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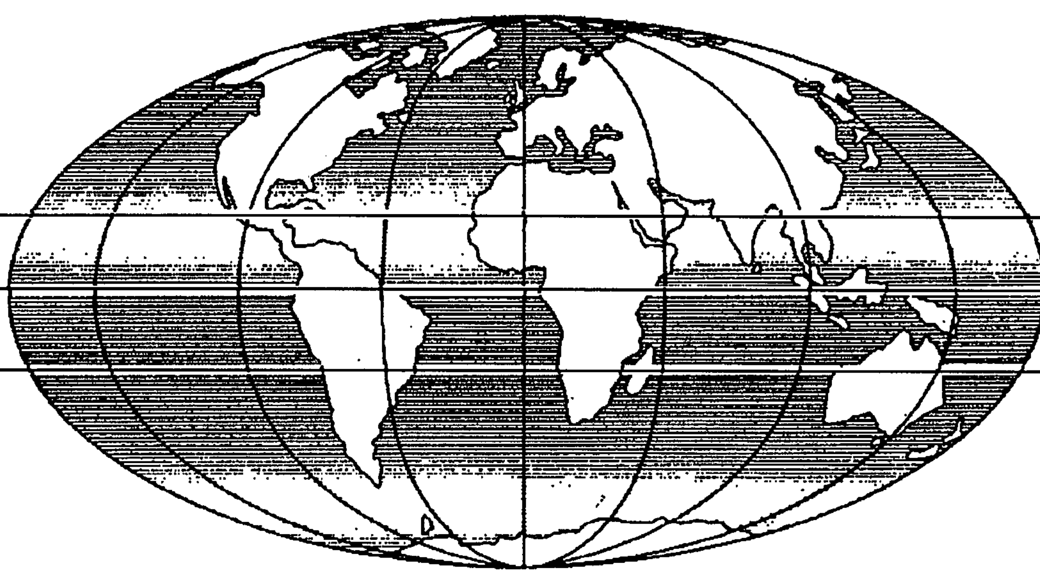
TITLE **The performance of a full-scale road pavement design experiment in Jamaica**

by J Rolt



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The performance of a full-scale road pavement design experiment in Jamaica

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This Paper describes the performance of eight experimental sections of road which were constructed using marly limestones and river shingle of marginal quality. The sections have carried nearly 5·0 million equivalent standard axles with an average unidirectional flow of 1300 vehicles per day. The primary mode of failure was found to be deformation in the wheeltracks which correlated well with early-life deflection. Models have been developed to relate the performance of the road to traffic and structural strength variables and show that the traditional relationship between pavement damage and axle load does not provide the best explanation of the observed behaviour. Strength coefficients for the various materials were derived and a design catalogue developed for the materials in question.

Introduction

In Jamaica, shortages of high quality stone have meant that marly limestones have been used for roadbases. These materials are very variable in quality and are usually 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 investigate this, a full-scale pavement design experiment was constructed to compare the performance of various unstabilized marly limestones with mechanically stabilized river shingle and a stabilized limestone.[1]

2. In this Paper the performance of the test sections after eight years of trafficking is described and the results analysed to provide a structural design guide for these materials under Jamaican conditions. Relationships have been developed between early-life deflection and traffic carrying capacity for comparison with other deflection-life curves and to provide a basis for performance prediction and overlay design. Various deterioration models have been developed which demonstrate unequivocally that the fourth power traffic damage law does not apply to the type of deterioration observed on the test sections and that a lower power provides the best explanation of behaviour.

3. The study also showed that, although deformation in the wheeltracks caused by inadequate load distribution was the primary mode of deterioration of the weaker sections of road, failure of the asphaltic concrete road surface, caused by ageing and embrittlement of the bitumen and subsequent brittle cracking, occurred on most sections after five or six years. The study thereby confirmed the importance of this type of failure which has been reported extensively elsewhere.[2–6]

Design of the experiment

4. The experimental sections were constructed as part of the May Pen bypass which is situated about 60 km west of Kingston. The layout of the experiment is shown in Fig. 1 and further details are given in Rolt et al.[1] A summary of the properties of the materials is shown in Table 1.

5. The experiment was designed to allow the following aspects of performance to be examined.

(*a*) the relative performance of different thicknesses of good quality marly limestone roadbase
(*b*) a comparison between a roadbase of poor quality marly limestone stabilized with cement and good quality unstabilized limestone of similar thickness
(*c*) a comparison between a full-depth pavement of river shingle, which was mechanically stabilized with fine graded limestone, and the same total thickness of conventional pavement comprising a roadbase of good quality limestone
(*d*) comparisons between conventional designs using different qualities of roadbase and sub-base materials.

Traffic

6. Regular classified traffic counts have been carried out since construction together with four three-day axle load surveys. By the time of the final road condition survey, the cumulative traffic loading had reached 1·3 million equivalent standard axles towards Kingston and 4·3 million towards Mandeville.

