TRANSPORT RESEARCH LABORATORY

Design of long-life flexible pavements for heavy traffic

Prepared for Highways Agency, British Aggregate Construction Materials Industries* and the Refined Bitumen Association

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Foreword

The current Department of the Environment,Transport and the Regions pavement design standards are described in Volume 7 of the Design Manual for Roads and Bridges (DMRB). For the particular case of fully flexible pavements, the Standard (DMRB 7.2.3) was established by considering the performance of a wide range of experimental pavements which formed part of the trunk road network. These pavements were constructed over a 20 year period starting in the early 1950s. The interpretation of the structural performance of these roads was based on theoretical design concepts. This led to a design method, developed in 1984 and described in TRRL Laboratory Report 1132, that took full advantage of theoretical methods of analysis, whilst making due allowance for their limitations.

A design life of 40 years was advocated in LR 1132 which was achieved for fully flexible pavements by strengthening the road after about 20 years. A detailed calculation of the cost of asphalt roads over 40 years, based on our understanding of pavement deterioration at that time, showed this to be the optimum economic solution. This calculation took into account variability of pavement performance, cost of traffic delays and other costs associated with maintenance and reconstruction. Since this method was introduced in the mid 1980s, traffic levels have continued to increase, with the consequent increase in user costs due to traffic disruption at roadworks. More recent work has indicated that it would be more cost effective to increase the design life of fully flexible roads for very heavily trafficked locations to at least 40 years, without the need for structural strengthening, to reduce future maintenance and the associated traffic delays.

Coupled with this, more technical knowledge has become available in recent years on the performance of heavily trafficked, asphalt roads. In particular, LR 1132 introduced a criterion that ensured that future roads were at no greater risk of fatigue cracking than roads constructed in the past. This was a conservative measure based on knowledge at the time. Subsequent experience has not detected evidence of fatigue cracking or damage in the main structural layers of the thicker, more heavily trafficked pavements. This, and other information, has indicated that deterioration, as either cracking or deformation, is far more likely to be found in the surfacing than deeper in the pavement structure; this evidence is in conflict with conventional theory. Also, it was found that the great majority of the thick pavements examined have maintained their strength or become stronger over time: they were not gradually weakened with trafficking. The overall conclusion of this project is that well constructed roads that are built above a threshold strength will have long lives provided that distress, in the form of cracks and ruts appearing at the surface, is treated before it begins to affect the structural integrity. Such roads should be referred to as *long-life* roads.

This is a major finding of this project, commissioned by the Highways Agency, British Aggregate Construction Materials Industries and the Refined Bitumen Association and carried out by the Transport Research Laboratory. A key element has been the contribution from other projects commissioned by sponsors but particularly those from within the Highways Agency's research programme. The Agency has provided access to the findings of completed projects and those emerging from current work with a total value of over £5 million, without which the conclusions from the project would have been far less robust. The detailed results of this project are contained in this report.

HIGHWAYS AGENCY

BACMI

Executive Summary

This report describes research sponsored by the Highways Agency, British Aggregate Construction Materials Industries and the Refined Bitumen Association. The overall objective was to review current design practice and information on flexible pavement performance that has accrued since the last revision of the design standards and to develop an improved design method for heavily trafficked, flexible pavements.

The main report is divided into sections with appendices being used to review much of the detailed work on which the main report is based. Short summaries are given at the end of the more important sections.

Design criteria and design concepts are reviewed in Section 2. The basic requirements to achieve satisfactory performance are defined. Information collected on the performance of trunk roads and motorways in the UK is discussed in relation to current pavement design concepts which assume that the main forms of structural deterioration are either fatigue, progressively weakening the structural layers, or structural deformation originating in the subgrade. This perception of gradual structural weakening has led to the concept of a *critical condition,* which is considered to be the last opportunity when the existing strength of the road can be used to good effect in the design of a strengthening overlay.

Section 2 is supported by Appendix A which summarises the ***evidence of deterioration*** and demonstrates that fatigue and structural deformation are not prevalent in well constructed roads provided that they are designed and built with sufficient strength to resist structural damage in their early life. Testing material extracted from old roads has established that the structural properties of asphalt change substantially over the life of the road and that these changes, which are referred to as *curing,* are crucial in understanding its behaviour. They help to explain why conventional deterioration mechanisms do not occur and why, provided the road is constructed above a minimum threshold strength, it will have a long structural life of 40 years or more. The investigations demonstrated that these *long-life* roads will deteriorate by rutting and cracking of the surface layers, but that this deterioration will not lead to serious structural deterioration, provided that it is treated in a timely manner.

In Sections 3, 4 and 5, the main elements of the road, i.e. the foundation, the structural layers and the wearing course, are considered in turn.

The design of the foundation is considered in Section 3 and, in particular, the requirements relevant to long-life roads. Experience has shown that foundations constructed to the current specifications are generally adequate, although they do not offer the Contractor the flexibility of an end-product specification. The advantages of an end-product specification for the road foundation, based on measured stiffness, are discussed. Results of pilot-scale and full-scale trials are summarised and comparisons of stiffness measurements using plate bearing tests are presented and the implications of adopting an end-product performance specification for the road foundation are outlined. The results of the foundation investigations are reported in more detail in Appendix B.

The design of the structural layers, presented in Section 4, recognises that pavement behaviour is complex and that it is not possible to quantify precisely all factors affecting pavement performance. The threshold level of pavement strength to achieve long-life, suggested by the results from experimental pavements, is adjusted conservatively to take account of higher present-day and future traffic flows, the possibility of surface cracking and the proposed increase in the maximum axle load. The methodology, described in Appendix C, leads to the conclusion that the existing design curves should be used for cumulative traffic up to 80 msa and that no additional thickness will be required for higher traffic levels. Less than 200 mm thickness for the asphalt layer is not recommended, even for lightly trafficked trunk roads, if these are to endure for 40 years or more.

Section 5 covers issues involved in the ***selection of wearing course materials*** for long-life roads. The wearing course serves several functions and no material is ideal for all situations. The various factors that affect performance are discussed and special reference is made to materials that are considered to be best suited for surfacing long-life roads. Section 5 is supported by Appendix D, which summarises the performance of numerous road trials involving bitumen modifiers used in hot rolled asphalt and porous asphalt.

There is frequently a need to adapt standard designs for use in new situations. Section 6 gives examples to illustrate the application of the design method. Consideration is given to introducing new or improved asphalt roadbases, developing designs for areas that are at high risk of surface rutting and building stronger roads in a limited construction depth.

Section 7 describes the ***construction of trial sections*** of the M65 motorway that were built according to the principles of long-life design. Thin wearing courses and high modulus base (HMB) were used and test methods were applied that have the potential for use in an end-product performance specification for the road foundation. The results of the various tests carried out on the pavement layers and the foundation are presented.

Road pavements will not survive for 40 years without regular checks on their condition and timely intervention to carry out the appropriate maintenance treatments. At some stage, maintenance will be required when the wear of the pavement is judged to adversely affect its structural integrity or the standard of service required by the road user. In Section 8 it is recognised that many of the existing procedures for pavement monitoring and investigation should continue but with changes in emphasis to reflect the different requirements of long-life roads.

In Section 9, an economic assessment of long-life pavements was carried out using the Highways Agency's

whole-life costing program, COMPARE. The whole-life costs of long-life roads are compared with the costs of conventional thicknesses to achieve a 40 year structural life. The possible reduction in costs resulting from maintaining the road by inlaying at night is discussed.

Risk and uncertainty are inherent in all construction projects. In Section 10 it is recognised that, generally, the relationship between materials and construction variables and pavement performance cannot be defined with any great precision. However, a qualitative analysis of risk for flexible pavements can be carried out by listing risks together with a qualitative assessment of how they may be reduced. The risks inherent in changing the design methodology and the risks incurred both during construction and during the life of long-life pavements are discussed.

The design method and concepts described in this report will have a considerable impact on the way pavement life, pavement deterioration mechanisms, pavement condition assessment and pavement maintenance are viewed. A key element has been the contribution from other projects commissioned by sponsors but particularly those from within the Highways Agency's research programme. The Agency has provided access to the findings of completed projects and those emerging from current work with a total value of over £5 million, without which the conclusions from the project would have been far less robust. For example, the current method of pavement condition assessment needs to be modified to be compatible with long-life designs. The changes will need to be managed in an orderly manner. The main areas of change are identified in Section 11 together with a discussion of the route to implementation.

The main conclusions of this work are that a well-constructed, flexible pavement built above a defined threshold strength will have a very long structural service life provided that distress, in the form of cracks and ruts appearing at the surface, is detected and remedied before it begins to affect the structural integrity of the road. There is no evidence that structural deterioration due to fatigue or cracking of the asphalt roadbase, or deformation originating deep within the pavement structure exists in roads that conform with the criteria for long-life.

1 Introduction

This report describes research sponsored by the Highways Agency, British Aggregate Construction Materials Industries and the Refined Bitumen Association. This study was enhanced by being able to draw on information from previous work jointly funded by the sponsors and from extensive research programmes that the Transport Research Laboratory has undertaken on behalf of the Highways Agency over many years.

The overall objective of this research was to review current design practice and information on pavement performance that has accrued since the last revision of the design standards in order to develop an improved design method for heavily trafficked, flexible pavements.

The pavement design method for fully flexible pavements that has been used in the United Kingdom since the mid 1980s was established by considering the performance of a wide range of experimental pavements which formed part of the trunk road network. These pavements were constructed over a 20 year period starting in the early 1950s. The method developed was based on an interpretation of the structural performance of these roads in terms of theoretical design concepts. This led to an analytically based design method, described by Powell et al (1984) in TRRL Laboratory Report LR 1132, that took full advantage of theoretical methods of analysis, whilst making due allowance for their limitations.

A design life of 40 years was advocated, which was achieved for fully flexible roads by strengthening the road after about 20 years. A detailed calculation of the cost of asphalt roads over 40 years, taking into account the variability of pavement performance, cost of traffic delays and other costs associated with reconstruction, showed this to be the optimum design strategy that minimises the costs over 40 years. Since this method was introduced in 1984, traffic levels have continued to increase, with the consequent increase in traffic disruption at roadworks. More recent work has indicated that it would be more cost effective to increase the design life of fully flexible roads for very heavily trafficked locations to at least 40 years, without the need for structural strengthening, in order to reduce future maintenance and the associated traffic delay costs. As a result of this, the option to consider a 40 year design life for heavily trafficked locations, that would not require major structural strengthening, was introduced in 1994. This option was based on a further extrapolation of pavement performance trends upon which the earlier designs were based. This requirement to extend design curves to accommodate much higher traffic levels without the need for costly intervention to carry out structural maintenance has prompted a review of the design standard.

Designs for future traffic levels have to be based on observations of pavement performance under the traffic levels experienced in the past. If the requirement is to design future roads for similar traffic flows, then this will not present a problem. However, if the anticipated future growth rate is substantially greater than historical levels, and there is economic advantage in designing the road for a longer structural life than in the past, then the problem is how can our current knowledge be used to best effect in designing future pavements expected to carry much higher volumes of traffic. In the previous revision of the design standards there was a mismatch between experimental data, covering traffic levels up to about 20 million standard axles (msa) and the design requirement for the most heavily trafficked motorways of several times this amount. Therefore, it was necessary to assume that the measured performance trends could be extrapolated to give realistic estimates of future performance. This problem is not overcome by using mechanistic models of deterioration because these models can only be calibrated and validated using performance data from present day roads. Their use then becomes a more sophisticated method of extrapolating performance trends. The question that needs to be considered is whether these extrapolations represent the best means of predicting future performance.

In the UK, the present position is that the most heavily trafficked motorways and trunk roads have carried in excess of 100 msa and now provide the opportunity to confirm the validity of the earlier extrapolations. Coupled with this, over the last 12 years more information has become available on the fundamental behaviour of road materials that can help to explain the observed performance of roads. The method described in LR 1132 introduced criteria to limit the degree of deterioration that was expected to occur deep within the pavement structure. These criteria ensured that future pavements were at no greater risk of fatigue cracking and subgrade rutting than the roads constructed in the past. This was a conservative measure based on incomplete knowledge at the time.

Subsequent investigations commissioned by the Highways Agency, entitled *Deterioration mechanisms for thicker flexible pavements and effects on design and maintenance*, have failed to detect evidence of deterioration in the main structural layers of the thicker, more heavily trafficked pavements (Leech and Nunn, 1997a). These have indicated that deterioration is far more likely to be found in the surfacing than deeper in the pavement structure. Also, it was found that the great majority of the thick pavements examined have maintained their strength or become stronger over time, rather than gradually weakening with trafficking as assumed in the current pavement assessment method based on deflection measurements (Kennedy & Lister, 1978).

