Research into the Design of Flexible Road Pavements

Recherches sur le calcul des revêtements souples

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SUMMARY

This paper considers the basic principles involved in the design of flexible road pavements and reviews the progress which has been made in the application of elastic theory to this problem. International co-operation in this field requires unified methods of evaluating traffic and the development of more widely acceptable criteria for judging pavement performance. In the latter connection the American concept of a present serviceability index is not considered fully satisfactory. British full-scale pavement design experiments and deflection studies are reviewed with particular reference to the major experiment now more than six years old at Alconbury Hill, Huntingdonshire, United Kingdom.

SOMMAIRE

Cette étude considère les principes de base en jeu dans le calcul des revêtements souples et examine les progrès faits dans l'application de la théorie élastique à ce problème. La coopération internationale dans ce domaine exige des méthodes unifiées pour évaluer le trafic et aussi le développement de critères plus largement acceptés pour juger le comportement d'un revêtement. Pour ce dernier, le concept américain du "present serviceability index" n'est pas considéré tout-à-fait satisfaisant. On passe en revue des expériences britanniques sur le calcul des chaussées et des études de déflexion, en tenant compte surtout de l'expérience majeure d'Alconbury Hill dans le Huntingdonshire au Royaume Uni, qui fut commencée il y a plus de six ans.

THE PROBLEM OF PAVEMENT DESIGN

THE METHODS OF PAVEMENT DESIGN at present in use are empirical by nature and stem principally from the prewar period. The method widely used in Europe, in one form or another, is the California bearing ratio (C.B.R.) method developed in the state of California in the 1930's. The proving of any method of design under road traffic conditions is necessarily a lengthy process since long periods of observation are involved. There has been a tendency, therefore, to retain methods like the C.B.R. which are based in some measure on early experience despite the revolution which has meanwhile taken place in relation to roadmaking materials and in the standard of performance demanded from our highways.

The traditional construction for main roads at the beginning of this century consisted of crushed or natural stone laid over the soil foundation. During the second and third decades, a thin bituminous surfacing made with a relatively soft binder was generally added. The function of this surfacing was largely to keep out water, to bind the surface, and to remove the dust problem associated with the increased speed of road vehicles. The main objective in the design of such a pavement was to produce a structure sufficiently thick to prevent any serious deformation of the soil and which could be maintained by regular surface treatment. The design thickness was then largely regarded as a function of the shear strength of the soil and early design methods such as the C.B.R. were therefore based on a measure of the shearing resistance of the soil. The main object was to produce a structure capable of progressive improvement and not one that would satisfy requirements for a specific period. It was not until the later 1920's that riding quality began to receive close attention.

The majority of the new roads now being built in Britain are motorways or major roads of similar standards. They are built in the expectation that they will be virtually maintenance free for a period of at least twenty years and that during this period they will retain a riding quality satisfactory for high-speed traffic. To attain these ends with flexible pavements, use is made of thick bituminous surfacings, bases bound with bitumen, tar or Portland cement, and sub-bases of crushed rock or stabilized gravel. Not only must the soil foundation not deform, but all the component layers of the pavement must be virtually free from deformation under the applications of heavy axle loading which the pavement will bear during its life. The soil must therefore be considered as one component only of a pavement, the performance of each layer of which reacts on that of the others. It is clear that a pavement of this type cannot be designed on the basis of the soil properties alone, particularly where these are assessed on the basis of a failure test such as the C.B.R.

The requirement of minimum pavement deformation suggests the use of elastic theory as the most likely starting point in any rational approach to pavement design. Burmister (1945) developed equations for the stress and deflections in two- and three-layer elastic pavements which were then used by himself, Fox (1948), and Acum and Fox (1951) in the solution of the specific case of pavements of varying elastic moduli thicknesses loaded uniformly over a circular area. The advent of the electronic computer has enabled the range of calculated values to be greatly extended (Jones, 1962; Kirk, 1961), and has made possible the extension of this approach to pavements of four or more layers.

Extension of elastic theory in this manner, or the use of more complicated visco-elastic models will be of limited use until much more information on elastic moduli, Poisson's ratios, and fatigue and deformation properties of pavement materials is available. Detailed studies of these properties are urgently required.