7. As the axle loads are considerably different in the two directions, the experiment provided an opportunity to examine the validity of the pavement damage law, which is generally used to express traffic damage in terms of

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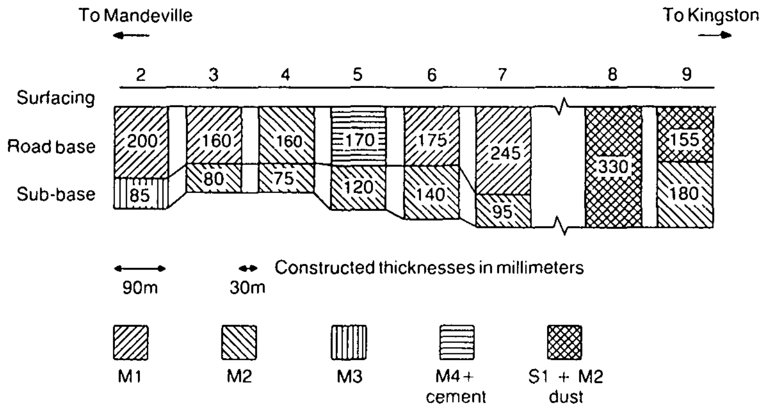


Fig. 1. Cross-section of experiment as-built

equivalent standard axles. This relationship is

$$N = \sum \left(\frac{Li}{8{\cdot}2}\right)^x$$

where

N is the total number of equivalent standard axles
Li is the axle load in tonnes of the ith axle
x is an exponent which depends on pavement structure
The summation is over all axles.

The value of x is usually assumed to lie between 4·0 and 4·5 but with a range of axle loads in each direction of travel it is not possible to analyse the data in such a way that the optimum value of the exponent can be derived directly. The best method is to express the traffic loading in terms of equivalent standard axles calculated using different values for the exponent, and then to determine which value provides the best fit to the observed road performance data. Table 2 shows the number of equivalent standard axles in each direction for each exponent value. The ratio of damage between the two directions of travel ranges from 1·5, when the exponent equals unity, to about 3·5 when the exponent is 5·0. If the best value of the exponent is less than 4, Table 2 shows that it should be possible to identify the optimum value with some precision.

Pavement investigations

8. During the first two years of the study, subsurface investigations were carried out by digging test pits in the road and removing samples of material for laboratory testing. In the subsequent five years, most of the subsurface investigations were carried out using a dynamic cone penetrometer (DCP).[7] This instrument measures the thicknesses and strengths of the unbound layers of the pavement and the strength of the subgrade to a total depth of up to 1·2 m. Its ease and speed of use enables sufficient measurements to be made to provide a level of statistical reliability which more than compensates for the need to check the relationship between DCP reading and strength (e.g. California bearing ratio (CBR) for the materials in question. During the three condition surveys that took place in the last five years of the study, 120 DCP tests were performed.

Layer thicknesses

9. The results from the DCP tests gave layer thicknesses which are accurate to within 10 or 15 mm. The mean thicknesses of the roadbases were found to be close to the design thicknesses but the sub-bases were usually thinner than expected. The results are summarized in Fig. 1.

10. Of equal importance for the analysis of performance is the variability within sections. The cumulative frequency distribution of the total thickness of roadbase plus sub-base relative to the mean thickness of each section is shown in Fig. 2. A high degree of variability is apparent. Variability of this magnitude almost always occurs with granular layers which are shaped by grader and must be taken into account in both performance analysis and pavement thickness design.

Strength of the clay subgrade

11. There is usually a degree of judgement required in characterizing a subgrade from DCP data. One problem arises because the subgrade almost invariably undergoes changes in

Table 1. Summary of material properties

Material	Liquid limit: %	Plasticity index: %	CBR @ OMC: %	Soaked CBR (4 days): %	Unconfined compressive strength: N/mm²
M1	Non plastic		125	100	—
M2	27	12	140	NA	—
M3	21	5	48	26	—
M4	41	27	—	15	—
M4 + 3% cement	—	—	>300	270	1·8
S1 + M2 dust	Non plastic		62	44	—

M = Marly limestone, S = River shingle

Table 2. Standard axles using different damage laws

Direction	Traffic	Power law exponent				
		1	2	3	4	5
To Kingston	esa/day	332	257	276	373	649
To Mandeville	esa/day	504	573	801	1283	2253
	Ratio	1·52	2·23	2·90	3·44	3·50

strength with depth. The computer program for the analysis of DCP data[7] takes this into account in a systematic manner and removes the need for subjective interpretation of the strength/depth profile. Table 3 summarizes the results of the subgrade strength surveys. The differences between surveys were not found to be statistically significant and a CBR value of 5% is indicated for structural design purposes.

12. DCP tests cannot be made at identical locations at each survey but some of the measurements made in 1988 were close to those made in 1985. These tests provide an opportunity to investigate more precisely any changes in subgrade strength because the variability within site can be largely removed.

13. Table 4 shows that a slight reduction in strength seems to have occurred, but the difference is not significant and could have arisen by chance. It can therefore be concluded that no measurable changes in subgrade strength have occurred after equilibrium was established early in the life of the road.

14. Further analysis of the subgrade strength data for each test section showed that the subgrade was effectively similar thoughout the site, thereby considerably simplifying the performance analysis.

In-situ strength of roadbases and sub-bases

15. The in-situ CBR values derived from the DCP tests showed that the majority of the materials were as strong as would be expected from the laboratory tests which were made on samples of the material prior to construction. For example, only two tests on roadbase material M1 showed the CBR to be less than 150% and these values were only slightly lower. Generally, between 5% and 10% of the results were found to be unsatisfactory, often only marginally so, but this variability is expected and was taken into account in the performance analysis.

