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THE STRUCTURAL DESIGN OF BITUMINOUS ROADS

by

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**The views expressed in this Report are not necessarily those
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CONTENTS

	Page
Foreword	
Abstract	1
1. Introduction	1
2. Design criteria	2
3. Design life	3
4. Traffic	4
5. The subgrade	4
5.1 Moisture conditions and foundation drainage	4
5.2 Assessment of subgrade strength and stiffness	5
5.3 Performance criteria	5
6. Capping layer	5
7. Sub-base	6
7.1 Granular sub-bases	6
7.2 Cement bound sub-bases	7
7.3 Assessment of the quality of the road foundation	7
8. Roadbase	8
8.1 Bituminous roadbases	8
8.2 Wet-mix granular roadbases	9
8.3 Lean concrete roadbases	9
9. The use of the design method to modify standard designs	10
9.1 New or improved bituminous roadbases	11
9.2 Design for heavily trafficked roads	12
9.3 Equivalence of bituminous and unbound granular material	12
9.4 Restriction on the application of the design method	12
10. Conclusions	13
11. Acknowledgements	13

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	Page
12. References	14
13. Appendix A: Variability in performance and its effect on design	24
13.1 Variability in pavement performance	24
13.1.1 Materials and construction practice	24
13.1.2 Laid thickness of bituminous materials	24
13.1.3 Sub-grade CBR	24
13.1.4 Traffic	24
13.1.5 Overall variability	25
13.2 Cost analysis	25
13.3 Reference	26
14. Appendix B: Assessment of traffic loading	32
14.1 Information required and available sources	32
14.2 Procedure to estimate cumulative standard axles	32
14.3 Sensitivity of traffic estimates to changes in input data	33
14.4 Reference	34
15. Appendix C: The road foundations	35
15.1 Subgrade	35
15.1.1 Soil strength	35
15.1.2 Soil stiffness	37
15.2 Sub-base	38
15.3 Capping layer	39
15.4 References	40
16. Appendix D: Roadbase design	45
16.1 Bituminous roadbases	45
16.1.1 Dense roadbase macadam	45
16.1.2 Design criteria	46
16.1.3 Rolled asphalt roadbase	46

	Page
16.1.4 Internal resistance to deformation of bituminous materials	47
16.2 Wet-mix roadbase	47
16.3 Lean concrete roadbases	48
16.3.1 Designs for limited life	48
16.3.2 Design for long life	49
16.4 References	50
Appendix E: Input data for the design model and an example of applying the design method	58
17.1 Data required for design models	58
17.2 Adapting the standard designs for bituminous roadbase to permit use of a different grade of bitumen	59
17.3 Reference	60

THE STRUCTURAL DESIGN OF BITUMINOUS ROADS

FOREWORD

For the last twenty-five years successive editions of Road Note 29 have provided the design background for the construction of new roads. Since the last Edition was published in 1970 there have been considerable further increases in the damaging power of commercial vehicles using the major road network and the most heavily trafficked roads are now designed for cumulative traffic of up to ten times that envisaged when the first motorways were opened. The consequential increase in need to repair older parts of the network has led to a major increase in reconstruction work, which presents distinctive design problems, particularly as there is also now interest in designing for longer lives in order to reduce future maintenance and the associated traffic delays. New and improved materials are being developed to satisfy these increasingly severe service conditions.

This document is not a simple replacement of Road Note 29. It is a research report that summarizes extensive research, stretching back over a thirty-five year period. As in Road Note 29, design guidance is based on the measured performance of the full-scale road experiments; these have now yielded a much greater quantity of data than at the last review in the late sixties, when several major experiments had only recently been built. There have also been considerable advances in interpreting the behaviour of road pavements as engineering structures. The results of research on mathematical modelling of pavement behaviour and the simulative testing of road materials and pavements have been used to re-analyse the behaviour of the full-scale experiments. Work has been carried out at the Laboratory and at other centres, both in this country and abroad. Standard designs developed for pavements with different types of roadbase constitute the reference structures to which the design methodology can be applied in order to provide the basis for assessing improved materials and other designs.

Design life is defined in terms of the cumulative traffic that can be carried before strengthening of the pavement by the application of a bituminous overlay is necessary. The road foundation is designed specifically to carry construction traffic, and the methodology provides the flexibility required for the design of reconstruction projects. The considerable uncertainties in relation to the estimation of traffic and sub-grade strength and the effect of construction conditions and material variability are taken into account in the design of the road-base.

The methodology provides the means of introducing innovations in design and materials. However, because of the limitations in theoretical models of pavement behaviour major changes require simulative testing in a pavement test facility before they can be considered for general use on the road.



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THE STRUCTURAL DESIGN OF BITUMINOUS ROADS

ABSTRACT

Since the 3rd edition of Road Note 29 was published in 1970, further information on road performance has been obtained and there have been important advances in research on materials, methods of construction and mathematical models of pavement behaviour. These advances have been used to develop a new method for the structural design of road pavements. It is based on the performance of experimental roads interpreted in the light of structural theory. The capping layer and sub-base is designed primarily as a construction platform. Design curves are given for the thickness of bituminous, lean concrete or wet-mix roadbase required to carry the traffic expected to use the road during its design life. The design method offers the means of adapting proven designs to take advantage of new materials, new design configurations and construction methods.

1. INTRODUCTION

In the third edition of Road Note 29 the Road Research Laboratory (1970) based structural design standards for bituminous roads on observation of the performance of a number of experimental roads built into the public road network. Design curves were developed to relate the overall pavement thickness and the thickness of each layer to the traffic to be carried and to the strength of the subgrade. These design curves formed the basis of the design standards issued by the Department of Transport (1976). Although developed from observations on pavements that had carried less than 10 million standard axles (msa), the designs have proved generally satisfactory in relation to cumulative traffic of up to about 40 msa, the maximum carried by lengths of the motorway network broadly in their original condition. However to accommodate the recent rapid growth in the number and damaging power of heavy goods vehicles, the busiest motorways now need to be designed to carry about 150 msa over 20 years. The empirical method of design used in developing Road Note 29 does not provide a satisfactory basis upon which to produce designs for traffic levels that are so far in excess of those observed on the experimental roads. Also the method does not facilitate a rapid design response to the availability of new or improved materials, to changes in methods of construction or to the need to modify designs when reconstructing roads.

Since the third edition of Road Note 29 was published in 1970 the experimental roads have carried more traffic, some over 20 msa, and a major programme to measure their structural properties has been completed. Moreover a theoretical interpretation of the much greater body of performance data now available from a greater number of experimental roads allows this valuable databank to be used to support an analytical approach to design and reconstruction of road pavements with full allowance for variability in material properties, strength of the subgrade and traffic.

Pavement design begins with a study of the response of the structure to transient loads generated by traffic and much work has been done, for example by Bleyenbergh, Claessen, Van Gorkum, Heukelom and Pronk (1977) and by Thrower, Lister and Potter (1972). A recent review by Monismith and Witczak (1983) concluded that for many practical conditions the dynamic stresses and strains generated within some types of pavement by traffic loading can be calculated with reasonable accuracy using a multi-layer linear elastic model. The stress history of each layer can be calculated for a given spectrum of wheel loads and total number of vehicles, taking into account the variation in stiffness of bituminous materials with changing temperature. These calculations are a pre-requisite for the prediction of long-term performance of the road pavement in terms of cracking and deformation.

Models of long term performance based on laboratory testing are less well developed. Laboratory tests that seek to simulate conditions in the road pavement are gross simplifications and although they yield results that are generally suitable for characterising the dynamic stress and strain behaviour, prediction of long-term performance from them is much less certain. Simplifying assumptions have also to be made in developing these models; for example, the materials of the pavement are treated as homogeneous, isotropic layers of idealised material. Consequently the many complex models that have been developed stretch structural theory in seeking to characterise the inhomogeneous particulate materials used in road construction. Performance testing of full-scale pavements, ideally under controlled conditions of wheel load and temperature, therefore plays an essential part in the development and calibration of the structural models and in the independent verification of new designs that are very different from present proven practice.

The approach adopted in the present work was to develop a standard set of designs based on the structural performance of the experimental roads. These standard designs were then interpreted in terms of theoretical design concepts leading to a design method that takes full advantage of advances in theoretical methods of analysis, whilst making due allowance for their limitations. The use of theoretical design concepts also demonstrates clearly the essential structural function of the pavement and should encourage pavement design and construction to proceed in a better informed environment.

2. DESIGN CRITERIA

To give satisfactory service a pavement must satisfy a number of structural criteria; some of these are illustrated in Figure 1. The more important design criteria are:

- (a) the subgrade must be able to sustain traffic loading without excessive deformation; this is controlled by the vertical compressive stress or strain at formation level,
- (b) bituminous materials and cement-bound materials used in roadbase designs for long life must not crack under the influence of traffic; this is controlled by the horizontal tensile stress or strain at the bottom of the roadbase,
- (c) in pavements containing a considerable thickness of bituminous materials the internal deformation of these materials must be limited; their deformation is a function of their creep characteristics,
- (d) the load spreading ability of granular sub-bases and capping layers must be adequate to provide a satisfactory construction platform.

In practice other factors have to be considered such as the effects of drainage and the ability of roads designed for light traffic to withstand occasional overloading.

All these criteria will be discussed in more detail later but consideration must first be given to the basic inputs for road design: the design life to be specified, the traffic to be carried and the strength of the subgrade on which the road is to be built.

3. DESIGN LIFE

Road Note 29 recommends a design life of 20 years for bituminous roads, because their life may be extended by a strengthening overlay: for concrete roads it suggests 40 years, because of the problems involved in extending the life and the small increase in thickness needed to secure the longer initial life. In Road Note 29 the end of the design life was associated with a surface rut of 20 mm or more or severe cracking and crazing and the pavement was considered to be in a failed state and in need of a major strengthening overlay or partial reconstruction. However it has subsequently been shown that strengthening a pavement in this damaged condition does not necessarily result in satisfactory subsequent performance.

Lister (1972) examined in more detail the expected changes in the surface state of a road that differentiate between sound and critical conditions in the pavement structure. Based on extensive comparisons between surface condition and structural integrity he defined the onset of critical structural conditions as rutting in the wheel paths of 10 mm or the beginning of cracking in the wheel paths. He concluded that these surface indications were normally the precursor of significant structural deterioration and marked the latest time when the application of an overlay could be expected to make best use of the original structural quality of the pavement in extending its life. In analysing the performance of the experimental roads, design life has therefore been determined at a surface condition when structural rutting in the wheel path exceeds 10 mm, or when cracking is observed; the onset of these critical conditions has also been estimated from deflection measurements, which would be the measure preferred in practice.

Pre-emptive overlaying at the onset of critical conditions is considered to be generally preferable to allowing the road to deteriorate further until extensive reconstruction becomes necessary; because it makes best use of the strength of the existing pavement it results in a pavement of more uniform strength, and because it is more readily anticipated by in-situ deflection measurement it can be more readily planned. Beyond the critical condition the nature and rate of deterioration becomes much less predictable but if overlaying is ruled out as being either impractical or uneconomic then the pavement can, with suitable monitoring to warn of impending failure, be allowed to deteriorate further prior to major reconstruction. The concept of design life adopted thus implies a further period of subsequent serviceable life that can be of considerable but uncertain duration.

From a detailed calculation of the cost of bituminous roads over forty years, Abell (1983) concluded that the initial design life to failure might, with advantage, be somewhat longer than the 20 years recommended in Road Note 29. A more recent analysis, summarised in Appendix A, extends Abell's work by taking into account variability of pavement performance, cost of traffic delays and other costs associated with reconstruction. As may be seen from Figure 2, the results suggest that for pavements with a bituminous roadbase the optimum design life that minimises the costs discounted over 40 years is close to 20 years, with 85 per cent probability that roads will survive that period without requiring a strengthening overlay to extend their lives. It should be noted that this design will give a substantially longer life than a Road Note 29 design for 20 years because it is based on the structural deterioration associated with a 10 mm rut in the wheel path rather than a 20 mm rut. With deflection as a guide, the life can be extended by the application of a strengthening overlay towards the end of the design life unless of course there are practical reasons that prevent the overlay approach.

The total cost of a road over 40 years is not very sensitive to a comparatively large increase in design life because of the relatively low initial cost of providing extra life and the discounting of future costs of reconstruction. On the other hand the economic consequences of specifying too short a life can be considerable as is shown in Figure 2. Nevertheless a shorter life may be specified if the designer believes that the road is at risk from factors that are not taken fully into account in the method of design; for example in an urban area frequent

openings by the public authorities and in a rural area road re-alignment or widening. An appreciation of costs and benefits tempered by engineering judgement should suffice to set design life at an appropriate level and ensure a uniform approach to design. In practice control over the quality of material used and the construction processes is also an important determinant of pavement performance, and hence value for money.

For roads with a lean concrete or wet-mix granular roadbase, the analysis is more complex but a twenty year design life would seem to be appropriate.

4. TRAFFIC

In Road Note 29, traffic is defined in terms of the cumulative number of equivalent standard 80 kN axles to be carried during the design life of the road; the same approach was used in the present work. Usually the designer has to estimate the number of standard axles from the number of commercial vehicles over the projected life of the road. The number of standard axles attributable to the average commercial vehicle is referred to as the vehicle damage factor and since 1973 axle surveys have indicated a considerable increase in this factor, particularly on motorways and some other heavily trafficked roads. This led Currer and O'Connor (1979) to propose revised vehicle damage factors for the design of new roads. This work was later extended by Addis and Robinson (1983) into the design method described in Appendix B, where the recommended processes for use in the estimation of cumulative standard axles for pavement design are described. Sources of data that may be helpful are listed and the sensitivity of the final estimate to varying input data is discussed.

5. THE SUBGRADE

The road pavement must reduce the stresses on the subgrade due to traffic loads to a level that ensures that there is only very limited deformation at the end of the design life. The magnitude of stresses in the subgrade at formation level is strongly influenced by the stiffness of the subgrade and the deformation caused by these stresses is related to the strength of the subgrade. Thus both stiffness and strength of the subgrade play a part in determining its performance; moreover subgrade stiffness affects the level of stresses generated in all the overlying pavement layers.

5.1 Moisture conditions and foundation drainage

Both stiffness and strength of the subgrade are strongly influenced by moisture conditions. In the UK there is little annual variation in these conditions. Current practice provides french drains at a depth of 0.6m in order to ensure as far as possible that the water table is maintained at least 0.3m below the level of the formation under the road. Apart from its effect on the strength of the subgrade, inadequate drainage can have very adverse effects on the stability of granular foundation layers subjected to prolonged saturation and thus negate the normal basis of design. Although the road experiments on which design is based did not include a systematic study of the effect of poor drainage on pavement performance the consequences were graphically demonstrated on several sections that failed quickly.

Black and Lister (1979) have shown that subgrades that become very wet during construction subsequently reach lower equilibrium strengths than those that are kept dry and well drained. The ability of the capping layer and sub-base to carry site traffic is also affected by wetting during construction. For the purposes of design it is assumed that the pavement and foundation are adequately drained with values of subgrade stiffness and strength corresponding to moisture conditions to be expected in the subgrade of the in-service pavement and the design

for a road expected to be built in wet conditions might therefore differ from one to be built in drier weather; this is discussed in Appendix C.

5.2 Assessment of subgrade strength and stiffness

The relationship between the various physical properties of a soil that control its stiffness and strength are complex and imperfectly understood. The California Bearing Ratio (CBR) test, although giving unrealistically low results on wet cohesive soils near saturation, is a generally accepted and practical measure that can be used to give an estimate of both stiffness and strength. For the design of new roads it can be applied to laboratory samples compacted at the appropriate field moisture content. It can also be used to test material in-situ at depths likely to be unaffected by seasonal variations of moisture when this strength condition is considered appropriate for design. Subgrade strength also needs to be measured when existing roads are being reconstructed. This can be done by in-situ CBR testing in pits or by making boreholes and using the cone penetrometer described by Black (1979). In either method care has to be taken to establish the relevance of the measurement to the proposed design for reconstruction and its method of execution.

Alternatively, the equilibrium subgrade strength for the cohesive soils that are most common in this country can be estimated with reasonable accuracy using a soil suction method described by Black and Lister (1979). Typical values of CBR for British soils that are representative of different construction and drainage conditions are given in Appendix C; values are quoted for conditions at the time of construction and also equilibrium values that correspond to subgrade conditions that should exist for the greater part of the in-service lives of the pavements.

When the value of CBR has been estimated it may be converted to subgrade stiffness or modulus as described in Appendix C.

5.3 Performance criteria

The upper limit of permissible vertical stress or strain in subgrades that are representative of those found in the UK has been established by examination of the behaviour of roads that failed early in life because of over-stressing of the subgrade. In properly designed roads the critical stress or strain is not exceeded and premature failure is not a problem; nevertheless gradual deformation of the formation under repeated traffic loading can contribute to the total deformation of the pavement and criteria in terms of the maximum permissible values of vertical stress and strain in the subgrade have therefore been developed. The level of these parameters generated by a standard axle load at a standard temperature has been estimated by analysing the elastic behaviour of 34 experimental road pavements with dense bitumen macadam roadbases; the computed values were then related to the traffic carried up to the onset of critical conditions to define the criteria. Details are described in Section 8.1 and Appendix D.

6. CAPPING LAYER

When the CBR of the subgrade is less than 5 per cent, the Department of Transport (1978) normally requires the use of a suitable capping layer of low cost, local material. This capping layer is designed to provide a working platform on which sub-base construction can proceed with minimum interruption from wet weather, and to minimise the effect of a weak subgrade on road performance. In addition to providing a firm platform for compaction of sub-base, the capping layer reduces the risk of damage during construction operations to any

cement-bound materials above the capping layer and can thus improve the structural contribution of these layers. Black and Lister (1979) observed that extra surcharge such as that of a capping layer also lessens any reduction in the strength of the subgrade during wet weather and any consequential adverse effect on its final equilibrium strength.

A wide range of materials has been used as capping layer but current knowledge of their structural properties is such that a theoretical analysis can be no more than indicative of their contribution to the load spreading ability of the working platform. However vertical stress or strain in the subgrade under construction traffic running on top of a combined capping layer and sub-base can be calculated to give a broad appreciation of the load spreading ability of this working platform, as described in Appendix C. The results suggest that current design thicknesses of capping layer are conservative. Because of the difficulty of estimating soil stresses and strains accurately and because of the uncertainty about actual soil strengths particularly during winter construction, this conservatism may be justified. Also the behaviour of the relatively diverse capping layer materials is not understood well enough to develop criteria for yield within these materials. On the other hand if capping layer is recognised as primarily a construction platform, the calculations indicate that there appears to be no justification for increasing capping layer thickness for roads designed to carry more traffic.

In Appendix C it is shown that if 150 mm of sub-base is laid on 350 mm of capping layer with a CBR of more than 15 per cent, the load spreading ability of the combined layers will meet the requirement for a satisfactory construction platform over subgrades with values of CBR of 2, 3 or 4 per cent. An additional thickness of capping layer of 250 mm will be required if the subgrade CBR is less than 2 per cent. The design of roadbase and surfacing is then the same as that for pavements constructed on soil of CBR 5 per cent or more and a sub-base of 225 mm.

Other methods of improving the road foundation are worthy of consideration, particularly cement or lime stabilisation. In all cases the design requirement is for an adequate working platform upon which to construct the sub-base and subsequent pavement layers, and a finished pavement equivalent in structural terms to pavements constructed on subgrade of CBR 5 per cent and a 225 mm sub-base.

7. SUB-BASE

The sub-base is a structurally significant layer, generally of granular material, that provides a working platform on which materials can be transported, laid and compacted; it also acts as a level regulating course and insulates the sub-grade against the action of weather. Any material within 450 mm of the road surface, including the sub-base itself, must be frost-resistant. Other types of sub-base have been claimed to perform these functions satisfactorily in other countries, including some with drainage properties as reported by Cedergren (1974). Open-graded cement and bituminous bound materials can provide a good working platform and act as a drainage layer that is particularly effective during construction, although their long term structural contribution is uncertain.

7.1 Granular sub-bases

The structural contribution of the sub-base is taken into account in the design of roadbase in Section 8 but its initial function is to serve as a working platform that protects the subgrade from overstress under construction traffic and limits the rate at which this traffic deforms the surface of the sub-base. The thickness requirements for granular sub-base were established in terms of the subgrade strength at the time of construction by two different methods.

The results of an extensive full-scale loading programme carried out by the US Corps of Engineers were adapted to suit a good quality Type 1 granular sub-base and a different failure criterion. The alternative approach was to relate values of vertical strain calculated by elastic analysis to measurements from roads that failed early due to overstressing of the subgrade. Details are given in Appendix C.

Undue weight should not be given to these analyses because, for example, the behaviour of the sub-base and subgrade materials is known to be inadequately represented by linear elastic theory. However, the close agreement of the two analyses based on very different assumptions adds confidence that both give a broad indication of the adequacy of a sub-base as a working platform.

The present specification places a practical upper limit of 225 mm on the thickness of a single sub-base layer. The analyses demonstrate that this thickness of well graded granular material should be satisfactory providing the length of roadbase and surfacing under construction with materials carried over the sub-base is less than about 1 km in length, a reasonable assumption in practice, and the soil CBR is not less than 5 per cent. Below this level of CBR a capping layer is normally specified. Otherwise an additional thickness of sub-base is needed and it would have to be compacted in two layers. The lower layer should be as thick as possible in order to avoid the danger of damaging the sub-base or subgrade during compaction. The thickness of the sub-base should be based on the likely CBR value at the time of construction; recommended values are indicated in Appendix C. The resulting thickness may then be taken into account in the final design at the estimated equilibrium design value.

7.2 Cement bound sub-bases

If good quality sub-base materials are not readily available, cement-bound sub-bases may be preferred, particularly for construction in wet weather.

The stresses generated by construction traffic and temperature in a cemented sub-base will generally cause it to crack; the extent of the cracking will be influenced by the amount of construction traffic, the strength and thickness of the sub-base and the stiffness of the subgrade and any capping layer. General cracking of the sub-base into small segments occurs when these factors are combined adversely.

The degree of subsequent deterioration of cement-bound sub-bases, and hence their contribution to structural performance under the less severe stress regime of the completed pavement is uncertain. Stronger materials will crack and abrade less than weaker ones but in general the structural contribution of a cement-bound sub-base cannot be guaranteed to be greater than that of a Type 1 granular sub-base and a conservative design assumption would be that its design thickness should be the same.

Stronger cement-bound materials constructed to the full recommended 225 mm thickness on a firm sub-grade or capping layer will exhibit their usual pattern of regular transverse cracks, but apart from this they should be only micro-cracked. Superior performance of the cement-bound sub-base would be a bonus in terms of extended life.

7.3 Assessment of the quality of the road foundation

Uncertainties about the structural properties of the subgrade, capping layers and sub-base make it desirable for the engineer to evaluate the road foundation up to sub-base level before constructing the main roadbase and surfacing layers. Such a direct assessment would give assurance as to structural adequacy and indicate any need to

improve the support provided by the road foundation. Determination of the stiffness of the foundation, measured on its surface, is not by itself sufficient; it would reflect only the strength condition of the subgrade at the time of construction and also of the possibly temporary weakness of capping layer and granular sub-base (if they were in a wet condition). Poor compaction of granular layers would also not necessarily be identified. To remedy these deficiencies stiffness testing needs to be accompanied by a rapid assessment of in-situ subgrade strength, for comparison with the expected equilibrium values, and by a separate check on the density of granular layers. This approach could be developed into a practical procedure for monitoring construction, but establishing it as a method of ensuring compliance with a specification is unlikely to be practicable.

