rammer method) (2).

The specifications for the asphalt concrete (AC) surfacing and the results of testing during construction are shown in Figure 4, in which it appears that quality control was good. That air voids were low is notable.

Layer thicknesses, in situ strengths of all layers, and variability in these values for each test section are discussed. The road was completed and opened to traffic by April 1981.



Specification		Measured range
Bitumen content, %	4.5 –7.0	5.8 – 6.3
VMA, %	14 max	13.0 – 13.7
Air voids, %	2 – 5	2.5 –3.4
Stability, kN lbs	>4.45 >1000	7.4 – 11.1 1650 – 2480
Flow, mm 1/100ins	2.0 – 4.6 8 –18	2.5 –3.3 10 – 13

FIGURE 4 Specification and results for asphalt concrete.

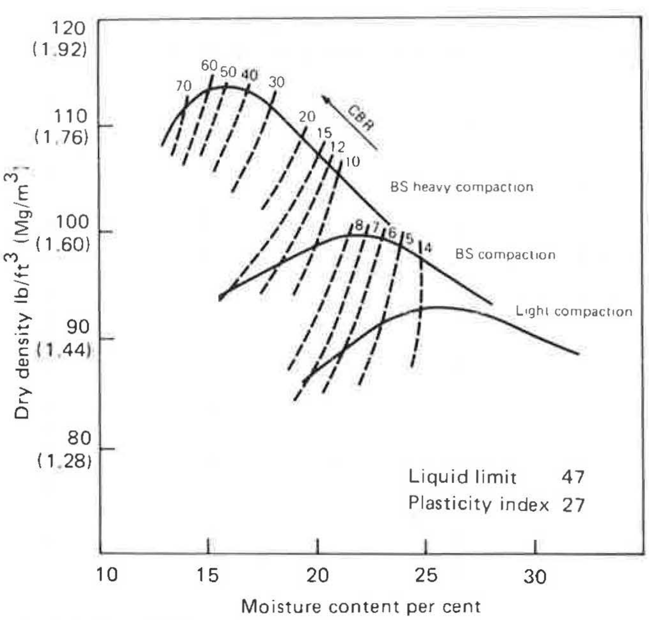


FIGURE 3 CBR-density-moisture content relationship for clay subgrade.

Traffic

The undirectional ADT since construction has been constant at approximately 1,300 vehicles/day; total commercial traffic reached 400,000 by June 1985.

Table 2 presents the percentage flow and average equiv-

TABLE 2 SUMMARY OF CLASSIFIED COUNTS AND AXLE LOAD SURVEYS FOR COMMERCIAL VEHICLES

AXLE CONFIGURATION	TOWARDS KINGSTON		TOWARDS MANDEVILLE	
	PER CENT	EF	PER CENT	EF
1.2	61	0.92	65	6.81
1.22	30	1.97	25	7.40
1.2 - 22	1	3.32	1	5.13
1.22 - 22	8	0.99	9	7.94

PER CENT = Per cent flow of the total number of commercial vehicles
EF = Equivalence factor per vehicle

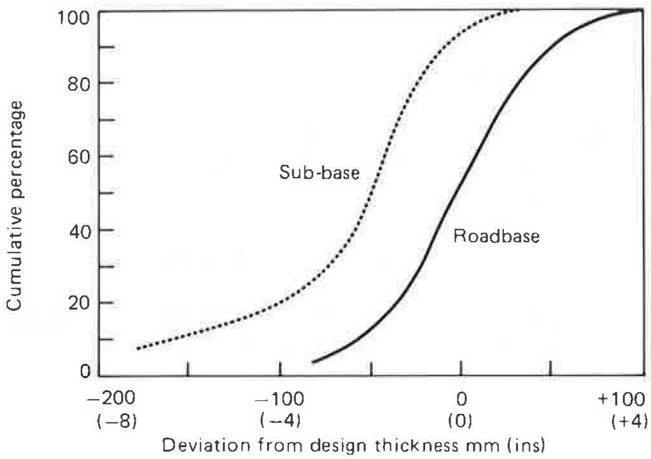


FIGURE 5 Distribution of roadbase and subbase thicknesses.

alence factor for each type of commercial vehicle for each direction. The majority of vehicles traveling toward Mandeville is heavily loaded, whereas the majority of vehicles traveling in the opposite direction is empty or only partially loaded. By June 1986, the cumulative traffic loading since construction reached 0.6 million standard axles toward Kingston and 2.9 million toward Mandeville.