Table 3. Subgrade strengths: CBR

Year of survey	1983	1984	1985	1988
Median	8·0	8·4	7·7	7·4
Mean	8·7	9·3	8·8	8·2
Standard deviation	2·3	2·3	2·7	2·5
Lower 10 percentile	5·6	6·0	5·4	4·9

Table 4. Subgrade strengths at similar locations

Year of survey	1985	1988
Median	8·1	7·0
Mean	8·1	7·1
Standard deviation	1·9	1·4

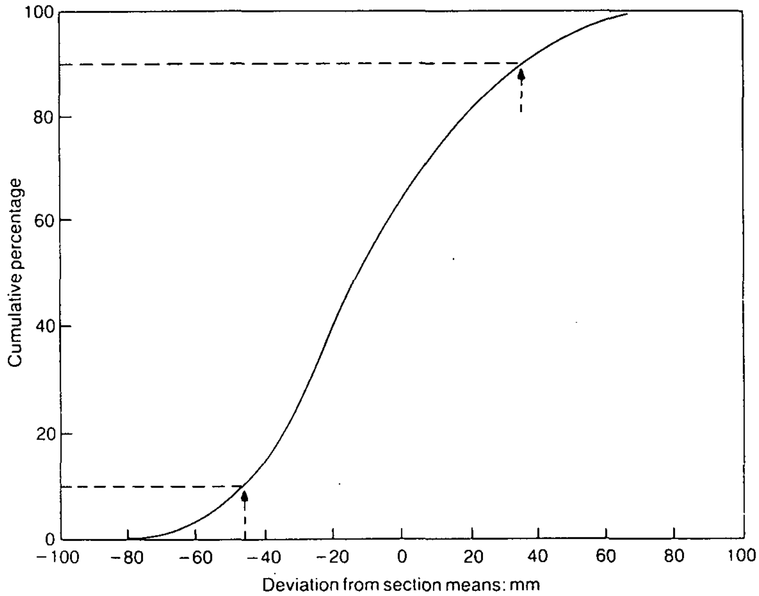


Fig. 2. Distribution of roadbase plus sub-base thicknesses for all sections

Overall pavement strength—structural number

16. The structural number (SN) is a useful method of comparing the overall structural 'strength' of similar types of road. Using the thicknesses and strengths from the DCP tests, the SN has been calculated using the following strength coefficients.[8]

(*a*) Surfacing: $a_1 = 0{\cdot}4$
(*b*) Granular roadbase: $a_2 = (29{\cdot}14\,(\mathrm{CBR}) - 0{\cdot}1977\,(\mathrm{CBR})^2 + 0{\cdot}00045\,(\mathrm{CBR})^3) \times 10^{-4}$
(*c*) Sub-base: $a_3 = 0{\cdot}01 + 0{\cdot}065\,(\log_{10}\mathrm{CBR})$

The contribution of the subgrade towards the overall pavement strength is taken into account by calculating the modified structural number (MSN) using the following.[8,9]

$$\mathrm{MSN} = \mathrm{SN} + 3{\cdot}51\,(\log_{10}\mathrm{CBR}) - 0{\cdot}85\,(\log_{10}\mathrm{CBR})^2 - 1{\cdot}43$$

The mean value and range of the Modified Structural Number for each section is shown in Table 5. The wide variability is notable.

Table 5. Structural numbers (vergeside only)

Section no.	Modified structural number	
	Mean	Range
2	3·6	3·0–3·8
3	3·3	3·0 3·6
4	3·4	2·9–3·8
5	3·6	3·1 4·0
6	4·0	2·6–4·2
7	4·0	3·8 4·2
8	3·7	3·2 4·4
9	3·7	3·1 4·1

Deflection measurements

17. Transient deflection measurements at 20 defined locations within each section have been carried out at regular intervals using the standard Transport Research Laboratory (TRL) method, by which a lorry has a rear axle loaded to 62·3 kN.[10] The results showed a significant increase over the first year but after June 1983 the deflections have remained reasonably constant on most sections indicating that the road pavements had reached an equilibrium condition and lending additional support to the conclusions based on the DCP results described above. The early life deflections for each test section, corrected to 35°C, are shown in Table 6. It is shown below that the performance of each test section correlated well with deflection values.

Visual condition surveys

18. Visual condition surveys were carried out to record the degree and extent of both rutting and cracking. During each survey, rut depths were measured at the deflection test points and the areas of surface cracks were mapped. Considerable cracking occurred between the last two surveys, especially in the vergeside wheelpaths of sections 8 and 9. Much of this cracking has been caused by the ageing and embrittlement of the surfacing rather than through either traditional fatigue of the asphalt or directly as a result of excessive deformations (see §§ 38–40).

19. The correlation between cracking and rutting is generally very poor except for a few isolated places where cracking occurred relatively early in the life of the pavement. In these locations, the cracking allowed water to penetrate the roadbase and sub-base over a sufficient period of time for deformations to have developed in the weakened material. The ranges of deformations recorded during the final survey are summarized in Table 7 and give a good indication of the relative performance of the sections.

Table 6. Range of early-life deflections (vergeside only)

Section no.	Deflections: mm	
	To Mandeville	To Kingston
2	0·36–0·61	0·40–0·51
3	0·56–0·67	0·47–0·57
4	0·37–0·56	0·49–0·69
5	0·38–0·57	0·35–0·66
6	0·46–0·60	0·46–0·55
7	0·43–0·56	0·42–0·62
8	0·45–0·65	0·43–0·83
9	0·44–0·67	0·50–0·59

Table 7. Rut depth values (mm) recorded during the final survey

Section no.	To Kingston				To Mandeville			
	Vergeside		Offside		Vergeside		Offside	
	Mean	Range	Mean	Range	Mean	Range	Mean	Range
2	4	0–8	1	0–3	16	2–34	4	2–9
3	15	7–23	3	0–5	27	16–45	2	0–4
4	18	0–32	0	0	17	5–27	2	0–8
5	3	0–11	0	0	5	0–9	3	0–8
6	8	7–10	1	0–3	14	5–23	6	0–17
7	6	4–10	0	0	10	6–15	8	6–10
8	16	0–29	0	0	9	0–15	0	0