8. ROADBASE

The performance of 144 sections of experimental road has been used to provide a basis from which to produce a standard set of designs that may be explained in terms of the theoretical design criteria described in Section 2. The production of the designs is described briefly below, and in more detail in Appendix D.

8.1 Bituminous roadbases

There are two main modes of deterioration associated with this type of roadbase, a gradual build up of deformation observable at the surface in the wheel paths and development of cracks in the bituminous material. The long term performance of 34 sections of experimental road with well compacted dense roadbase macadam manufactured to British Standard BS 4987 (1973a) and containing 100 pen bitumen, was monitored in terms of deflection measured under a rolling wheel load moving at creep speed using a deflection beam, and rut depth. The design curve thus developed is shown in Figure 3; it relates the thickness of roadbase and surfacing to the life at which a strengthening overlay will be needed if the life is to be extended. The results show a very considerable amount of scatter for a subgrade CBR of 5 per cent and a sub-base thickness of 225 mm. The design curve corresponds to target thicknesses specified; the variability in thickness of pavement layers in the experimental roads is considered to be typical of construction practice in the UK. It is drawn so that 85 per cent of the roads will achieve the design life, this value being chosen because it provides a typical basis upon which maintenance interventions are judged.

The effect of subgrade CBR on design thickness was derived from a statistical analysis of the performance data. With a design CBR of less than 5 per cent a capping layer would normally be required beneath the sub-base; the roadbase would then be designed directly from Figure 3. If a capping layer is not used then the sub-base would need to be thicker to carry the construction traffic (as described in Appendix C) and the design thickness of roadbase modified where necessary and when it is practical to do so. On strong soils the effect of subgrade CBR on design thickness is small but some adjustment is possible as indicated in Appendix D.

The design curve in Figure 3 was produced using the results of laboratory fatigue tests as an aid to interpreting the results from sections of experimental road. It is widely known that laboratory fatigue tests grossly underestimate performance in the road and that large conversion factors have to be used in purely theoretical design methods. However Lister, Powell and Goddard (1982) deduced from repeated loading tests on a full-scale pavement in a circular road machine and from field evidence that dense coated macadam pavements would reach the design life given by Figure 3 without appreciable cracking. The design curve has therefore been used with some confidence to generate conservative design criteria for the maximum permissible vertical strain in the subgrade and the maximum permissible horizontal strain at the bottom of the macadam roadbase. Figure 4 gives the relationships between permissible strain and design life, calculated with computer programs based on those

originally devised by Thrower (1968 and 1979). These relationships provide a basis upon which to introduce improved and new materials or to design for heavier traffic, as described in Section 9.

The performance of 29 sections of experimental road with rolled asphalt roadbases was investigated in the same manner as described for dense roadbase macadam. The composition of the rolled asphalts covered a wide range of aggregates and gradings, but all complying with British Standard BS 594 (1973b). The performance of these roads with rolled asphalt roadbase was indistinguishable from others with dense bitumen macadam roadbase. Figure 3 may therefore be taken as the design curve for both. This does not mean necessarily that both types of pavement deteriorate in the same manner, as described in Appendix D.

For lightly trafficked roads a minimum thickness of bituminous layers of 110 mm is recommended. This thickness is close to the minimum that can be laid and is sufficient to avoid excessive subgrade stress under the occasional very heavy wheel load on hot days; a rapid failure could otherwise occur.

As the design thickness increases, deformation within the bituminous layers becomes increasingly important for design lives in excess of, say, 80 msa, a method of analysis based on a laboratory creep test can be used to check that these layers have adequate resistance to deformation, as outlined in Appendix D.

8.2 Wet-mix granular roadbases

Results from experimental roads provide a picture of the behaviour of pavements with wet-mix roadbase and rolled asphalt surfacing. The performance in terms of deflection and rut depth of 42 sections of experimental road, that included a range of crushed rock aggregates and graded slag was used to develop the standard design in Figure 5 for a CBR of 5 per cent and a sub-base thickness of 225 mm. Minimum layer thicknesses at low traffic levels are dictated by practical considerations.

For a design CBR of less than 5 per cent a capping layer would normally be required beneath the sub-base. If a capping layer is not used then the sub-base would need to be thicker to carry the construction traffic (as described in Appendix C) and the design thickness of roadbase modified, as indicated in Appendix D, where it is necessary and practical to do so. On strong soils the effect of CBR is small: some adjustments are possible.

Analysis of the behaviour of wet-mix roadbases in mechanistic terms presents considerable problems. There is uncertainty as to the effective elastic modulus, which has been shown to be highly stress dependent by many workers including Brown and Pappin (1981) and Barksdale (1972). Also little is known about the characterisation of failure in the material. However there is good agreement between observed and predicted behaviour if the performance criteria developed from the study of the experimental pavements with bituminous roadbases are applied to those with wet-mix roadbase, providing that a modulus of 0.5 GPa is assigned to the wet-mix roadbase. Based on this analysis the expected mechanism of failure in pavements designed to carry less than 20 msa is by deformation. Above 20 msa the designs may be inadequate in relation to fatigue cracking if the bitumen content of the basecourse is less than specified in British Standard BS 594 (1973b) for rolled asphalt.

8.3 Lean concrete roadbases

This type of roadbase is widely used but analysis of its structural behaviour under the stresses generated by traffic and vertical temperature gradients is far from straightforward. Transverse shrinkage cracks occur very early in the life of all lean concrete pavements, typically at about 4m intervals. Thereafter design for heavily trafficked roads should ensure that the roadbase is not further cracked by the combined effects of traffic and temperature.

Otherwise the expensive and time-consuming operation of replacing the roadbase will usually be necessary when structural strengthening is carried out. For less heavily trafficked roads, where the effects of traffic delays associated with reconstruction work may be less serious, it may be more economic to allow a thinner roadbase to crack gradually and break up under traffic. This leads to the concept of one design for a limited life and a second design for a long but indeterminate life during which no significant deterioration of the lean concrete roadbase is expected.

For roads designed for limited life, in which the roadbase gradually cracks and deteriorates under the influence of traffic and temperature, the performance of 39 sections of experimental road has provided the basis for design. These pavements included lean concrete roadbase containing crushed rock or gravel aggregate with 28-day compressive strengths ranging from 4 to 20 MPa. Surfacing consisted of dense basecourse macadam and/or rolled asphalt. The design thickness of surfacing has been established primarily from the observation of cracking whereas that of the lean concrete roadbase was derived from deflection measurements supported by observation of the onset of critical conditions. The resulting relationship between the thickness of lean concrete and life is shown in Figure 6, with the thickness of lean concrete matched to a sub-base thickness of 225 mm and a subgrade CBR of 5 per cent. With a CBR of less than 5 per cent a capping layer would normally be required beneath the sub-base. If a capping layer is not used then the sub-base would need to be increased according to Appendix C to carry the construction traffic and the design thickness modified where it is necessary and practical to do so. Details are given in Appendix D.

For lightly trafficked roads the minimum thickness of both surfacing and lean concrete base is determined by engineering tolerances and the need to avoid severe cracking of the roadbase under construction traffic. For traffic in excess of 20 msa this design approach requires a thickness of roadbase greater than can be laid in a single lift.

For heavily trafficked roads the objective has been to design a lean concrete roadbase that would not become generally cracked under the combined effects of traffic and temperature. When sections of experimental road were examined by coring and visual observation after carrying traffic equivalent to between 8 and 20 msa, it was found that some had longitudinal cracks in the wheel paths while others had not. In most of those that had longitudinal cracks the combined stresses due to traffic and temperature had exceeded the mean 28-day flexural strength. Flexural strength at 28 days has thus been taken as an indicator of the ability of lean concrete roadbases to resist forces to which they are subject in the road. Calculation of stresses within the lean concrete indicates that, with material properties and traffic conditions similar to those encountered in the experimental roads, 250 mm of lean concrete covered by 200 mm of surfacing is required to resist cracking and these thicknesses have been adopted for designs for heavy traffic in Figure 6; these relate to a design CBR of 5 per cent on a standard thickness of sub-base of 225 mm. Again, adjustments can be made for different foundation strengths as described in Appendix D. There is little evidence to prove that this thickness of lean concrete can be compacted satisfactorily in a single layer and the performance of two layer construction has been shown to be more variable. Some possible solutions to this problem are discussed in Appendix D which also describes the development of the designs in more detail.

9. THE USE OF THE DESIGN METHOD TO MODIFY STANDARD DESIGNS

Figures 3, 5 and 6 provide a standard set of designs that meet the requirements of most current situations for new roads. They may be reduced easily to the form of catalogue favoured by most highway authorities, where designs are presented for four or five traffic categories. This simplifies the design procedure considerably and is a sensible

response to inherent variability in material properties and construction methods and acknowledges that pavement design is not a precise process. Appendix A demonstrates that the consequent additional variability in design life resulting from this simplification has no significant effect on whole life costs.

There is an increasing need to adapt the standard designs already described for use in new situations, a need that has been highlighted frequently during the reconstruction of heavily trafficked roads. In these situations, the available construction depth is usually restricted by clearances under bridges, and by the need to maintain drainage levels. As a result it is often impossible to accommodate the greater thicknesses of sub-base and bituminous materials required to carry the much greater volume of traffic for which reconstructed pavements must normally be designed. Either a bituminous roadbase material of superior performance must be used or new designs are required. The analytical design method described in Section 8 for bituminous roadbases can be used to adapt standard designs to take advantage of improved materials and new developments in design and construction methods.

The application of similar design techniques to wet-mix and lean concrete roadbases is, in principle, perfectly feasible but in practice less certain and would require an awareness of the limitations of the approach and comprehensive laboratory and full-scale tests to verify the performance of any new design.

The following examples illustrate the application of the design methods and conclude in Section 9.4 with some cautionary remarks.

9.1 New or improved bituminous roadbases

If a pavement with a bituminous roadbase is constructed from materials whose properties can be shown to differ from those of the standard materials given in Appendix E then it is possible to use the design criteria presented in Figure 4, the horizontal strain at the bottom of the roadbase and the vertical strain at the top of the subgrade, to determine the traffic the pavement can carry before it is expected to reach the end of its design life, because of the onset of fatigue cracking or excessive deformation. The shorter life derived from the application of these two criteria determines the design life.

The critical strains in the pavement are strongly influenced by the modulus of the roadbase. The sensitivity of the design thickness of bound layers to changes in modulus of the bituminous layers is given in Figure 7. The fatigue strain criterion of Figure 4a assumes that the fatigue characteristics of the improved material are the same as those of the standard material, ie the only difference is the change in strain level brought about by a change of modulus. Any difference in the laboratory fatigue characteristics, eg because of a change in binder content, would have to be allowed for in a modified fatigue criterion. An example of this approach was the determination of the benefit of improved compaction of bituminous materials reported by Powell, Lister and Leech (1981) who showed that the density of coated macadam roadbase and basecourse could be increased by at least 3 per cent by improving compaction specifications. The consequent increase in stiffness might be used to justify a reduction in thickness of the roadbase of about 9 per cent; alternatively the benefit could be taken as a longer life. A further example was the development of an improved dense roadbase macadam containing a harder binder and higher filler content reported by Leech (1982) and described further in Appendix E.

These examples refer to improvements in well proven materials. For any new material laboratory testing will be required to determine its resistance to deformation and the necessary laboratory fatigue testing will require calibration against fatigue performance in the pavement by testing under controlled conditions in an accelerated life testing facility.

9.2 Design for heavily trafficked roads

Traffic delays are particularly undesirable on heavily trafficked roads. In designs for these roads it is therefore desirable to avoid cracking in the roadbase, which would necessitate the replacement of both roadbase and surfacing. Goddard (1982) and Lister, Powell and Goddard (1982) described a design, already implemented that avoids these problems by using a standard thickness of hot-rolled asphalt lower roadbase under a thickness of well-compacted dense bitumen macadam of proven high resistance to internal deformation. The laboratory fatigue life of rolled asphalt at a given level of tensile strain is about twice that of dense bitumen macadam because of its higher binder content, and the fatigue criterion of Figure 4a has therefore been adjusted as described in Appendix D. This adjusted criterion was then used to derive a design with adequate resistance to fatigue cracking (expressed in terms of the thickness of upper roadbase). Application of the load spreading criterion based on permissible strain in the subgrade leads to a thicker design curve as shown in Figure 8. Selection of the thicker of these curves for design purposes should ensure that the structure deteriorates primarily through deformation, with little risk of fatigue cracking in the roadbase. The resulting design curve for the total thickness of bituminous material is the solid line in Figure 8; the curve is seen to be a continuation of the standard curve for dense bitumen macadam roadbases given in Figure 3. The main difference between the two designs is the greater assurance against fatigue cracking of the composite design for heavy traffic.

Although Section 8 explains that a completely satisfactory design method for pavements with a lean concrete roadbase is not currently available for heavily trafficked roads, Appendix D describes some possible solutions.

9.3 Equivalence of bituminous and unbound granular material

With experience of local materials and site conditions the engineer may wish to select a granular sub-base whose thickness is other than the 225 mm recommended in Section 7.1. For example, in reconstruction work the existing sub-base may be thinner than 225 mm but known to be adequate as a construction platform. An adjustment to the thickness of roadbase given in the standard designs is then necessary.

Multi-layer elastic analysis has been used to establish thicknesses of sub-base and bituminous roadbase that give equivalent performance, when bituminous roadbase is substituted for granular sub-base. From such an analysis it appears that for a given design life, 10 mm of dense roadbase macadam is equivalent to about 30 mm of Type 1 sub-base material. The equivalence varies little with thickness and is in good agreement with the empirical data collected from full-scale experiments.

A similar equivalence between roadbase macadam and wet mix suggests that 10 mm of dense roadbase macadam is equivalent to about 20 mm of wet mix. However the analysis for wet mix roadbase is less certain and greater caution is required in adopting this equivalence.

9.4 Restriction on the application of the design method

The procedure for adapting standard designs to new situations has been developed furthest for roads with bituminous roadbases; design criteria for lean concrete and wet-mix are less certain so that there is more risk in changing the standard designs, which are based firmly on the performance of experimental roads.

In all cases the extent of work required to prove a new material or design concept will depend upon how far it departs from the proven standard designs. The process of proving a design change requires extensive laboratory and field testing and a good understanding of the analytical process.

Improvements in existing materials, such as minor changes in hardness of a conventional bituminous binder or in aggregate grading can be introduced with reasonable confidence provided the effect of these changes on stiffness modulus is known. However, to take account of variability of stiffness modulus and the degree of age hardening of binder in the road, the determination of a representative value of modulus requires extensive laboratory testing of samples of laid material. Some laboratory testing would also be required to check the resistance to fatigue cracking of the new material and, particularly for thick pavements, its resistance to deformation.

For radically new materials or design concepts more extensive laboratory tests would be necessary and, to give complete confidence, particularly in relation to fatigue cracking, the new design would require proving in an accelerated life testing facility or in a full-scale road trial.

10. CONCLUSIONS

1. A standard set of designs for bituminous roads incorporating bituminous, lean concrete and wet-mix granular roadbases has been developed. The designs are based on the systematic observation of the performance of numerous sections of experimental road and the interpretation of the results using theoretical design concepts. The analysis takes account of the effect of variability in road performance.
2. Design life has been defined as the traffic carried up to the time when pre-emptive strengthening of the pavement is necessary. This contrasts with the approach adopted in Road Note 29 where the design life was defined as the traffic associated with failure of the road.
3. Recommendations are made for the design thicknesses of sub-base and capping layers. These are based on estimates of the ability of these layers to carry construction traffic.
4. The interpretation of the performance of the experimental roads using structural design concepts offers the means to adapt the standard designs for new situations, such as reconstruction, taking full advantage of better materials and new design configurations. The approach has been developed furthest for roads with bituminous roadbases. Design concepts for wet mix and lean concrete are less certain.
5. To realise the benefits of this more flexible, and hence more powerful, approach to design, the specifying authority would require testing to prove a non-standard material or design; the extent of the proving tests required would depend on the degree of departure from the standard designs developed in this report.

11. ACKNOWLEDGEMENTS

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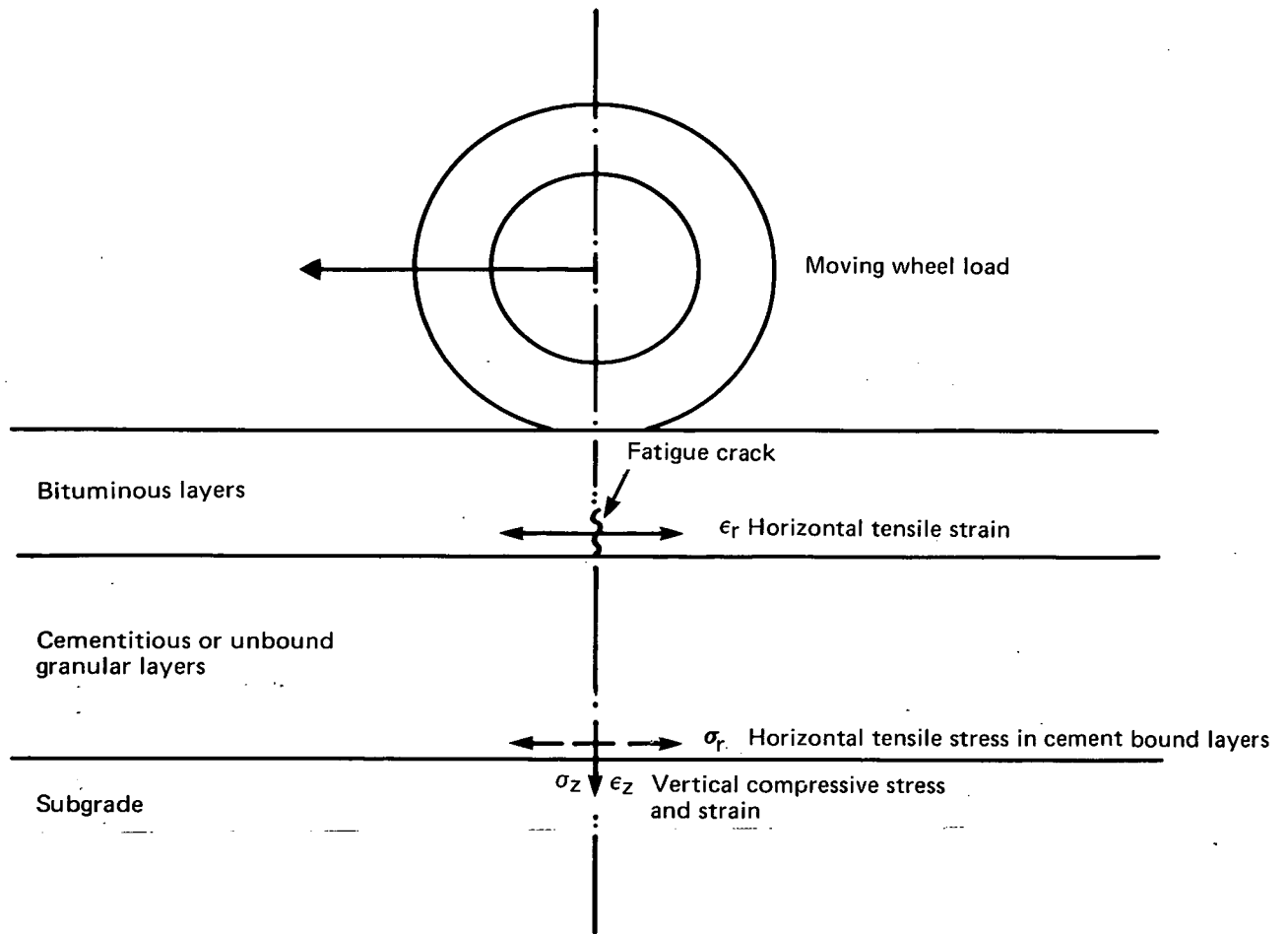


Fig. 1 Critical stresses and strains in a bituminous pavement

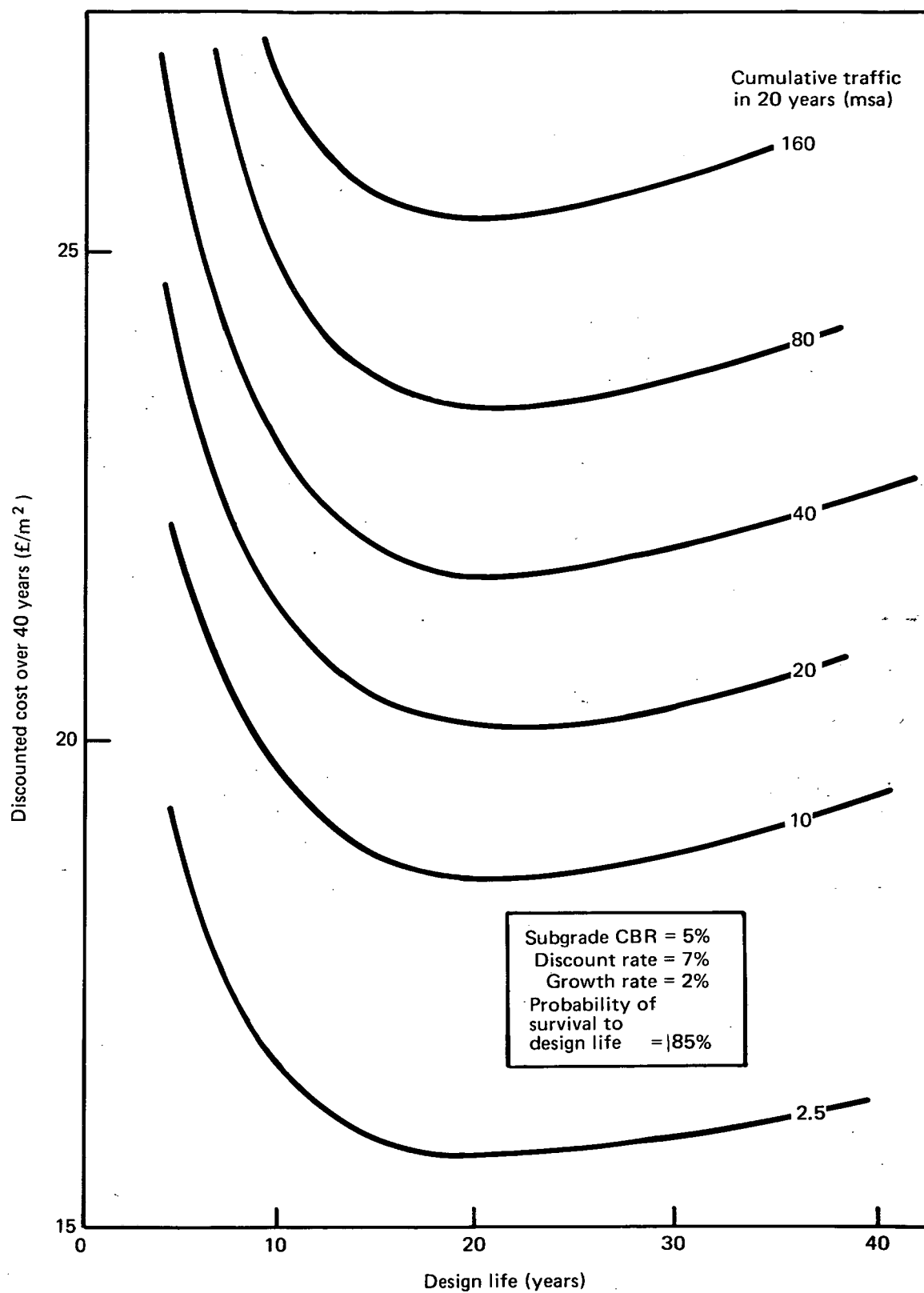


Fig. 2 Discounted cost over 40 years for different design lives of roads with a bituminous roadbase