PAVEMENT INVESTIGATIONS

In 1982 and in 1983, subsurface investigations were carried out by digging test pits. Between 1983 and 1985, a total of 72 nondestructive tests were performed using the dynamic cone penetrometer (DCP) to measure layer thicknesses and in situ strengths to a depth of 800 mm *(3)*.

Thickness of Pavement Layers

Although the mean thicknesses of the roadbases were revealed to be close to the design values, there was a shortfall in the thickness of subbase for Sections 2–6; in particular, Sections 4, 5, and 6 were especially thin relative to the design, having thicknesses of 73, 130, and 110 mm, respectively, instead of 200 mm. Of equal importance for the analysis of performance is the variability of thicknesses within sections. Cumulative frequency distributions are shown in Figure 5. The median roadbase thickness equals the design value, but 10 percent of the pavement has a roadbase thickness of 60 mm (2.4 in.) less. The median thickness of subbase is about 50 mm (2 in.) below the original design value, and 10 percent of the pavement has a subbase thickness 150 mm (6 in.) less than the design. If the variability of the subbase is translated into expected life, other things being equal, the lesser thickness implies that 95 percent of the pavement will fail before reaching its expected design life and 10 percent of the pavement could fail before reaching 10 percent of its design life. Variability of this magnitude often occurs for granular layers laid with a grader; it needs to be taken into account explicitly in any structural design method.

In Situ Strength of the Pavement Materials

The in situ strengths of the pavement materials as measured by the DCP are summarized in Table 3.

The minimum CBR values suggested by Road Note 31 *(1)* of 80 and 25 percent for roadbase and subbase, respectively, are soaked values, and therefore some of the in situ readings correspond to material that is weaker than specified, especially for Limestone M2 and the cement-stabilized material M4. The lower CBR value of 53 percent for M2 when used as a subbase material is a result of the lower densities.

The cement-stabilized roadbase used in Section 5 shows a particularly high variability in strength and, although the section appears to be performing well, the test pit results indicate poor mixing of the stabilizer during construction.

Laboratory investigations in 1983 indicated that there was no measurable breakdown of any of the limestones during the first 2 years of traffic.

Strength of the Clay Subgrade

Control testing for density during construction showed that the mean dry density was 96 percent of the maximum dry density achieved in the BS standard compaction test with a relatively

TABLE 3 IN SITU CBR VALUES (DCP METHOD) OF THE PAVEMENT LAYERS

MATERIAL	ROADBASE				SUBBASE			
	NO OF TESTS	$\bar{x}$	SD	RANGE	NO OF TESTS	$\bar{x}$	SD	RANGE
M1	18	119	22.1	88-150	NOT USED			
M2	5	107	34.6	67-150	21	53	18.0	27-99
M3	NOT USED				6	41	13.5	25-62
M4 + 3% Cement	5	103	55.3	33-150	NOT USED			
S1 + M2 Dust	6	97	9.1	80-105	NOT USED			

small standard deviation. Samples of clay subgrade obtained in 1982 had a mean moisture content of 20.3 percent with a standard deviation of 3.1 percent. Results of the in situ strength tests using the DCP showed that the subgrade had a mean CBR value of 9.1 percent in 1983. Using the CBR-density-moisture content relationship (Figure 3), the in situ CBR results at the given field density corresponded to a mean moisture content of 20.2 percent with a standard deviation of 1.4 percent, indicating no change from 1982 and confirming that equilibrium conditions were reached.

The DCP results were analyzed by section. At least five results available for each section showed that the subgrade along the site was particularly uniform with no significant differences at the 5 percent level between any sections. However, there was some indication at the 10 percent level that the subgrade of Section 3 was weaker than that of Section 4, Section 3 being the weakest and Section 4 the strongest along the site, mean CBR values being 7.9 and 9.5 percent, respectively. The distribution of in situ CBR values for the complete site is shown in Figure 6. The overall median value is 8.2 percent; 90 percent of the values are greater than 6.2 percent.