This study has examined the requirements of all the component layers of the pavement to achieve good service. An end-product performance specification for the road foundation, the design of the main structural layers and the roles of various types of wearing course material are considered. Design concepts are reviewed and up-to-date information on pavement performance from full-scale experimental pavements are presented. Studies have been made of deterioration mechanisms occurring on the road network, of long-term deflection monitoring of motorways and of condition assessment reports prepared to aid the design of structural maintenance. This information is required to produce a design method for roads expected to last at least 40 years without the need for structural strengthening. Such roads will be referred to as *long-life* roads.

Throughout this report, the word asphalt has been used as a generic term for a bituminous mixture. However, at times it is used to describe a specific material, for example, hot rolled asphalt or asphalt concrete.

2 Design considerations

The mechanical behaviour of road materials under traffic is too complex to model in detail and a rigorous validated mechanistic method of pavement design has yet to be developed. The analytically based methods that have been developed generally use simple linear elastic theory to determine stresses or strains at critical locations in the pavement structure and these are compared with maximum permissible values to determine whether the pavement is adequate to carry a given amount of traffic. The permissible values are obtained from the back-calculation of stresses and strains for structures that are known from experience to perform well. This approach ensures that the method gives realistic predictions for conditions similar to those under which it was developed and it provides a rational procedure for adapting designs to new situations or for introducing improved materials.

To give satisfactory service, the road must also be well constructed using good quality materials to ensure that potential problems are not built into the road structure and that the pavement satisfies a number of performance criteria. The more important of these are:

a the road must be able to sustain traffic without excessive deformation,

b the asphalt layers must not crack under the influence of traffic and of climate,

c the load spreading ability of the road foundation, consisting of granular sub-base and capping layers, must be adequate to provide a satisfactory construction platform,

d the finished road surface must provide the road user with a smooth ride and maintain good skid resistance.

After a road designed for long-life has been constructed, its condition must be checked at regular intervals and any deterioration must be remedied before it has any serious influence on the structural integrity of the road.

These requirements to ensure that a road has a long structural life will be discussed later, but first, consideration must be given to the mechanisms of deterioration that control performance.

2.1 Deterioration mechanisms

The objective of pavement design is to control the observed mechanisms of deterioration. The design method for fully flexible pavements, developed by Powell et al (1984) and reported in LR 1132, was based on the assumption that asphalt roads deteriorate due to the accumulation of small amounts of damage caused by the passage of each commercial vehicle. This method considered that the damage mechanisms were either a fatigue phenomenon that caused a gradual weakening and eventual cracking at the underside of the roadbase, or structural deformation originating in the subgrade.

Information is reviewed, in Appendix A, on the performance of fully flexible pavements that form part of the trunk road and motorway network in the United Kingdom. This information comes from full-scale experimental roads, studies of deterioration mechanisms on the road network, long-term deflection studies and condition assessment reports prepared to aid the design of structural maintenance. No evidence of conventional roadbase fatigue or structural deformation was found in well constructed flexible pavements. The observed rutting and cracking were found to originate in the surface layers and, as long as it is treated in a timely manner, it is unlikely to lead to structural deterioration. The Highways Agency are currently sponsoring research to develop improved mechanistic models of flexible pavement behaviour to gain greater insight into the way flexible pavements deteriorate.

One of the findings of this research was that changes occurring in asphalt over the life of the road are crucial in understanding its behaviour. These changes, which are referred to as *curing*, can help to explain why conventional mechanisms of deterioration do not occur and why, provided the road is constructed above a minimum threshold strength, it should have a very long, but indeterminate, structural life. The increase in the stiffness of the asphalt roadbase causes the traffic-induced strains in the pavement structure, which control fatigue and structural deformation, to decrease with time. Therefore, a road will be more vulnerable to structural damage in its early life, before curing has increased the structural strength of the material. If the road is designed and constructed with sufficient strength to prevent structural damage in its early life, it has been found that curing doubles the stiffness of DBM roadbase in the first few years in service and this will substantially improve the overall resistance of the pavement to fatigue and structural deformation. The improvement in the bearing capacity of the road, as determined by deflection measurements, provides confirmation of this improvement.

2.2 Design life

In LR 1132, Powell et al (1984) adopted an approach to the design life that minimised costs over the life of the road. More recent work by Bowskill (1993) and Abell (1993) indicated that it might be more beneficial to set the design life of fully flexible roads greater than the 20 year initial structural life set at present.

The future requirement is to design roads for higher traffic levels than those accommodated by the current design method. It seems reasonable to base future designs, for these higher traffic levels associated with longer design lives and increased traffic flow, on a further extrapolation of the design curves given in LR 1132. In fact, confidence in the robustness of the extrapolation is increased by the form of the design curves; large increases in life are achieved for relatively modest increases in pavement thickness. Although extrapolation is fairly reliable, the reliability of the designs inevitably diminishes as the design curve moves further away from the range of the data on which the relationship was based. Information on the performance of roads that have carried cumulative traffic of up to about 100 msa, given in Appendix A, shows that it is not necessary to continuously increase pavement thickness in order to achieve longer life provided that the pavement is sufficiently thick and well constructed so that problems are not built in from the outset. Existing roads that do not fulfil these conditions will require strengthening in order to achieve long-life.

In LR 1132, the road was designed for the onset of a critical condition after about 20 years. This was defined as the latest time when the application of a strengthening overlay could be expected to make best use of the original structural quality of the pavement in extending its life for a further 20 years. Lister (1972) linked the onset of the critical condition with rutting in the wheel path of 10 mm or the occurrence of cracking in the wheel paths. This was based on extensive comparisons between the surface condition and the structural integrity of relatively thin, lightly-trafficked constructions built predominately in the 1950s and 60s. Lister also demonstrated that these roads gradually weakened with cumulative traffic and that the remaining life of the road, to the onset of the critical condition, could be determined from a knowledge of the cumulative traffic the road had carried and by measurement of the deflection of the pavement under a standard wheel load.

The review in Appendix A found no evidence that this concept of critical condition could be applied to thicker, well-constructed roads. This has important and far reaching implications for the design and the condition assessment of fully flexible pavements.

2.3 Traffic

In LR 1132, traffic is defined in terms of the cumulative number of standard 80 kN axles to be carried over the design life of the road. The same approach has been used in this work. The estimated daily 24 hour commercial vehicle flows have to be converted to annual flows and allowance made for the proportion of commercial vehicles travelling in the nearside lane. The annual numbers of commercial vehicles are then multiplied by the estimated damaging effect of an average commercial vehicle, the vehicle wear factor, to give an estimate of the cumulative number of standard axles. The details of this calculation are given in Design Manual for Roads and Bridges (DMRB 7.2.1).

The fourth power law is used in the determination of the vehicle wear factor. This empirical law, which relates the structural damage in the road to the fourth power of the wheel load, was a major development of the AASHO Road Test (Highway Research Board, 1962). However, more recent work on the comparison of trafficked pavements in accelerated test facilities (OECD, 1991) recognises that this law is only a general description and approximation of the relative pavement damaging power of axle loads. Wide variations to this general rule were found with the powers varying between 2 and 9 depending on the degree and mode of deterioration and the condition of the pavement at the time the comparison was made. The design criteria used in LR 1132, which have a fourth power dependence on the traffic induced strains at critical locations in the road, provide some justification for this law. However, for thicker roads, in which the mechanisms of structural deterioration that form the basis of the design criteria are not observed, this law is unlikely to be applicable. A conservative estimate, given in Appendix C, indicates that present designs for cumulative traffic greater than 80 msa can be considered to be long-life roads.

2.4 Pavement construction

Good engineering practices and quality control are essential requirements for pavements designed for long-life in order to ensure that the construction does not contain potential problems. For example, many of the problems associated with poor compaction of asphalt have been eradicated with the introduction of site testing to control the level of compaction (Powell & Leech, 1987) and, more recently, a specification clause has been implemented to prevent a deformation susceptible roadbase macadam being produced that contains an excessive amount of bitumen (Clause 929, MCHW 1). Failure to do this may result in asphalt roadbases with inferior structural properties.

Most of the more common materials problems could be eliminated by specifications based on the relevant performance properties. These would provide assurance that pavement layers and materials are properly designed for the functions that they are intended to perform. End-product, performance-based specifications for asphalt wearing course and roadbase are currently under development (Nunn & Smith, 1994). The development of a performance specification for the pavement foundation is considered in this report. Its introduction would result in foundations being constructed to a more uniform standard without weak areas that may result in localised poor performance.

The qualitative process of identifying possible risks associated with construction and the sensitivity of these risks to changes in the pavement variables are examined in Section 10.

2.5 Summary

1 Well constructed asphalt pavements built above a threshold strength will have a very long life.

2 Deterioration in these pavements is confined to the uppermost layers of asphalt and manifests itself as surface cracking or deformation.

3 Fatigue and structural deformation are not prevalent in these structures.

4 Good construction practice and materials are required to ensure long-life. Furthermore, regular monitoring and timely remedial action are essential to prevent surface deterioration affecting the structural integrity of the

pavement.

5 The structural properties of the asphalt roadbase change considerably over the life of the road. This *curing* results in an overall improvement in the load-spreading ability of the roadbase, which can explain why thicker pavements have a *long-life*.

3 Pavement foundations

There is an interdependence between all layers of a flexible road construction. Structural layers, designed for long-life, will be ineffective without good foundation design. Stiffer road foundations will contribute to smaller pavement deflections under a loaded axle and reduced traffic induced strains in the pavement structure. One method of ensuring an adequately stiff foundation is to specify its surface stiffness as an end-product requirement for the completed foundation. Although the advantages of specification by end-product have been generally recognised, the problem has always been to find a practicable method of measuring the end-product.

As part of this project, test methods for measuring two properties that can be related to foundation performance, and have good potential to be used in an end-product specification, were assessed. The most appropriate properties identified for specification purposes were the compacted density of the foundation layers and the surface stiffness. The test methods selected to measure these properties were the nuclear density test (NDT) (British Standard BS 1377: Part 9, 1990) and the Dynamisches Plattendrückgerät, referred to in this report as the portable dynamic plate bearing test (PDPBT), which was developed in Germany (Kudla et al, 1991). Both these methods are fast, cost-effective and practicable. Appendix B describes trials carried out to investigate the potential of these methods and discusses the practical implications of adopting an end-product specification.

Experience in the UK has demonstrated that foundations constructed to the prevailing specifications are generally adequate, although they do not offer the Contractor the flexibility of an end-product specification. Until an end-product specification is fully developed and tested, the current advice for subgrade assessment and foundation design must remain but more emphasis needs to be placed on factors, such as drainage, that influence foundation strength over 40 years. The current foundation design procedures are summarised in this Section.

3.1 Foundation design

Foundations are designed to withstand loading from construction traffic, and to act as a construction platform for the laying and compaction of subsequent layers. In addition, the foundation protects the subgrade, should adverse weather occur during construction, and contributes to the structural strength of the completed pavement. All UK foundations are designed to HD25/94 (DMRB Vol.7). This design is based upon a relationship derived from the performance of experimental roads (Powell et. al. 1984) and was confirmed by results obtained from trafficking experiments on unsurfaced roads by Potter and Currer (1981) and Ruddock, Potter and McAvoy (1982).