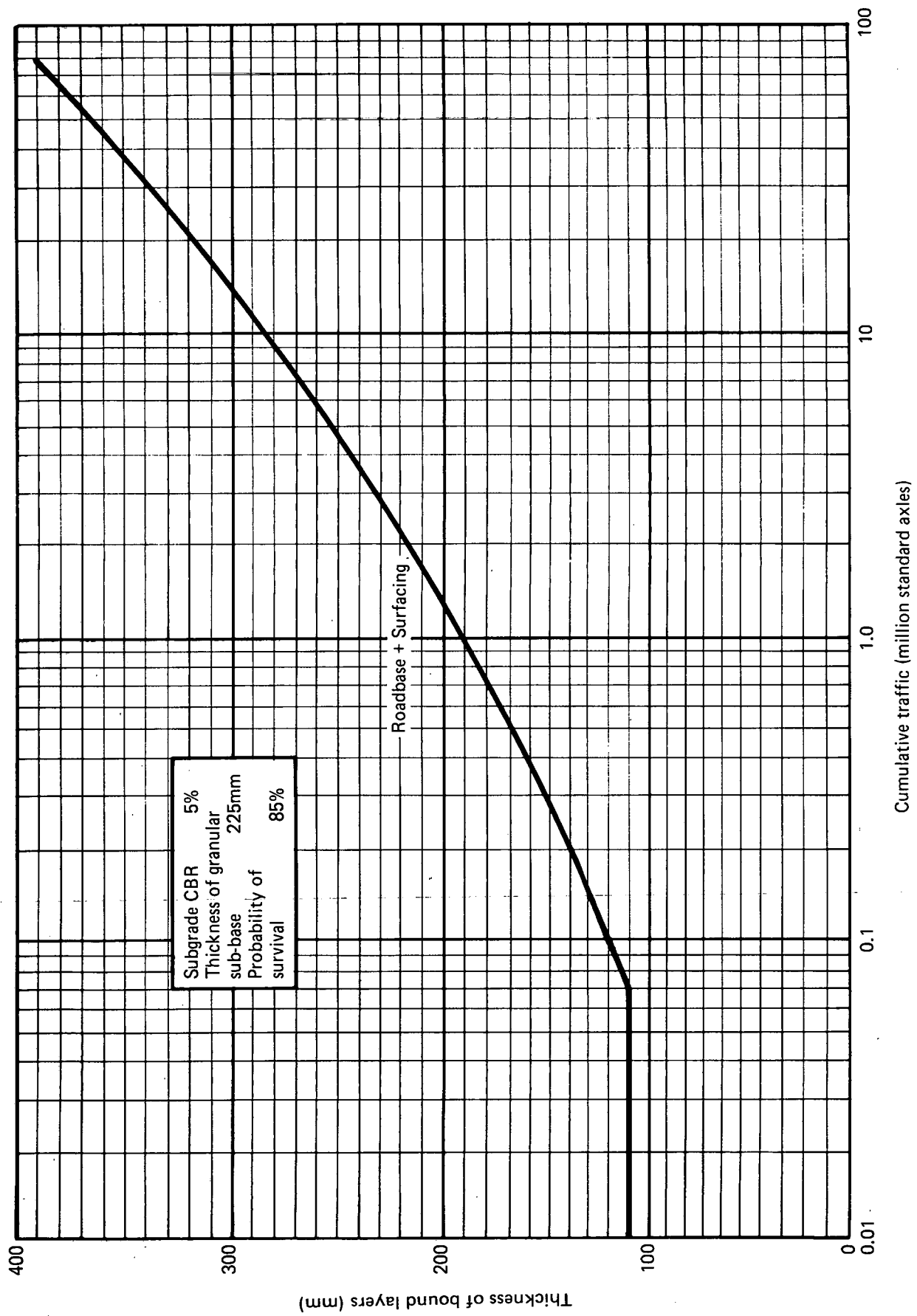
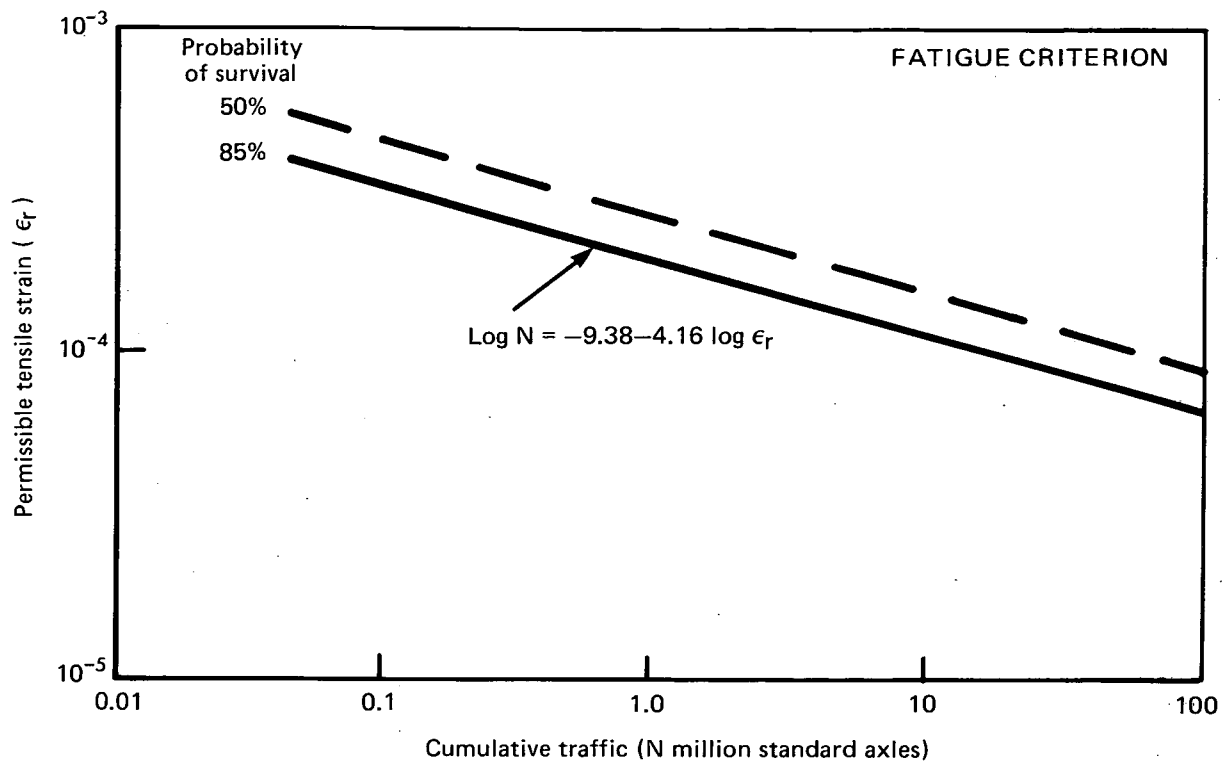
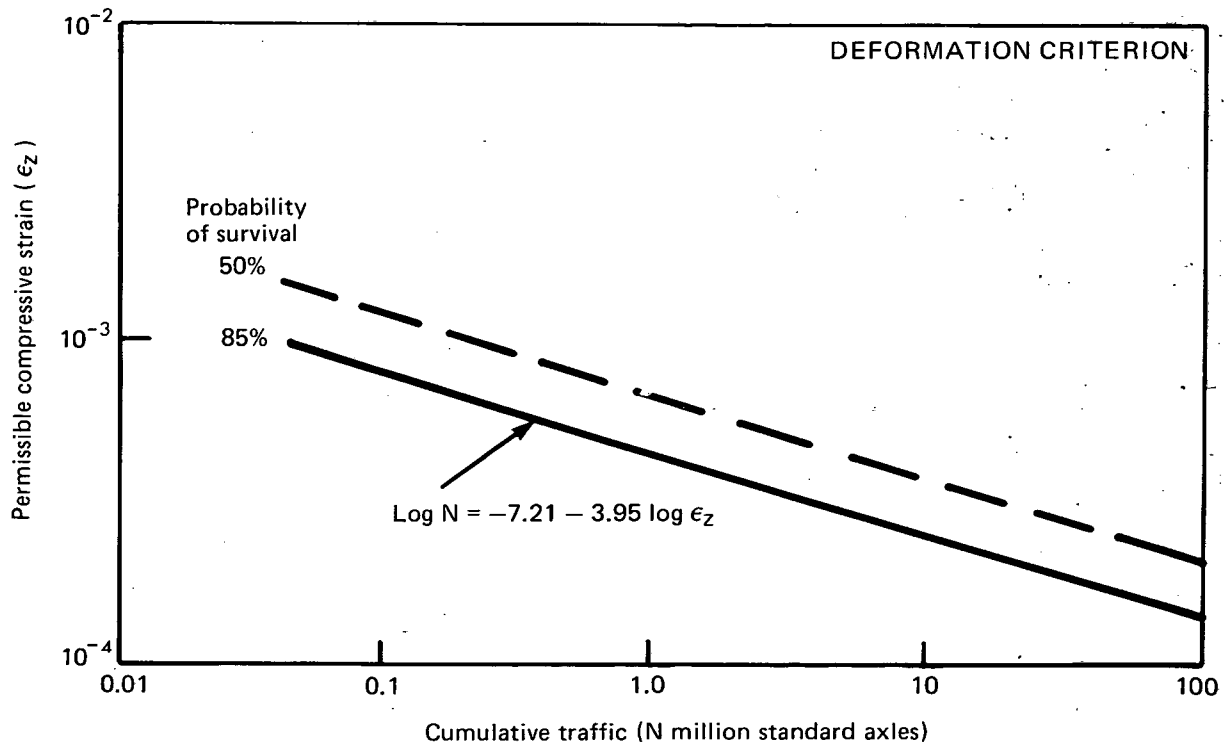


Fig. 3 Design curve for roads with bituminous roadbase



(a) Bottom of dense bitumen macadam roadbase



(b) Top of the subgrade

Fig. 4 Permissible strains induced by a standard 40kN wheel load at a pavement temperature of 20°C

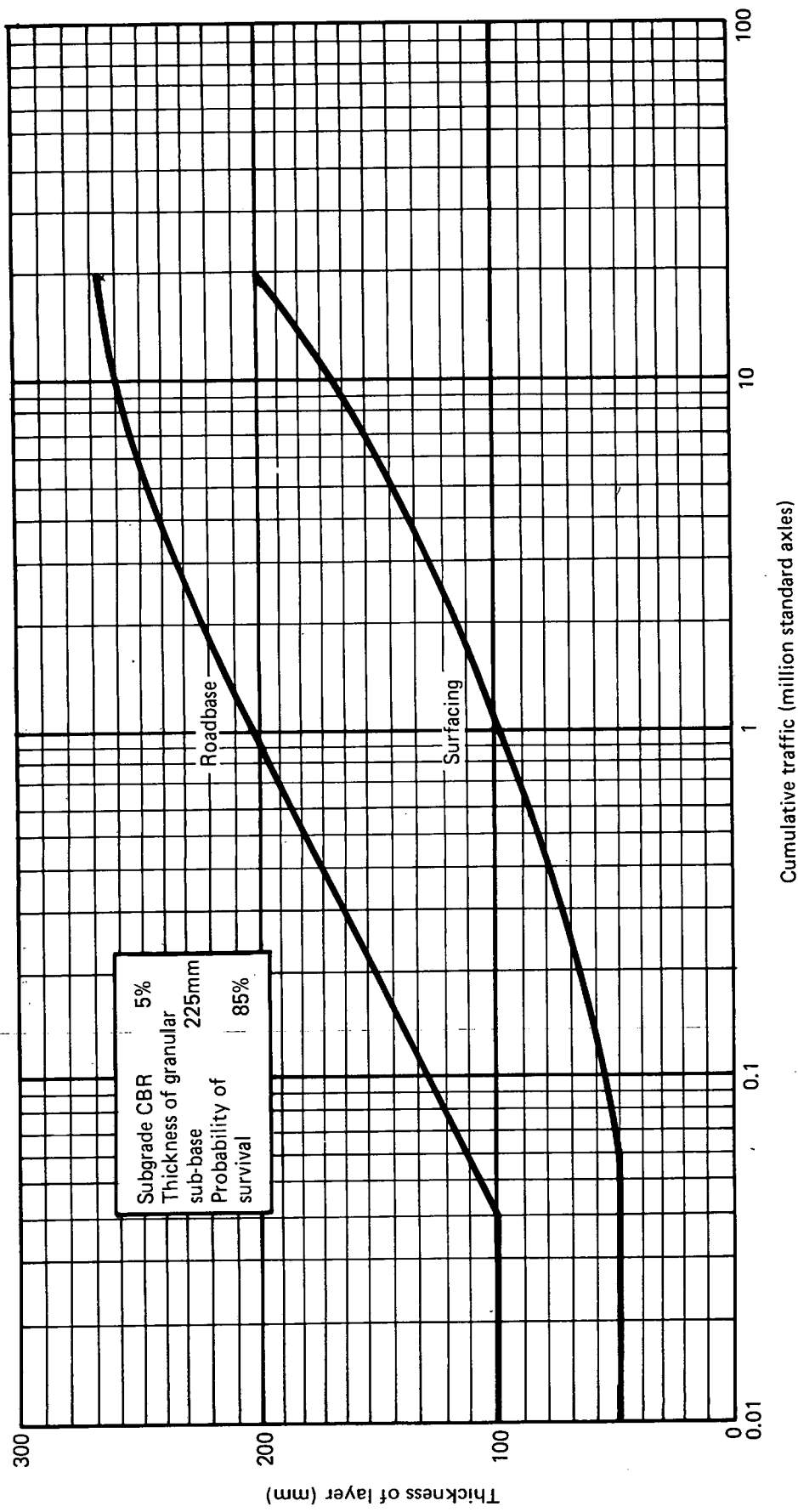


Fig. 5 Design curves for roads with wet mix roadbase

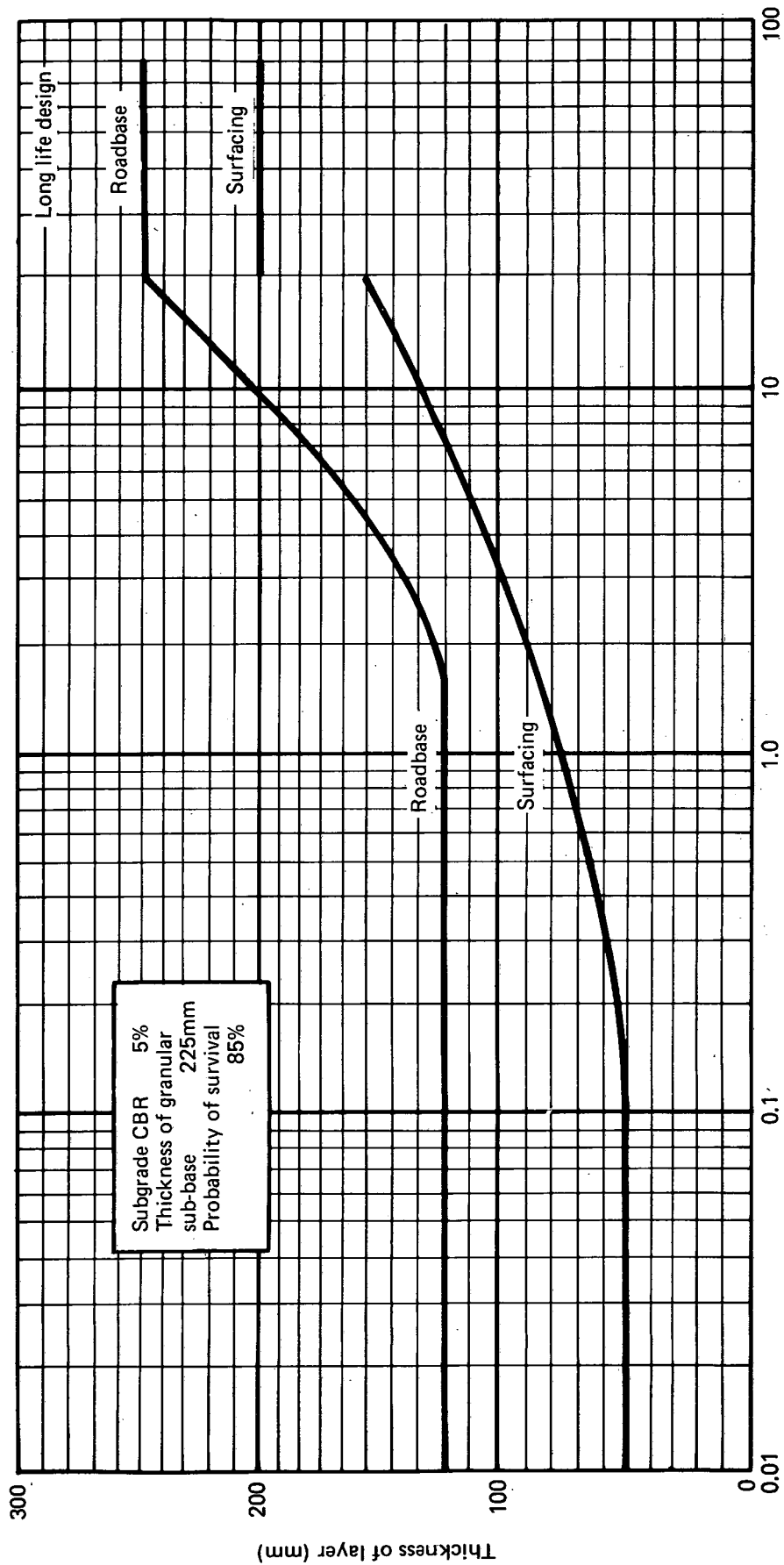


Fig. 6 Design curves for roads with lean concrete roadbase

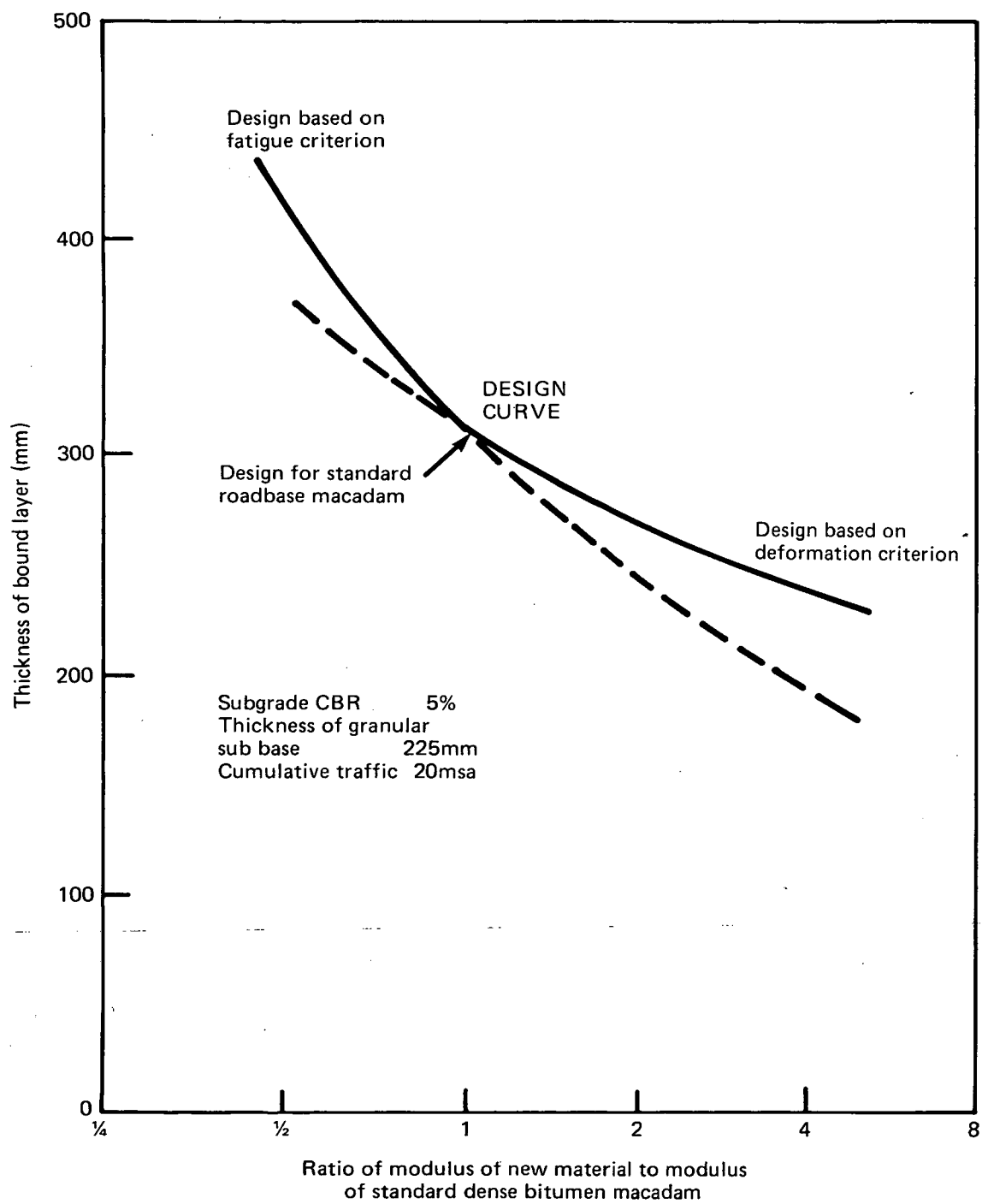


Fig. 7 Effect of changes in modulus of roadbase macadam on design thickness

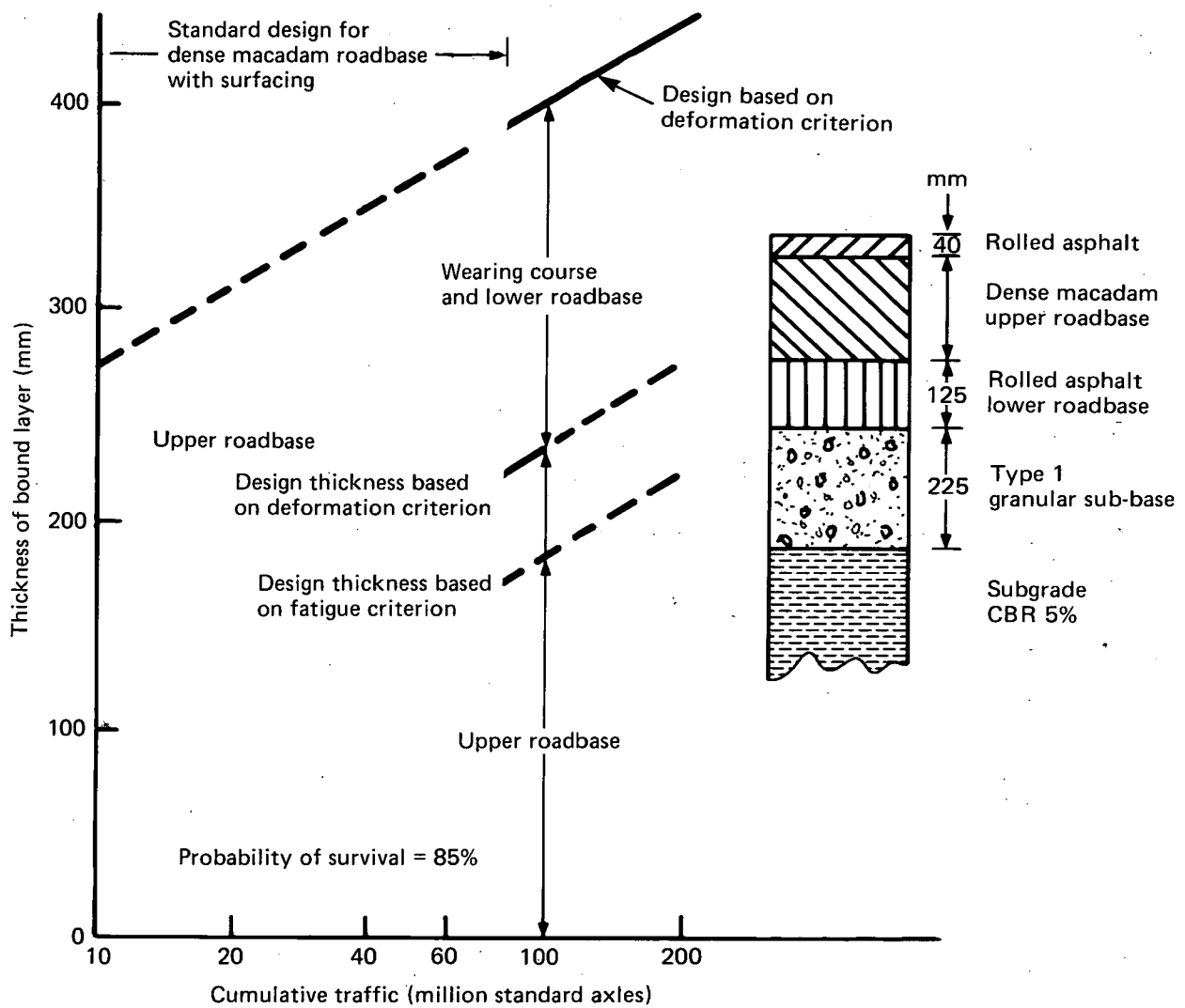


Fig. 8 Design for traffic in excess of 80msa

13. APPENDIX A

VARIABILITY IN PERFORMANCE AND ITS EFFECT ON DESIGN

The performance of a road is influenced by a number of parameters, the properties of the laid materials and their thicknesses, the strength of the subgrade and the traffic carried. Because all are subject to inherent variability or uncertainty, a wide range of performance by a given nominal design is therefore to be expected. If the design of a road pavement is based on mean values there would be no indication as to the likely range of expected lives, and in particular to the extent of premature failures. Neither would it offer a basis upon which to investigate and optimise the total cost of constructing and maintaining roads. A simple model of the structural behaviour of pavements with a bituminous roadbase has therefore been developed to take into account the inherent variability in the factors influencing pavement life. By discounting construction and reconstruction costs, the model has been used to determine the pavement design life that results in the minimum cost over the whole life of the road.