* Surface failures were too advanced on much of section 9 for reliable analysis

Analysis of performance

20. The primary mode of failure of five of the test sections (2, 3, 4, 6, 7) was rutting in the wheeltracks. Little or no rutting occurred on sections 5, 8 and 9 until the surface had cracked and therefore the primary mode of failure of these sections differed from that of the others. The deformation was not confined to the bituminous surfacing and is known to depend on pavement deflection.[1] The mode of failure is therefore structural in nature and depends on overall pavement thickness and the strengths of the pavement layers and the subgrade. It was therefore possible to derive relationships between pavement performance and structural variables. These relationships have been used to draw up thickness design charts for the types of materials and traffic levels covered by the experiment. For this purpose the variables must be either measurable before construction begins or controlled by means of standard specifications. Variables which depend on composite structural properties, such as surface deflection, can rarely be used for design because their values cannot be known accurately beforehand.

21. However, once a road is built the prediction of performance often proves difficult because the detailed structural information available at the time of construction is rarely available at a later date. Also, the properties of the materials may have changed with time. To obtain this information the engineer must usually open test pits in the road. This is expensive and time consuming and does not provide sufficient statistical information concerning the variability along the site. In this situation, non-destructive test methods are preferred and composite structural variables, especially those derived from surface deflections, are especially useful.

22. It is therefore desirable to develop performance relationships between deterioration and both types of structural variables. The two sets of variables should be related and there-

fore these interrelationships have also been explored.

23. Road pavements are notoriously variable in physical properties over quite small distances. To determine relationships between structural variables and performance, it is usually essential that measurements of both sets of variables correspond to the same locations; sectional averages cannot usually be used for detailed analysis because too much detail is lost. For this reason the performance of each section was initially analysed individually and each measurement point was considered as a separate pavement characterized by variables measured only at that point (the effective area of influence around each point is of the order of the area of a deflection bowl or about 10–15 m^2. To satisfy these requirements, rut depths and surface condition were always measured both at the deflection points and also at the points where DCP measurements were made.

Performance models based on deflection

24. For each section, models were developed for the deformation in the vergeside wheeltracks as a function of traffic, age, deflection and combinations of these variables. Some of the results are summarized in Table 8 for the two directions of travel. The general form of the models is

$$\text{Rut depth (RD)} = a_0 + a_1 \times \text{ESA} + a_2 \times \text{Age} + a_3 \times \text{DEF} + \sum a_i \times \text{Interaction terms.}$$

Only those models in which all coefficients were significant, as measured by Student's 't' test, have been included. No suitable models could be derived for sections 8 and 9. The section with the cemented roadbase, section 5, behaved quite differently from the others and has been excluded.

25. For some of the sections, especially in the heavily loaded direction towards Mandeville, some of the models are good, with R^2 values greater than 0·7. However, comparison between sections of the models with the same variables illustrates that there are marked differences between them. For example, the models show that, for the same deflection and traffic level, section 6 deteriorates more rapidly than section 7. On the other hand section 2 is very similar to section 4.

26. A comparison of the models which have Age as the only explanatory variable (Table 9) reveals that the fourth power law for pavement damage does not hold. According to this law, the traffic in the direction of Mandeville should be 3·4 times more damaging than the traffic in the direction of Kingston (Table 2), or, in other words, the number of vehicles required to generate the same depth of rut in the Kingston direction should be 3·4 times greater than in the Mandeville direction. Table 9 shows that, with the exception of section 2, the traffic seems to be only about 50% more damaging in the Mandeville direction. This is in good agreement with a power law of unity, as indicated in Table 2.

27. To study this further, all the traffic data were reanalysed using different power law .

Table 8. Rut depth models by section (vergeside only)

	To Mandeville	
Section	Equations	R^2
2	RD = −3·4 − 2·19 (ESA) + 0·118 (ESA × DEF)	0·61
3	RD = 1·5 − 17·1 (ESA) + 0·355 (ESA × DEF)	0·73
4	RD = −2·5 + 0·085 (ESA × DEF)	0·63
6	RD = −0·9 + 10·8 (ESA) + 0·268 (ESA × DEF)	0·93
7	RD = 0·2 − 5·4 (ESA) + 0·151 (ESA × DEF)	0·90
2	RD = −2·8 − 0·13 (Age) + 0·064 (Age × DEF)	0·63
3	RD = 2·3 − 0·93 (Age) + 0·0191 (Age × DEF)	0·73
4	RD = −1·8 + 0·0044 (Age × DEF)	0·62
6	RD = −0·3 − 0·585 (Age) + 0·0144 (Age × DEF)	0·91
7	RD = 0·6 − 0·3 (Age) + 0·0081 (Age × DEF)	0·89
	To Kingston*	
3	RD = −1·5 + 0·274 (ESA × DEF)	0·51
4	RD = −2·4 − 37·6 (ESA) + 0·957 (ESA × DEF)	0·63
6	RD = −1·8 + 9·0 (ESA)	0·55
7	RD = −0·7 − 7·15 (ESA) + 0·246 (ESA × DEF)	0·51
3	RD = −2·9 + 0·0033 (Age × DEF)	0·61
4	RD = −4·8 − 0·365 (Age) + 0·10 (Age × DEF)	0·69
6	RD = −2·8 + 0·11 (Age)	0·67
7	RD = −1·3 − 0·063 (Age) + 0·0025 (Age × DEF)	0·56

* No suitable models could be developed for section 2 in the Kingston direction
ESA: millions
DEF: 0·01 mm
Age: months