3.1.1 Thickness

The thickness of capping and sub-base is obtained from

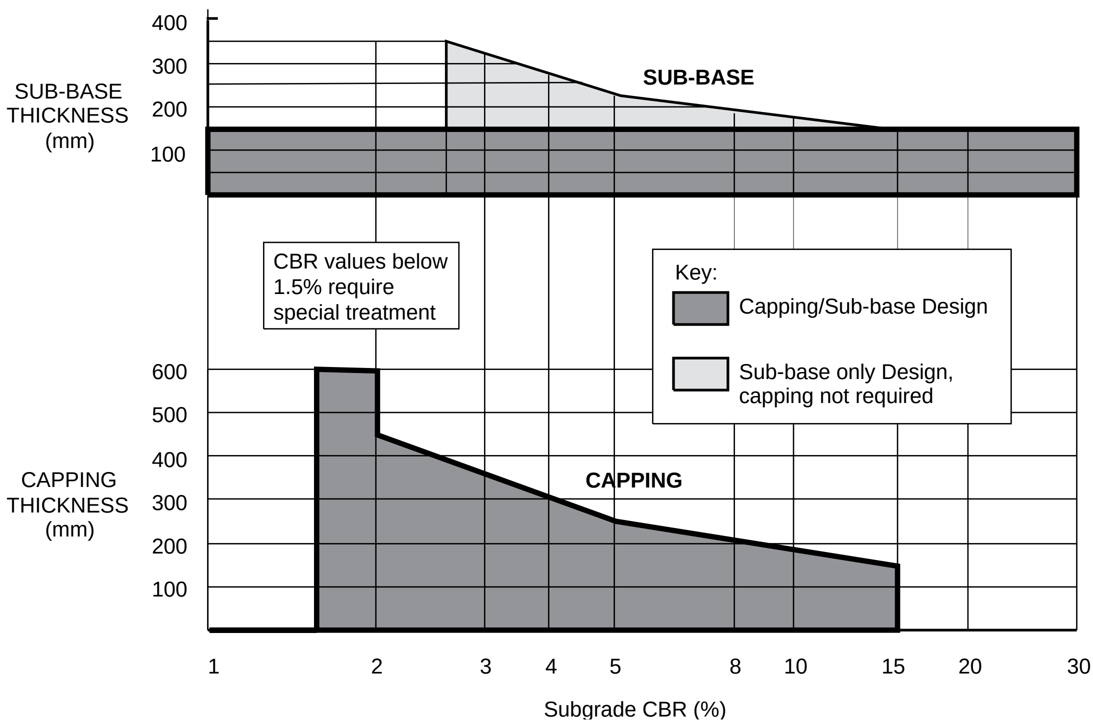

Figure 1 Capping and sub-base thickness design

Figure 1. The subgrade design CBR used should be the lower value of either the equilibrium CBR or the construction CBR. The method of subgrade assessment for the design of long-life pavements is the current method described in HD25/95. For subgrades with a CBR of greater than 15% a sub-base thickness of 150 mm is required, as this is considered to be the minimum thickness to ensure satisfactory compaction. When the CBR is between 2.5 and 15%, there are two options available:

- 150 mm of sub-base can be used on a thickness of capping that depends on the subgrade CBR value, or
- an increased thickness of sub-base can be used again depending on the subgrade CBR value.

All pavements to be constructed on a subgrade CBR below 2.5% must use the first option. For subgrade CBRs of less than 2%, there are a number of treatment options available prior to the construction of the foundation. These are described in more detail in HD25/94 (DMRB Vol.7).

The curves shown in Figure 1 (reproduced from DMRB 7.2.2) are used to design foundations to limit the deformation to a maximum of 40 mm for 1,000 passes of a standard axle. This is the maximum that can be tolerated if the sub-base surface is to be reshaped and recompacted effectively and serious rutting is to be avoided in the subgrade. The robustness of these foundation designs was demonstrated in a pilot-scale trial carried out at TRL as part of this project. In this trial, which is described in Appendix B, 1,000 standard axles of construction traffic produced ruts depths that were much less than the permissible maximum of 40 mm. Thinner foundations exhibited greater rutting and stronger designs produced as little as 2 mm rutting after 1,000 axles. This work and the earlier studies by Potter et al (1981) and Ruddock et al (1982) demonstrated that the design curves given in Figure 1 are robust.

3.2 End-product performance specification

3.2.1 Introduction

Current practice for the construction of pavement foundations employs a method specification which defines the materials to be used for each of the constituent layers of the foundation and how they should be compacted. This can restrict the Contractor’s choice of materials and prevent him from seeking more economic solutions. An end-product specification based on the specification of foundation requirements at both formation level and top of the foundation will provide the Contractor with greater scope in the use of his materials, improve consistency and ensure that the requirements of the foundation are met. A move to end-product specifications would require the Contractor to have a greater responsibility for the quality of the foundation produced.

From the review of practices elsewhere, and of current UK practice (Appendix B), the next development in the construction of road foundations is likely to be a move toward end-product specification.

To assess the performance of a foundation in-situ, it is necessary to test appropriate characteristics. Laboratory tests can provide values of the resistance to permanent deformation of a material but, there are currently no satisfactory in-situ methods of directly assessing resistance to permanent deformation. However, Chaddock and Brown (1995) have demonstrated that there is a broad relationship between foundation stiffness and its subsequent performance. It is therefore proposed that the surface stiffness of the foundation be measured together with the state of compaction of the constituent materials. To ensure consistent quality of construction for a long-life pavement, minimum values for the state of compaction and elastic stiffness are best specified at formation level and on the completed foundation.

3.2.2 State of compaction

Construction of any layer of the pavement requires the constituent materials to be well compacted to ensure good performance. The in-situ measurement of dry density would indicate the state of compaction, when compared with the maximum dry density achieved using a standard test.

The dry density of compacted, in-situ foundation materials can be referred to as a percentage of the maximum dry density achieved in a standard laboratory test (BS1377: Part 4, 1990). The in-situ, bulk density can be measured using two methods:

Nuclear Density Gauge - A small radioactive source is placed in the foundation material with a detector positioned on the surface. The intensity of radiation reaching the detector can be related to the bulk density of the material, the higher the bulk density the lower the intensity of radiation. This is a quick and efficient way of measuring in-situ bulk density from which dry density can be calculated. Further details of the testing procedure are given in BS1377: Part 9 (1990).

Replacement methods - This requires the removal of a sample of compacted material which is then replaced by a material of known density (commonly sand). Details of this procedure are given in BS1377: Part 9 (1990). The mass of the material needed to fill the hole is known, therefore the volume can be calculated. The mass of the sample of foundation removed can then be measured and its density calculated. The method is relatively slow and labour intensive and is not recommended for routine measurements.

3.2.3 Elastic stiffness

Foundation trials reported by Chaddock and Brown (1995) have been carried out as part of a Highways Agency project (Project title: *Tests for pavement foundation design and assessment*). These trials, in which the elastic stiffness of foundations were measured prior to trafficking with a loaded lorry, have produced a broad relationship between the stiffness and the performance of the foundation.

There are a number of different devices available which claim to measure, either directly or indirectly, the stiffness properties of the foundation in-situ and these are considered in Appendix B. The results from these devices cannot be compared directly because of the differing areas loaded, stresses applied and duration of loading pulses. All these factors will affect the elastic stiffness obtained for the completed foundation due to its stress dependent nature and the differing properties of constituent materials.

The Portable Dynamic Plate Bearing Test from Germany (Dynamisches Plattendrückgerät) (Kudla et al, 1991) was selected as a possible method of end-product testing for the foundations of long-life pavements. It is quick and easy to operate and it is used elsewhere in Europe. It was evaluated alongside a more widely used device - the Falling Weight Deflectometer.

3.2.4 Results of full scale trial

Results from the pilot-scale trial, described in Appendix B, suggested that average values of 30 MPa and 50 MPa for elastic stiffness using the PDPBT and 95 and 97% of the vibrating hammer maximum dry density (BS1377: Part 4, 1990), are appropriate for the capping and sub-base layers respectively. Following this pilot-scale assessment, a full-scale trial was carried out on the M65 under contractual conditions to examine the practicability of an end-product system, and to identify potential problems to be resolved before this form of specification can be introduced.

This trial demonstrated that a combination of specifying density and elastic stiffness at both the formation and top of the completed foundation could be used as the basis of an end-product specification for foundations. This would remove the Contractor’s obligation to use specific materials and methods of compaction.

However, more information is required from a wide range of materials and site conditions before authoritative end-product criteria can be defined. The pilot-scale trials carried out as part of this research indicated that minimum thicknesses of the foundation layers may be required even though the measured elastic surface stiffness may be adequate. A foundation incorporating a thin, stiff layer may suddenly fracture under the higher stresses induced by construction traffic and its load spreading ability would reduce dramatically resulting in a failure.

These, stiff layers would probably remain intact under the low stresses induced by the end-product test and consequently a relatively high, unrepresentative surface stiffness would be measured.

Generally, provided that they are not seriously degraded by construction traffic, stiffer materials resulting from treated layers enable the thickness of the foundation layers to be reduced (Chaddock and Atkinson, 1997).

3.3 Construction practice

Potential foundation problems can be avoided by adopting good construction practice. The following issues should be considered during foundation design and construction:

- subgrade drainage
- application of suitable compactive effort
- intrinsic material properties (grading, moisture content etc.)
- the amount of construction traffic to be carried.

Bound foundation materials, asphalt substitution and full-depth pavements, can be considered for long-life pavements. The use of stronger foundation materials, such as CBM1 and CBM2 and the strengthening of Type 1 with cement can also be considered for a long-life pavement foundation design. These materials will reduce the risk of damage from construction traffic especially under adverse wet weather conditions.

3.4 Summary

1 Road foundations constructed to the current specification are generally adequate for long-life pavements.

2 Trials have demonstrated that surface stiffness and dry density can be used as the basis of an end-product performance specification for the road foundation.

3 The portable dynamic plate bearing test (PDPBT) and the nuclear density test (NDT) are practicable and economic methods that have potential to be used in an end-product specification.

4 Structural layers

The deterioration of thick, well-constructed, fully-flexible pavements is not structural and generally occurs at the pavement surface as cracking and rutting. Evidence presented in Appendix A shows that changes which occur in asphalt over the life of the road are crucial to understanding why roadbase fatigue and structural deformation are not the prevalent modes of deterioration. These changes, which are referred to as *curing*, can explain why a road that is constructed above a minimum threshold strength should have a long, indeterminate, structural life. The behaviour of pavements and the implications for the design of *long-life*, fully flexible, pavements are considered in more detail in Appendix C.

Pavement behaviour is very complex and it is not possible to quantify precisely all the factors affecting pavement performance. Therefore, the threshold level of pavement strength to achieve a long but indeterminate life, which is suggested by TRL experimental pavements, is adjusted conservatively in Appendix C. These adjustments take into account higher traffic flows, the possibility of surface cracking, and the proposed future increase in the maximum legal axle load that were not fully accounted for in the experimental pavements.

4.1 Design thicknesses

The analysis given in Appendix C leads to the conclusion that roads do not need to be constructed thicker than that required by the current standard for 80 msa to achieve a very long structural life. The proposed new designs, using standard roadbase macadams, are illustrated in Figure 2; they relate the thickness of asphalt, laid on a foundation

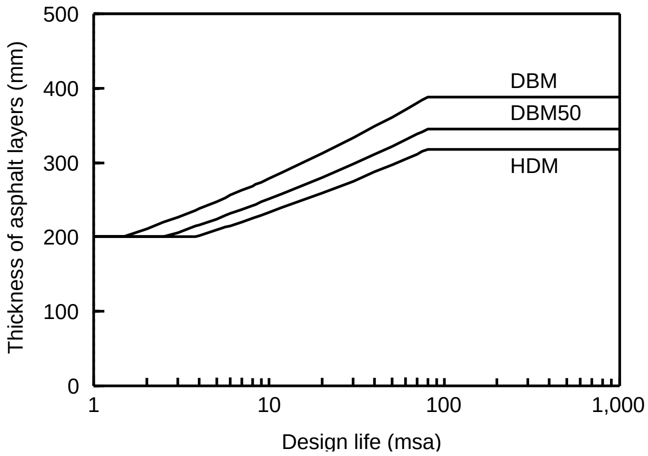

Figure 2 Design curve for roads with asphalt roadbase

equivalent to or better than 225 mm of Type 1 sub-base laid on a subgrade CBR of 5 per cent, to the cumulative traffic predicted over a 40 year period. A foundation of this quality is expected to have a surface stiffness somewhat greater than 50 MPa, measured using the portable dynamic plate bearing test. Further work is required to define this criterion more precisely. Designs involving improved roadbase materials are dealt with in Section 6.1.

The designs given in Figure 2 are for a 40 mm thickness of conventional rolled asphalt wearing course with a basecourse or upper roadbase material having nominally similar material properties to those of the lower roadbase. If an alternative wearing course material or layer thickness is used, then the overall design thickness of the asphalt may need to be adjusted. These adjustments are dealt with in the Section 5.4.

The life of a road constructed to the proposed standard is not known, but it is expected to be at least 40 years. Premature structural failure may be brought about by poor construction practice, or by failure to remedy surface distress. Therefore, to ensure a long-life, the pavement has to be well constructed and procedures need to be developed in which regular inspections of pavement condition are carried out and timely action taken to remedy any surface deterioration detected. This requires defining methods of monitoring at the network level, intervention levels for detailed investigation and preventative maintenance.