13.1 Variability in pavement performance

In determining the overall probability distribution for the life of a pavement each of the following independent variables was defined by a mean and a standard deviation.

13.1.1 Materials and construction practice. The performance of the experimental roads provided information about the effect on performance of combined variabilities in material quality and construction practice and those associated with shortcomings in the concepts of CBR and standard axle as appropriate input parameters. The standard deviation associated with this variability was derived from the scatter of observations shown in Figure A1. This source of variability provides the most significant contribution to the overall variation in pavement life.

13.1.2 Laid thickness of bituminous materials. Information collected to monitor compliance with a specification for surface level indicated that on average the total thickness of all layers of bituminous material is equal to the specified thickness, with a standard deviation of 10 mm.

13.1.3 Sub-grade CBR. In the absence of comprehensive information about the variability of CBR the standard deviation of the difference between the design and actual CBR of the sub-grade was assumed to be CBR one per cent.

13.1.4 Traffic. Results of the rotating census mentioned in Appendix B suggest that the initial flow of commercial vehicles may be described by a normal distribution with a standard deviation of about 20 per cent of the mean value.

The annual rate of growth of traffic observed on individual roads, averaged over the past 20 years, shows a large scatter about the mean annual growth rate for all roads. For most calculations the average growth rate was taken to be 2 per cent with a standard deviation of 2 per cent.

The work of Currer and O'Connor (1979) suggests that the expected variation in vehicle damage factor over the trunk road network at any one time can be described by a standard deviation of about 20 per cent.

13.1.5 Overall variability. The individual factors that influence pavement performance were combined to estimate the overall variability. Figure A2 gives an example of how the thickness of the bound layer influences the probability of achieving a particular design life specified as the onset of critical conditions in the road pavement. A thickness of bound layer of 275 mm gives about 40 per cent chance of survival to 20 years, ie there is a 60 per cent probability that the road will not reach the 20 year design life. To increase the probability of survival to 85 per cent the thickness of the bound layer would have to be increased to 350 mm. The gradual nature of the deterioration process is evident in Figure A2: 3 per cent of the pavements with 350 mm of bound layer, will have reached a critical condition after 10 years, 15 per cent after 20 years and 28 per cent after 30 years.

13.2 Cost analysis

A simple analysis of costs over 40 years has been carried out to help in the selection of a suitable design life. The analysis takes account of the variability of pavement performance and also of the cost of traffic delays and other consequences of reconstruction.

In practice strengthening by overlay at the onset of critical conditions is the preferred maintenance strategy, but a direct analysis of costs was found to be excessively complex. Fortunately, although the timing of reconstruction is uncertain because of the less predictable deterioration of the road after it has reached a critical condition, a simpler analysis based on reconstruction at a level of cumulative traffic 50 per cent greater than the nominal design life was found to be a satisfactory indicator of design life options, and gave results similar to the more complex analysis.

Plant, labour and material costs were obtained from unpublished studies carried out on new construction contracts. These costs were also applied to reconstructions, but augmented also by the cost of traffic control measures and those that result from delays or diversions. The extra costs incurred because of traffic delays varies widely from one site to another depending on the type of road, the volume of traffic it carries and the availability of diversionary routes. Typical costs were derived by examining the cost of traffic delay for roads at the mid-point of their design range in traffic terms, and with no diversionary routes available during reconstruction. These extra costs incurred at reconstruction were about the same magnitude as the costs of the engineering work. All costs, which were in terms of January 1983 prices, were discounted back to the year of initial construction at 7 per cent; this is consistent with the approach adopted for all Trunk Road investment appraisals.

Discounted costs have been calculated for the construction and reconstruction over 40 years for pavements with bituminous roadbase, designed to carry traffic from 2½ to 160 msa over 20 years. From Figures A3 and A4 it may be seen that, regardless of traffic volume, the selection of a short design life would incur large cost penalties. Beyond about 15 years, design life has little influence on costs discounted over 40 years; typically there is an increase of only about one per cent for an increase in design life from 20 to 30 years. However, a standard design life can help greatly to match the allocation of road construction funds to the needs of forecast traffic and ensure a reasonable time between major maintenance interventions. A design life of 20 years and an 85 per cent probability of survival seems to balance the potential economic benefits of a longer design life against failures to achieve a long design life for reasons other than traffic loading, for example deterioration of drainage, openings by public utilities and road widening or realignment. It should be remembered that at the design life the road can be strengthened by an overlay, or it can be allowed to proceed towards eventual failure and reconstruction, which would not usually be necessary for a number of years.

A sensitivity analysis suggests that the assumptions made in the computations are not critical and that the conclusions concerning the choice of design life can be applied widely. For example the consequences of changing

the growth rate and discount rate can be seen in Figure A5. If the discount rate is other than 7 per cent, a choice of design life of 20 years and an 85 per cent probability of reaching this design life does not incur large cost penalties. This choice of design life seems reasonable also for a zero growth rate. Similarly, because of discounting, the optimum design life is not very sensitive to estimates of construction costs and traffic delay 20 or 30 years in the future. However, because of the contribution of traffic delays to whole life costs, there are considerable benefits to be gained by reducing delays at reconstruction.

If the maintenance strategy selected does not involve reconstruction but consists of strengthening by overlay at the time when the pavement condition is critical, the analysis is more complex but a limited study confirms the choice of a 20 year design life and an associated 85 per cent probability of achieving that life. On motorways factors other than traffic loading are unlikely to be responsible for failure to achieve a long design life and costs of traffic delays are high. Hence design lives of longer than 20 years could be chosen if they were thought to be technically feasible under the expected very heavy traffic.

Pavement design is not a precise process, neither is the estimation of future traffic loading; furthermore there are practical limits due to the variability of the construction process. For these reasons it is normal practice to simplify the continuous design curves by adopting a catalogue divided into four or five categories or discrete steps in thickness; the appropriate thickness can be selected to suit each design requirement. Remembering that the method of specifying design life is conservative in that 85 per cent of the roads are expected to survive longer than their design life, it is sufficient to adopt an approximate method whereby the discrete steps in thickness correspond to performance that will centre on the nominal design life.

13.3 Reference

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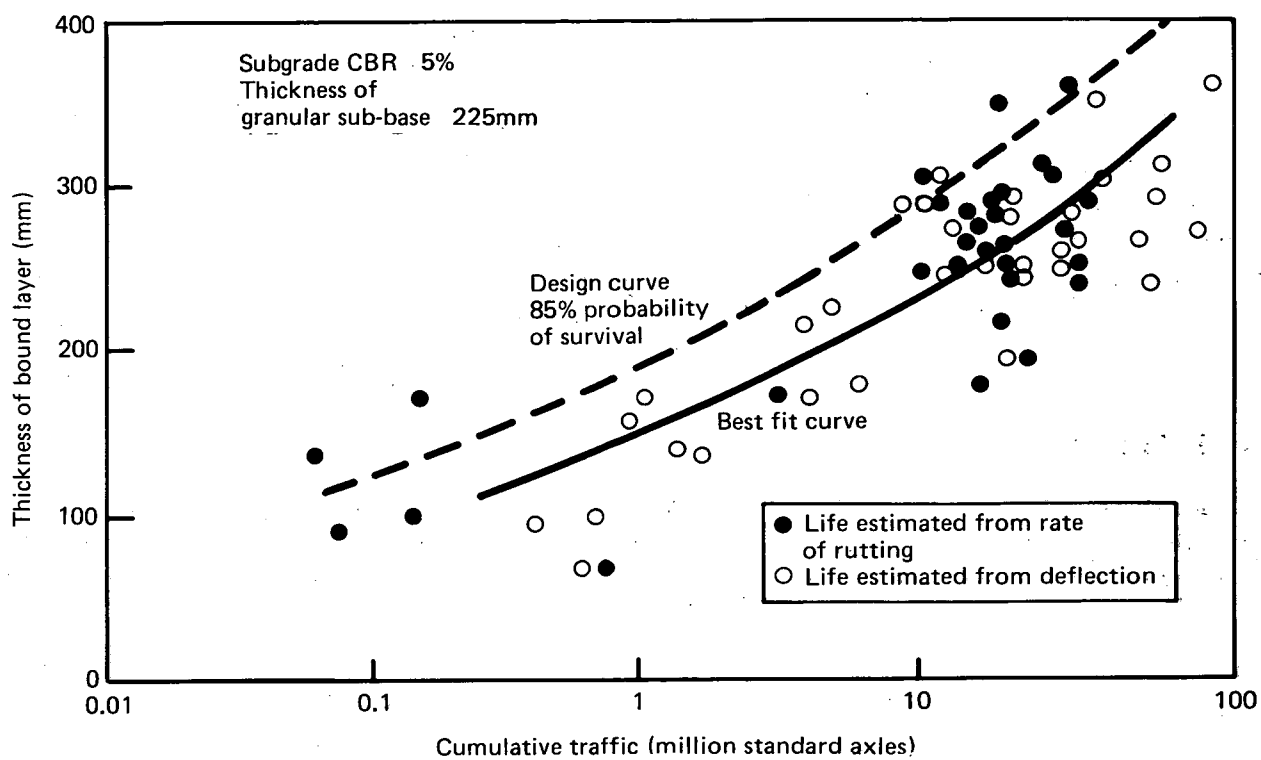


Fig. A1 Relation between thickness and life of experimental roads with dense bitumen macadam roadbase

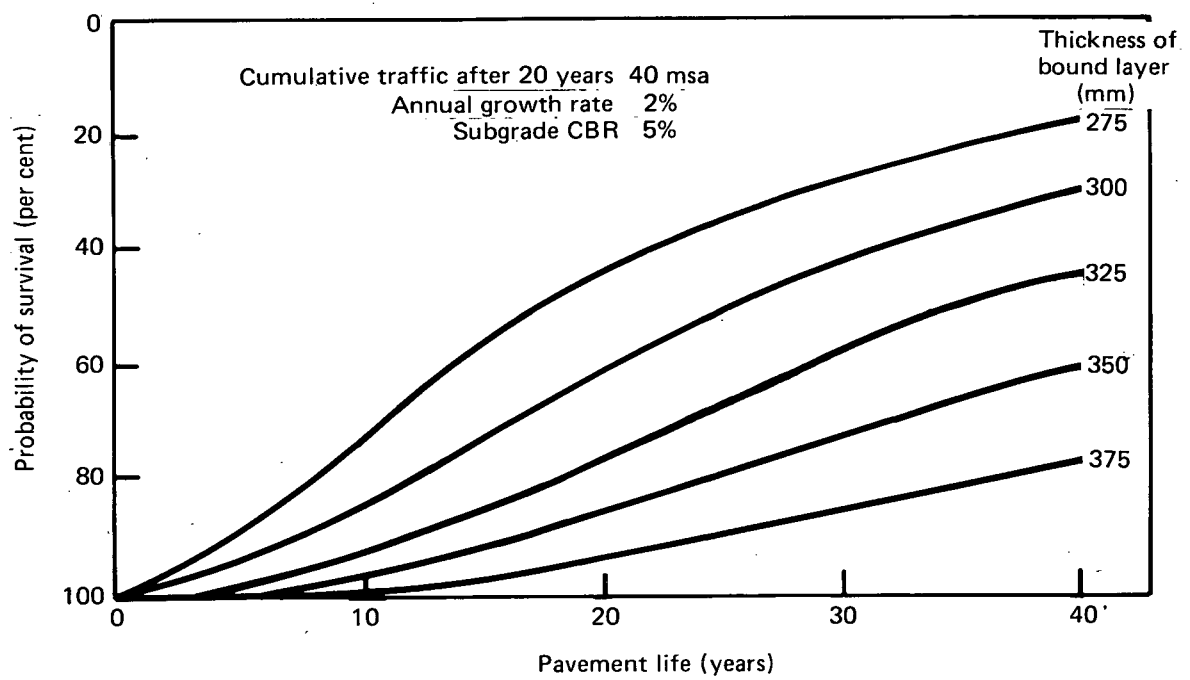


Fig. A2 Probability of survival for different thicknesses of bituminous pavement

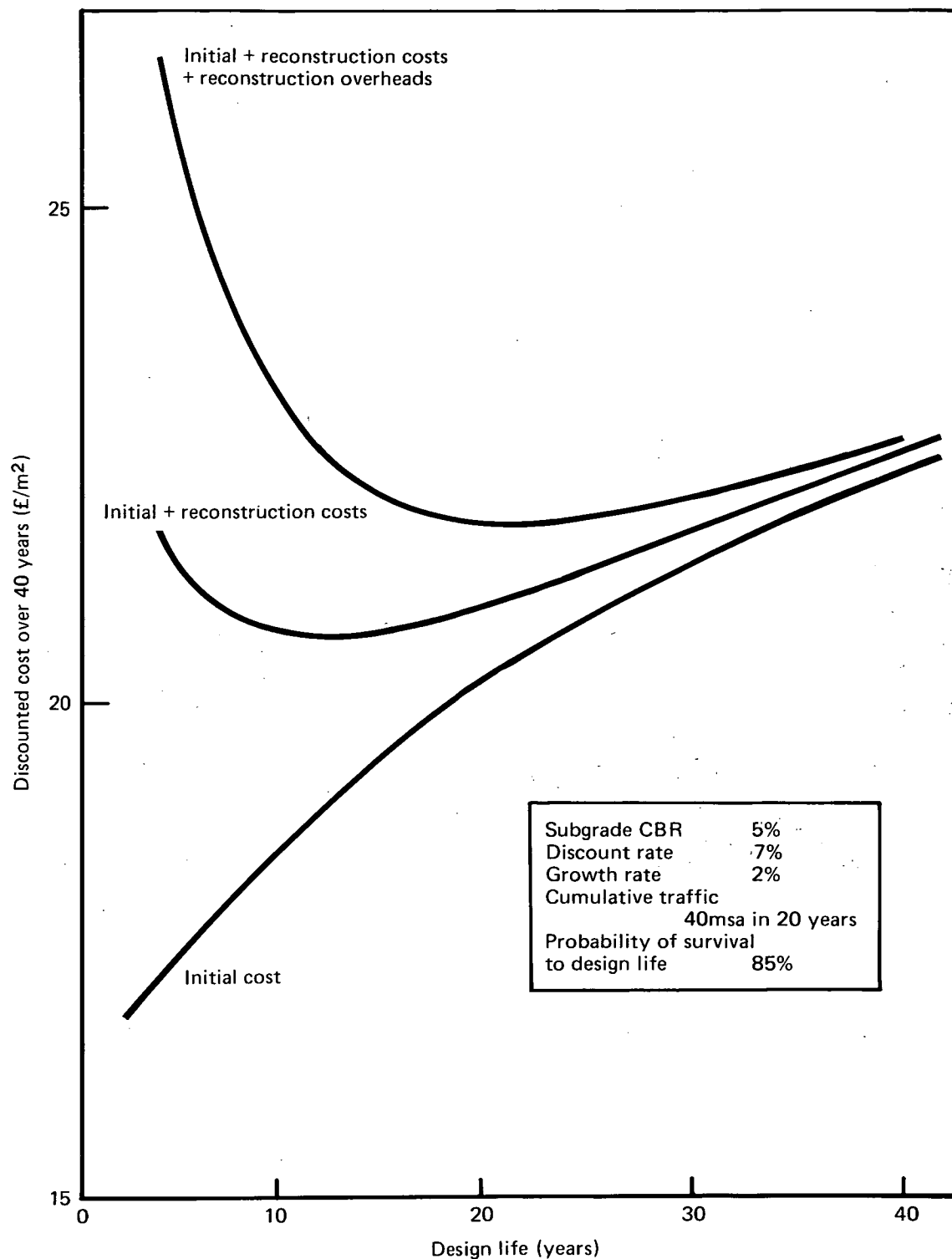


Fig. A3 Costs of construction and reconstruction discounted over 40 years for roads with bituminous roadbases

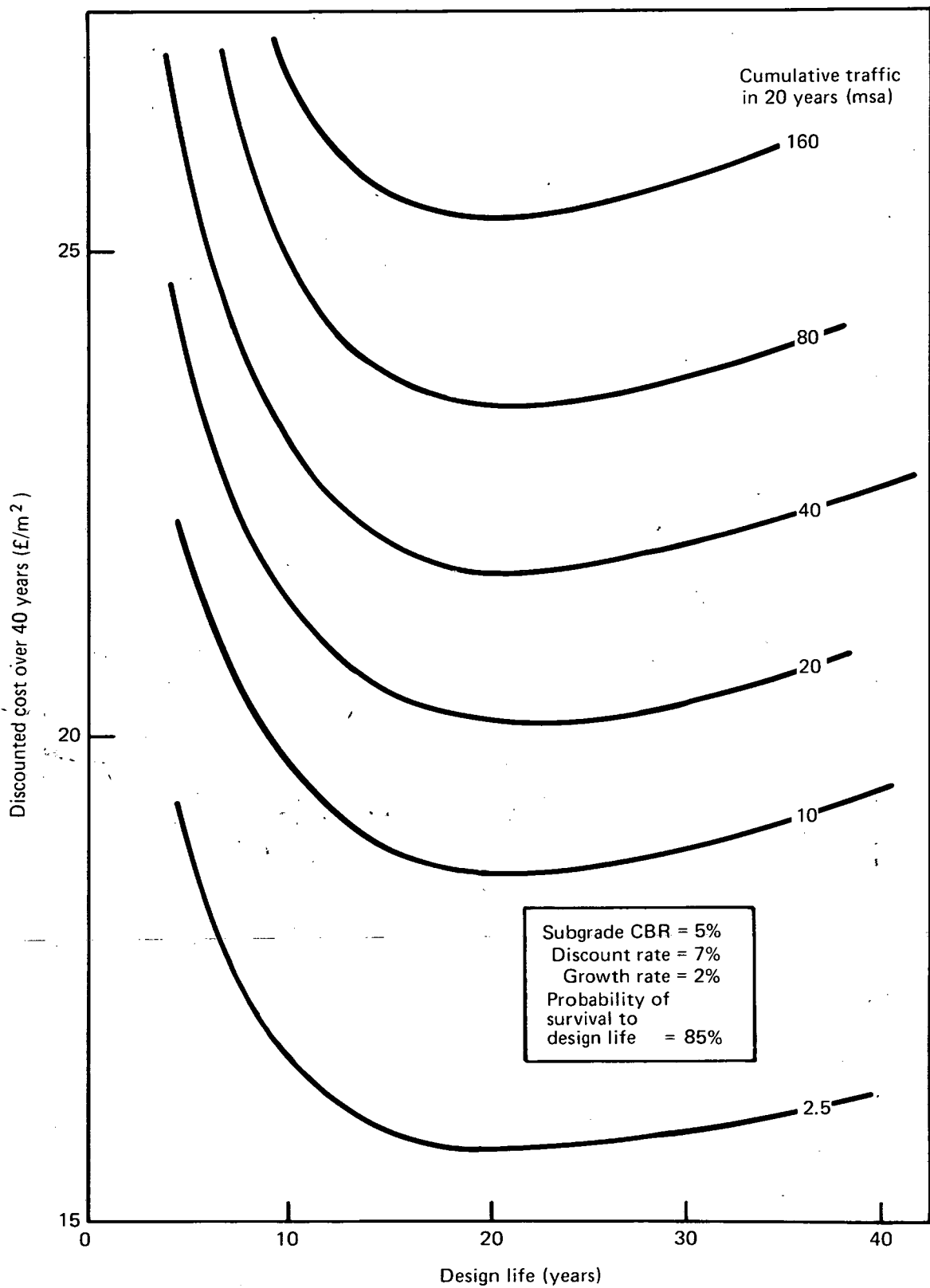


Fig. A4 Discounted cost over 40 years for different design lives of roads with a bituminous roadbase