Table 9. Coefficients of the Age term in Age only models

Section no.*	To Mandeville	To Kingston	Ratio
2	0·186	0·054	3·4
3	0·246	0·176	1·4
4	0·200	—	—
6	0·159	0·110	1·5
7	0·104	0·073	1·4

* Good models could not be obtained for all sections

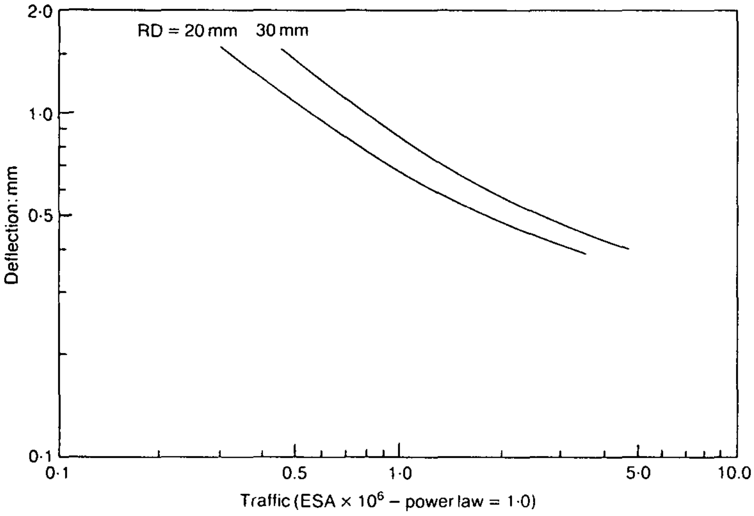


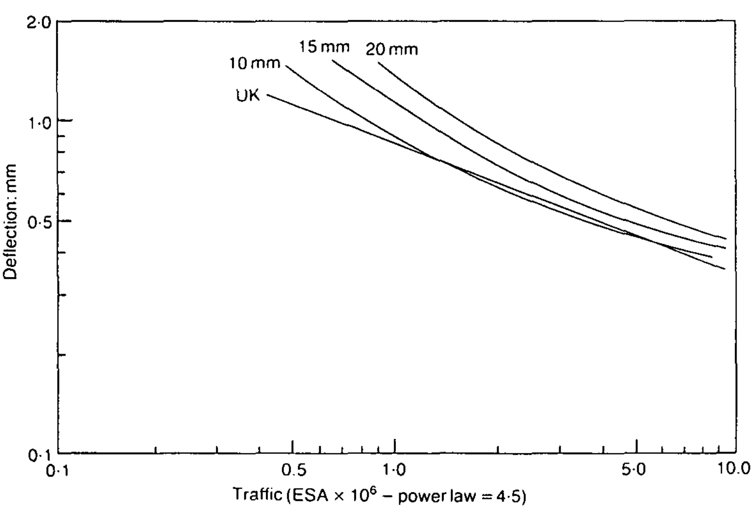
Fig. 3. Deflection–life curves (combined model)

relationships. Models were then developed for each section by combining the data for both directions of travel. For sections 2, 6 and 7 the exponent which explains the data most satisfactorily was found to be between 2 and 3. For sections 3 and 4, which were the weakest sections (Tables 5 and 7), the optimum value was less than unity. The discriminatory power of this method of analysis is poor for exponents above 3, however, there is strong evidence that the best value for the exponent is considerably less than 4·5 under the conditions of the experiment and for the type of failures observed.

28. Combining the data for both directions of travel and for sections 2, 3, 4, 6 and 7 leads to a series of models for each exponent value. The R^2 coefficients for those models were all between 0·60 and 0·64 and the best model was for an exponent of unity (R^2 = 0·64) although the model for an exponent of 2 was almost as good. The model is

$$\mathrm{RD} = -2{\cdot}3 - 16{\cdot}5\,(\mathrm{ESA}_1) + 0{\cdot}583\,(\mathrm{ESA}_1 \times \mathrm{DEF}) \quad (1)$$

Fig. 4. Deflection–life curves



29. This compares with the model for an ESA exponent of 4·5 (R^2 = 0·60) which is

$$\mathrm{RD} = 0{\cdot}50 - 6{\cdot}4\,(\mathrm{ESA}_{4.5}) + 0{\cdot}193\,(\mathrm{ESA}_{4.5} \times \mathrm{DEF}) \quad (2)$$

where ESA_x = millions of equivalent standard axles based on an exponent of x.

30. In deformation studies, especially in the laboratory, it has often been found that models of the following form provide the best fit to deformation data,

$$\mathrm{RD} = AN^x$$

where N is the number of stress repetitions and A and x depend on material properties. Attempts to fit models of this kind were not particularly successful, with R^2 values always consistently lower than the linear models.

Deflection–life relationships

31. Where structurally based deformation failures have been predominant, it has been found useful to express the performance data in the form of deflection–life curves to allow predictions of traffic carrying capacity to defined levels of deterioration. These can be derived for any level of deterioration but the selection is usually based on either a critical level at which overlaying with a new structural surface is the most appropriate method of repair, or a failure level at which major reconstruction is likely to be necessary.

32. Figure 3 illustrates the results obtained using the combined model (eq. 1, i.e. excluding sections 5, 8 and 9) for terminal rut depths of 20 and 30 mm. For comparison with the UK deflection–life curves, the combined model based on an exponent of 4·5 (eq. 2) has been used for terminal rut depths of 10, 15, and 20 mm (Fig. 4). It can be seen that over the range for which the models were derived, namely deflections between 0·3 and 0·8 mm, the results are very similar to the UK curve which is based on a critical rut depth of 10 mm.