A thickness of less than 200 mm for the asphalt layer is not recommended for even lightly trafficked roads that are required to endure for 40 years. Thin roads will be at risk of structural deformation and the rapid propagation of any surface initiated cracks through the full thickness of asphalt (Schmorak & Dommelen, 1995; Leech & Nunn, 1997).

4.2 Design criteria

Appendices A and C indicate that elastic stiffness modulus, which is a measure of load spreading ability, is the most important structural property of the roadbase and that the calculated level of stress and strain induced in the pavement structure when it is first open to traffic is the main indicator of future performance. Most of the current information on the initial elastic stiffness modulus of asphalt roadbase, and its subsequent behaviour over the life of the road, is obtained from measurements using the indirect tensile stiffness modulus (ITSM) test (Cooper & Brown, 1989) to British Standard Draft for Development 213 (British Standards Institution, 1993). This test method is an important development. It is an economic and practical method of measuring the structural properties of asphalt. The Highways Agency are currently sponsoring work that has led to improving the precision of the ITSM test and currently a draft end performance specification (Clause 944), involving this test, is being developed for asphalt roadbase and basecourse in a series of road trials. Trials carried out on the M53, M56, A2 and A13 have demonstrated the ITSM test method to be practical and realistic and subject to further successful trials later this year, the Highways Agency will be in a position to implement Clause 944.

At the present time, information on the initial stiffness of a wide range of asphalts and its subsequent behaviour over the life of the pavement is incomplete. To remedy this, the Highways Agency have made resources available to help in the systematic collection of information on the structural properties of roadbase macadams.

In the meantime, the design method developed by Powell et al (1984) can be used to establish criteria for long-life, as well as for the thinner designs that have limited life. In this method *effective in-service* stiffness values are used. At the time this method was formulated, the detailed curing behaviour of asphalt roadbase was not understood and the values used for DBM were determined by testing a large number of sawn beams of materials extracted from roads of various ages using a 3-point bending test. Furthermore, the reference sinusoidal loading frequency was 5 Hz using this test compared to a frequency of approximately 2.5 Hz using the ITSM test. Although the 3-point bending test can obtain very high quality data, it is not practicable to use it to obtain the quantity of information required to rapidly assess material and to investigate a phenomenon such as curing. The indirect stiffness modulus test, although restricted to measuring stiffness at a lower effective load frequency, has been demonstrated in a previous research contract jointly sponsored by the Highways Agency, British Aggregate Construction Materials Industries and the Refined Bitumen Association to be a practical and economic method that is ideally suited to this role (Nunn & Bowskill, 1992). In Section 6, the use of this test to assess new materials is discussed; the relationship between early-life stiffness using the indirect tensile test and the *effective in-service* stiffness measured under sinusoidal conditions is discussed in Appendix E.

The method developed by Powell et al (1984) uses a multi-layer, linear elastic model of the pavement to calculate the strains at critical locations in the pavement structure that are considered to be responsible for deterioration. The equivalent in-service structural properties of the constituent material in the pavement and the loading conditions required for this calculation are given in LR 1132 (Powell et al, 1984). These criteria, material properties and pavement loading conditions are reproduced in Appendix C.

TRRL Laboratory Report LR 1132 contains criteria to guard against fatigue cracking as well as structural deformation. The fatigue criterion can be regarded as a conservative measure, particularly for the thickest pavements. For roads constructed with a roadbase containing a 100 penetration grade binder, the predicted fatigue life is broadly comparable to the deformation life. For roads constructed using stiffer materials the design life is controlled by the deformation criterion. As the thicknesses approach the long-life designs, for traffic in excess of 80 msa, these criteria become increasingly conservative, but this can be justified on the grounds that heavily trafficked roads need to involve less risk.

4.3 Summary

1 The road is more vulnerable in its early life before the structural properties of the asphalt roadbase have been increased by curing. For example, curing will result in a doubling of the elastic stiffness of DBM in the first few years of service. This results in a substantial improvement in the road's resistance to structural deformation and the perceived risk of fatigue damage to the roadbase.

2 Well constructed roads that are designed above a threshold strength will have a life in excess of 40 years. These roads are referred to as *long-life* roads.

3 Conservative calculations show that it is not necessary to construct roads thicker than that required by the current standard for 80 msa to achieve *long-life.*

4 An end-product performance specification based on early life indirect tensile stiffness modulus is a promising development.

5 Wearing course

The majority of the information presented in this Section is drawn from previous research sponsored by the Highways Agency.

5.1 Surface requirements

The wearing course provides the running surface for traffic and has a marked effect on the safety and comfort of the road user. In the context of long-life roads it will need to be replaced on a number of occasions over the life of the road.

The development of improved surfacing materials has occurred along two, quite different, routes. Following research sponsored by the Highways Agency (Project title: *Adverse weather working - laying bituminous materials*) into thick paver-laid wearing courses, it is becoming more common to specify a 45 mm or, in adverse weather conditions, a 50 mm thickness rather than the traditional 40 mm to achieve better compaction and thereby improve durability. In contrast, there is also growing interest in the use of thinner layers of paver-laid material and *veneer* treatments, some as thin as 5 mm, that meet the current specifications for surface characteristics. This has resulted in a much greater choice of materials being available for the wearing course than there is for any other asphalt layer. Therefore, it is necessary to consider carefully the role of surfacing materials in the design of long-life roads and the choices available to the highway engineer.

The wearing course has to serve many functions and it is clear that no single surface treatment will provide all the desired characteristics. Ideally, the riding surface should:

- offer good skid resistance
- allow for rapid drainage of surface water
- minimise traffic noise
- resist cracking and rutting
- withstand traffic turning and braking forces
- protect the underlying road structure
- require minimal maintenance
- be capable of being re-cycled or overlaid
- be durable and give value for money.

Many treatments are available which provide some of these requirements but none offers them all. Therefore, the selection of the wearing course is a matter of identifying the most appropriate material for each application.

Bitumen is a visco-elastic material, which at low road temperatures approaches elastic behaviour and at high road temperatures approaches viscous behaviour. Thus, under hot weather conditions and heavy traffic loading, asphalt surfacing will deform more rapidly and will highlight inadequacies in the material composition.

Modification to either the bitumen binder or to the mixture almost invariably leads to an increased initial cost. The Client will need to be assured that the use of these more expensive treatments will actually provide improved performance which justifies the higher initial cost. While laboratory testing of modified materials will often show performance improvements, compared to conventional mixtures, caution needs to be exercised in interpreting the results for in-service performance. It is for this reason that road trials using the modified materials are important to validate the laboratory-based studies.

At the design stage, it is essential to prioritise the desired requirements for any one site. At the same time consideration should be given to the maintenance requirements of the wearing course over the design life of the road to arrive at a *whole-life cost* using the Highways Agency program COMPARE. This concept is covered in more detail by Abell (1993).

In the UK, the trunk road network is required to be maintained at the appropriate level of skidding resistance (DMRB 7.3.1). These requirements are site dependent and are aimed at achieving a consistently safe driving environment. Investigatory levels are set, below which an investigation is carried out to ascertain whether a treatment is appropriate. These *skidding standards* have contributed to UK roads being among the safest in Europe (European Conference of Ministers of Transport, 1993).

Table 1, which is the consensus opinion of a panel of experts representing the asphalt industry, provides an indication of the relative ability of the various types of surfacing available to meet and maintain the desired

Table 1 Effectiveness of different treatments in meeting desired properties

Material#		suitability for re-profil -ing	defor- mation resistance	resistance to cracking	spray reduc -ing	noise reduc -ing	skid resis -tance	texture depth	initial cost	durab -ility	speed of construc -ion	quality of ride
		Desired property (for fuller description, see below)										
Thick wearing course	Rolled asphalt	✔✔✔✔	✔✔✔ (✔✔✔✔*)	✔✔✔✔	✔✔	✔✔	✔✔✔✔	✔✔✔✔	✔✔✔	✔✔✔✔✔	✔✔	✔✔✔✔
	Porous asphalt	✔✔✔	✔✔✔✔✔	✔✔✔✔	✔✔✔✔✔	✔✔✔✔✔	✔✔✔✔	✔✔✔✔✔	✔✔	✔✔✔	✔✔✔	✔✔✔✔✔
	Asphalt concrete/ dense bitumen Macadam	✔✔✔✔	✔✔✔✔	✔✔✔	✔✔	✔✔✔	✔✔✔	✔✔	✔✔✔✔	✔✔✔	✔✔✔	✔✔✔✔
	Mastic asphalt/ Gussaphalt	✔✔✔	✔✔	✔✔✔✔✔	✔	✔✔✔	✔	✔	✔	✔✔✔✔✔	✔✔	✔✔✔
	Stone mastic asphalt	✔✔✔✔✔	✔✔✔✔✔	✔✔✔✔	✔✔✔✔	✔✔✔✔	✔✔✔✔	✔✔✔✔	✔✔✔	✔✔✔✔✔	✔✔✔	✔✔✔✔
Thin wearing course	26 - 39 mm thick	✔✔✔	✔✔✔✔✔	✔✔✔	✔✔✔✔	✔✔✔✔	✔✔✔✔	✔✔✔✔	✔✔✔✔	✔✔✔	✔✔✔	✔✔✔✔
	18 - 25 mm thick	✔✔	✔✔✔✔	✔✔✔	✔✔✔	✔✔✔	✔✔✔✔	✔✔✔	✔✔✔	✔✔✔	✔✔✔✔	✔✔✔✔
	< 18 mm thick	✔	✔✔✔	✔✔	✔✔	✔✔✔	✔✔✔✔	✔✔✔	✔✔✔	✔✔	✔✔✔✔	✔✔✔✔
Veneer treat- ment	Surface dressing	n/a	n/a	✔✔✔	✔✔✔	✔	✔✔✔✔	✔✔✔✔✔	✔✔✔✔✔	✔✔	✔✔✔✔✔	†
	High-friction systems	n/a	n/a	✔✔	✔✔	✔✔	✔✔✔✔✔	✔✔✔✔	✔	✔✔✔	✔	n/a
	Slurry surfacing	✔✔‡	n/a	✔✔	✔✔	✔✔	✔✔✔	✔✔	✔✔✔✔✔	✔✔	✔✔✔	n/a

✔ = least advantageous to ✔✔✔✔✔ = most advantageous

\# *Some of these materials will have a limited laying season.*

* *The deformation resistance of hot rolled asphalt can be enhanced by designing to conform to Clause 943 of the* Specification for Highway Works.

† *The quality of ride for surface dressing will depend on the design of surface dressing, the aggregate size(s) employed and the evenness of the substrate.*

‡ *Slurry surfacing can give a useful improvement to the profile of the type of surface to which it is applied, for which this rating is appropriate - for other types of surfacing, it may not be appropriate.*

properties. It is considered that the maintenance of texture, skidding resistance and surface profile are critical for the satisfactory performance of the surfacing layer. Of necessity, the ranking is subjective and many factors, other than the surfacings themselves (for example, the condition and properties of the underlying layer), will affect performance. Therefore, Table 1 should only be used as a first guide to the type of material that will provide particular properties: a full understanding of the materials and of the appropriate circumstances for its use will be needed to optimise the selection of material type.

Desired properties in Table 1:

suitability for re-profiling — *The suitability of the material to be used for regulating or re-profiling an existing surfacing.*

deformation resistance — *The ability of the material to resist the effects of heavy traffic to create ruts in the wheel-paths during hot weather.*

resistance to cracking — *The ability of the material not to crack or craze with age, particularly in cold weather and in areas of high stress.*

spray reducing — *The ability of the material to form a surfacing which minimises the amount of water thrown up by the wheels of passing traffic into a drivers line of sight in wet conditions.*

noise reducing — *The ability of the material to form a surfacing which reduces the noise generation at the tyre/surfacing interfaces and/or increase the noise absorbed.*

skid resistance — *The ability of the material to form a surfacing which can achieve a high mean-summer SCRIM coefficient.*

texture depth — *The ability of the material to form a surfacing which can achieve a high texture depth, with particular reference to the requirement for high-speed trunk roads of greater than 1.5 mm using the sand-patch method.*

initial cost — *The initial cost to supply, lay and compact an area with the material.*

durability — *The ability of the material to*

	remain in place and retain its other properties under the prevailing traffic and climatic conditions.
speed of construction	*The time required between closure and re-opening of the road when surfacing it with the material.*
quality of ride	*The ability of the material to form a surfacing which gives a driver a comfortable ride.*

For long-life designs, hot rolled asphalt, porous asphalt, stone mastic asphalt and thin surfacings are considered as potential surfacing materials at the initial construction stage. The other surface layer types, listed in Table 1, are either unlikely to be used or would be best considered as maintenance treatments.