ELASTIC AND DEFORMATION PROPERTIES OF ROAD MATERIALS

The condition of stress and strain under which road materials can be regarded as behaving elastically can only be investigated experimentally either in the laboratory or *in situ*. The most severe conditions of loading directly beneath a loaded wheel give rise to vertical compressive stresses and radial stresses which may be either compressive or tensile depending on the location of the point considered. In materials of high elastic modulus such as cemented bases and dense asphalt surfacings laid on flexible bases, appreciable tensile stresses are likely to be generated at the bottom of the layer (Whiffin and Lister, 1962) and for such materials a laboratory repeated loading flexural test or tensile strain measurement in the field, to establish acceptable stress or strain levels, appears to be most appropriate. For unbound bases and subgrade materials repeated loading triaxial tests or *in-situ* measurement of subgrade stress and deflection are more applicable.

Using a repeated loading triaxial machine based on a cam-spring system which is capable of applying both vertical and all-round stresses (Grainger and Lister, 1962) heavily over-consolidated clays tested under adverse water-table conditions have shown no permanent deformation under repeated deviator stresses of 1 to 4 lb/sq.in. Using piezo-electric gauges, stresses in this range have been measured in clay subgrades of modern "high quality" pavements designed according to British trunk road specification when subjected to the normal legal maximum wheel-load of 10,000 lb. This indicates that subgrade failure in these clays is unlikely to be the first cause of pavement deterioration.

TRAFFIC EVALUATION

The area and intensity of loading associated with the passage of a single wheel-load over a pavement can be approximated to circular loading. Normal road traffic, however, constitutes commercial wheel-loads ranging between 1,000 lb and more than 10,000 lb, each of which imposes a very different stress condition on the pavement. Some method is necessary for expressing mixed traffic of this type in terms of an equivalent number of applications of a selected wheel-load, before the elastic theory can be fully applied to the design of flexible pavements.

The AASHO road test (Liddle, 1962) has provided equivalence factors for one type of pavement structure employing a crushed-stone base and granular sub-base laid on a soil subject to frost action. It was shown, for example, that one application of a 15,000-lb wheel-load was approximately equivalent to 10 applications of a 9,000-lb wheel-load and 50,000 applications of a 1,000-lb wheel-load. It is very unlikely, however, that the same equivalence factors would apply to pavements with bound bases as apply to those with unbound bases and this is a matter which needs further investigation. A road machine employing a track 110 ft in diameter is now being used in Great Britain to compare the deformation and deterioration of five different road structures under repetitions of wheel-load in the range 2,000 to 10,000 lb. The structures employ unbound, cemented, and bitumen-bound bases and they are laid on a sub-base and subgrade, the level of the water table within which can be varied. The sections are fully instrumented so that the stress and transient deflections measured can be examined in relation to elastic theory, and also the long-term performance.

PERFORMANCE CRITERIA FOR PAVEMENTS

Any procedure for designing road pavements implies the adoption of a criterion of performance in terms of which satisfactory behaviour and failure can be specified. The absence of definite criteria of this type has made the correlation of pavement research throughout the world very difficult.

The pavement must fulfil two functions; it must remain structurally sound to satisfy the engineer, and it must provide a satisfactory ride to satisfy the road user. The riding quality of a road that deteriorates in the structural sense will, in general, also deteriorate. However, a structurally sound road can be built with a poor riding quality. It is clear, therefore, that the two factors are not always interdependent. For evaluating the results of the AASHO road test the American engineers introduced the concepts of present serviceability rating (P.S.R.) and present serviceability index (P.S.I.). A panel of road users travelling over selected roads in the United States in vehicles of their own choice assessed the serviceability of those roads on a scale of 0 to 5. The panel were also allowed to inspect the pavements but their final assessment appears to have been mainly a measure of the "rideability" of the pavements. This was defined as the P.S.R. The P.S.I. which was correlated with it was expressed by means of a statistical model as a function of the longitudinal slope variance (SV), rut depth (RD), and the percentage of cracking and patching present ($C + P$). This index was used to describe the condition of the experimental pavements during the road test.