Structural Number

The structural number (SN) is a useful method of comparing the overall structural thickness of similar types of road. Using the thickness and strengths from the DCP tests, the SN has been calculated using the following strength coefficients *(4)*.

1. Surfacing:

$a_1 = 0.4$

2. Granular roadbase:

$a_2 = [29.14\,(\mathrm{CBR}) - 0.1977(\mathrm{CBR})^2 + 0.00045(\mathrm{CBR})^3] \times 10^{-4}$

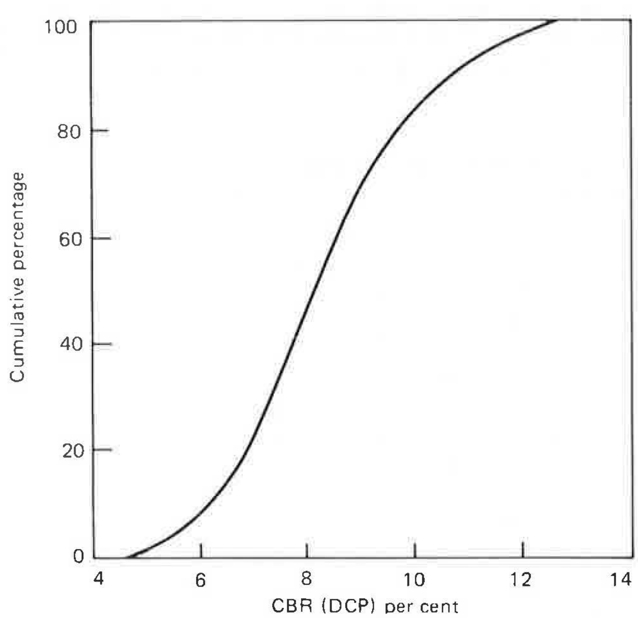


FIGURE 6 Distribution of subgrade strength.

3. Subbase:

$a_3 = 0.01 + 0.065(\log_{10}\mathrm{CBR})$

The contribution that the subgrade makes towards the overall pavement strength is taken into account by calculating the modified structural number (SN_1) using the following equation.

$$SN_1 = SN + 3.51(\log_{10}\mathrm{CBR}) - 0.85(\log_{10}\mathrm{CBR})^2 - 1.43$$

The mean value and range of the modified structural number (SN_1) for each section are shown in Figure 7.

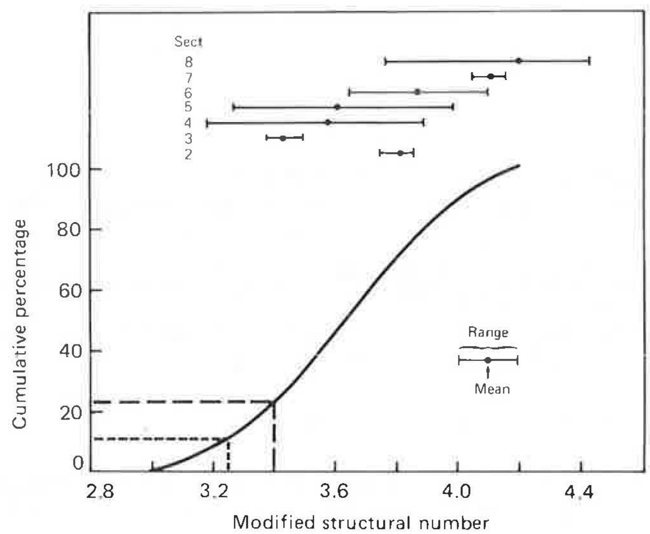


FIGURE 7 Distribution of modified structural number.

Deflection Measurements

Transient deflection measurements at 20 defined locations on each section have been carried out at regular intervals since construction using the standard TRRL method *(5)*. The results show a significant increase over the first year caused by the wetting of the imported clay subgrade. After the first year, the deflection values have remained reasonably constant, indicating that the pavement materials reached equilibrium, thereby lending additional support to the conclusions based on the subgrade strength values measured using the DCP.