For satisfactory performance and durability, all asphalt layers must be adequately compacted. For the lower, and thicker, road layers there are compaction criteria, but for the surfacing layer none are specifically given. Therefore, good volumetric design of the mixture is essential to ensure that the voids content is within an acceptable range when fully compacted; a mixture will be susceptible to fretting if the voids content is high, while a low value will indicate a susceptibility to deformation. A high voids content will also increase the permeability of the layer to air and water which will make it more prone to in-situ ageing, and ultimately to brittle fracture at low pavement temperatures.

For gap-graded materials such as hot rolled asphalt, the stiffness of the mortar is critical in limiting the rate of deformation; for continuously graded materials, the aggregate interlock generally limits the degree of deformation. However, if continuously graded mixtures have been over-filled with binder, they too will suffer high deformation rates under unfavourable conditions.

The other major deterioration mode in asphalt surfacings is cracking. Whilst not considered to be a major problem, studies summarised in Appendix A have shown that cracking, caused by a combination of traffic induced stresses, thermal extremes and oxidative hardening, can affect the longer term durability of the surfacing. The cracks initiate at the surface and, if left untreated, they may eventually damage the underlying layers.

5.2 Thick paver-laid wearing course

Thick paver-laid wearing courses are defined as materials laid to a thickness of 40 mm or more (typically up to 50 mm). Hot rolled asphalt, porous asphalt, stone mastic asphalt and mastic asphalt fall into this category, although mastic asphalt is only used in specialist locations such as bridge decks and is not considered further in this report.

5.2.1 Hot rolled asphalt

Hot rolled asphalt (HRA), containing 30 or 35 per cent coarse aggregate and 50 penetration grade bitumen, is the principal asphalt surfacing material specified for motorways and trunk roads in the UK. The advantages of this material are that:

- it is widely used and understood
- it can be designed using a wide range of locally available materials (British Standards Institution, 1992)
- high PSV aggregate is not required for the full layer depth
- it is relatively tolerant to small changes in binder content
- it can be laid and compacted within a wide climatic envelope
- experience has demonstrated that hot rolled asphalt performs satisfactorily, providing adequate deformation resistance, skid resistance and durability for a wide range of traffic and climatic conditions.

Hot rolled asphalt also has potential for improvement by better mixture design or by the use of modifiers.

As traffic volumes have increased during the last 20 years, attention has focused on the need to design mixtures with improved resistance to deformation. While all asphalts are susceptible to rutting, it is probably the major mode of deterioration associated with hot rolled asphalt. The reason for this lies in the nature of hot rolled asphalt itself. It relies on a stiff mortar (fine aggregate plus binder) to resist flow. The choice of fine aggregate affects both the optimum binder content and the resulting stability of the mixture. Design of this material primarily involves optimising the binder content for the range of properties including stability (British Standards Institution, 1990). Binder viscosity is an important variable that governs the behaviour of hot rolled asphalt. Increasing binder viscosity produces a stiffer mortar which reduces susceptibility to rutting but may adversely affect other properties. It is for this reason that additives, to modify the binder, have been pursued.

Many studies and trials, mostly carried out on behalf of the Highways Agency, using modifiers in hot rolled asphalts ranging from polymers (such as ethylene vinyl-acetate (EVA), styrene-butadiene-styrene (SBS) and synthetic rubber (SR)) to epoxy-resins, oxidised bitumens, curing agents (effecting binder hardening over time and in addition to normal ‘age-hardening’), sulphur and others are summarised in Appendix D.

As a result of research for the Highways Agency (Project title: *Surface treatments*) and contractual experience, it is considered that a rutting criterion for mixture design may be better than a Marshall stability requirement for in-situ performance. This has led to the specification of a rutting rate at 45°C or 60°C measured using a wheel tracking test as a compliance test in some contracts. The test, originally developed at TRL, is now a British Standard (British Standards Institution, 1996). The original test temperature was 45°C, but this was not sufficiently discriminatory for very stiff mixtures and, for work carried out on modified mixtures in particular, the wheel-tracking test is regularly carried out at 60°C to demonstrate the benefit of the modification (Gershkoff et al, 1997). Therefore, a maximum wheel-tracking rate at 60°C is used as the requirement for very heavily trafficked roads, where polymer-modified mixtures are often required because they are effective in producing more deformation resistant materials at high ambient temperature.

Hot rolled asphalts have an air voids content in the range of about 2 to 8 per cent, with the minimum value achievable being dependent on the mixture composition (Road Research Laboratory, 1963). For a given hot rolled asphalt aggregate grading, the voids content (in the compacted state) and the binder content are inversely related; a very low voids content is associated with an excessive binder content and ‘over-filling’ of the aggregate skeleton, which leads to a reduction in resistance to deformation. Research sponsored by the Highways Agency (Project title: *Test for voids content of rolled asphalt*) identified that the specification of a voids content less than 4 per cent, at the design stage, would be appropriate to maximise durability (Daines, 1995). This should not present a problem because the majority of hot rolled asphalts have an optimum voids content below this value. Daines (1985) has also demonstrated that sufficient time to achieve adequate compaction under adverse conditions can be obtained more consistently by specifying a 50 mm rather than a 40 mm thick wearing course layer.

The trials reported in Appendix D, for modified hot rolled asphalt with pre-coated chippings containing either 30 or 35 per cent coarse aggregate, show that the use of modifiers generally improves deformation resistance. Ideally, the modifier should give enhanced behaviour over the road surface temperature range without adversely affecting the mixing and laying characteristics. A modifier which stiffens the binder at all temperatures (for example, polyethylene) will produce a more deformation- resistant material but it may cause workability problems. Research for the Highways Agency (Project title: *Adverse weather working - Laying bituminous materials*) showed that, with the performance of the laid material depending also on the degree of compaction achieved, consideration has to be given to the effective workability of the mixture and the time window available for compaction (Nicholls & Daines, 1993). Stiffening the binder may also lead to higher energy costs and, in some instances, it may be more cost effective to use a harder grade bitumen.

However, harder binders can lead to a more rapid loss of the pre-coated chippings and, possibly, to low-temperature cracking. There were problems in earlier trials with high penetration index bitumens and crushed rock fines mixtures. A more carefully balanced approach (for example, ‘designing’ binders more appropriately and blending crushed rock fines with sand) has led to better in-service performance.

Where the modification has relied on a curing process to improve the performance of hot rolled asphalt, the results have been variable. The reason is that the curing process is partly dependent on the presence of air. If the hot rolled asphalt has a low voids content (which is usually the case) the curing effect may either not proceed or be delayed. If the voids content is too high, the curing may be extensive and lead to cracking. This uncertainty and the consequent variable performance led to the withdrawal of manganese oleate (Chemcrete) as a modifier in December 1990.

The longevity of hot rolled asphalt will, of course, depend on factors other than deformation resistance. Within the context of long-life design, hot rolled asphalt surfacing, in common with all surfacing materials, must also maintain adequate skidding resistance. However, this is largely dependent on the resistance to polishing of the aggregate used for the pre-coated chippings. For hot rolled asphalt materials, improved deformation resistance should reduce the rate of loss of texture through embedment. Surface cracking is not normally considered to be a problem with hot rolled asphalt wearing course but with increasingly stiff binders being used (including modified binders) the position should be kept under review.

5.2.2 Porous asphalt

Porous asphalts have carefully selected gradings that produce about 20 per cent voids contents when fully compacted. The composition in the UK was developed in research for the Highways Agency (most recent project titles: *Durability of porous asphalt on heavily trafficked roads*; and *Durability of pervious macadam surfacing*). They enhance the environment and improve safety (Colwill et al, 1993) by reducing tyre generated noise in both wet and dry conditions, by reducing spray thrown up by vehicle tyres and reflected glare at night from vehicle headlights in wet conditions. Safety is improved because, with less water on the road surface, better tyre/road grip is achieved in wet conditions.

Together with the significant advantages in using porous asphalt, there are some disadvantages. It has a relatively low structural strength (Potter & Halliday, 1981), due to its high voids content, and there is a perceived greater risk of poor durability. Being relatively weak in shear, the material is particularly vulnerable at high stress sites. Furthermore, careful consideration needs to be given to providing the drainage path to allow water passing through the layer to escape. Porous asphalt also uses good quality aggregate throughout its thickness which, together with the preferred use of modifiers (to prevent binder drainage during transportation from the mixing plant and to improve durability), increases the overall cost of the material. Additionally, the lower bound layers and foundation layers need to be protected from the ingress of water, so porous asphalt must be laid on an impermeable basecourse. However, most of these disadvantages can be overcome by careful planning.

The following points arise from a series of road trials of porous asphalt in the UK carried out for the Highways Agency over several decades (Nicholls, 1997a):

- High quality aggregates (in terms of having a ten percent fines value greater than 180 kN) must be used for porous asphalt because tyre-induced stresses are applied to relatively few point-to-point contact areas within the essentially single-size coarse aggregate skeleton rather than being dissipated over the large internal surface area in a dense mixture.
- Durability is improved at higher binder contents which

provide a thicker binder film (Daines, 1992). However, binder contents are limited by the onset of binder drainage; the use of fibres or binder modifiers can reduce binder drainage and hence improve durability.

- Although 10 mm gradings, similar to those on airfields, have been investigated as well as 14 mm open-textured macadams, 20 mm gradings are considered to be superior for spray reduction and in maintaining their open texture. The smaller size aggregate should provide better noise reduction, but this has not been found to be the case on the M1 trial. Since 1980, a standard 20 mm grading has been used, which has better spray reducing ability and a longer spray-reducing life. Nevertheless, the 20 mm grading, proven in many trials, is a compromise between durability and hydraulic conductivity (spray-reducing life), particularly when using modified binders at a higher binder content.
- Improved durability is obtained if a 200 pen bitumen is used in preference to a 100 pen bitumen. However, some early closing-up of the surfacing may be experienced under traffic and, therefore, the use of the softer grade is not recommended for the most heavily trafficked sites. The extra durability is related to the time taken for the binder to harden to a critical condition, so that the binder can no longer accommodate the traffic induced strains at low temperatures. For this reason, grades of bitumen harder than 100 pen should not be used.

The overall life of porous asphalt is governed by progressive binder hardening, until the binder can no longer accommodate the strains induced by traffic. Brittle fracture occurs after several years during low winter temperatures; if a surfacing has survived a cold period during winter, it will usually remain satisfactory during the following warmer months. Some indication of imminent failure may be provided by the onset of fretting which is usually indicated by an increase in texture depth which can be monitored using the High-Speed Texture Meter (HSTM). At this stage, core samples can be taken from the surfacing, the binder recovered and its condition compared with a critical condition in terms of binder hardness, currently considered to be when the binder penetration has reduced to 15.

5.2.3 Stone mastic asphalt

Stone mastic asphalt, also known as Splittmastixasphalt and Stone Matrix Asphalt, was first developed in Germany nearly 25 years ago as a proprietary product to resist wear from studded tyres but recognition of its excellent resistance to deformation led to its standardisation. Today, it is the most widely used type of wearing course on heavily trafficked roads in Germany and many other countries have, or are in the process of, developing variants. Depending on the maximum stone size, it can be laid as either a thick or a thin wearing course layer.

Stone mastic asphalt, described by Kast (1985), contains about 70 percent of coarse aggregate and between 6 and 8 per cent by weight of a bitumen in the grade ranging from 50 to 100 pen. An additive is required to prevent binder drainage; cellulose fibres are used in about 90 per cent of cases with modified binders being used in the remainder. Modified binders are used to improve performance as well as prevent binder drainage.

The coarse and open surface texture of stone mastic asphalt is claimed to provide good surface drainage and to generate tyre noise levels comparable to those of porous asphalt; the surfacing should be able to provide adequate texture depth and skidding resistance. However, the long-term retention of the higher levels of texture required for UK roads, obtained by modifying the aggregate grading, needs to be validated in service and the initial skidding resistance may be lower until the thicker binder film wears off the surface. In common with porous asphalt, the material uses high quality aggregates throughout its depth.

Stone mastic asphalt is a relatively new material in the UK so there is limited experience of its use. It was introduced into the UK through work jointly sponsored by the Highways Agency, British Aggregate Construction Materials Industries and the Refined Bitumen Association (Project title: *European mixes for road construction and maintenance*) (Nunn, 1994). Its relatively high materials cost compared with hot rolled asphalt is a potential drawback, although this may be partly redressed by the cost savings in not applying pre-coated chippings. Nevertheless, interest is being shown in stone mastic asphalt for the UK market and road trials are currently being funded by the Highways Agency (Project title: *Surface treatments*) to assess its potential as both a thick and a thin surfacing layer. If these and future trials confirm the successful performance of stone mastic asphalt, it will be well suited for use in long-life pavements.