The basis for the statistical model used to relate P.S.R. and P.S.I. is not completely clear, since the roads on which the original survey was carried out apparently exhibited no transverse deformation and the RD term did not appear in the original model (AASHO Report No. 5, 1962). This factor was in fact introduced later when it was found that the AASHO pavements exhibited considerable rutting. Apart from this difficulty, however, it appears very unwise to attempt to use one criterion to cover both riding quality and structural condition since such a criterion is unlikely to be completely satisfactory in describing either structural condition or surface characteristics.

Present British practice is to define structural performance of flexible pavements in terms of the total deformation in the wheel-tracks and the degree of cracking, and to define riding quality separately in terms of an irregularity index. Structural repairs (apart from surface treatment) are normally regarded as necessary when the total deformation in the wheel-tracks reaches 1 in. or when about 50 per cent of the area of the wheel-tracks is subject to cracking of a serious nature. In this connection it is interesting to note that the majority of the pavements in the AASHO road test which deteriorated to the failure level of P.S.I. = 1.5 had a rut depth in the wheel-tracks (measured under a 4-ft straight edge) of 0.65 to 0.85 in. These values would in fact correspond to a total deformation from the original level of about 1 in.

In Great Britain riding quality is measured by means of a 16-wheel profilometer described by Scott (1948). The numerical assessment or q value is a summation of the size and number of longitudinal irregularities of the road surface expressed in inches per mile. All unevenness of less than 0.1 in. amplitude is ignored to eliminate any contribution of coarse surface texture which would not influence riding quality. Satisfactory correlation between q value and riding quality assessed subjectively by a road user panel has been obtained on British roads. The classification is given in Table I, in which roughometer values using a machine based on the American Public Roads Association machine are also included for comparison.

TABLE I. CORRELATION BETWEEN IRREGULARITY INDEX AND RIDING QUALITY

Irregularity index (in./mile)		
Profilometer q	Roughometer r	Description of riding quality
<75	<130	Good or better
75–130	130–180	Fairly good to very fair
130–200	180–240	Indifferent to rather poor
>200	>240	Poor to bad (deserves remedial treatment)

The function slope variance (*SV*) used in assessing the AASHO road test when derived from profilometer records shows reasonable correlation with the q value for the wide range of pavement types included in the Alconbury Hill experiment referred to later in this paper. It would seem reasonable therefore to regard the Chloe type profilometer used in the AASHO experiments and the multi-wheel profilometer as alternative methods of assessing riding quality.

The question of whether a combined serviceability index involving riding quality and structural condition should be more widely adopted is clearly one which requires early consideration.

FULL-SCALE PAVEMENT DESIGN EXPERIMENTS

Experimental pavements built into new highways serve a dual purpose in relation to road research. They furnish direct evidence on which to base standards of design for the particular type of road under investigation, and if adequately instrumented and observed they provide the detailed information required to check theories of pavement design. Twelve experimental roads of this type have been constructed in Britain. These experiments cover about 10 miles of heavily trafficked road. In addition regular observations were made on the performance of a number of non-experimental roads constructed during the past ten years. The results of three early experiments were discussed previously (Croney and Salt, 1961). It is proposed to consider only the information obtained so far from the major experiment at Alconbury Hill, Huntingdonshire, which has been under observation for over 6 years.