That the deflection values correlated extremely well with the performance of each section is shown in the following paragraphs.

PERFORMANCE

Sections with Unstabilized Marly Limestone Roadbases

Pavement deterioration has been monitored by measuring the intensity of cracking and the deformation or rutting under a 2-m straightedge at the 20 deflection test points in each section. The results obtained in June 1985 by wheelpath for Sections 2–8 are summarized in Table 4.

TABLE 4 DEFORMATION AND CRACKING, JUNE 1985

SECTION NO	TOWARDS KINGSTON						TOWARDS MANDEVILLE					
	VERGE SIDE			OFF SIDE			OFF SIDE			VERGE SIDE		
	DEFORMATION mm		CRACKING m/m^2	DEFORMATION mm		CRACKING m/m^2	DEFORMATION mm		CRACKING m/m^2	DEFORMATION mm		CRACKING m/m^2
	MEAN	RANGE		MEAN	RANGE		MEAN	RANGE		MEAN	RANGE	
2	0.4	0-2	0	0	-	0	1.4	0-3	0	6.2	0-9	0
3	4.8	2-7	0	0.4	0-2	0	0.4	0-2	0	15.2	7-21	0
4	3.3	0-8	0.22	0	-	0	0.8	0-4	0	9.0	3-16	0
5	1.6	0-5	0	0	-	0	1.4	0-4	0.37	1.0	0-5	0
6	0.6	0-3	0	0	-	0	2.4	0-6	0	9.0	2-18	0
7	1.8	0-6	0	0	-	0	5.4	4-7	0	6.4	3-10	0
8	1.4	0-7	0.11	0	-	0	0	-	0	0	-	0.01

The predominant mode of failure to date has been deformation (Table 4). Except for Sections 5 and 8, which are discussed in more detail later, the deformation measured in the verge-side wheelpath in the direction toward Mandeville is three to four times greater than in the other, more lightly loaded, direction.

Deformation in the wheelpath depends on vertical stresses in the pavement and the number of times the stresses are repeated. Deflections also depend on vertical stresses, especially those at subgrade level. Provided all the deformation does not occur in the upper layers of the pavement as a result of inadequate shear strength, it would be expected that after a given time or cumulative traffic, deformation would correlate reasonably well with deflection. This correlation was revealed to be true in this experiment. The magnitude of the deformations is expected to increase with traffic or load repetitions. Usually a model is found to fit the data of the form

$$RD = AN^B \qquad (1)$$

where

RD = rut depth (mm),
N = number of load repetitions, and
A, B = constant dependent on deflection and other variables.

Attempts were made to fit this and other models to the data, but the model that best explained the variability was

$$RD = -0.51 + TD(0.0027D - 0.080); \quad \text{for } 35 < D < 70 \text{ and } 0 < T < 3.0, \qquad (2)$$

where D is deflection (10^{-2} mm) and T is the cumulative number of standard axles (millions).

The multiple correlation coefficient for this model was 0.84, giving an R^2 value of 0.70. An important characteristic of this model is that for no traffic the deformation is close to zero for all values of D. The predicted versus measured deformation is plotted in Figure 8. The model should not be extrapolated for deflection values less than 0.35 mm or greater than 0.70 mm.

Assuming a deformation criterion of 10 mm for defining critical pavement condition as is done in the United Kingdom, the model can be used to predict the life of other road pavements with a similar structure. Figure 9 shows results from Equation 2 superimposed on the deflection performance curve established for roads having naturally cementing roadbases in the United Kingdom. Where the curve has been extrapolated outside the range of data available at present from this study, it should be treated with caution. However, within the present range of data there is reasonable agreement between the two curves, illustrating that the deflection criterion curve established for the Jamaica test sections is similar to that developed in the United Kingdom.