5.3 Thin surfacings

Thin surfacings are classified as proprietary asphalt wearing course mixtures laid at nominal thicknesses of less than 40 mm. It is generally recognised that thin surfacings contribute relatively little to the overall strength of the road, but they can provide a fast maintenance treatment to restore skid resistance and riding quality. In this respect, they can be specified where there is evidence of rutting and, because of the nature of their surface texture, many of them also provide some reduction in tyre noise.

Traditionally, thin surfacings in the UK were based on either specifications for friction course materials or modified wearing course mixtures. Laid thicknesses were typically around 20 mm, with polymer modified binders sometimes being used. In order to achieve a good bond to the existing surface, a tack coat is applied. Binder contents of thin surfacings were generally between 4.2 and 4.8 per cent.

Since 1980, proprietary thin surfacing materials have been developed and used extensively in France (Tessonneau, 1985) mainly to restore surface characteristics and to surface pavements that have received a strengthening overlay. A number of these thin surfacing materials have been introduced into the UK and TRL are currently monitoring the performance of some proprietary thin wearing courses (Nicholls et al, 1995) for the Highways Agency (Project title: *Surface treatments*) and for individual companies offering a proprietary product. The earliest trial is only about 6 years old and long-term performance is not established, but there is sufficient experience available from other countries for thin surfacings to be considered as an option for long-life roads. Lives in excess of 10 years have been claimed for thin surfacings (Litzka et al, 1994).

5.4 Further considerations

With stiffer, lower bound layers now being specified for heavily trafficked roads, the requirement that the wearing course layer must, of necessity, also contribute to the overall structural strength of the road is under review (Preston, 1994). The primary function of the wearing course is to provide the specified surface characteristics (see section 5.1) while load-spreading is the primary function of the lower asphalt layers. To this end, a road consisting of high modulus roadbase and basecourse with a very thin wearing course would be an efficient structure, provided durability could be guaranteed. The very deformation-resistant high modulus material, laid to within 20 or 30 mm of the pavement surface, would contribute to a rut resistant structure. Use of thinner wearing courses would also require closer tolerances for the surface profile of the lower asphalt layers.

These considerations suggest that, in future, the wearing course layer is likely to be regarded as a *renewable layer* to be replaced at intervals, whereas the underlying layers will be regarded as permanent. Thus, rehabilitation costs would be generally limited to the thin surface layer with potential saving in maintenance costs and consideration needs to be given to its likely life, the ease and speed with which it can be replaced and its potential for recycling. However, for this to happen there has to be a significant change in the accuracy of the construction profile of the lower layers.

The structural contribution of the wearing course needs to be taken into account. It is assumed that stone mastic asphalt and thin surfacings provide the same structural contribution as the same thickness of hot rolled asphalt. Figure 2 gives the total thickness of asphalt required to achieve the specified design life, which includes a 40 mm hot rolled asphalt wearing course. The wearing course does contribute to the overall stiffness of the road and if a wearing course thickness other than 40 mm is used, then the overall thickness of the asphalt layer will need to be adjusted according to the type of roadbase as given in Table 2 and illustrated in Figure 3.

Table 2 Increase in roadbase thickness for type of surfacing and roadbase

*Thickness of surfacing**	*Increase in roadbase thickness (mm)* DBM	DBM50	HDM	HMB15
50	-10	-9	-8	-6
40	0	0	0	0
30	+10	+9	+8	+6
20	+20	+18	+16	+13
10	+30	+27	+24	+20
0	+40	+36	+32	+26

* *Surfacing includes HRA, SMA and thin surfacings. Porous asphalt is not included. The Design Manual for Roads and Bridges (DMRB 7.2.4) recommends that a layer of porous asphalt is structurally equivalent to a layer of HRA of half the thickness (Potter & Halliday, 1981). However, recent work (Nicholls, 1997a) indicates that the structural equivalence of porous asphalt relative to HRA may be closer to 1.5:1 rather than 2:1.*

5.5 Summary

The findings from the review of surface layers within the context of long-life design are as follows:

1 The primary requirement of the wearing course is to provide a safe riding surface for traffic. Therefore, the provision and maintenance of appropriate surface characteristics are the main priority. If a thick asphalt roadbase is specified the contribution of the wearing course to the structural strength of the road pavement becomes less important.

2 The wearing course serves several functions and no material is ideal for all situations. Therefore, selection of the wearing course is a matter of identifying the most appropriate material for each application.

3 There is scope to consider modification of surfacing materials and to consider alternative design methods (for example, based on voids and deformation resistance).

4 From the hot rolled asphalt trials reported, various modifiers appear to offer improvements in deformation resistance. Within the context of long-life design, modified materials with improved deformation resistance may also help in maintaining satisfactory texture and skid resistance.

5 Porous asphalts with modified binders have generally

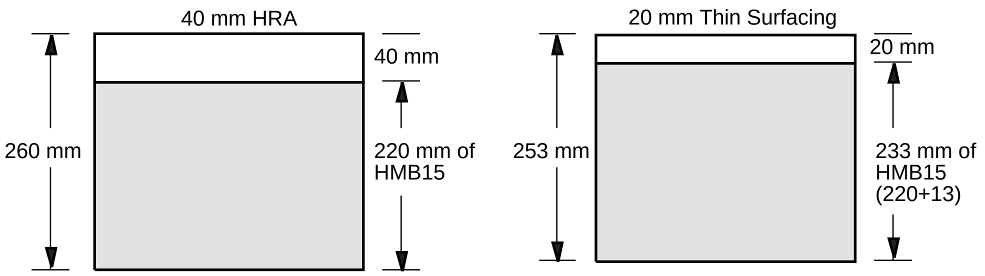

Figure 3 Illustration of roadbase adjustment using a thin surfacing

been found to have longer in-service lives. Principally, this has been due to the higher binder contents that have been achieved using some polymers and other additives. There is a view that certain modified binders also aid performance by their ability to improve adhesion, but the evidence from the A38 trials is that, provided this higher binder content can be safely achieved without binder drainage, the type of polymer or modifier used is relatively unimportant.

6 Under adverse weather conditions more consistent compaction can be obtained by specifying a 50 mm rather than a 40 mm thick HRA wearing course layer.

7 Thin surfacing treatments have increased the surface maintenance options available to the highway engineer. Assuming that closer tolerances of the profile of the lower asphalt layer can be achieved, a case could be made for using thin surfacings in new construction.

8 Whatever surfacing material is used in the design of long-life roads, the critical performance parameter may well be the maintenance of surface characteristics, assuming that rutting and fretting have been reduced to a minimum. While modifiers may contribute to the maintenance of texture, the frictional properties will principally depend on the aggregate properties.

6 Modifications to standard designs

There is frequently a need to adapt standard designs for use in particular situations. For example, modified designs may enable reconstruction of heavily trafficked roads to be carried out more efficiently where construction depths are restricted by clearances under bridges and where drainage levels have to be maintained. In this situation, it is often not possible to accommodate the thickness of asphalt and sub-base required to carry the greater volume of traffic. In such cases the designs can be adapted to take advantage of improved materials and new developments in design and construction. The following examples illustrate the application of the design method.

6.1 New or improved asphalt roadbase

Roadbase materials with improved structural properties are usually produced using lower penetration grade binders. Roadbase materials that use a 50 penetration grade binder have only been in wide-spread use in the UK since 1987 (Nunn et al, 1987) and, consequently, less is known about their long-term curing behaviour than dense bitumen macadam (DBM) made with 100 penetration grade binder. Roadbase asphalt with a harder bitumen can be viewed as a material which, when first laid, contains some of the benefits of a conventional material after curing. However, the threshold thickness for long-life was estimated conservatively, assuming no curing occurs, therefore, the long-life designs for the improved macadams will still be adequate even if curing takes place at a much lower rate compared with DBM.

A new material must also be well compacted and have good deformation resistance and durability. Compliance with the relevant clauses of the Specifications for Highway Works (MCHW 1), will ensure that the material is compacted adequately and is designed with sufficient binder to be durable without being unstable due to an excess of binder.

In a separate element of research sponsored by the Highways Agency, British Aggregate Construction Materials Industries and the Refined Bitumen Association, Nunn and Smith (1997) have shown that the load spreading ability of a dense macadam incorporating a low

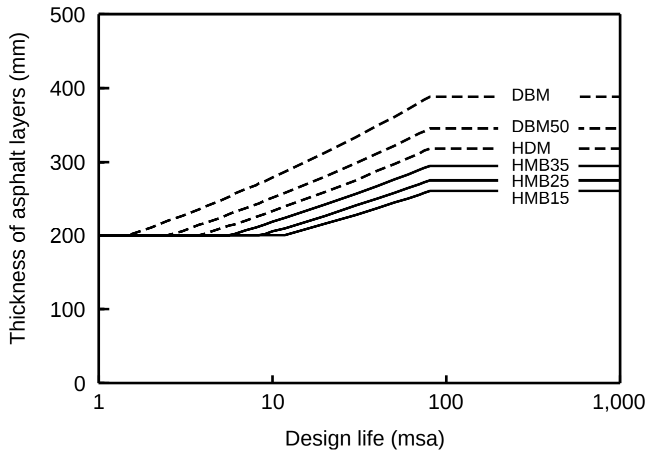

Figure 4 Design curve for high modulus roadbase (HMB)

penetration grade binder is increased significantly compared with, say, a dense bitumen macadam manufactured with a 100 penetration grade binder. This material, known as high modulus base (HMB), also has improved resistance to deformation without any loss of resistance to cracking. Because of variability in material composition and laying practice, the increase in stiffness can vary considerably. It is shown in Appendix E that, at 20°C and 5 Hz, a stiffness modulus of HMB made using 15 pen binder (HMB15) is 4 times greater than that of standard DBM and is, therefore, appropriate for pavement design purposes at the 5 per cent level of significance. For a macadam incorporating a 25 or 35 penetration grade binder (HMB25 or HMB35), the corresponding improvements are 3.3 and 2.6 respectively. This increase in stiffness can be used to justify a substantial reduction in the thickness of the roadbase. For a given level of traffic, design thicknesses can be determined using equations C1 and C2 of Appendix C. The effect of this improvement in the stiffness modulus over the full range of cumulative traffic is compared with the designs for standard materials in Figure 4.

An example of long-life construction using HMB in a contractual situation is given in the next Section, in which other examples of long-life pavement constructions were also evaluated. These illustrate that the highly effective load spreading properties of HMB can be maximised by using it to form the entire thickness of the asphalt layer apart from a thin wearing course layer.

Nunn and Smith (1997) have demonstrated that the use of HMB results in more economic road construction. The higher cost of HMB is more than offset by the reduction in

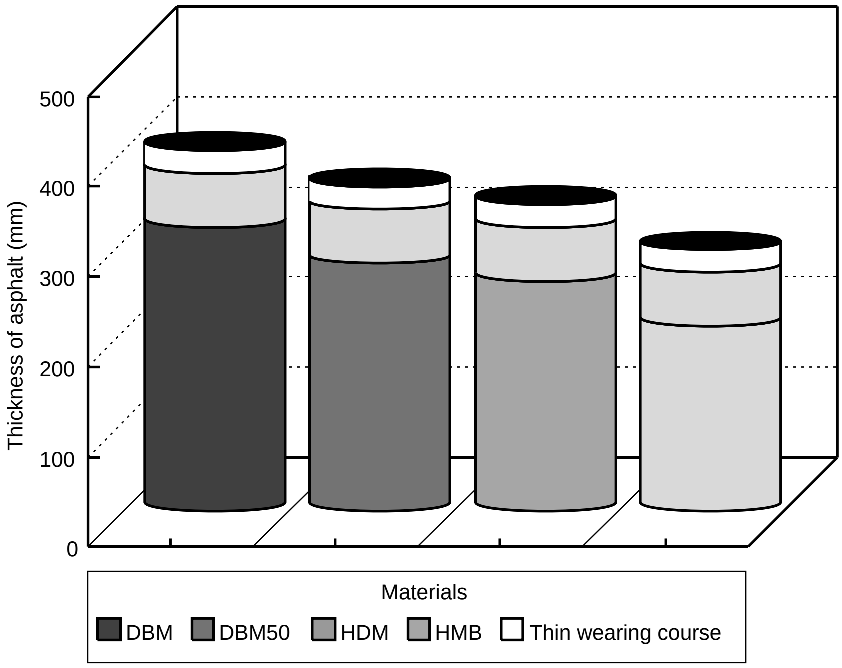

Figure 5 Pavement designs to resist surface rutting

material thickness required to construct a pavement with the same design life. For example, savings in material cost for the roadbase layer will be over 25 per cent using HMB15 in place of DBM.