Performance models based on layer properties

33. As with the relationships between performance and deflection, it is again important to take account of the wide variability of performance and strength within each experimental section by treating each test or measurement point separately. In addition, each wheelpath and each direction of travel must also be treated separately, being combined only when the form of the models show that it is possible and appropriate to do so.

34. The deformation in the wheeltracks for each section should depend on measurable strength variables. The SN and MSN are simple methods of combining both layer thicknesses and strengths which have been used extensively for analysis and design. In this study the

DCP tests have provided contiguous data on the strengths of the pavements to a depth of at least 750 mm. It was, therefore, thought possible to relate performance to an improved measure of pavement strength based on a more detailed interpretation of these strength profiles.

35. For the Mandeville direction, several models were derived which were considerably better than models based on MSN but, invariably, the opposite was true for the Kingston direction. Excluding the section with the cement-stabilized base from the analysis and selecting models with both a high value of R^2 in the Mandeville direction and a reasonable value in the Kingston direction, the best models were of the form illustrated in Table 10. With data of this kind it is possible to derive a large number of models of similar form and with similar R^2 values. Although extrapolation beyond the range of the data is not recommended, it is prudent, other things being equal, to select the final model on the basis of its extrapolation properties. The final choice of model, shown in the last row of the Table, was based on a good fit within the range of the data combined with engineering judgement to ensure that extrapolations outside the experimental range gave realistic answers. This was done by choosing a model that gave rise to a relationship between MSN and traffic carrying capacity that followed the same general shape in the areas of extrapolation as similar relationships derived from accepted design charts such as Road Note 31[11] (see §§ 36–37).

Structural design models

36. Using the relationships between rut depth and MSN derived above and the relationships between rut depth and traffic described in §§ 24–40, it is straightforward to derive relationships between MSN and the traffic required to produce any specified level of deformation. Such relationships can then be used to draw up a catalogue of structures or a design chart. Once again it is clear that the 4·5 power law does not express the relative damaging power of the traffic correctly. For it to do so, the carrying capacity of the pavement should be independent of the direction of travel, and the relationships for the two directions should be congruent. It is found that a power law close to unity provides the best fit for the data, with the relationships for the two directions becoming indistinguishable. The results are shown in Fig. 5.

37. Unfortunately, since the traffic damage law in these experiments has been shown to differ considerably from the 4·5 power law normally used, direct comparisons of traffic carrying capacity with design charts in common use are difficult to make. There is no absolute measure of pavement damage that will permit cross calibration to be made. It is clear that the difference in damaging power between traffic in the two directions is not as large as predicted but it could be argued with equal validity that the damage in the more lightly trafficked direction is greater than expected or that the damage in the heavily loaded direction is less.

Table 10. Deformation: SN models for sections 2,3,4,6,7 and 8 (vergeside wheelpath)

To Mandeville		To Kingston	
Equation	R^2	Equation	R^2
$111-76{\cdot}9 \log_e$ MSM	0·80	$74{\cdot}7 - 51{\cdot}8 \log_e$ SN	0·50
$(1050.\text{MSN}^{-3}) - 11{\cdot}4$	0·83	$(736.\text{MSN}^{-3}) - 9{\cdot}1$	0·55
$(740.\text{MSN}^{-2{\cdot}6}) - 15{\cdot}0$	0·82	$(260.\text{MSN}^{-1{\cdot}9}) - 15$	0·45
$(1220.\text{MSN}^{-3}) - 15$	0·79	$(1220.\text{MSN}^{-3}) - 19$	0·30

Surface cracking

38. During the first five or six years of the study, deformation was the primary mode of failure on most sections but extensive cracking has subsequently occurred on some sections as discussed in §§ 18–19.

39. Samples of asphaltic concrete were obtained from areas of pavement displaying different surface characteristics and were analysed to determine the properties of the bitumen in the mix. The samples were sliced so that the bitumen in the top 5 mm could be examined separately from the bulk of the sample. The viscosity of the recovered bitumen was measured at a temperature of 45°C using a sliding-plate viscometer at shear rates in the range 5×10^{-1} to 5×10^{-3}. Some of the results are illustrated in Table 11 and confirm that severe ageing has taken place in some areas of the road. The viscosity ranged from $2{\cdot}2 \times 10^4$ poise in samples from sound areas or surfacing which were smooth and rich in bitumen up to 4×10^6 poise in samples from cracked areas which were dry and rough.

Table 11. Viscosity of recovered bitumen (shear rate $= 5 \times 10^{-1}$ and $T = 45°C$)

Sample area	Description of surfacing	Viscosity (Log_{10} Poise)	
		Bulk	Top 5 mm
1	Very dry, loss of material and badly cracked	6·45	6·45
2	Dry, loss of material and cracking	5·95	6·00
3	Rough and slightly dry. No cracks	4·92	5·95
4	Smooth, no cracks, veins of bitumen evident	4·34	5·58

40. The results show that bitumen from the top 5 mm of each sample was up to twenty times more viscous than that from the bulk of the sample. This hardening and embrittlement is the cause of the cracking which occurred between the last two condition surveys. The results from Jamaica are similar to those obtained elsewhere[2,4,5,6] and further confirm that the kind of failure is predominant in much of the tropics.

Comparative performance of the test sections

41. The models described in §§ 33–35 above are based on an analysis which includes all the sections, or all except sections 5 and 9. Although some of the models appear to be quite good, it is useful to examine each section in more detail to identify any distinctive behaviour.

42. Section 5 has performed better than any other although its Modified Structural Number is not high. Excluding this section, which is the only one with a cement-stabilized base, the other sections have performed largely as expected from their Modified Structural Numbers, at least as far as deformation is concerned.