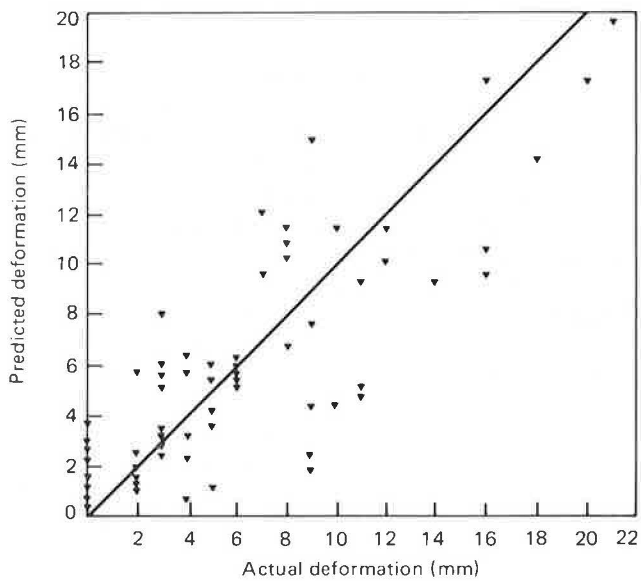


FIGURE 8 Predicted versus actual deformation for Equation 2.

Comparative Performance of Sections

Sections 2, 3, and 7 allow comparison among the performance of different thicknesses of limestone roadbase. The thinnest

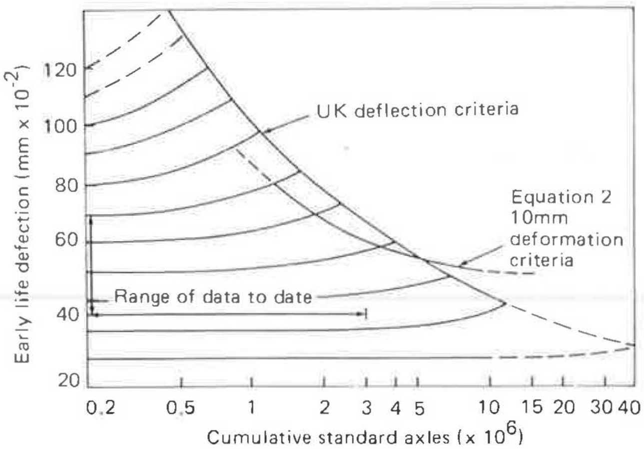


FIGURE 9 Relation between earlier life deflection and life for pavements with granular roadbases whose aggregates exhibit a natural cementing action.

section, Section 3, has performed least well, as expected, but Section 2 has performed at least as well as Section 7 despite being considerably thinner (Figure 1). The reason for this anomalous behavior is not known. The subbase of Section 2 consisted of a different limestone from that used on Sections 3 and 7, but standard tests indicated that its properties were inferior. The DCP tests indicated that the subgrades of Sections 2 and 7 were similar, but the deflections on Section 7 were slightly higher than those on Section 2. Further testing and monitoring are necessary to resolve this apparently anomalous behavior. Both sections performed well and deterioration did not progress far enough to characterize the trends.

Comparison of performance between the section with the stabilized roadbase, Section 5, and the sections with unstabilized roadbases (Sections 2, 3, 4, 6, and 7) shows that Section 5 has performed slightly better than Section 2 and considerably better than Sections 3, 4, 6, and 7, which have deformed extensively.

The mean SN_1 values of Sections 2 and 5 (Figure 7) are similar, but the range of SN_1 is greater on Section 5 with a low minimum value of 3.27 that reflects the poor mixing of the cement. Despite this poor mixing, the roadbase of Section 5 has performed better than that of Sections 6 and 7, which are considerably thicker. Unfortunately, because of the anomalous behavior of Section 2 relative to Sections 6 and 7, it is not possible to derive equivalencies between stabilized and unstabilized limestone. Comparison between Sections 2 and 5 indicates that 1 mm of stabilized limestone is equivalent to 1 mm of unstabilized but better-quality material. However, comparison between Sections 5 and 7 indicates that 1 mm of unstabilized limestone is equivalent to less than 0.75 mm of stabilized material.

It might be expected that the difference in base stiffness would be reflected in the deflection results, but Section 5 is not noticeably different from Sections 2, 3, 4, or 6. Deflection is sensitive to subgrade strength but is not particularly sensitive to the stiffness of the upper layers. It is anticipated that differences between Section 5 and others could be identified from radius of curvature measurements.