6.2 Areas at high risk of surface rutting

HMB is very resistant to deformation and when used with very thin wearing course it is suitable for situations in which the risk of excessive rutting is high, for example, areas with heavy, canalised traffic or steep, south-facing gradients (crawler lanes) on heavily trafficked routes.

Appendix A shows that deformation in the asphalt layers occurs mainly in the upper 100 mm of a strong pavement. If the objective is to reduce the risk of premature rutting in the asphalt layers, it may be sufficient to construct a pavement using a layer of HMB as a base course or upper roadbase layer immediately below a thin wearing course or a wearing course specifically designed to resist rutting. These designs are illustrated in Figure 5 for pavements expected to carry more than 80 msa.

6.3 Asphalt substitution

In a conventional road construction, a sub-base of well graded granular material is used to protect the earthworks from the effects of weather and provide a construction platform for the asphalt layers. In situations in which construction depth is limited there may be a technical and economic case for replacing some or all of the sub-base with a structurally equivalent, but less thick, asphalt layer. This form of construction is often referred to as full-depth asphalt, but is more correctly described as asphalt substitution because it may be neither necessary nor desirable to replace all of the granular sub-base.

The need to construct strong pavements to withstand increased traffic loading, coupled with limitations on the depth of construction, identifies a niche ideally suited to designs incorporating a larger proportion of asphalt roadbase. There are also practical advantages in reducing the exposure of the subgrade and considerable economies and benefits could be realised as a result of reduced tonnages of excavation and haulage and reduced traffic delays resulting from shorter contract durations. Asphalt substitution was evaluated in pilot-scale and full-scale road trials in earlier collaborative work funded by the sponsors of the current work.

Full-scale trials showed that, in new construction, reducing the thickness of the sub-base subjects the road foundation to considerable risk of serious rutting during the laying of the first asphalt layer. To make substitution designs acceptable, measures would be required to reduce this risk. The implementation of any risk reduction measure will add to construction costs and reduce or even negate the small materials cost advantage that asphalt substitution has over conventional sub-base design. Therefore, this form of construction is unlikely to be generally cost effective for new road construction. Methods of reducing risk are discussed by Nunn and Leech (1986).

However, road trials of asphalt substitution (Leech and Nunn, 1990), demonstrated that large savings are possible in reconstruction, especially if the existing foundation is utilised. Further savings would result from faster construction times and reduction in consequential work if the original surface level is not raised. However, it is not possible to estimate the construction time for asphalt substitution relative to conventional construction; the circumstances of an individual contract need to be considered. This is best assessed by the Contractor when tendering for a contract; substitution could result in large savings in costs, especially in lane rental schemes.

The road trials showed that, provided the foundation was strong enough to support the construction traffic with negligible deformation, asphalt substitution based on an equivalence ratio of 0.30 for the thickness of dense bitumen macadam containing 100 penetration grade binder relative to that of Type 1 sub-base material, was structurally equivalent to conventional designs. The equivalence ratio varies little with thickness and it is in good agreement with a comparison between predictions using linear elastic theory and measurements of traffic induced stresses and strains in pilot-scale pavements. Consequently, asphalt substitution can be based on linear elastic theory with the choice of design depending on the cost relative to conventional construction. Consideration is also required of the problems that might arise from using a thinner granular foundation, especially in adverse weather.

6.4 Summary

1 The method proposed for long-life design is as versatile as the current method for the introduction of innovations.

2 Improved roadbase macadams incorporating harder bitumens (HMB) can be used to construct more cost effective pavements.

3 These materials can also be used to construct more rut resistant pavement structures.

4 Asphalt substitution or full-depth asphalt can produce economic, technical and environmental benefits. However, additional risks may be incurred at construction.

7 Example of long-life design

The essential elements of long-life asphalt pavement design methodology were implemented on a trial basis as part of an on-going motorway construction contract. The trial was carried out on the M65 extension near Blackburn, for which Tarmac Construction Ltd was the main Contractor. It was designed to determine whether construction using the proposed long-life design is feasible, to examine any difficulties that might occur and to assess the practicability of introducing end-product performance criteria for the road foundation. Two test sections were constructed that included thin surfacing materials, high modulus roadbase using 15 penetration grade bitumen (HMB15) and heavy duty macadam (HDM). Potential end-product performance tests were also used to assess the road foundation. These constructions are illustrated in Figure 6.

Two proprietary surfacing materials were chosen by the Contractor; 10 mm Masterflex and 14 mm Masterpave. Masterflex is a thin wearing course material, while Masterpave is a stone mastic asphalt that can be laid as either a thin or thick wearing course layer. Contractual constraints made it impossible to reduce the overall thickness of the asphalt, nevertheless, this did not detract from the value of the trial. The test sections, which were laid in a cutting with a subgrade CBR of 3 per cent, were 500 m in length and 3 lanes wide.

Table 3 Measurements on capping

Measurement		*Capping* Well graded	*Capping* Segregated
Surface modulus (MPa)	Mean	52.2	37.1
	Std dev	7.2	5.5
	Sample No.	24	12
In-situ density (Mg/m^3)	Mean	1.971	1.740
	Std dev	0.043	0.095
	Sample No.	13	9

7.1 Foundation

The trial offered the opportunity to assess the practicability of end-product performance tests for the road foundation and to investigate whether these tests could successfully identify construction weaknesses. TRL staff conducted the end-product testing that was carried out in addition to the normal acceptance requirements. The Contractor was responsible for satisfying the conventional specification requirements only. The principal end-product tests, which are described in more

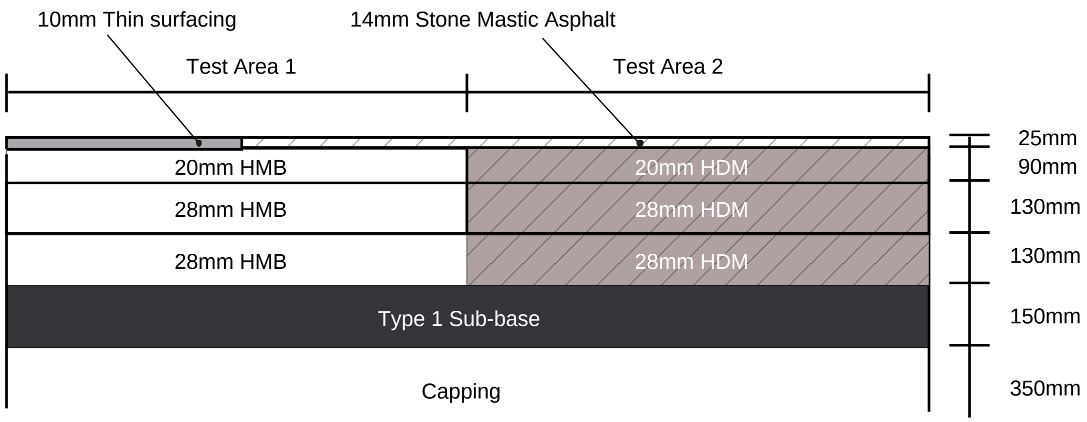

Figure 6 Construction thicknesses in M65 trial

detail in Appendix B, were the portable dynamic plate bearing test (PDPBT) to measure the surface modulus of the foundation layer and the nuclear density test (NDT) to measure the in-situ density.

The capping laid throughout the trial area had been used as a haul road for construction traffic for the previous 12 months. The capping material was of good quality throughout and produced a very stiff layer; areas of weakness had been identified during the previous 12 months of trafficking and had been reconstructed. However it was possible to carry out tests on the more typical material that had recently been placed and compacted on the opposite carriageway. The area selected showed segregation along approximately a third of its length. Results of the PDPBT and NDT measurements carried out on this area are summarised in Table 3.

Both areas exceeded the expectation of 30 MPa for the surface stiffness of a good capping layer. The difference between the test results for the two areas is statistically significant at the less than 1 per cent level, which demonstrates the capability of the PDPBT to discriminate between layers of different quality.

PDPBT, FWD and density measurements were carried out on the compacted sub-base. A mean surface stiffness of just over 50 MPa was obtained using the PDPBT and approximately 70 MPa with the FWD, confirming that the relationship between measurements with these two devices is material and site specific. The average dry density was slightly greater than 96 per cent of the Proctor compaction method (BS 1377: Part 4, 1990). The sub-base complied with the current specification for Type 1 sub-base and the mean surface stiffness measured with the PDPBT was close to the provisional end-product criterion of 50 MPa. No problems were encountered with laying the lower roadbase layer which confirmed that the foundation was fit-for-purpose.

Table 4 Mean properties of structural layers

Material property	*HDM 20 mm base -course*	*HDM 28 mm road -base*	*HMB 20 mm base -course*	*HMB 28mm road -base*
ITSM (GPa)	5.2	6.1	12.6	15.6
Creep stiffness (MPa)	9.5	-	58.5	-
Binder content (M/M%)	4.4	4.2	4.7	4.2
Nominal pen of binder	50	50	15	15
Pen of recovered binder	34	34	9	9
Voids content (V/V%)	2.3	0.8	2.5	2.3
Percentage of refusal density (%)	96.1	99.3	97.2	97.7

7.2 Structural layers

Heavy duty macadam (HDM) and high modulus base with a 15 penetration grade binder (HMB15), designed to Clause 929 (MCHW1), were laid in the two test sections. Good construction practice was observed throughout the trial and, on completion, cores were taken for subsequent laboratory measurement of the indirect tensile stiffness modulus (ITSM) and creep characteristics. These measurements, which were carried out according to British Standards Drafts for Development, BS DD 213 (1993) and BS DD 226 (1996), are measures of load spreading ability and resistance to deformation. Table 4 records the measured properties of the materials.

Table 5 Texture depth measurements

	Measurement	*Thin surfacing*	*SMA*
Sand-patch (mm)	Mean	2.53	2.36
	Standard deviation	0.49	0.26
MTM (mm)	Mean	0.93	1.12
	Standard deviation	0.07	0.09

Table 4 illustrates the good quality of these materials. The ITSM of HDM roadbase is typically expected to be in the range 3.2 to 6.7 GPa (Nunn, 1996), while the stiffness of HMB compares well with the values obtained by Nunn and Smith (1997) in a series of road trials. The 150 per cent greater stiffness of HMB compared with HDM would allow the thickness of structural layers to be reduced by about 20 per cent.

A further advantage of HMB is its excellent resistance to deformation. This is especially desirable when used in combination with a thin wearing course where the basecourse is in close proximity to the wheel load and there is less insulation from solar radiation.

Table 6 Wheel tracking results

Measurement		*Thin surfacing*	*SMA*
Wheel tracking rate at 45°C (mm/h)	Mean	0.84	0.93
	Standard deviation	0.44	0.46
Wheel tracking rate at 60°C (mm/h)	Mean	1.10	1.30
	Standard deviation	0.23	0.21

7.3 Wearing course

The thin surfacing and the stone mastic asphalt were laid 25 mm thick. The surface textures of these materials were measured at 10 m intervals using the sand-patch test and the mini-texture meter (MTM) was used over the whole trial area. The results in Table 5 show that both materials achieved the requirement of 1.5 mm texture depth, measured using the sand-patch test, specified for high speed roads.

The wearing course of a long-life road has to resist rutting from repeated passages of commercial vehicles and, with the increased use of *super-single* tyres, very deformation resistant surfacing systems are required. Thin surfacing laid over HMB basecourse or upper roadbase has the potential for superior rut resistance. Twelve, 200 mm diameter cores were cut from each of the wearing course types laid on HMB. The top 50 mm consisting of the surfacing and HMB was wheel-tracked in accordance with BS 598: Part 110. The results of these tests, in Table 6, illustrate the potentially good rut resistance of these surfacings.

These results compare favourably with the maximum wheel-tracking rates of 2 mm/h at 45°C for sites requiring high rut resistance, and 5 mm/h at 60°C for sites requiring very high rut resistance, specified in the Draft Clause 943 of MCHW 1 - Hot Rolled Asphalt Wearing Course (Performance-Related Design Mix). As can be seen, both materials show only a small increase in rutting rate at the higher test temperature.

7.4 Summary

The following conclusions can be drawn from the trial of long-life design:

1 Additional site testing will ensure that all pavement layers have good performance related properties and that the pavement is unlikely to deteriorate prematurely.