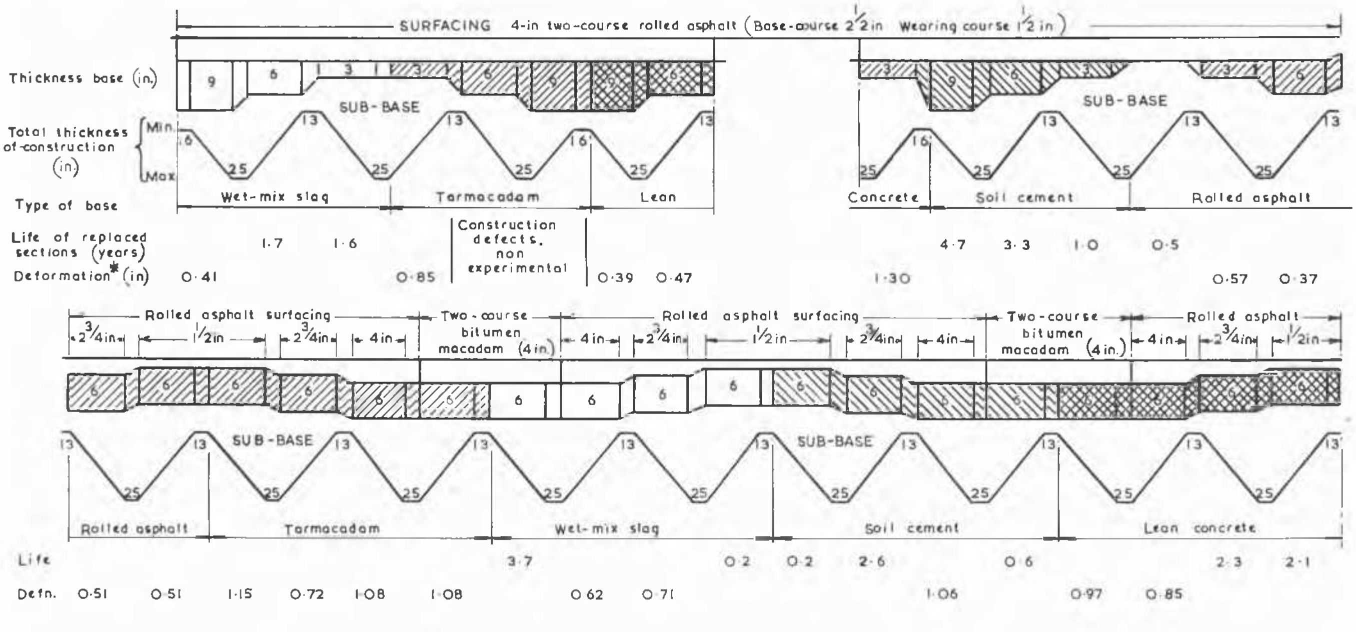
At this site 33 flexible sections were laid on one carriageway of a new dual carriageway road. Details of the types and thicknesses of structure used are given in Fig. 1. The soil along the site was a heavy clay having pockets of a lighter boulder clay, the equilibrium California bearing ratio ranging from 3 to 5 per cent.

A fine sand sub-base (of particle size distribution between No. 200 and No. 72 British Standard sieve) was used and the formation of each section was sloped to give a thickness of sub-base varying along the section in the manner shown in Fig. 1. The sections were either 150 or 200 feet long. The *in-situ* C.B.R. value of the sand sub-base as placed was generally between 20 and 30 per cent, the corresponding laboratory C.B.R. values being about 10 per cent higher. In practice this sand proved difficult to compact; an average dry density of about 98 lb/cu.ft. was obtained at an average moisture content of about 11 per cent.

The wet-mix bases were made with a graded slag of 2 in. maximum size mixed with 6 per cent of water. The material was compacted in 3-in. layers to an average dry density of 135 lb/cu.ft.

The fine sand used for the sub-base was mixed with 8 per cent of Portland cement to provide the soil-cement bases. The average field strength at 7 days was about 140 lb/sq.in. For the lean-concrete bases an aggregate was used of maximum size 1½ in. The mix proportion used was 1:15 with a water content of 5 per cent. The average compressive strength attained was 2750 lb/sq.in. at 28 days.

The tarmacadam bases were of the open-textured type made from crushed-rock aggregate of maximum size 1½ in. The tar content was 2 to 3 per cent by weight and the viscosity 50° e.v.t. The rolled-asphalt bases were laid with base-course material conforming to B.S. 594: 1950, Table I, Schedule 4.



* FOR SECTIONS STILL IN SERVICE DEFORMATION IN THE NEARSIDE WHEELPATH OF THE SLOW TRAFFIC LANE IS QUOTED

FIG. 1. Layout and performance of experimental sections at Alconbury Hill.