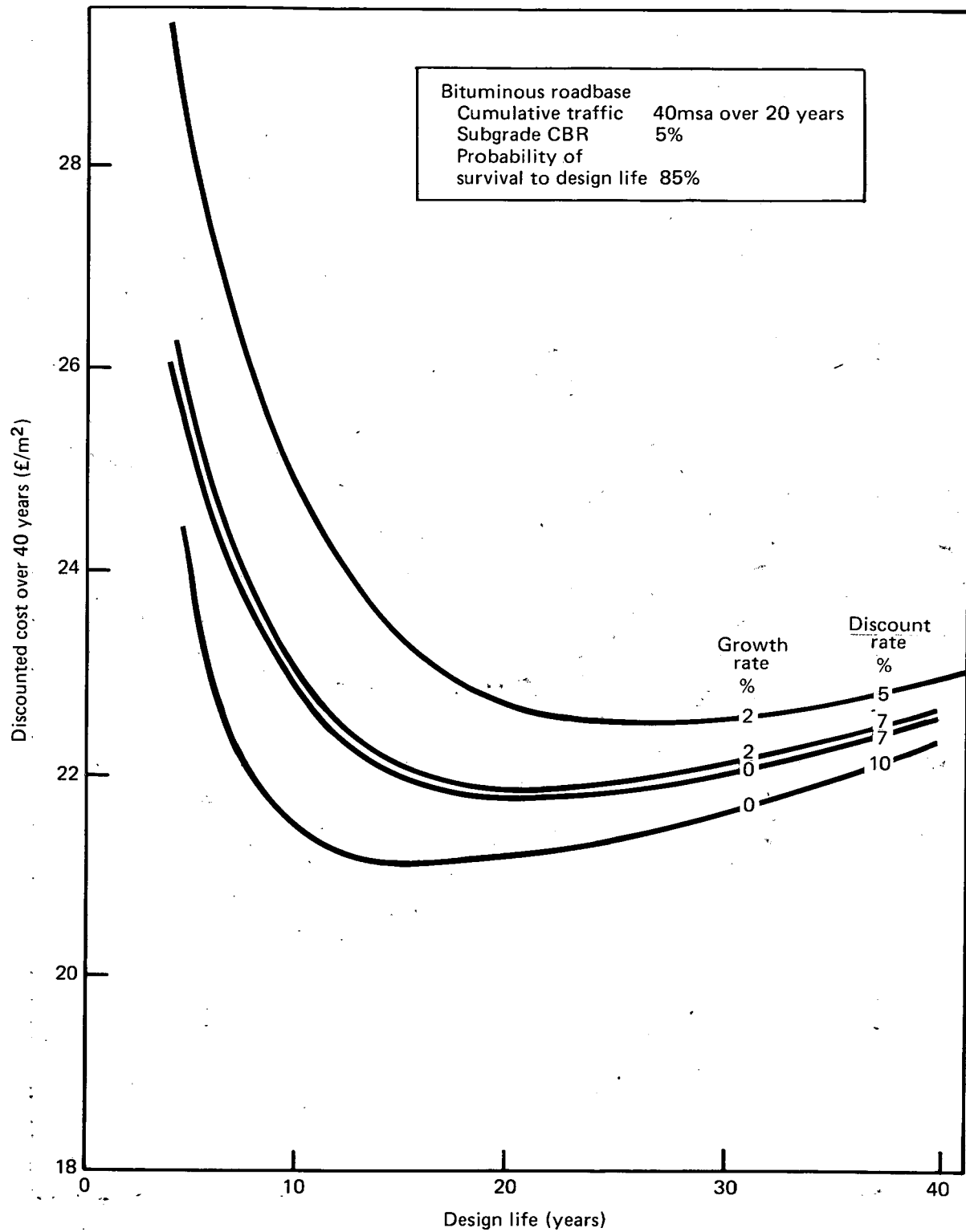


Fig. A5 Consequence of changing growth rate and discount rate on the total cost discounted over 40 years

14. APPENDIX B

ASSESSMENT OF TRAFFIC LOADING

The calculation of standard axles for design purposes is based on a forecast of the commercial traffic expected to use the road over its design life. Estimated annual 24 hour daily commercial vehicle flows have to be converted to annual flows and allowance made for the proportion of commercial vehicles travelling in the nearside traffic lane. The annual numbers of commercial vehicles are then multiplied by the estimated damaging effect of an average commercial vehicle, the vehicle damage factor, to give an estimate of the cumulative number of standard axles; in most cases it will be sufficiently accurate to use the vehicle damage factor applicable at the mid-term of the design life.

In this Appendix the process of estimating cumulative standard axles is described, together with sources of data that support the calculations and the sensitivity of the estimates to varying input data.

14.1 Information required and available sources

The core or rotating census provides twenty-four hour annual average daily flows (24hr AADF) of commercial vehicles. The core census consists of 170 randomly selected sites for which traffic is counted on 3 days in each month of the year. Some 110 of these sites are on motorway, trunk or principal roads. The rotating census was started in 1979 and will cover, over a six-year cycle, all links of the motorway, trunk and principal road network. Each link (ie each length of road between major intersections) is counted one day in each cycle and converted to AADF values for each vehicle type. Where the rotating census cannot provide suitable data, it may be necessary to mount a short survey.

Where possible the directional split in the flow of commercial vehicles should be calculated from census data. In the absence of specific information a 50/50 split should be adopted.

The growth rate applying to commercial traffic will not necessarily be the same as that applying to other traffic. Where census counts are available it may be possible to derive a growth rate applying specifically to commercial traffic. However, unless there are special circumstances, for example local industrial or commercial development or the opening of a feeder route, an annual growth rate of 2 per cent may be used without serious loss of accuracy for trunk roads and motorways.

14.2 Procedure to estimate cumulative standard axles

Addis and Robinson (1983) have shown that the total number of commercial vehicles T_n using the slow lane over the design life, n years, can be expressed in terms of the initial daily flow F_o , the growth rate r and the proportion of commercial vehicles using the slow lane P as follows:—

$$T_n = 365F_o \frac{((1+r)^n - 1)}{r} P \dots\dots\dots (B1)$$

In order to convert this cumulative number of commercial vehicles to equivalent standard axles, it must be multiplied by the number of equivalent standard axles per commercial vehicle at the mid-term of the design life. The factor is calculated from the following formula which is based on the results of dynamic weighbridge

measurements carried out over a number of years. It gives an estimate of the vehicle damage factor D for any mid-term year t based on the 24 hour AADF of commercial vehicles for that year F; the base year is 1945, so that the year 1984 corresponds to t = 39.

$$D = \frac{0.35}{0.93^t + 0.082} - \left(\frac{0.26}{0.92^t + 0.082} \right) \left(\frac{1.0}{3.9(F/1550)} \right) \dots\dots\dots (B2)$$

As an example, the equation generates vehicle damage factors that vary with time and AADF of commercial vehicles as shown in Table B 1.

TABLE B 1
Vehicle Damage Factors

Year	Daily flow of commercial vehicles AADF			
	250	1000	2000	4000
1985	0.78	1.64	2.17	2.49
1990	0.93	1.89	2.49	2.84
1995	1.18	2.12	2.76	3.14
2000	1.22	2.31	3.00	3.40
2005	1.34	2.47	3.18	3.60
2010	1.43	2.60	3.33	3.76

Estimates of vehicle damage factor calculated from Equation B 2 will normally be sufficiently accurate for design purposes.

14.3 Sensitivity of traffic estimates to changes in input data

Table B 2 is an example to show the percentage changes in cumulative standard axles because of changes in input data that are representative of observed errors. Over a 20 year period growth rate and vehicle damage factor are more important than initial traffic flow or the proportion of commercial vehicles in the slow lane; over a period of 40 years, growth rate is the dominant factor and vehicle damage factor is less important.

TABLE B 2

Percentage change in cumulative standard axles for various changes in input data

Input parameter (amount of change)	20 year design (83 msa)	40 year design (232 msa)
Initial daily flow (400 commercial vehicles per day)	8 %	7 %
Growth rate (2 per cent per annum)	17 %	35 %
Proportion of commercial vehicles in slow lane (5 per cent)	6 %	7 %
Vehicle Damage Factor (0.5)	17 %	14 %

Baseline conditions: Type of road – Dual 2-lane carriageway
 Year opened to traffic – 1982
 Initial Daily Flow of commercial vehicles – 4000
 Growth rate – 2 per cent per annum

14.4 Reference

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15. APPENDIX C

THE ROAD FOUNDATIONS

15.1 Subgrade

In the design of road pavements the strength and stiffness of the subgrade are required to be estimated or measured for two design functions:

- (a) to characterise the subgrade as the foundation of a haul road to carry construction traffic; for this purpose the strength of the subgrade just before placing the capping layer or sub-base is needed.
- (b) to establish the likely in-service long-term strength and stiffness of the subgrade after the disturbance of the construction phase and when moisture equilibrium has been established within it.

15.1.1 Soil strength. The California Bearing Ratio (CBR) test has been widely used to determine the strength of road subgrades. In spite of its limited accuracy, it remains the most appropriate generally accepted measure, and has the advantage that it can also be related within reasonable limits to subgrade stiffness. The subgrade of the Laboratory's road experiments have been characterised in this way. More sophisticated equipment that simulates the stress conditions under a pavement by repeated triaxial loading has been developed as a research tool and this type of test increases our knowledge of subgrade behaviour, but unless it can be simplified it is unlikely to replace the CBR as a design test. Also it cannot be used for testing soil in-situ.

The CBR value of a soil at a given moisture condition depends on the structure of the soil. Some completely remoulded soils at a given moisture condition can exhibit CBR values that are little more than half those of the same soil in an undisturbed overconsolidated state. In practice construction operations ensure that no subgrades are completely undisturbed and many are substantially remoulded, at least at depths close to formation that determine subgrade performance. Embankments contain soils partially remoulded by compaction, and relatively undisturbed deep cut can exhibit strength characteristics that resemble those of a remoulded soil. There is therefore no certainty as to the structural state of the soil in a new road; estimation or measurement of remoulded values represents a sensible and somewhat conservative approach.

A guide to CBR values of cohesive subgrades was obtained from relations between the suction of a soil, its plasticity index and its strength using methods described by Black and Lister (1979). The strength of the subgrade is critically dependent on its moisture content, and hysteresis in the relation between suction and moisture content ensures that soil that has become wet at the time of construction will remain wetter and therefore weaker in its equilibrium condition under the completed pavement than soil that has been kept relatively dry; this effect is particularly marked in soils of low to medium plasticity.

Typical moisture conditions during construction have been considered when estimating subgrade CBR for the design of capping layers and sub-bases. After very wet weather on a site with poor drainage weak soil would be removed before attempting to construct either a capping layer or sub-base. However heavy rain while the capping layer or sub-base is being laid must be considered. With only a thin layer in position when rainfall occurs the lower line of Figure C1 shows that soil fully wetted on a poorly drained site can approach the low CBR value of one per cent. In practice only very light clays subjected to these poor construction conditions will reach this value before further construction protects the subgrade from the effects of rain and the middle line on Figure C1 is a more realistic estimate.

Under average construction conditions of variable weather on a well drained site the earthworks should be protected promptly with capping layer or sub-base. If delay is unavoidable it is particularly important that the site be well drained with soil laid to a good fall, and perhaps also waterproofed. Under these conditions the likely value of CBR is given by the upper line of Figure C1. The rapid placing of a substantial capping layer or sub-base would produce a considerable further improvement as would construction in good weather with the soil always drier than its likely final equilibrium condition.

The predicted equilibrium values of CBR attained under the completed pavement are influenced by moisture conditions at the time of construction. They also reflect the depth of the water table with which sub-grades come into equilibrium, a high water table giving a lower subgrade strength. The total surcharge applied by the pavement is also an important factor, with heavier pavements incorporating a capping layer leading to a higher subgrade strength. All these influences are most pronounced on soils of low and medium plasticity.

Table C 1 indicates reasonable estimates of equilibrium values of CBR for combinations of poor, average and good construction conditions, high and low water tables and thick and thin pavements. Good conditions result in subgrades never getting wetter than their equilibrium moisture contents beneath the finished road. A high water table is one 300 mm beneath formation level and is consistent with effective sub-soil drainage: the lower water table is 1 metre down. A thick pavement is of 1200 mm depth including a 650 mm capping layer and is typical of a motorway construction: a thin pavement is 300 mm deep. For pavements of intermediate thickness founded on plastic soils an equilibrium value of CBR may be interpolated.

TABLE C 1
Equilibrium suction-index CBR values

TYPE OF SOIL	PLASTICITY INDEX	HIGH WATER TABLE						LOW WATER TABLE					
		CONSTRUCTION CONDITIONS:						CONSTRUCTION CONDITIONS:					
		POOR		AVERAGE		GOOD		POOR		AVERAGE		GOOD	
		THIN	THICK	THIN	THICK	THIN	THICK	THIN	THICK	THIN	THICK	THIN	THICK
HEAVY CLAY	70	1.5	2	2	2	2	2	1.5	2	2	2	2	2.5
	60	1.5	2	2	2	2	2.5	1.5	2	2	2	2	2.5
	50	1.5	2	2	2.5	2	2.5	2	2	2	2.5	2	2.5
	40	2	2.5	2.5	3	2.5	3	2.5	2.5	3	3	3	3.5
SILTY CLAY	30	2.5	3.5	3	4	3.5	5	3	3.5	4	4	4	6
SANDY CLAY	20	2.5	4	4	5	4.5	7	3	4	5	6	6	8
	10	1.5	3.5	3	6	3.5	7	2.5	4	4.5	7	6	>8
SILT *	—	1	1	1	1	2	2	1	1	2	2	2	2
SAND (POORLY GRADED)	—	←————— 20 —————→											
SAND (WELL GRADED)	—	←————— 40 —————→											
SANDY GRAVEL (WELL GRADED)	—	←————— 60 —————→											

* estimated assuming some probability of material saturating

Figure C2 illustrates the effect of weather and construction practice on the equilibrium CBR. The lower band represents the range of values to be expected for the range of pavement thicknesses and water table depths under poor site conditions: the middle and upper are the equivalent bands for average and good conditions respectively. CBR values in the upper band are also appropriate in a reconstruction operation, if a subgrade of known high equilibrium strength can be protected from ingress of moisture and not left without at least a substantial proportion of the surcharge of the pavement for any appreciable length of time; in the case of clays of low plasticity this should not be more than a few days.

In light clay soils of plasticity index less than about 25 per cent, theory implies that only a fraction of the overburden pressure, based on the compressibility factor defined by Black, Croney and Jacobs (1958), is effective in influencing the equilibrium suction value. However limited direct measurement of the soil strength of subgrades of this type indicates that strengths are higher than predicted by theory. Table C1 represents a reasonable compromise.

The estimates of CBR strength illustrate the wide range of possible values, and in particular the considerable influence of conditions during construction. In recognition of this uncertainty the engineer will usually choose to simplify the results for design purposes.

When the equipment and experience is available CBR values can be measured in the laboratory on recompacted samples in accordance with British Standard 1377 (1975). To obtain equilibrium values the conditions of moisture content and density should reproduce as far as possible conditions in the completed road. To avoid spurious effects from pore water pressure, cohesive soils should be compacted to a density corresponding to no less than 5 per cent air voids. Equilibrium moisture content can be deduced from measurements on a suction plate.

Careful consideration of likely causes and consequences is required when in-situ measured values of CBR exceed those predicted from Table C 1 by a large margin, particularly when the measurement implies the omission or reduction in thickness of any capping layer.

15.1.2 Soil stiffness. Measurements of the stiffness modulus of the subgrade soil are required for use in calculating the stresses and strains in the road pavement and in the subgrade soil. Although simulative triaxial test methods are available as research tools there are no readily applicable techniques for routine application.

The value of CBR is a measure of stiffness, but at a much larger strain and lower strain rate than that generated by traffic. Because soil stiffness varies with stress to an extent that is influenced by the degree of overconsolidation, as shown by Brown, Lashine and Hyde (1975) and by the residual structure of the soil in the subgrade a single correlation between modulus and CBR would be unexpected. Design stiffnesses have been established primarily from the comprehensive data relating stiffness measured by wave propagation to in-situ CBR tests on both remoulded and undisturbed soil subgrades reported by Jones (1958).

Account has been taken of the likely effect of the unrealistically low strain levels generated in the wave propagation technique and of other information obtained from repeated load triaxial testing carried out at realistic stress levels and in-situ measurements of transient stress and strain in experimental pavements. It was not possible to differentiate in practical terms between the stiffness of remoulded and undisturbed soils of the same CBR. The design equation developed is:

$$E = 17.6(\text{CBR})^{0.64} \text{ MPa} \dots\dots\dots (C1)$$

where CBR is in per cent; this is a lower bound relationship, valued between 2 and 12 per cent.

15.2 Sub-base

The ability of the sub-base to carry construction traffic has been investigated by two types of analysis. The first involves the provision of a stable platform that will not rut excessively under the construction traffic; either at its surfacing or at formation level; whereas the second is associated only with the protection provided by the sub-base in avoiding excessive dynamic strains within the subgrade that could cause its deformation. Both methods assume complete canalisation of traffic; this will normally be a more severe condition than is encountered in practice.

An extensive test programme on unpaved roads was carried out by the US Corps of Engineers to determine the thickness h (mm) of granular material required to limit rut depth to 75 mm for a given traffic level and sub-grade CBR. Charts representing these data were prepared by Webster and Alford (1978) and characterised by Giroud and Noiray (1981) as follows:

$$\log N_{75} = \frac{h(\text{CBR})^{0.63}}{190} \dots\dots\dots (C2)$$

where N is the number of standard 80 kN axles.

However pilot-scale trials in the UK by Pike, Acott and Leech (1977) and by Potter and Curren (1981) have demonstrated a greater resistance to rutting of good quality sub-base (Type 1) than that predicted by the above equation. An additional factor has therefore been introduced to take account of this and also for the different criterion for the sub-base to provide a good working platform; the deformation has been limited to 40 mm, the minimum that can be tolerated if the sub-base surface is to be reshaped and recompacted efficiently and serious rutting is to be avoided in the subgrade. The adapted equation is:

$$\log N_{40} = \frac{h(\text{CBR})^{0.63}}{190} - 0.24 \dots\dots\dots (C3)$$

This equation has been used to establish the thickness of sub-base required for different levels of cumulative construction traffic, as indicated in Figure C3.

The second approach determined only the thickness of sub-base required to reduce the dynamic vertical strain in the subgrade at formation to a level that would not bring about major deformation within it; the method is based on an elastic analysis. Adequate characterization of the non-linear elastic response of granular materials is particularly difficult when they are unsurfaced and laid on foundation materials or subgrades that are relatively weak. Shook, Finn, Witczak and Monismith (1982) and others have shown that the dynamic stiffness of these lower layers imposes an upper limit on the stiffness of overlying granular layers and in this analysis the stiffness of each successive sub-base layer that has been compacted separately has been taken to be three times that of the layer beneath it, up to an upper limit of stiffness of 150 MPa. On weak subgrades of up to CBR of 5 per cent the lower sub-base layer thickness of 225 mm is assumed.

The levels of acceptable dynamic subgrade strain for different amounts of cumulative traffic were derived from the elastic analysis of the performance of experimental roads described in Appendix D. Sub-bases carrying

construction traffic support fewer loads than even lightly trafficked roads: the experimental relation of Figure D3(b) corresponding to an 85 per cent probability of achieving the pavement life without undue subgrade deformation, was therefore extrapolated back to give Figure C4. There was some confirmation of the conservatism of this relation in the results obtained from trafficking experiments on unsurfaced roads by Potter and Currer (1981) and Ruddock, Potter and McAvoy (1982).

Strains in the subgrade under a standard wheel load were computed and related to the criterion in Figure C4 to establish sub-base thicknesses required for different levels of cumulative traffic; these results are compared with those of the first analysis in Figure C3.

Given the imprecise nature of both analyses there is good agreement between the two methods in predicting sub-base thickness required for haul roads to carry traffic of between 100 and 1000 standard axles, a range catering for the majority of both minor and major road works. The guide to the length of road that can be constructed on a given thickness of sub-base that is also indicated on Figure C3 is obtained from the traffic versus sub-base thickness characteristic using the typical wheel loads and vehicle routes on construction sites established by Hardman, Heaton, Jordan and Abell (1976).

The thickness of sub-base should be based on the likely CBR value at the time of construction. Recommended values are given in Figure C1. The resulting thickness may then be taken into account in the final design at the estimated equilibrium design value of CBR, as described in Appendix D, wherever it is necessary and practical to do so.

When a capping layer is not employed on weak subgrades of less than 5 per cent the thickness of sub-base exceeds 225 mm and must be laid in two layers. The lower layer should be as thick as practically possible, and preferably 225 mm, in order to avoid the danger of damaging the sub-base or subgrade during compaction.

15.3 Capping layer

Apart from its function as a protection for earthworks, the use of a capping layer beneath the sub-base has structural advantages on weaker soils. Because the stiffness of an unbound granular layer is strongly influenced by the stiffness of the layer beneath it, the full potential stiffness of good quality sub-bases cannot be properly mobilised when they are laid on weaker subgrades. Also, on subgrades weaker than 5 per cent CBR the required thickness of sub-base is often greater than 225 mm; they therefore must be built in two thinner layers and any thin lower layer may prove difficult to compact without damaging sub-base and subgrade. It can therefore be economical and sound engineering practice to use a lower quality material than Type 1 sub-base to replace the lower sub-base layer, provided the material can be compacted and is then stable. However the primary requirement for the capping layer and sub-base remains that the combined structure shall be able to act as a haul road and construction platform under all normal weather conditions.

The calculation of dynamic stresses and strains within the sub-base, capping layer and subgrade can be no more than broadly indicative of the behaviour of the foundation under vehicle loading on the sub-base surface because the stiffness properties of the capping layer cannot be well defined and are expected to be non-linear. Nevertheless an analysis similar to that used for the sub-base design allows comparison of the subgrade vertical strains for various combinations of capping layer stiffness and thickness. The results of this analysis, made for a 150 mm sub-base, the thickness currently recommended for use with a capping layer, is illustrated in Figure C5. The vertical strain in the subgrade is most sensitive to capping layer thickness, while being relatively insensitive to capping layer stiffness over the probable range of stiffness indicated in Figure C5. However a capping layer material of minimum CBR of 15 per cent is recommended in order to reduce the possibility of failure within the capping layer.

The ratio of permissible subgrade vertical strain under sub-base alone, to that under the combination of sub-base and capping layer is a measure of the factor of safety for the latter design. From Figure C5 it can be seen that for the standard arrangement of 150 mm of sub-base on a 350 mm capping layer over a subgrade of CBR greater than 2 per cent there is a factor of safety of at least 2. For subgrades of CBR 2 per cent or less the increase in capping layer thickness to 600 mm maintains or enhances this factor; uncertainty in material properties and in the analysis make the choice of thinner capping layers imprudent.