Comparison of the performance of the full-depth mechanically stabilized river shingle on Section 8 with that of Sections 6 and 7, which have approximately the same overall mean pavement thickness comprising limestone base and subbase, shows that the river shingle has performed remarkably well, being particularly stable and showing no deformation after 2.9×10^6 equivalent standard axles (esa) in the heavily loaded direction. Neither the DCP CBR values nor the deflection results have identified this effect; indeed the deflection in the verge-side wheelpath toward Mandeville is higher on Section 8 and the SN_1 values show a similar range (Figure 7).

Finally it is not possible to differentiate between the performances of the two roadbase materials used on Sections 3 and 4. Although there is no significant difference between the thickness of material used on Sections 3 and 4, there appears to be a slight difference in subgrade strength as described. This confounding effect masks any differences in performance that could be ascribed to the different materials. Further testing and monitoring are necessary before firm conclusions can be drawn.

Comparison with Appropriate Design Guides

Early-life deflection of the sections with unstabilized limestone roadbases is a good indicator of performance; deformation has been the primary mode of failure. It is also of interest to compare the performance of the sections with that expected from structural design guides.

No in situ thickness or layer strength tests apart from deflection tests were carried out at the precise location of the defined test points because such tests are either destructive or could disturb the equilibrium conditions within the road. Test pits were therefore excavated and DCP tests carried out between test points. It is a reasonable assumption that within each test section the areas showing the highest deformation correspond to the thinnest sections of road, or more accurately, to partly take account of strength differences, to areas of lowest SN_1. If it is assumed that the distribution of SN_1 corresponds to the distribution of deformation (Figures 7 and 10), it is possible to obtain a relationship between SN_1 and traffic-carrying capacity to a defined terminal condition. For example, let a rut depth of 15 mm be chosen as a critical condition. In June 1985 after the passage of 2.9 million esa, 11 percent of the pavement had a rut depth of 15 mm or more (Figure 9). This area of pavement (i.e., the weakest 11 percent) corresponds to the area of pavement with SN_1 values of less than 3.25 (Figure 7).

The expected traffic-carrying capacity of the areas of pavement that have not yet reached 15-mm deformation can be obtained by using the relationship between rut depth and traffic defined by Equation 2. The relationship is linear with traffic; therefore the expected traffic-carrying capacity of an area of pavement currently displaying a rut depth of, say, 10 mm will be 2.9 million × 15/10, that is, 4.35 million esa. Using this method and the distributions shown in Figures 7 and 10, the relationship between traffic-carrying capacity and SN_1 shown in Figure 11 was obtained.

The results show that sections were performing well in comparison with both the Road Note 31 design guide *(1)* and the NAASRA Interim Guide to Pavement Design *(6)*. Two aspects of Figure 11 are important, namely the increased traffic-carrying capacity and the steeper nature of the curve.

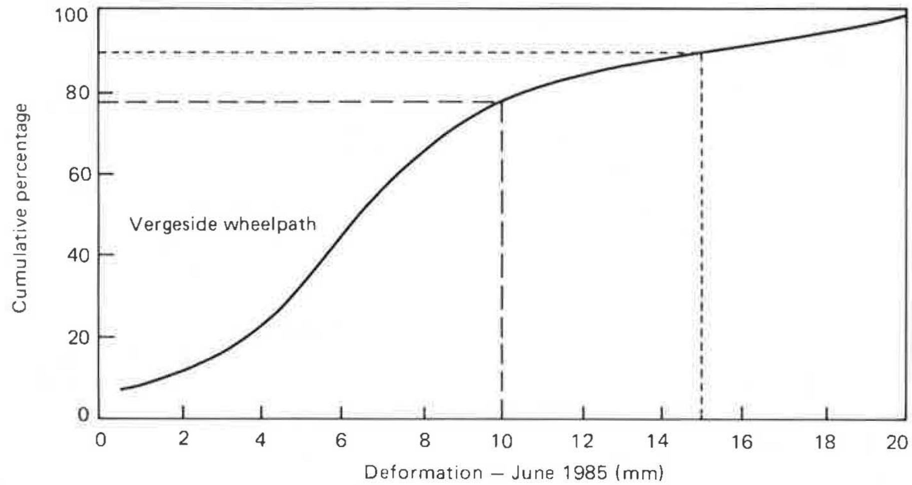


FIGURE 10 Distribution of deformation.