The wearing course of the asphalt surfacing complied to

B.S. 594: 1950, Table V, Schedule 1, stone content 35 per cent, 40 to 60 pen. lake asphalt/asphaltic bitumen mixture. The asphalt base-course was of the same material used for the asphalt bases referred to above. The base-course of the two-course open-textured bitumen macadam surfacing complied to B.S. 1621: 1964, Table III, 1½-in. nominal size single-course material; binder 300 pen. bitumen. The wearing course was to the same standard, Table IV, ¾-in. nominal size; binder 300 pen. bitumen.

The performance of the pavements used in the Alconbury Hill experiment has been assessed by: (*a*) regular measurements of the transverse deformation which has occurred under traffic made by the optical levelling of transverse rows of metal studs set into the road surface at 1-ft intervals across the carriageway; (*b*) regular detailed inspections to study the development of cracking and other faults; (*c*) regular measurements with the deflection (Benkelman) beam.

In addition, gauges were installed in certain sections to determine the proportion of the measured total deformation that occurred in the various layers of the structures. Some pressure gauges were also built into the subgrade to measure subgrade stresses. Measurements of the water-table level along the site have shown that it varies between depths of 1 ft (winter) to 3 ft (summer) below the pavement level at the nearside verge. Axle-loads using the pavements are measured on an electronic weighbridge incorporated in the experiment. The traffic at the site is very heavy and includes 4,000 commercial vehicles per day (two-way flow).

On the sections with 2¾- and 4-in. surfacings the criterion used for judging performance has been the deformation in the nearside wheelpath (3 to 4 ft from the nearside verge) where experience shows deformation is always greatest. Failure is adjudged to have occurred when the deformation reaches 1 in. On some of the sections with 1½-in. asphalt surfacings severe cracking and break-up of the surfacing occurred before very serious deformation was apparent and for these sections cracking was taken as the failure criterion. Of the 33 sections 13 failed in the slow lane during the first 6 years' traffic; details of these are given in Fig. 1. The figure also shows the average deformation in the nearside wheelpath of the slow lane for those sections still in service after 6 years. (In general deformation in the fast lane, which carries only about 10 per cent of the commercial vehicles, has been small and replacement of the slow lane only has been necessary, except on one section with a thin surfacing.)

On the basis of the number of sections reconstructed it is clear that the type of material used in the base and surfacing has had a profound influence on performance for the conditions at this site. Six of the seven sections have been replaced with the stabilized sand base, four of the seven sections with the wet-mix slag base, and two of the seven sections with the lean-concrete base. No replacements have been necessary on the sections with rolled asphalt or tarmacadam bases although certain of the latter are now considerably deformed. The best performance has been given by the sections consisting of a 6-in. base of rolled asphalt or a 9-in. base of lean concrete under a 4-in. rolled asphalt surfacing.

According to the method of flexible pavement design at present in use in Great Britain, 24 in. of construction would have been required on the soil at this site assuming an average C.B.R. value of 4 per cent. This was the thickness used at the end of each section where the sub-base was thickest. At the other end of the sections the total thickness of construction was only about half this value. There is no no clear evidence from the results to suggest that increasing the thickness of sub-base improved the performance. Of the 13 sections which failed and were replaced in the slow traffic lane (including the one in which the 4-in. asphalt surfacing was laid directly on the sand sub-base), seven had deformed at the thick end at the time of failure, and six at the thin end. Transient deflection under heavy wheel-loads, as observed by the deflection beam, tended to be greatest at the thin ends particularly for the weaker sections and there was rather clearer evidence of a tendency for cracking to originate first at the thinner ends.

The gauges used to measure the permanent deformation in the various layers of some of the pavements showed that deformation of the subgrade was by no means the major factor in the total deformations measured. Where the pavement was weak, e.g. the 4-in. bitumen macadam surfacing on a 6-in. wet-mix slag base, or the 4-in. rolled asphalt surfacing laid directly on the sand sub-base, about one-half the deformation occurred in the subgrade, but for the other pavements the layers contributed approximately equally to the deformation. In the one instrumented section which has remained in good condition, although underdesigned, the subgrade movement reflected only changes in water-table levels and no deformation due to traffic was recorded, confirming the experimental evidence of the repeated triaxial tests already mentioned.