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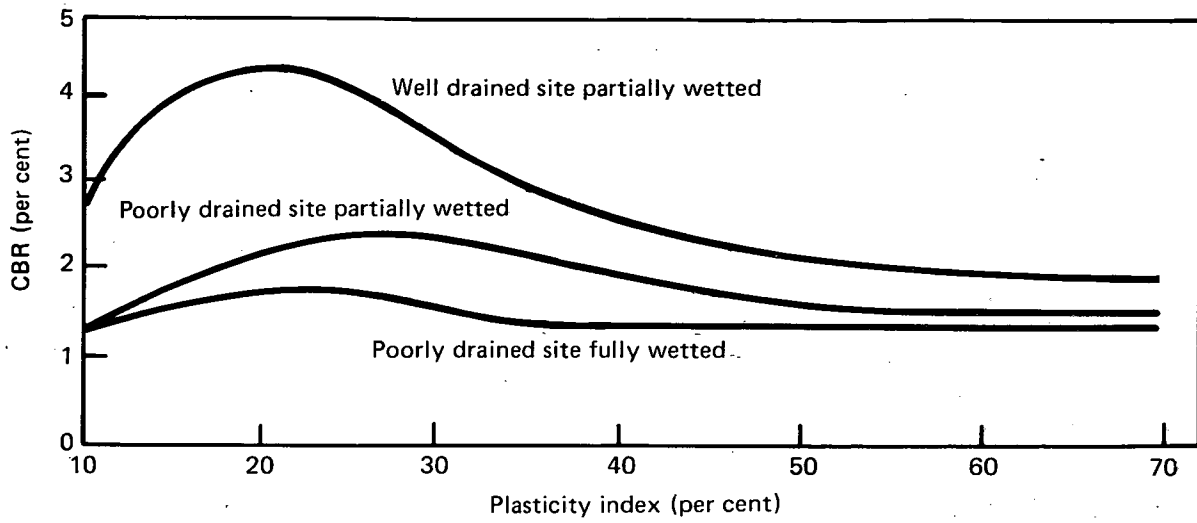


Fig. C1 Subgrade CBR during construction

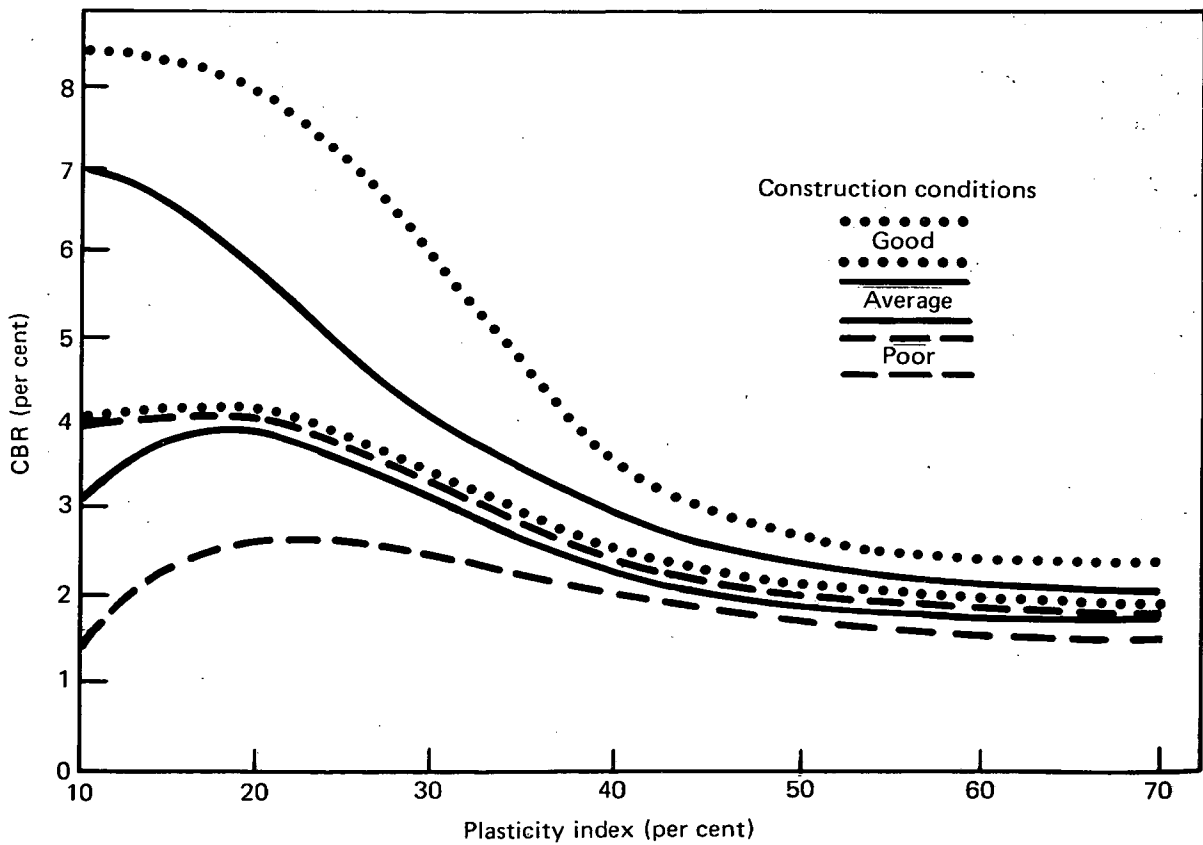


Fig. C2 Equilibrium CBR

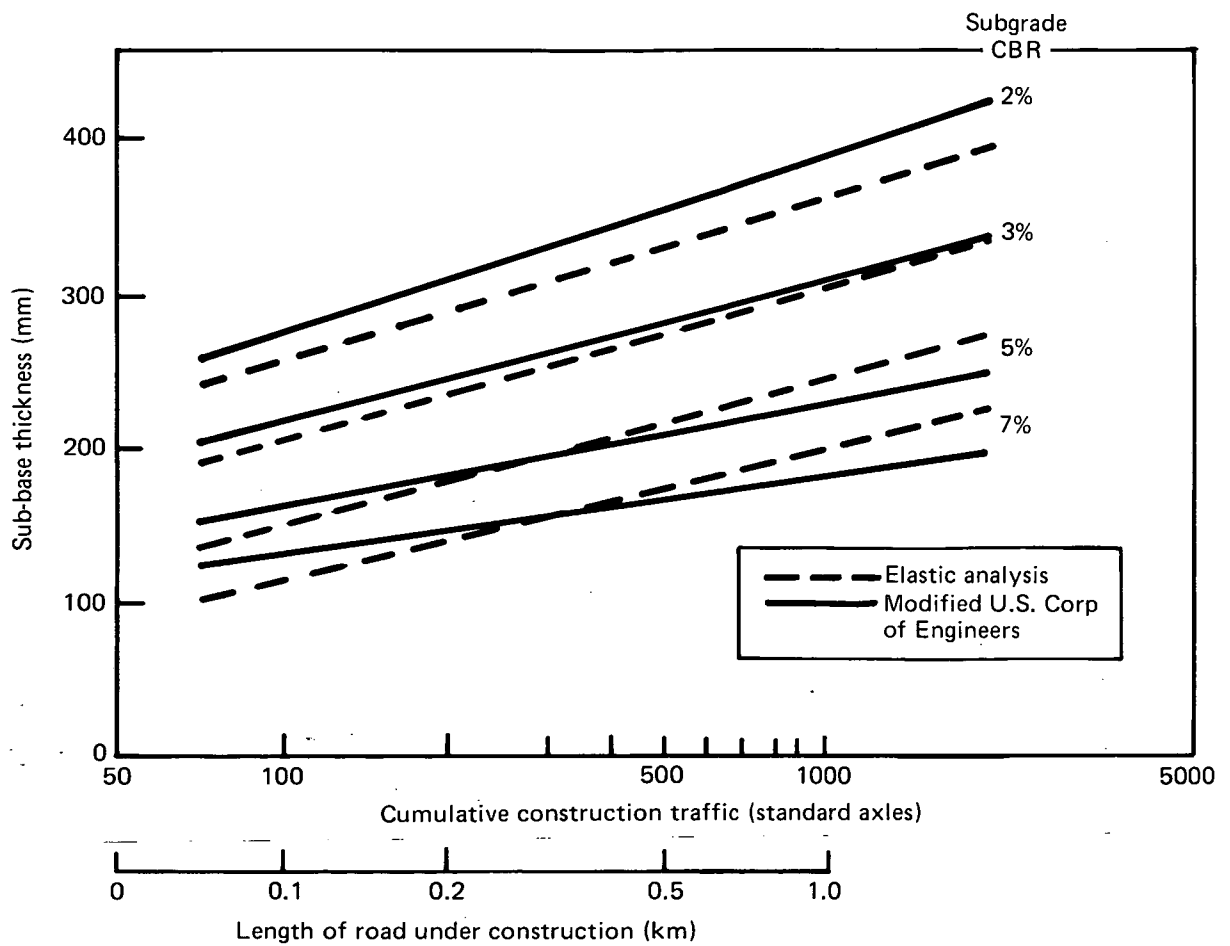


Fig. C3 Thickness of sub-base required to carry construction traffic

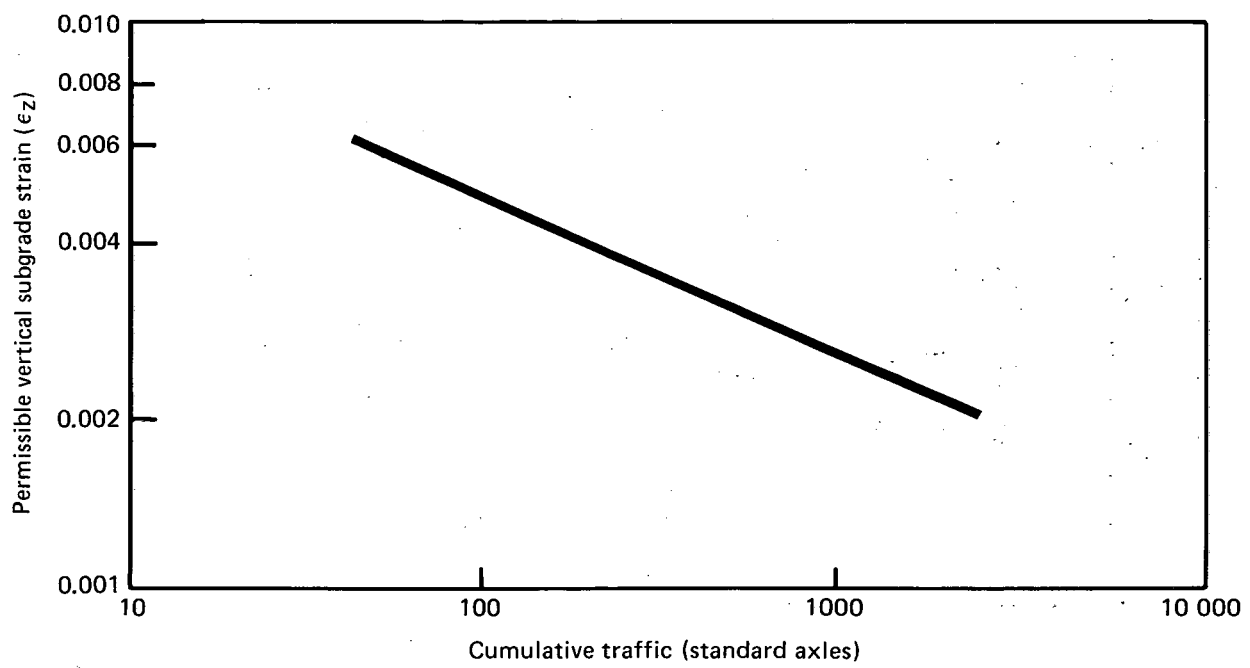
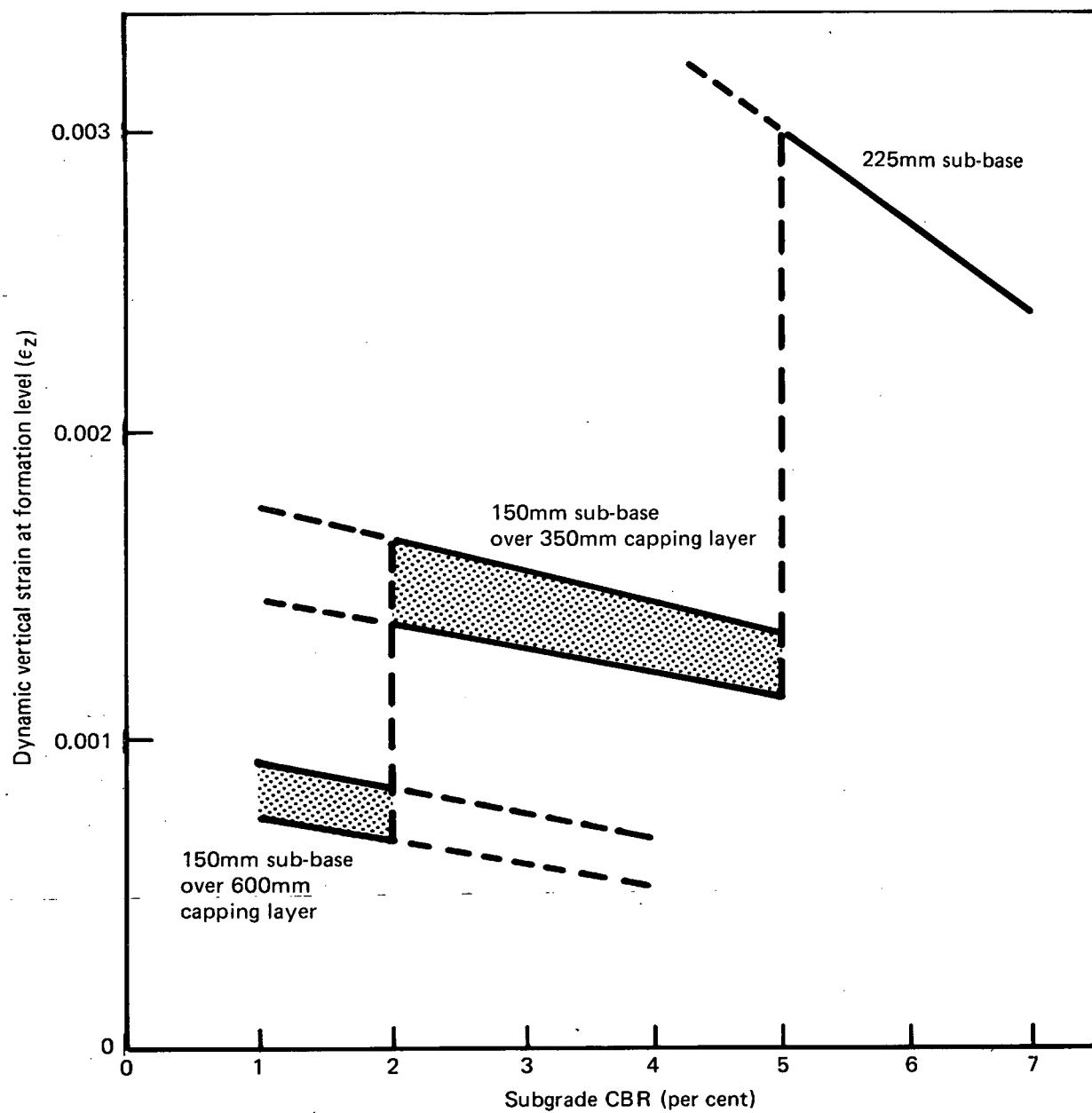


Fig. C4 Relation between permissible subgrade vertical strain and cumulative construction traffic



**Fig. C5 Dynamic vertical strain at formation level
beneath a 40kN wheel load**

16. APPENDIX D

ROADBASE DESIGN

The performance of experimental roads forms the basis of the design method and offers the means of adapting proven designs to take advantage of new materials and the development of new designs. This approach has proved satisfactory for bituminous roadbases, but corresponding design criteria for wet-mix and lean concrete roadbases are less well established, so that it is more difficult to adjust the standard designs for new situations.

The standard designs described in this work were based on the long-term performance of 144 sections of experimental road whose construction details and structural properties were carefully measured. The structural performance of each section was assessed by monitoring systematically the development of rutting and cracking of the road surface and by measuring deflection using a deflection beam. The rut depth in the nearside wheelpath was calculated from periodic measurements of the transverse profile at specific locations in the test sections and the development of rutting was related to the cumulative traffic in msa. The deflection measurements were made at these same locations.

Section 3 of the report defined life in terms of critical structural conditions whose onset is associated with a 10 mm rut or the beginning of cracking in the wheelpath. For those experimental roads still in sound condition, pavement life was predicted either from deflection measurements or from the average rate of rutting. Multiple regression techniques were then used to correlate pavement life with the thickness of pavement layers and CBR of the subgrade. From the resulting equations, designs for the roadbases were developed.

16.1 Bituminous roadbases

16.1.1 Dense roadbase macadam. Figure D1 summarises the results for those experimental roads containing dense roadbase macadam with 100 pen bitumen. It relates the thickness of bound layer to the design life with all the results adjusted to a subgrade CBR of 5 per cent and a thickness of granular sub-base of 225 mm. Predictions of fatigue life based on laboratory tests grossly underestimate the observed life in the road pavement but the functional form of the best fit curve in Figure D1 is in close agreement with that derived from laboratory fatigue tests reported by Goddard, Powell and Applegate (1978). The design curve proposed in Figure D1 is conservative in that only 15 per cent of the test sections reach a critical condition before the specified design life. Evidence from motorways and from full-scale tests under controlled conditions of wheel load and temperature in a road machine confirms that pavements would reach the design life given by Figure D1 without appreciable cracking. Onset of major deterioration in this type of pavement therefore appears likely to be by a combination of deformation and the beginning of fatigue cracking.

The effect of design CBR on pavement thickness for a sub-base of 225 mm is shown in Figure D2. For design CBRs of less than 5 per cent a capping layer will normally be used: low values of design CBR will generally be associated with low or lower values at the time of construction. When a capping layer is not employed a thickness of sub-base in excess of 225 mm will be required to carry the construction traffic, according to Figure C3. The final design thickness based on the equilibrium CBR value should then take into account this additional sub-base thickness. Similarly, where CBR values are greater than 5 per cent it may be worthwhile to reduce the thickness of sub-base required for construction and incorporate this change in the final design. Figure D2 can be used to carry out these adjustments in conjunction with the layer equivalence between bituminous roadbase and unbound granular sub-base materials given in Section 9.3.

Figure D2 also includes curves showing the expectation of reaching the design life lower than the 85 per cent probability used in the standard case. These curves can be used for the design of less important roads where greater risks of early failure can be tolerated, or where it is anticipated that openings for public utilities and the effect of road widening or realignment are likely to limit the effective life. In these cases the probability appropriate to the importance of the road should be used for the design.

16.1.2 Design criteria. From the curves in Figure D1 design criteria were derived that defined resistance to fatigue cracking at the bottom of the bituminous roadbase in terms of horizontal tensile strain and resistance to deformation in the subgrade in terms of the vertical compressive dynamic strain or stress. The two subgrade criteria represent conditions that are sufficiently similar for practical purposes; the stress criterion is however a function of the strength of the subgrade in terms of CBR, and therefore the simpler unique strain criterion has been adopted.

The design criterion for fatigue cracking was developed by first calculating the dynamic strains in pavements of various thicknesses. These were represented by a multi-layer elastic system subjected to realistic spectra of wheel loads, temperatures and temperature gradients in the bituminous layers, which were measured in the road experiment at Alconbury Bypass. This was part of an investigation of the variation of climatic conditions within the UK that demonstrated that there is no justification for adopting more than one set of temperature conditions, given the other uncertainties in the design process. The accumulation of fatigue damage under these repeated dynamic strains was then computed by means of Miner's hypothesis: the approach was developed by Thrower (1979). The fatigue relationships used were of the form obtained by laboratory testing over a range of temperatures and levels of dynamic strain. As noted in Section 8.1, the curves derived in the laboratory require considerable adjustment to match observed road performances.

An iterative computation of cumulative fatigue damage was then used to establish the equivalent temperature, and hence a representative stiffness of the full depth of bituminous material, that results in the same damage under a repeated 40 kN standard wheel load as was generated by the full range of combinations of wheel loads and pavement temperatures. The experimental relations between pavement thickness and critical life given in Figure D1 were then converted into a simple criterion represented by a curve relating dynamic tensile strain at the equivalent temperature to cumulative traffic in standard axles. The criteria representing 85 per cent probability of survival are shown in Figure D3 at the equivalent temperature of 20°C.

From the design curve of Figure D1, the corresponding criterion for deformation resistance of the subgrade was determined. The vertical compressive strains at formation level were established under the whole range of wheel loads and temperatures and the cumulative damage evaluated. The equivalent temperature of the bituminous layers giving the same cumulative damage was established and the experimental relation between thickness and traffic converted into the strain criterion curve shown in Figure D3.

16.1.3 Rolled asphalt roadbase. Performance data from experimental pavements with a rolled asphalt roadbase were indistinguishable from those in Figure D1 for dense roadbase macadam. Although rolled asphalt contains a harder binder, nominal 50 pen bitumen, than the dense bitumen macadam used in the experimental roads and also has a different aggregate grading, the range of stiffnesses of the two types of material in-service has been found to be very similar. This may be partly explained by the additional hardening of the binder that takes place soon after dense bitumen macadam has been laid. Analysis of cores cut from the experimental roads after several years in service has confirmed that the penetration at 25°C of recovered bitumen in a dense bitumen macadam will be reduced from its initial value at mixing of say 100 pen to about 30 pen, ie close to that for the bitumen in rolled asphalt, which hardens less after it is laid. Therefore the stiffness modulus for dense bitumen

macadam gradually increases for a year or so after being laid and eventually its value is similar to that of rolled asphalt.

Because the stiffness of rolled asphalt and dense bitumen macadam roadbases have been found to be broadly similar and their design thicknesses for a given traffic are also the same, there is equal expectation of yield developing in the subgrades under both types of roadbase. Rolled asphalt is however considerably less fatigue susceptible and the fatigue strain criterion for dense bitumen macadam has been modified for rolled asphalt on the basis of systematic laboratory testing; this curve is also shown in Figure D3. The mode of deterioration for rolled asphalt pavements is therefore by deformation.

16.1.4 Internal resistance to deformation of bituminous materials. Although Figure D3 defines permissible compressive strains at the top of the subgrade as being a parameter to control subgrade deformation, this does not take into account the deformation within the bituminous layers themselves, particularly in thick pavements. Assessment of deformation in the bituminous layers will be important for heavily trafficked roads and for the introduction of new bituminous materials. A model that predicts deformation in bituminous layers has been developed by Nunn (1984) from previous work by Thrower (1975) and Shell (1978). The model assumes that the pavement structure can be represented by idealised multi-layer viscous elements with the required viscous coefficients derived from laboratory creep tests. This model has been used to define a procedure to check that bituminous materials have adequate resistance to deformation. This procedure is based on the use of a uniaxial creep test to compare a new material with the material on which the standard design is based. The standard test procedure is similar to that described by Shell (1978) but in order to facilitate the comparison of the rates of deformation of different materials instead of a single test temperature, a range of 20°C–30°C is examined: the relationship between deformation and temperature for other materials cannot be assumed to be the same as that for conventional dense bitumen macadam.

16.2 Wet-mix roadbase

Results for experimental roads containing wet-mix roadbase and rolled asphalt were analysed in terms of rut depth and deflection in the same manner as for bituminous roadbase. The statistical analysis yielded the design curves given by Figure 5 of the main text; the curves are for a design CBR of 5 per cent and a sub-base thickness of 225 mm. For design CBRs of less than 5 per cent a capping layer will normally be used. When a capping layer is not employed a thickness of sub-base in excess of 225 mm will be required to carry construction traffic, according to Figure C3. In adverse conditions represented by 2 per cent CBR both at the time of construction and at design equilibrium, an extra 15 mm of bituminous surfacing and the same thickness of wet-mix are required, as well as the extra thickness of sub-base required for construction traffic. For CBRs greater than 5 per cent the reduction in design thicknesses are insignificant with a sub-base of appropriately reduced thickness.