43. Overall, the deformation on section 5 was equivalent to a pavement with a Modified Structural Number of about 4·1. This is 0·6 higher than the measured value and indicates that the effective strength coefficient for the roadbase is rather higher than estimated from the in-situ CBR. Using the actual roadbase thickness of 168 mm, the effective strength coefficient is about 0·22 instead of 0·13. In other words, 100 mm of cement-stabilized roadbase is equivalent to about 160 mm of unstabilized, good quality, marly limestone, at least as far as the deformation resistance of the composite structure is concerned.

44. It might be expected that the difference in roadbase stiffness would be reflected in the deflection results, but perusal of Table 6 indicates that section 5 is not noticeably different from sections 2, 4 or 6. Deflection is sensitive to subgrade strength but is not very sensitive to the stiffness of the upper layers. It is expected that differences between section 5 and other sections could have been identified from radius of curvature measurements but suitable equipment was not available.

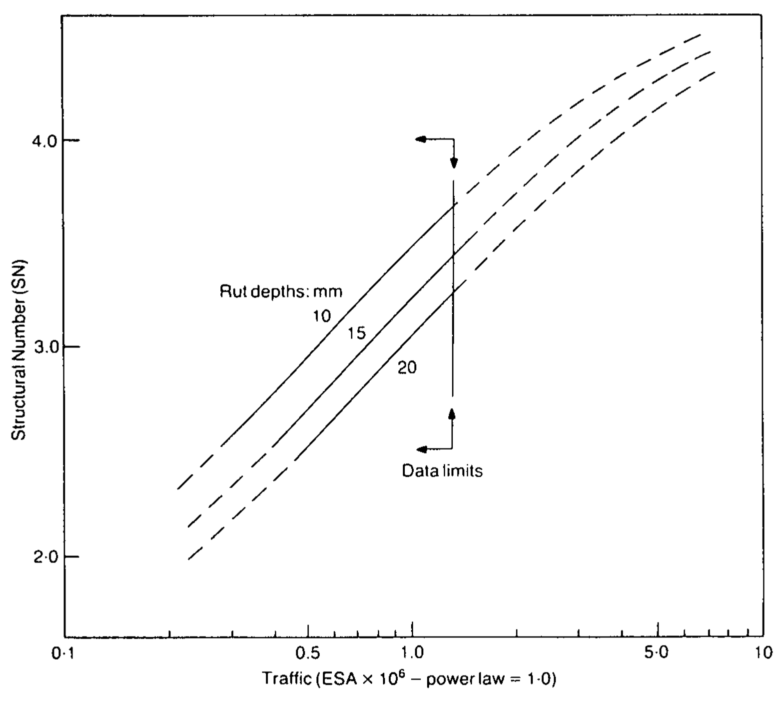
45. It has not proved possible to distinguish between the performance of the two unstabilized limestones used for roadbase, M1 (used for sections 2, 3, 6 and 7) and M2 (used for section 4) (see also Table 1) except in so far as their performance is reflected in their SNs. The roadbase strength coefficient, a_2, is not sensitive to the CBR values when these are high, hence, as far as deformation is concerned, the two materials appear to have performed identically. However, M2 contains plastic fines and under wetter conditions is not expected to perform as well as M1. Indeed, the final survey showed evidence that, in the Kingston direction, horizontal shear had occurred between the asphaltic concrete surface and the roadbase of section 4. The reason for this horizontal movement could not be determined with certainty but must have been exacerbated, if not actually caused, by the presence of the plastic material.

46. The results from the sections with mechanically stabilized river shingle, namely sections 8 and 9, show that performance was good until the surface began to crack. Indeed, during the first five years no deformation occurred on section 8. Unfortunately, by the time of the last survey, the advanced level of cracking on parts of these sections reduced the amount of useable data and decreased the reliability of the performance models for these sections. The effective strength coefficient of the stabilized shingle is probably about 0·15–0·16 rather than 0·13 but there is insufficient data to provide a precise value.

Design charts

47. Using the results illustrated in Fig. 5, design charts have been prepared for the conditions and materials in Jamaica. The chart for road structures using marly limestones for roadbases and sub-bases is illustrated in Fig. 6. The designs are based on the fact that 85% of the rut depth data which were used to develop the design equations (Table 10) lies within

Fig. 5. Structural design models



±50% of the mean rut depth calculated from the regression equations. Thus, by using a rut depth of 15 mm for the mean rut depth at the end of the design period, less than 10% of the road will reach a rut depth of 22 mm during that time. This, combined with the selection of subgrade strength based on the lower 10 percentile value expected under equilibrium conditions produces structural designs with an acceptable degree of safety. Nevertheless, some engineering judgement will always be required during the design process.

48. The as-built thicknesses of unbound pavement layers are usually more variable than desired and this leads to another source of uncertainty. If the thicknesses in the charts are specified as minimum values, the resulting designs will be unduly conservative. It is recommended that the thicknesses of unbound layers are specified as lower 10 percentiles. This is based on the reasonable assumption that the areas of road which are relatively thin are very unlikely to coincide with areas within the lower 10 percentile for subgrade strength.

49. The other major uncertainty is the estimation of traffic. If the predicted traffic is close to the boundary between one traffic class and the next *and* if the subgrade strength lies towards the lower limit of the subgrade strength category, it would be prudent to opt for the higher traffic class for design purposes.