At the low end of the curve, around $SN_1 = 3.2$, the traffic-carrying capacity is 3 to 4 times greater than expected. Although this factor seems large, the relationship is so sensitive to SN that this improvement in capacity is equivalent to an increase in SN of only about 0.3, similar to the range measured over one test section of supposedly uniform thickness. This degree of variability is normal in road construction as demonstrated in numerous full-scale trials and pavement investigations carried out by the Overseas Unit; it must be taken into account by the road designer. Nevertheless, in this analysis the thickness variations are specifically accounted for; hence other explanations for the differences must be sought.

One possible source of systematic error is the measurement of subgrade strength using the DCP. The relationship between DCP and CBR depends to some extent on moisture content, density, and soil type. The maximum likely systematic error for this subgrade material has been estimated to be about 3 percent of CBR; if the subgrade strength has been underestimated by this amount, SN_1 would increase by about 0.3.

The other notable aspect of Figure 11 is that the slope of the relationship is steeper than expected from the design charts. Although the position and scale of the relationships are somewhat different, the slope accords with the analysis of pavement structural designs based on HDM III deterioration relationships carried out by Cox and Rolt *(7)*. However, most of the curve shown in Figure 11 is an extrapolation of existing trends. The analysis has shown that deformation is linearly related to traffic to 3.0 million esa and only 11 percent of the road has reached the 15-mm failure condition up to the present time. There is no guarantee that a similar linear behavior would apply in the future to the areas of pavement that were deforming more slowly. The effect of age hardening of the surface is expected to become apparent and the rate of deformation may therefore begin to decrease, thereby extending the life of the rest of the pavement, at least as far as deformation failure is concerned. This increase in pavement life will flatten the curve in Figure 11 provided no other form of failure (such as cracking) becomes the more dominant mode. Continued monitoring is planned to study the subsequent behavior of the trials.

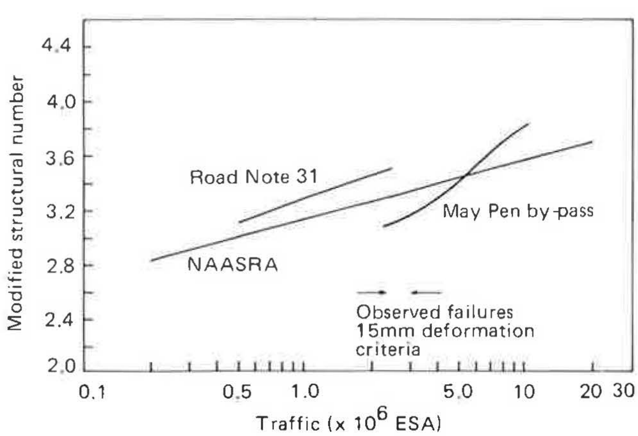


FIGURE 11 Comparison with design guides.

CONCLUSIONS

To date, the experiment has shown that with good-quality AC surfaces, marly limestones can be used to make successful road pavements that deteriorate through the gradual buildup of permanent deformations and whose performance can be predicted reasonably accurately by means of deflection measurements. A model describing the relationship between rut depth, traffic, and deflections has been derived that is applicable up to 3 million esa and for deflections between 0.35 and 0.70 mm measured under an axle load of 62.3 kN (14,000 lb).

The experiment has demonstrated the superior performance of full-depth mechanically stabilized river shingle, but the inconsistent behavior of two sections of standard construction using limestone roadbases and subbases has prevented the estimation of thickness equivalences between unstabilized good-quality limestone, cement-stabilized limestone of lower quality, and mechanically stabilized river shingle.

The overall performance of all sections is better than expected from existing design guides and reliable information for thickness designs capable of carrying up to 10 million esa is expected to be available as the experiment progresses.

The experiment has demonstrated the necessity for taking proper account of variability in subgrade strength and layer thicknesses within the structural design method.

The results have also underlined the importance of good quality control in the manufacture of the premix. The thin premix surface has been both flexible and durable, displaying no fatigue cracking and has also withstood high deformations in the other pavement layers without cracking.

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