An analytical approach to the design of wet-mix roadbases is uncertain because there is no authoritative failure criterion associated with yield within the wet-mix; the analysis is also complicated by the non-linear elastic behaviour of unbound granular material, which form the major structural layers of the pavement. If linear elastic behaviour is assumed for the wet-mix and the design criteria derived from the performance of dense roadbase macadam are adopted for this material, then the selection of an appropriate modulus for the wet-mix is crucial, as is demonstrated in Figure D4. For an elastic modulus of wet-mix of 250 MPa, fatigue is generally more critical than deformation and the predicted lives are much shorter than those observed in the experimental roads.

On the other hand, if the modulus is increased to 500 MPa the corresponding design curves are in good agreement with results of observed performance. A gradation of stiffness is assumed for the granular sub-base as described in Appendix C (15.2). This analysis demonstrates that the pavements designed to carry less than 20 msa would be expected to fail by deformation. Above 20 msa the designs may be inadequate in relation to fatigue cracking if the bitumen content of the base-course is less than that specified in British Standard BS 594 (1973b) for rolled asphalt.

Because of the uncertainties involved in applying the analytical method of design to wet-mix roads, any major change in the standard design would require testing of full-scale structures to confirm performance predictions.

16.3 Lean concrete roadbases

16.3.1 Designs for limited life. An analysis of the structural performance of the pavement test sections in the experimental roads yielded designs for limited life, with the roadbase gradually deteriorating under the influence of traffic and temperature. In all test sections the roadbase aggregate gradings were similar to the present specification, and the roadbase was laid either in a single layer or there was good bonding between separate layers. The surfacings consisted of rolled asphalt or dense macadam basecourse and rolled asphalt wearing course. In designing the roadbase, the critical life of the experimental roads was based either on visual conditions or predicted from deflection measurements. Regression analysis showed surface rutting to be less important in predicting performance, presumably because the ruts were associated mainly with the condition of the surfacing only; they were therefore not used in the analysis. Figure D5 relates the thickness of lean concrete roadbase to design life with all results adjusted to a surfacing thickness of 150 mm, a sub-base thickness of 225 mm and a subgrade CBR of 5 per cent; the 85 per cent confidence limit curve forms the basis of the design. The surfacing design in Figure D6 was based on two approaches. The inspections of visual condition provided evidence to identify the relationship between the thickness of surfacing and traffic carried when cracks were first observed at the surface. This resulting design in Figure D6 is more conservative and structurally more plausible than the alternative based on rutting measurements; Figure D6 also shows this latter thickness of surfacing necessary to prevent the rut depths from exceeding 10 mm with the data adjusted to a subgrade CBR of 5 per cent, a sub-base thickness of 225 mm and a design thickness of lean concrete as indicated by Figure D5.

Finally the roadbase thickness given by the 85 per cent probability of survival curve in Figure D5 was adjusted to match the recommended thickness of surfacing to produce the standard design curves for traffic up to 20 msa described in section 8.3; these are for a design CBR of 5 per cent and a sub-base thickness of 225 mm.

For design CBRs of less than 5 per cent a capping layer will normally be used. Such low values of design CBR are normally associated with lower values of CBR at the time of construction, so that when a capping layer is not employed a thickness of sub-base in excess of 225 mm will be required to carry the construction traffic, according to Figure C2. In the adverse conditions represented by a 2 per cent CBR, both at the time of construction and at design equilibrium, the extra thickness of sub-base required for construction traffic provides the additional design strength required and no extra thickness of roadbase and surfacing is necessary. Similarly for CBRs greater than 5 per cent the thickness of sub-base may be reduced but the thickness of roadbase and surfacing remains the same.

Construction practice limits the minimum thickness of both surfacing and lean concrete roadbase at low traffic levels.

For traffic in excess of 20 msa the thickness of lean concrete exceeds the maximum that can be laid in a single lift while maintaining satisfactory levels and good compaction. The performance of two-layer construction is more variable because of many instances of poor bonding at the interface between layers.

16.3.2 Design for long life. An intensive programme of coring and observation of surface cracking of experimental roads, after they had carried between about 8 and 20 msa, revealed that only the thickest sections and those with higher strength concrete had not cracked longitudinally. With one exception, as shown in Figure D7, those sections with lean concrete of a 28 day flexural strength in excess of the calculated combined horizontal tensile stresses due to traffic and temperature effects at the bottom of the lean concrete had not cracked longitudinally.

The traffic stress was calculated for a standard wheel load of 40 kN and a temperature of bituminous surfacing of 20°C; the thermal component of the stress, due to restrained warping, was calculated for a temperature gradient associated with a warm day using a method described by Thomlinson (1940) with allowance for the influence of the slab dimensions according to Westergaard (1927). The suggestion by Lister (1972) that flexural strength is a useful indicator of resistance to cracking, is strongly supported by the evidence from Figure D7, and this provides a basis on which to design pavements to avoid longitudinal cracks in lean concrete bases. The rapid reduction in the combined thermal and traffic stresses with increasing thickness of lean concrete is such that the lean concrete roadbase can be designed for a long but indeterminate life.

Table D1 shows the effect of thickness of roadbase and surfacing on the ratio of flexural strength to combined stresses within the lean concrete made with gravel aggregate. It was derived by computing traffic and thermal stresses for a wide range of pavement structures for a subgrade CBR of 5 per cent, a 225 mm sub-base and the same load and temperature characteristics as those adopted in calculations for the experimental roads. For all aggregates 200 mm of bituminous surfacing over 250 mm of lean concrete should be sufficient to resist cracking: provided the current minimum average compressive strength requirement of 11 MPa (corresponding to 1.2 MPa flexural strength) is met, and these thicknesses have been adopted for designs for long but indeterminate life. Table D1 indicates that these thicknesses imply some margin of safety: this was considered advisable in order to take into account changes in the relationship between strength and age of cement that have taken place since the road experiments were built.

TABLE D1

Ratio of flexural strength to combined thermal and traffic stresses
in underside of lean concrete roadbase (CBR = 5 per cent)

Mean Lean Concrete Flexural Strength at 28 days MPa	Bituminous Surface Thickness mm								
	100			150			200		
	Lean Concrete Roadbase Thickness mm								
	220	240	260	220	240	260	220	240	260
1.5	0.81	0.96	1.15	1.03	1.20	1.36	1.25	1.41	1.57
1.25	0.69	0.83	0.98	0.89	1.03	1.18	1.09	1.23	1.36
1.0	0.58	0.69	0.81	0.75	0.85	0.98	0.91	1.04	1.16
0.75	0.47	0.54	0.64	0.59	0.48	0.77	0.72	0.82	0.91

Where a capping layer is not used for design in adverse conditions, represented by a CBR of 2 per cent for both construction and design, an extra 12 mm of lean concrete is required in addition to the increased thickness of sub-base necessary for construction. On subgrades of greater than 5 per cent CBR some small reduction is possible: 12 mm for a CBR of 10 per cent associated with a sub-base of appropriately reduced thickness. The engineer may well consider these changes to be not worthwhile.

These thicknesses of lean concrete exceed the maximum standard that can be laid in a single lift. An increase in the present minimum 28-day compressive strength from 11 to 15 MPa should ensure a mean flexural strength of at least 1.5 MPa for all aggregates and might permit a reduction in the required thickness of lean concrete but there would be an associated increase in the width of transverse cracks. Another solution is suggested by the work of Potter, Sherwood and O'Connor (1984) on lean concrete with additions of pulverised fuel ash. There is therefore the possibility of single-layer construction that is sufficiently thick to carry the heaviest traffic.

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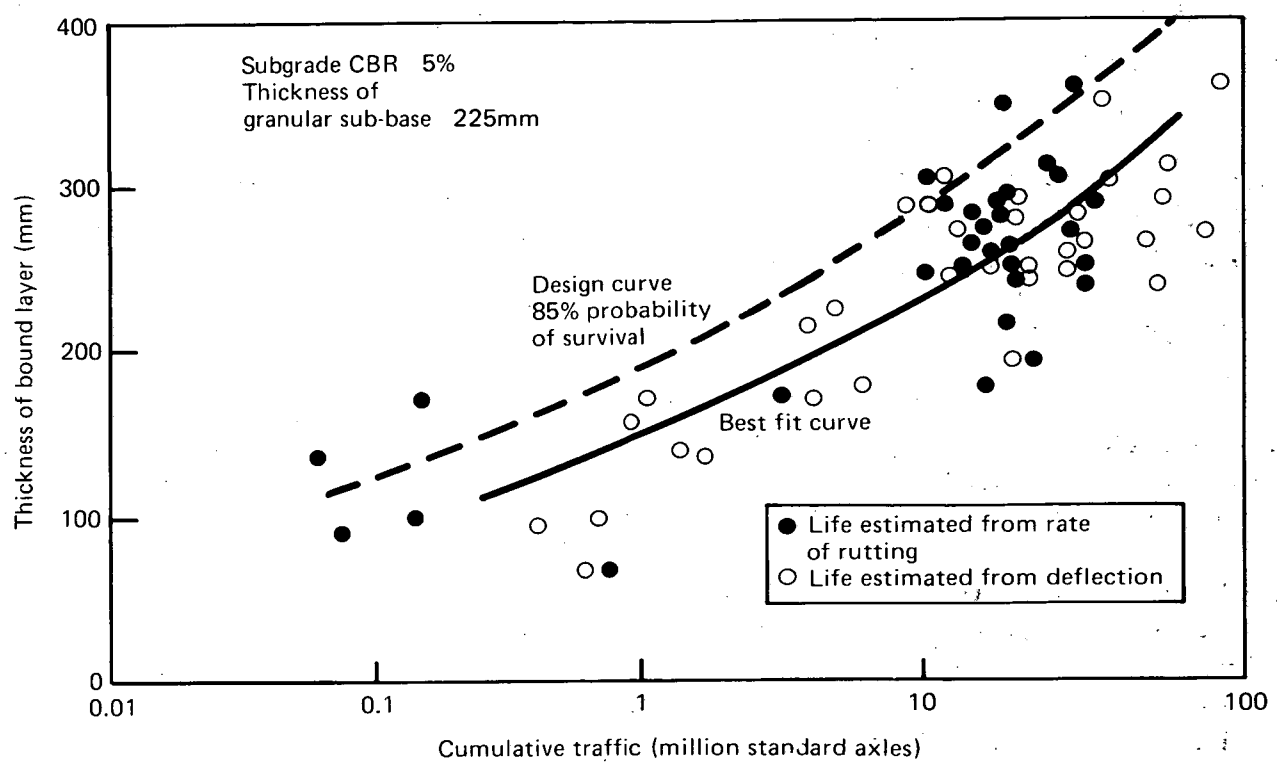


Fig. D1 Relation between thickness and life of experimental roads with dense bitumen macadam roadbase

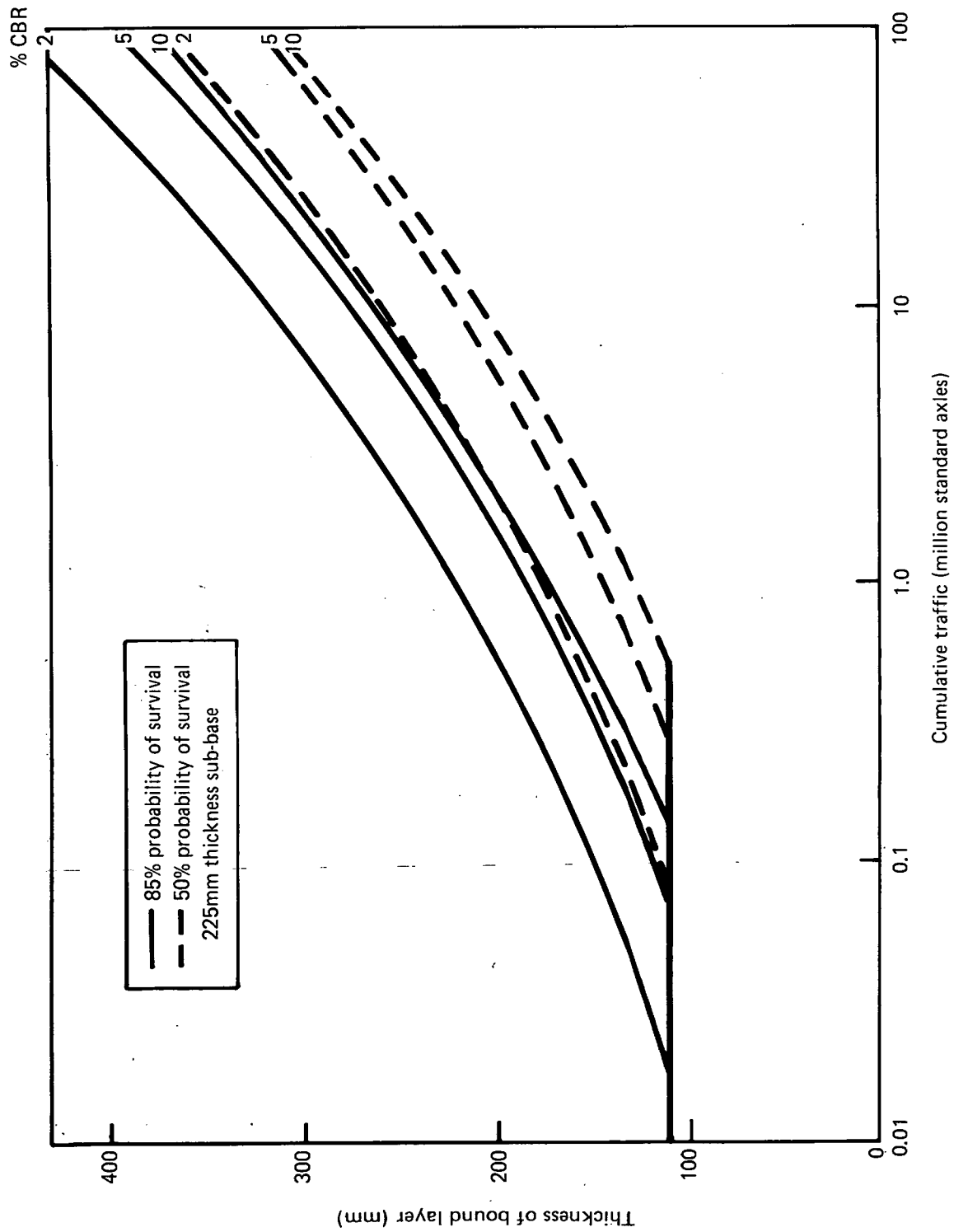
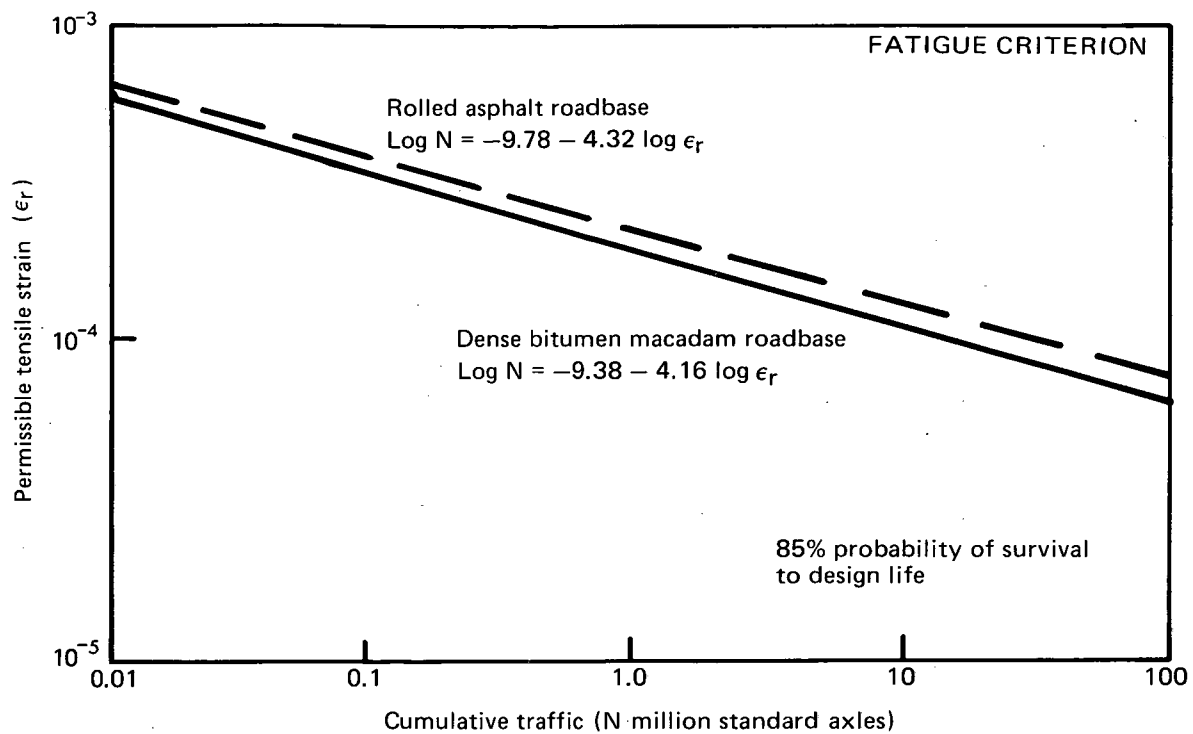
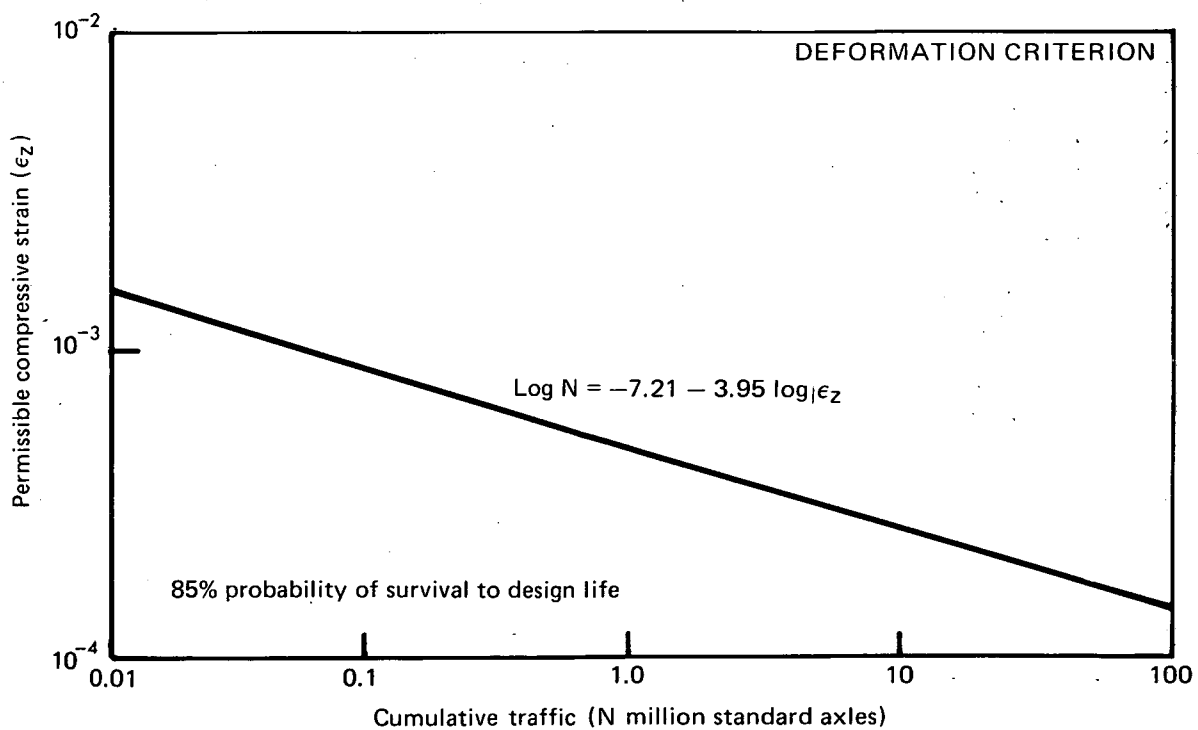


Fig. D2 Design curves for roads with dense bitumen macadam and rolled asphalt roadbase at different levels of subgrade CBR and probability of survival



(a) Bottom of the roadbase



(b) Top of the subgrade

Fig. D3 Permissible strains induced by a standard 40kN wheel load at a pavement temperature of 20°C

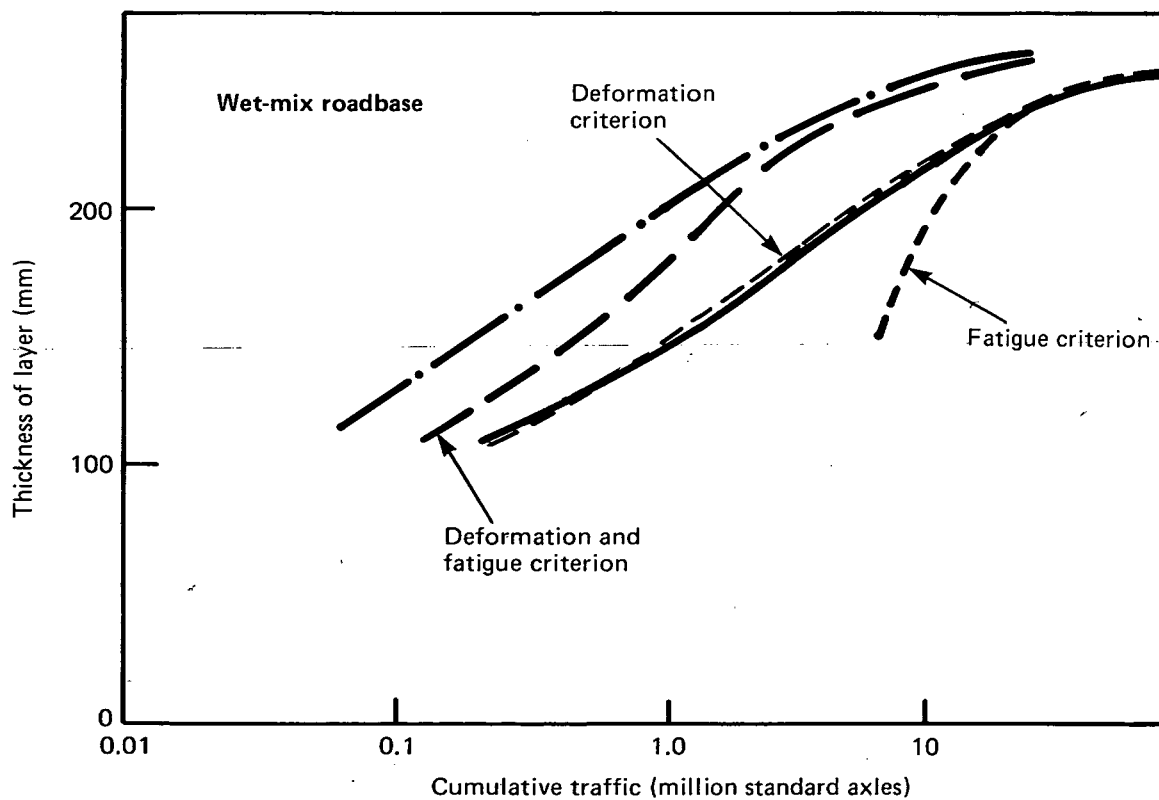
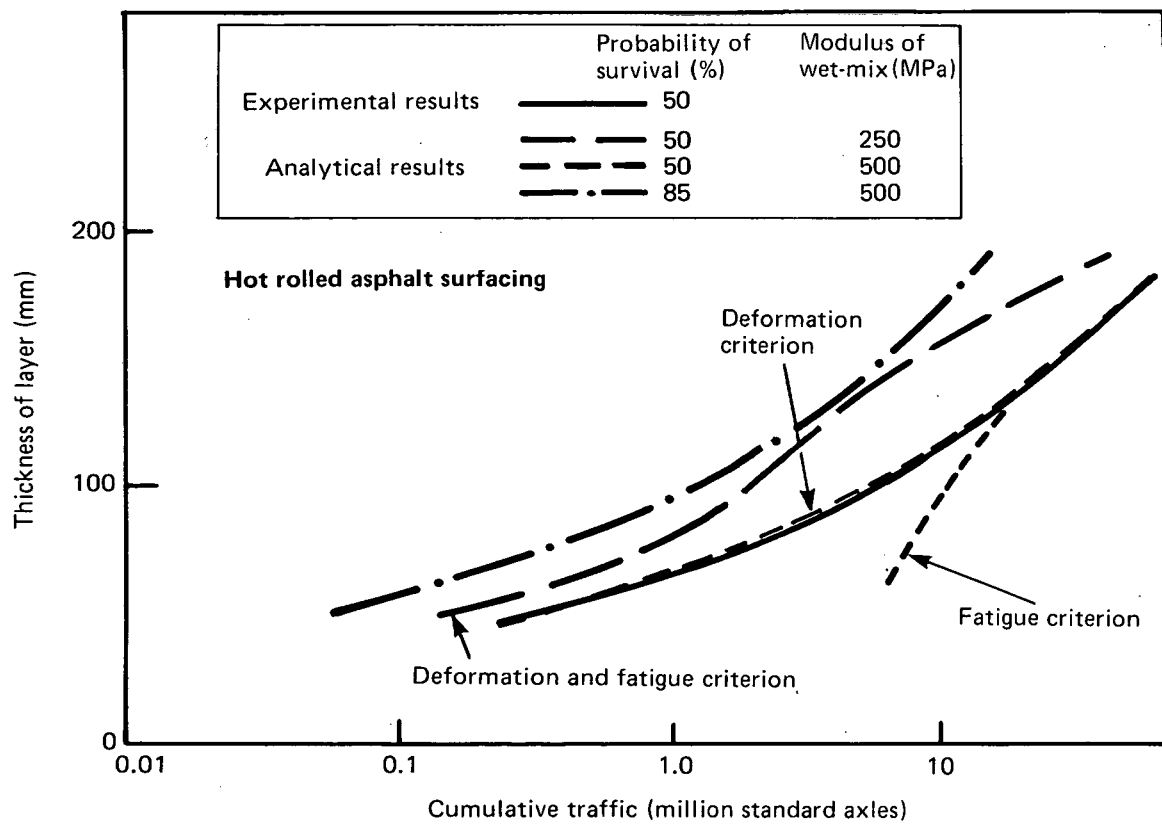