50. Finally the designer needs to address the problem of the pavement damage law. The charts have been drawn up using both the traditional 4·5 power law as well as the power of unity derived during this study. Clearly it would not be sensible to abandon the 4·5 power law based on the result of one experiment. Nevertheless, serious errors could occur in design if the axle load spectra differ from that of these experiments and the wrong damage law is used. Although there is evidence from elsewhere that the 4·5 power law may need to be reconsidered for some types of structure and for certain modes of failure,[12] it is recommended that axle load data are analysed using both laws and the structural design for each method of analysis is identified from the charts. The design which is the most conservative should then be selected.

51. Despite the success of the experiment in developing structural designs, the eventual mode of failure for much of the trial site was cracking in the bituminous surfacing caused by the ageing and embrittlement of the asphaltic concrete surface. This type of failure is extremely common in hot countries and is the subject of an extensive research programme worldwide. It is advisable to design the surfacing mix to be as flexible as possible and to minimize the risk of brittle failure by suitable surface maintenance such as regular surface dressings.[5]

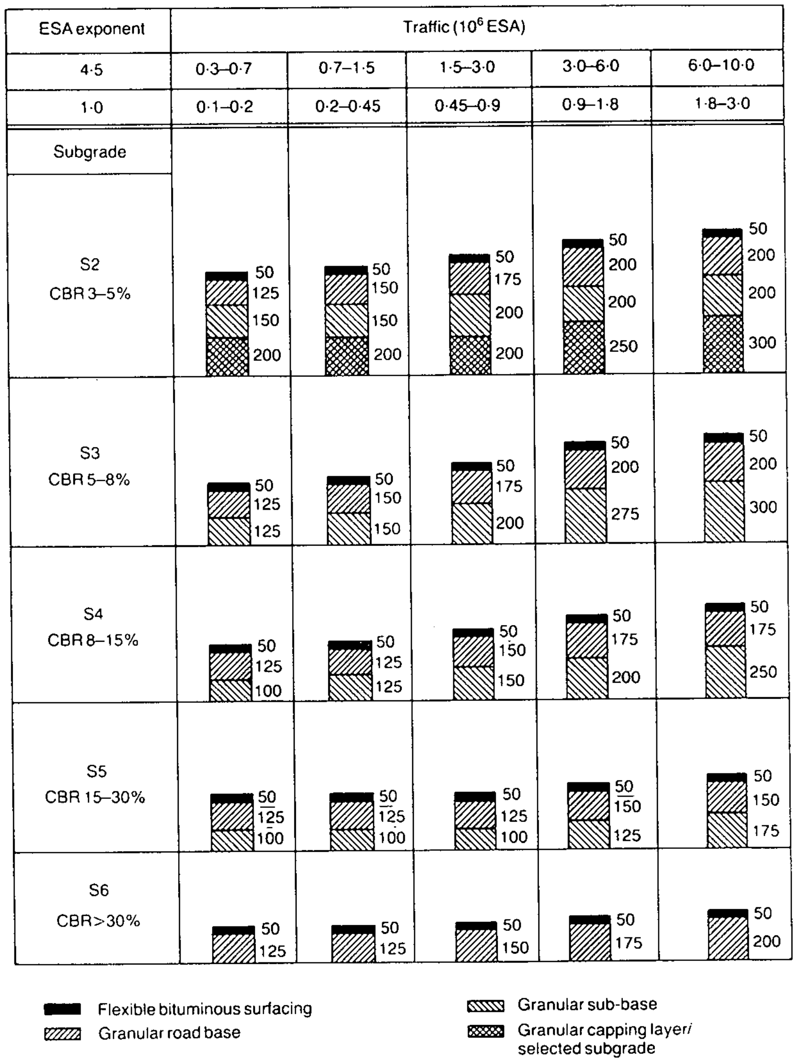


Fig. 6. Pavement designs—catalogue of structures

Conclusions

52. The following conclusions were made from the experiment.

(*a*) With good quality asphaltic concrete surfaces, marly limestones can be used to make successful road pavements the performance of which can be predicted reasonably accurately by means of deflection measurements.

(*b*) A model describing the relationship between rut depth, traffic and deflections has been derived which is applicable up to about 5·0 million standard axles and for deflections between 0·35–0·80 mm measured under an axle load of 62·3 kN. From this model, deflection–life curves have been produced which can be used to predict performance from non-destructive deflection tests.

(*c*) A structural design chart for roads with unstabilized marly limestone roadbases has been derived based on the SN concept.

(*d*) The experiment found that 100 mm of cement-stabilized marly limestone roadbase was found to be equivalent to about 160 mm of unstablized limestone roadbase.

(*e*) River shingle roadbases which were mechanically stabilized with marl dust performed slightly better than unstabilized marly limestones roadbases.

(*f*) For the type of deformation failures observed on most of the sections, the traditional 4·5 power law derived during the AASHO Road Test was found to be inappropriate and a power of unity was found to provide the best explanation of the data.

(*g*) The experiment has demonstrated the necessity for taking proper account of variability in subgrade strength and layer thicknesses within the structural design methods.

(*h*) The thin premix surface has been both flexible and reasonably durable, displaying no fatigue cracking and withstanding high deformations in the other pavement layers without cracking. However, after five to six years many areas of the surfacing became extremely brittle through age hardening of the bitumen and this resulted in cracking which developed extensively during a relatively short period of time and which propagated downward from the brittle surface. This type of cracking is extremely common in the tropics and cannot be prevented by additional thickness or strength in the underlying layers.

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54. The project formed part of the research programme for the Overseas Development Administration but any views expressed are not necessarily those of the Administration, the Department of Transport or the Jamaican Ministry of Construction.

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56. The Paper is published by permission of the Chief Executive of the Laboratory. Crown Copyright.

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