The behaviour of the pavements is qualitatively in agreement with that expected from the multi-layer elastic theory. For example, in the section with 6-in. wet-mix slag bases the use of a 4-in. asphalt surfacing of high elastic modulus gave rise to lower base, sub-base, and subgrade stresses and deformations than was the case where the lower modulus bitumen macadam surfacing of the same thickness was used. On the other hand, in the sections where the lean concrete bases of very high elastic modulus were employed the type of surfacing had much less influence on the stresses and deformation in the layers below. The excellent performance of the rolled asphalt bases is probably associated with the relatively high elastic modulus combined with the ability of this material to accept comparatively large tensile strains without cracking. Cemented materials while they may have a much higher elastic modulus than rolled asphalt crack at lower tensile strain levels. In this experiment a 3-in. asphalt base has performed satisfactorily without cracking while a thickness of lean concrete of between 6 and 9 in. was necessary under the same conditions to ensure that rapid deterioration due to cracking did not occur. (Measurements with the deflection beam did in fact indicate some slow progressive cracking even with the 9-in. base although cracks through the surfacing have not become apparent.) A thickness of 9 in. of the cement-stabilized sand was insufficient to prevent progressive cracking under the same circumstances.

DEFLECTION STUDIES

In the absence of enough adequate information on elastic and fatigue properties of road materials to permit a truly analytical approach to pavement performance, an empirical evaluation of performance related to the easily measured pavement parameter of road surface deflection is being carried out in the United Kingdom using the Benkelman deflection beam test in which the deflection of the road surface caused by a dual wheel-load of 7,000 lb. is measured using a standardized routine.

In pavements in which the base is the main structural component, the type of base material and the intensity of commercial traffic are the dominant factors influencing

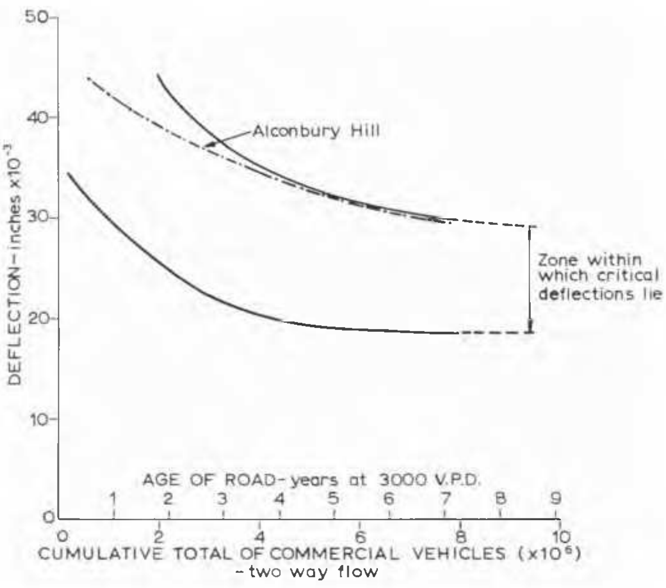


FIG. 2. Relation between critical deflection and cumulative traffic for roads with crushed-stone bases.

the magnitude of deflection criteria. Fig. 2 shows the zone within which lie criterion curves related to the cumulative total of commercial vehicles obtained on a large number of experimental trunk roads incorporating crushed-stone bases. The position of a curve within the zone is governed by subgrade strength type and thickness of surface and base thickness. Results obtained at Alconbury Hill are superimposed. Few failures with bituminous-bound bases have so far been recorded but the available information suggests criteria of the same size.

On roads constructed with cemented bases the size of any criterion depends on whether the pavement is capable of carrying traffic satisfactorily with the base in a cracked condition. Deflection of less than 0.012 in. are associated with the satisfactory performance of such bases for traffic of up to 5×10^{-6} vehicles. On bases which require to be substantially uncracked deflections of less than 0.005 in. are found to correspond to this condition. The measurement of deflections of this size does not necessarily preclude the development of cracking at a later date but does indicate that early failure is unlikely because the onset of major cracking of the base can always be detected as a considerable increase in deflection.

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