Fig. D4 Comparison of experimental and theoretical designs for wet-mix roadbases

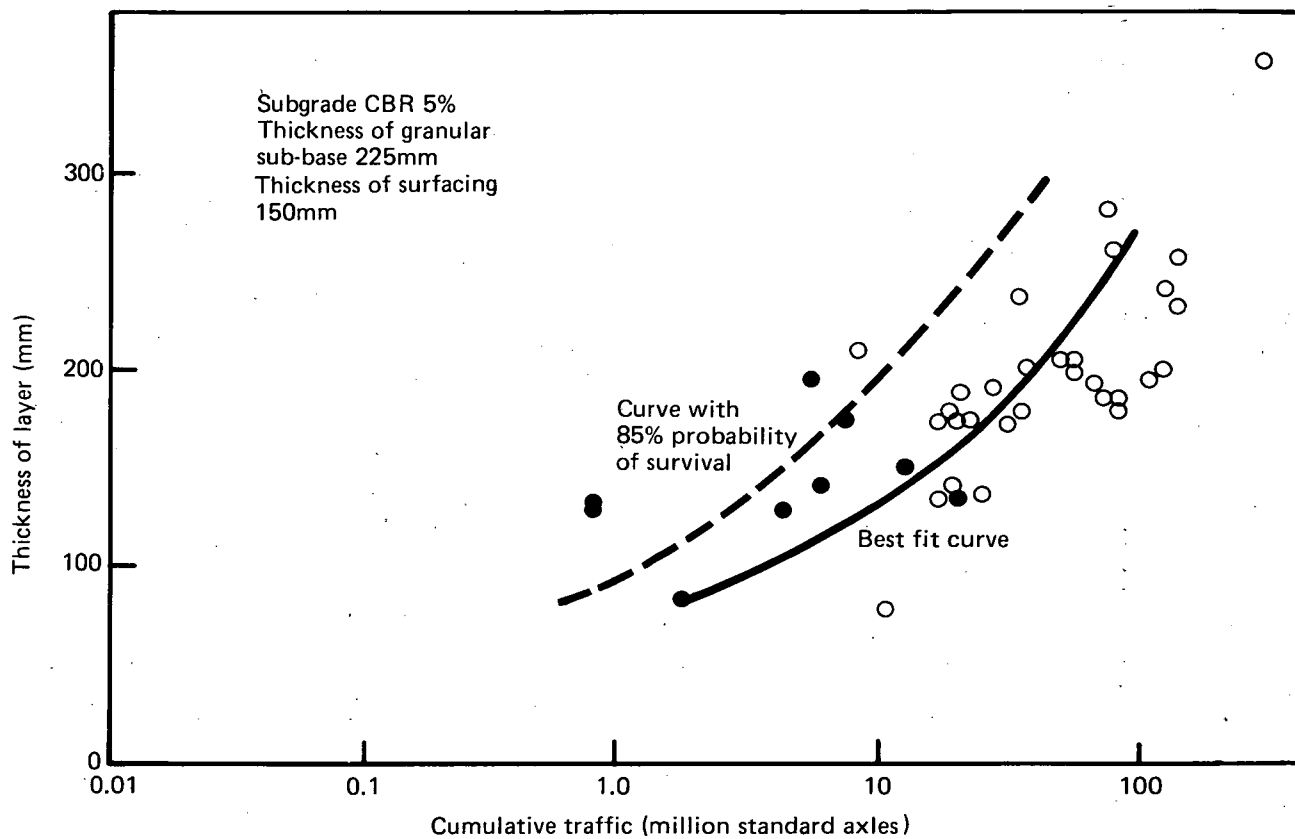


Fig. D5 Relation between thickness and life of experimental roads with lean concrete roadbase

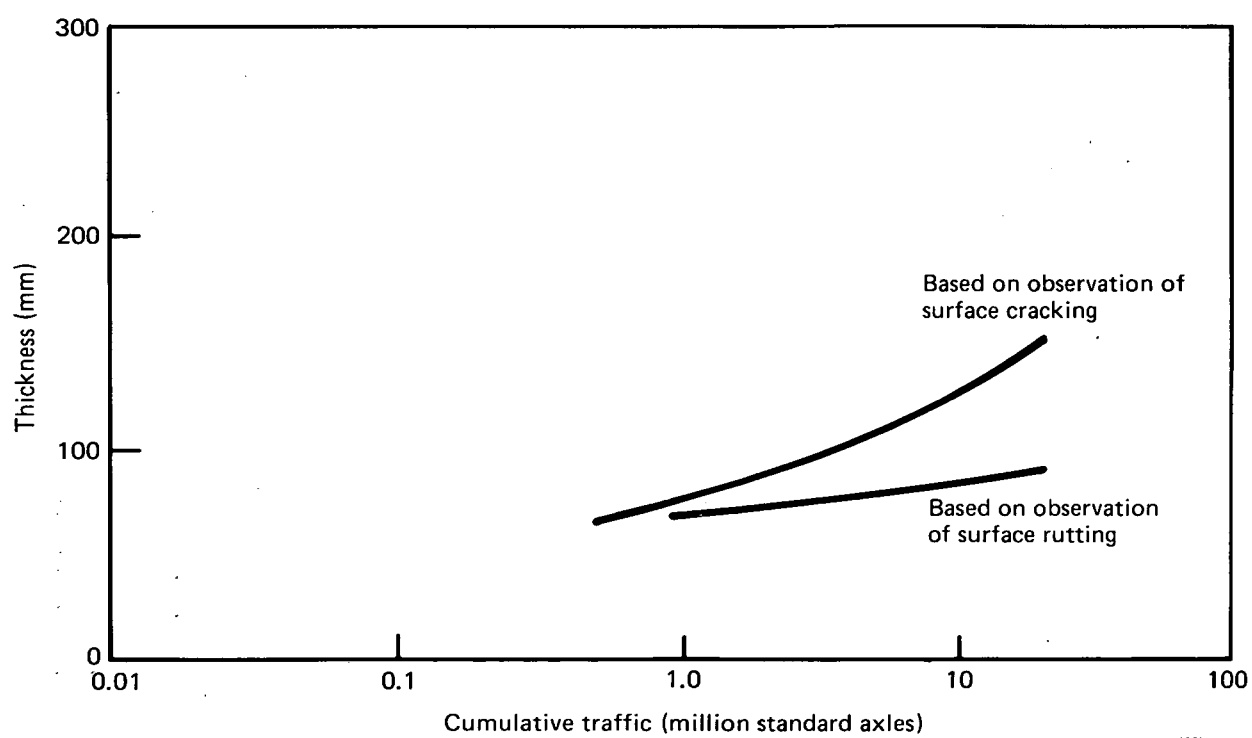


Fig. D6 Design of bituminous surfacing for roads with lean concrete roadbases



Fig. D7 Flexural strength as an indicator of ability to resist longitudinal cracking

17. APPENDIX E

INPUT DATA FOR THE DESIGN MODEL AND AN EXAMPLE OF APPLYING THE DESIGN METHOD

17.1 Data required for design models

To make best use of the design method, the structural properties of the pavement materials and subgrade must be known. The values assigned to each property to calculate critical strains in the standard designs are given below:

Bituminous material

Loading frequency	5 Hz
Equivalent temperature	20°C
Modulus of dense bitumen macadam (100 pen)	3.1 GPa
Modulus of hot rolled asphalt (50 pen)	3.5 GPa
Poisson's ratio	0.35

Fatigue criterion:

For dense bitumen macadam (100 pen) $\log N_f = -9.38 - 4.16 \log \epsilon_r$

For hot rolled asphalt (50 pen) $\log N_f = -9.78 - 4.32 \log \epsilon_r$

where N_f is the road life in standard axles and ϵ_r is the horizontal tensile strain at the underside of the bound layer under a standard wheel load.

Deformation criterion

$$\log N_d = -7.21 - 3.95 \log \epsilon_z$$

where N_d is the life of road in standard axles and ϵ_z is the vertical compressive strain at the top of the subgrade under a standard wheel load.

Sub-base (Type 1)

Modulus

The modulus of each layer that has been compacted separately is given by:

$$E_n = 3E_{n+1} \quad \text{for } E_{n+1} \leq 50 \text{ MPa}$$

$$E_n = 0.15 \text{ GPa for } E_{n+1} > 50 \text{ MPa}$$

where E_{n+1} is the modulus of the underlying layer and the upper limit for the thickness of a compacted layer is 225 mm.

Poisson's ratio	0.45
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Capping layer

Modulus	Range between 50 and 100 MPa.
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Poisson's ratio	0.45
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Subgrade (cohesive soil)

Modulus	$E = 17.6 (\text{CBR})^{0.64} \text{ MPa}$
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Poisson's ratio	0.45
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Standard wheel load

Load	40 kN
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Contact radius	0.151m
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17.2 Adapting the standard designs for bituminous roadbase to permit the use of a different grade of bitumen

The standard designs for bituminous roadbase were derived from the observed performance of experimental roads that incorporated dense bitumen macadam roadbase. The binder in the roadbase material was 100 pen bitumen. If the bitumen penetration is changed, the design criteria given in Section 8.1 offer the means to adjust the standard designs.

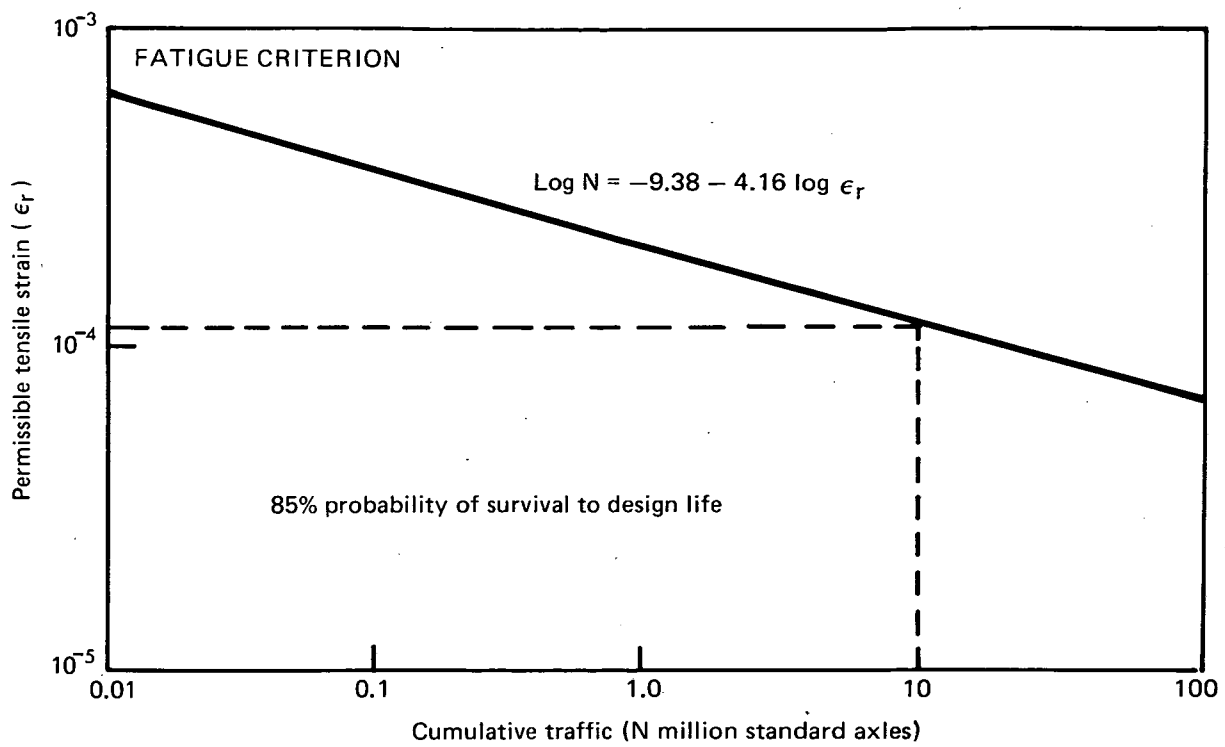
Leech (1982) has shown that a change in nominal bitumen penetration from 100 to 50 pen has no significant effect on the laboratory fatigue or deformation characteristics of the bituminous material, but increases the stiffness modulus of the material significantly. Because of variability in material composition and laying practice, this increase can vary considerably but an extensive programme of testing suggested that at 20°C the stiffness modulus is approximately doubled. For a given level of traffic, appropriate thicknesses of the new material can be calculated for each of these criteria using elastic theory and the permissible subgrade strains and tensile strains at the bottom of the bituminous material given in Figure E1.

The greater of these thicknesses would be selected for design purposes. For example, at 10 msa, the permissible strain in the subgrade at formation level given by Figure E1 is $2.5 \cdot 10^{-4}$ whereas the corresponding horizontal tensile strain at the bottom of the bituminous roadbase is $1.1 \cdot 10^{-4}$. For a standard wheel load of 40 kN, an effective temperature of 20°C and a foundation consisting of a subgrade of CBR 5 per cent and a 225 mm granular sub-base, elastic theory predicts that the thickness of bituminous material corresponding to these strain levels are 235 mm and 210 mm assuming that the stiffness modulus of the bituminous roadbase and basecourse is double that of dense bitumen macadam containing 100 pen bitumen. The effect of this change in stiffness modulus on thickness of bound layer over the full range of design traffic is shown in Figure E2. The subgrade strain criterion demands the greater thickness over the full range of traffic and is therefore appropriate for design purposes. For a given design life in msa, the introduction of 50 pen in place of 100 pen bitumen leads to a marked increase in stiffness modulus of the material and the appropriate reduction in design thickness of bound material is typically between 15 and 20 per cent. Alternatively the benefit could be taken as a longer life.

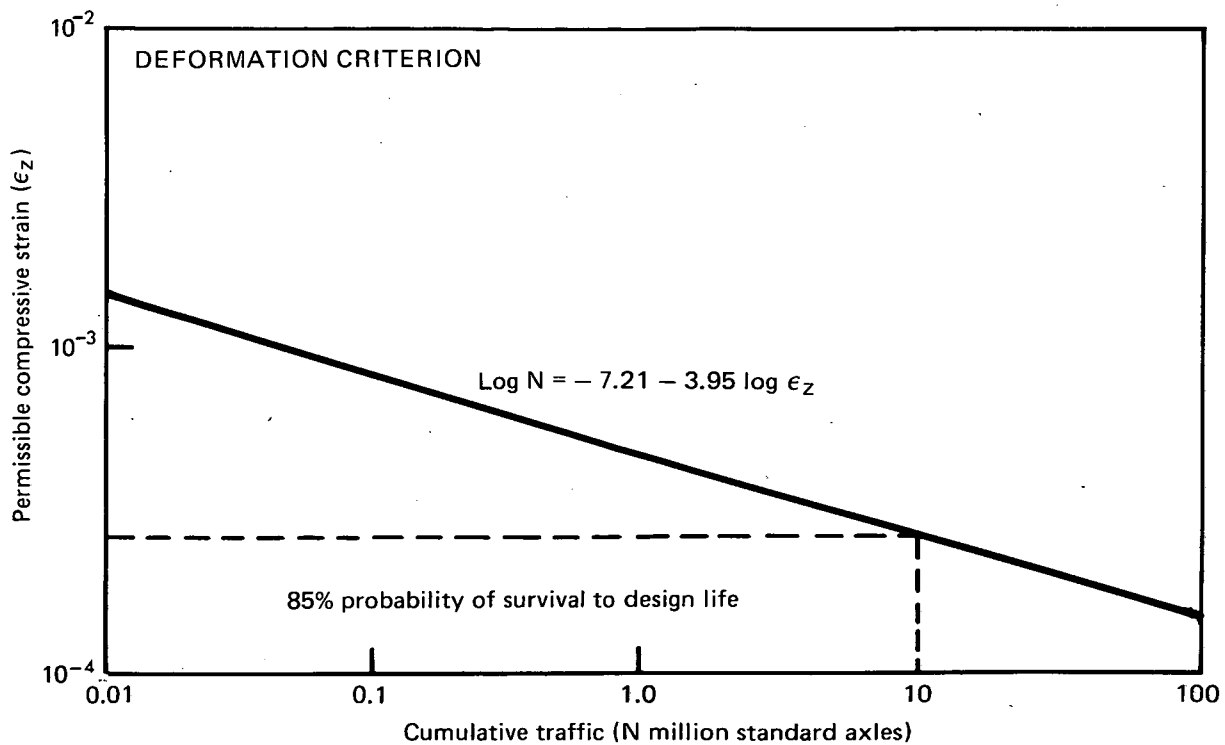
Figure E2 also shows the corresponding design curve if both roadbase and basecourse macadam contain 200 pen bitumen assuming that the stiffness modulus of this material at 20°C is half that for 100 pen bitumen.

17.3 Reference

LEECH, D, 1982. A dense coated macadam of improved performance. *Department of the Environment Department of Transport, TRRL Report LR 1060*. Crowthorne (Transport and Road Research Laboratory).



(a) Bottom of dense bitumen macadam roadbase



(b) Top of the subgrade

Fig. E1 Permissible strains induced by a standard 40kN wheel load at a pavement temperature of 20°C

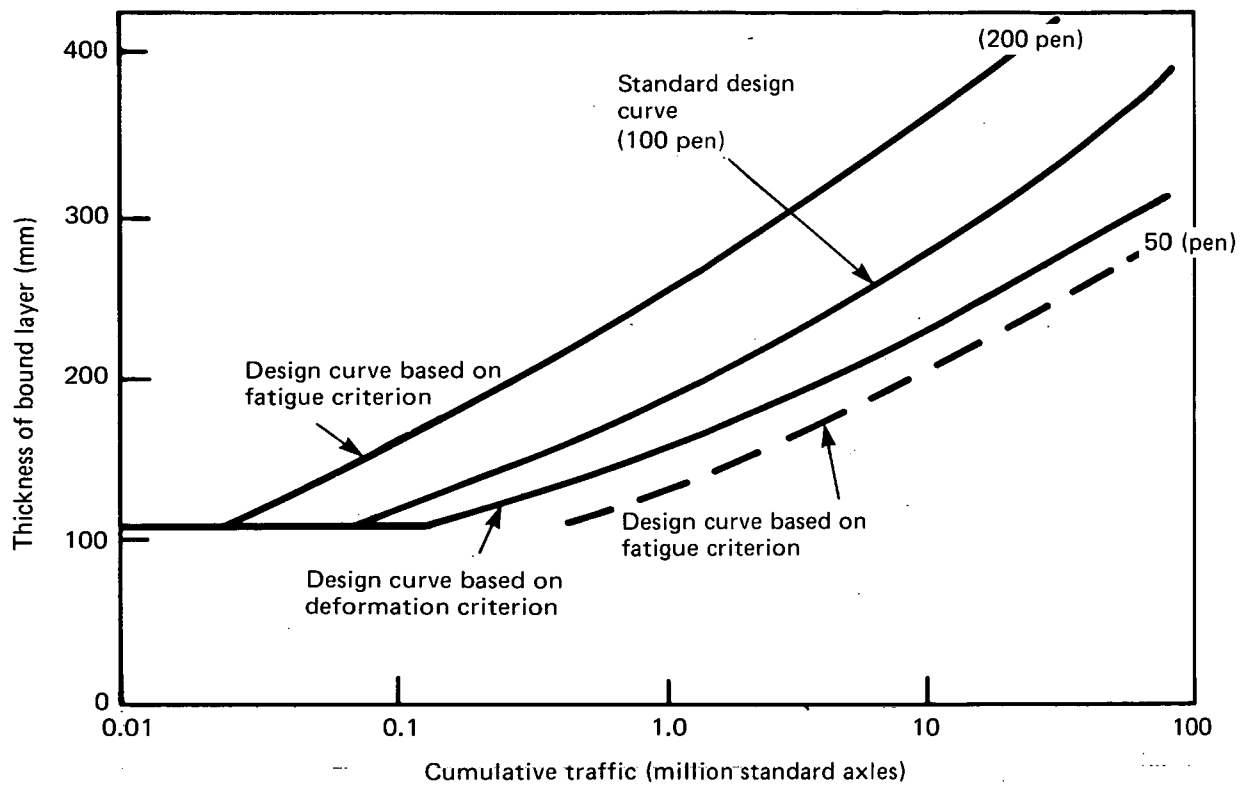


Fig. E2 Effect of change in penetration of binder on design thickness of pavement containing dense bitumen macadam roadbase and basecourse

ABSTRACT

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