2 An end-product specification for the road foundation is capable of identifying areas that do not comply with the proposed requirements for long-life pavements. More information is required before a stiffness criterion for the completed foundation can be established.

3 High modulus base is an alternative material for the structural layers that has excellent load spreading ability and deformation resistance. It offers a stable substrate for thin and very thin surfacing. Providing appropriate mixing and laying temperatures are achieved, the material handles similarly to conventional roadbase macadam.

4 Thin surfacing can meet the current texture requirements for wearing course. A road potentially very resistant to surface rutting can be produced when a thin surfacing or well designed HRA is laid in combination with a very stable basecourse or upper roadbase.

5 Overall, the trial demonstrated that Long Life Pavement Design is practicable and that considerable benefits can be expected from its introduction.

8 Condition assessment

Long-life pavements will not survive for 40 years without timely intervention to carry out the appropriate maintenance treatment. At some stage, maintenance will be required either when pavement wear is judged to threaten the structural integrity of the pavement or the standard of service provided to the road user, including safety.

The investigations, summarised in Appendix A, have shown that the deterioration of thick, well constructed, flexible pavements is normally confined to the surfacing layers. Cracks normally develop from the surface downwards and, even if they begin to extend into the main structural layers of the road, irreversible damage to the pavement may be avoided by the timely replacement of the surface layers. The investigations have also shown that significant deformation in the lower layers of well constructed pavements is not common.

In view of the importance of arresting surface deterioration before it can affect the main pavement structure, more attention will need to be paid to determining the depth to which cracking and rutting extend into the pavement layers to arrive at the most appropriate maintenance treatment.

The Highways Agency’s Design Manual for Roads and Bridges (DMRB 7.3.2) already gives guidance on assessment and structural examination of pavements along with interpretation of results and consequences for the design of strengthening. In this assessment method deflection is considered to be a good indicator of structural condition, but it recommends that further supporting evidence is required before structural maintenance is recommended. This is good advice, and in dealing with long-life roads, many of the existing procedures for pavement monitoring and investigation should continue but there needs to be a change of emphasis and the introduction of some new techniques. The most important change will be the reduction of the influence of pavement deflections to predict the remaining life of the pavement. The strategy for condition assessment is currently under review by the Highways Agency.

8.1 Summary

1 The existing pavement assessment strategy should be modified to enable it to deal adequately with long-life roads.

2 Clear identification of the causes and extent of any deterioration must be used in making decisions on maintenance.

3 Deflection should be used to indicate when to investigate pavement condition rather than the existence of a *critical condition* which is now known as *investigatory condition* (DMRB 7.3.2, HD29/94).

9 Whole life costing

The whole-life costs of long-life designs have been compared, in Appendix F, with 40 year designs based on existing relationships (HD 24/96) using the Highways Agency’s computer program, COMPARE. Whole-life costs are used to compare alternative pavement design and maintenance options. This is a complex process involving the prediction of the timings of the maintenance treatments for different classes of road carrying different quantities of traffic, and includes the calculation of the durations of maintenance works and the costs of construction. In this process the user costs during maintenance and the costs of the maintenance itself are discounted to present values and totalled, together with the initial construction costs.

In Appendix F, a three-lane motorway (D3M) and a two-lane dual carriageway (D2AP) have been considered at two traffic levels. Pavements designed with roadbases of dense bitumen macadam (DBM), heavy duty macadam (HDM) and high modulus base using 25 penetration grade binder (HMB25) have been considered. HMB is manufactured using a penetration grade binder between 50 and 15 dmm. A mid-range HMB was used in this analysis. With these roadbases, conventional hot rolled asphalt and alternative surfacing materials such as stone mastic asphalt

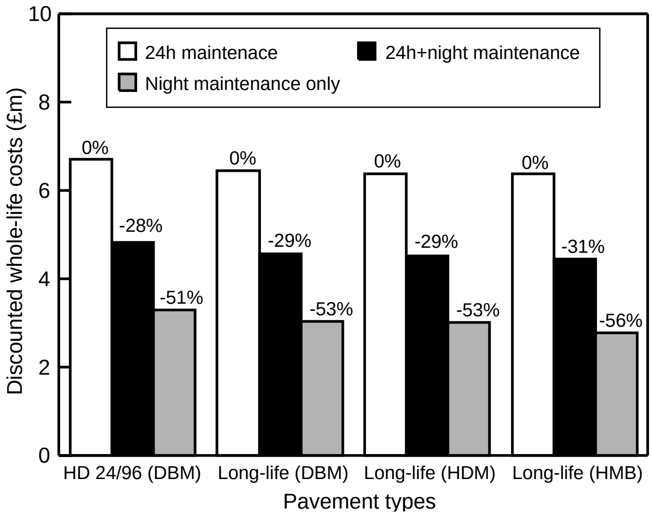

Figure 7 Whole-life costs for D3M with day and night maintenance

(SMA) and thin surfacings may be used. However all the costs in this study were determined for normal hot rolled asphalt surfacing. It is known that in Germany, SMA can achieve a service life in excess of 20 years but it is not known if it would still satisfy UK performance criteria, particularly for skidding resistance, over this period. The effect of these long surfacing lives would be worthy of consideration in future.

The total discounted costs for the long-life designs are all lower than the 40 year pavements designed in accordance with DMRB 7.2.1 HD 24/96 with the thinner HMB pavement having the lowest whole-life costs.

The maintenance for long-life roads will be predominately resurfacing that can be carried out by inlaying at night. The effect on whole-life costs of carrying out this resurfacing overnight and re-opening the road each day was examined. Two alternative maintenance procedures were compared with maintenance being carried out over a 24 hour period throughout the life of the road. In the first case, 24 hour closures were assumed for the first 20 years and thereafter, after predicted traffic levels had increased substantially, maintenance was carried out during night closures only. In the second case, maintenance was carried out using night closure over the full 40 year period. The results, for the D3M, illustrated in Figure 7, show that substantial savings in discounted costs are possible. The cost of the maintenance works increases but this is more than offset by the substantial reduction in user costs.

A sensitivity analysis was carried out to demonstrate the benefits of using surfacing materials that performed better than the default cases assumed by COMPARE. The effect of one year longer intervals between resurfacing on total costs resulted in reductions of between 13 and 17 per cent in the whole-life costs for the design scenarios considered. These savings are principally due to lower user costs. This illustrates the benefits of good construction practice and of improving existing surfacing materials and developing new materials.

9.1 Summary

1 Analysis of the 40 year designs based on existing relationships in DMRB and the new long-life designs show reductions in whole-life costs for the new long-life designs. This is principally a result of lower initial construction costs. The HMB roadbase option has the lowest whole-life cost but the differences are small.

2 These analyses have considered normal surfacing lives. Existing surfacings can provide longer lives and new surfacing treatments are being designed to make it easier to achieve long lives. Long-life pavements with longer periods between resurfacing have the potential for reduced maintenance and user costs, and these need to be considered in future analyses.

3 Substantial reductions in user costs can be achieved for the higher trafficked roads if some or all the maintenance is carried out with night closures. This option is only practicable with resurfacing by inlay when there is no raising of the level of the road surface.

10 Risk analysis

Risk and uncertainty are inherent in all construction projects. A full risk analysis for pavements includes both the qualitative process of identifying possible risks and an understanding of the sensitivity of the risk to changes in the pavement variables. Generally the relationship between these variables and pavement performance cannot be defined precisely, thus precluding a rigorous quantitative analysis of risk. However, a qualitative analysis of risk for flexible pavements can be carried out by listing risks together with a qualitative assessment of how they may be reduced. Information can then be assembled on observed pavement performance in relation to the form of construction and traffic to support the qualitative assessment.

When considering a new design method, changes in risks and the reliability of the resulting pavement need to be considered in relation to current practice. To be acceptable, any increase in risk must be outweighed by cost savings.

10.1 Perceived risks

Risks are associated with the pavement design process, the construction of the pavement and the pavement during service. Some of these risks are well known, whilst a few are of more recent concern. For example, there have been a number of hotter summers in recent years which have aggravated the problems caused by increasing traffic flows and the trend towards the use of super-singles. If such a change in weather patterns continues these factors will impose increasing demands on asphalt surfacing.

The move towards functional specifications or end-product performance specifications for the pavement layers has been gathering momentum over recent years. This form of specification produces a redistribution of risk, with the risk of poor performance being reduced for the Client while the Contractor's responsibility for providing more uniform quality of material is increased. Although a full performance specification has yet to be finalised for any of the pavement layers, a specification is currently in force that ensures a good in-situ level of compaction of asphalt roadbase and basecourse. It gives the Contractor some freedom to design his product and ensures that the material does not contain excess binder (Clause 929, MCHW). This has reduced the risk of problems arising from inferior load-spreading ability and poor resistance to rutting of the main structural layers of the road. The Highways Agency is currently sponsoring projects to develop end-product performance specifications for all pavement layers.

Changes are also occurring in the procurement of road construction and maintenance management. The Public Finance Initiative (PFI) through the Design, Build, Finance and Operate (DBFO) programme will transfer part of the risk from the public to the private sector.

10.1.1 Design methodology

The greatest uncertainty in the design methodology for long-life roads is probably created by the much higher volume of traffic they are expected to carry compared with the current design method. Traditionally, it has been assumed that fatigue weakening or cracking of the roadbase and structural deformation are the major determinants of the life of flexible roads. This is contrary to the observations summarised in Appendix A, which show that well constructed roads, provided they are built thick enough, stiffen over time and can be considered to have a very long life. The design methodology given in Appendix C, which permits the calculation of design thicknesses required to achieve this long life, is based on conservative assumptions and roads constructed somewhat thinner than these are still likely to have long lives. The risk of premature failure for long-life designs, which is discussed in Appendix C, is regarded as low, provided that their condition is regularly checked and appropriate maintenance applied to remedy any deterioration that threatens the structural integrity of the pavement.

Poor estimates of the inputs required for design impose a risk. The design method requires an estimate of the cumulative number of standard axles. For cumulative traffic above 80 msa, pavements will be designed to the conservative threshold strength required to achieve a very long life without further strengthening. The life of this design does not depend on the cumulative traffic and, as a consequence its performance will not be sensitive to the level of cumulative traffic. In this case, the risk to the pavement structure resulting from a poor traffic estimate will be negligible while for pavement designed for less than 80 msa, the risk will be no greater than the current risk. These pavements are expected to have a better than 85 per cent probability of achieving their design life.

10.1.2 Construction risks

The importance of a consistently sound foundation for long-life pavements has already been identified. This requires the use of suitable materials, laid and compacted in an appropriate manner. Experience has demonstrated that Type 1 sub-base, when graded correctly, provides a suitable foundation material for long-life pavements. However, stiffer foundations will result in smaller pavement deflections under a loaded axle and a reduction in the potential risk of structural deterioration caused by traffic induced strains.

Laying and compaction is currently carried out to a method specification (MCHW 1), with a roller of known mass providing a specified number of passes to a known thickness of laid material. The segregation of unbound materials and the compaction in *over-dry* conditions can result in an inadequate long-life pavement foundation. The introduction of end-product testing of the degree of compaction (density) and the overall strength (elastic stiffness) of the completed foundation would reduce this risk. The use of bound foundation materials would again reduce this risk further.

The foundation is generally designed to carry 1,000 standard axles of construction traffic and is permitted to develop a surface rut of up to 40 mm under these loadings. If the foundation is required to carry a larger amount of construction traffic, the Contractor may need to consider constructing a stronger foundation. In any event, the overall thickness of the foundation layers should be carefully controlled, as the trials described in Appendix B showed that a 15% reduction in the unbound material thickness increased the rutting after 1,000 standard axles from 30 mm to 50 mm. However, if the foundation is to be completely constructed from bound materials, a saving in material thickness may be achieved (Chaddock and Atkinson, 1997).

Another risk encountered during the construction of any pavement is the ingress of moisture from both rainfall and water run-off from adjacent ground. This is particularly critical at certain stages of construction when moisture susceptible materials are exposed or the drainage is absent, or ineffective. Although drainage is designed to carry away surface and sub-surface moisture when the pavement is completed, the stage at which these drainage systems are installed is not specified. It is the Contractor's responsibility to provide *adequate* drainage during construction. He may decide to delay the installation on economic grounds, which will increase the risk of achieving adequate performance in adverse weather conditions.

10.1.3 In-service risks

Traffic estimation, material quality, climatic factors and other external influences are all sources of risk in flexible pavements. The structural integrity of a pavement constructed according to the criteria for long-life pavements will not be sensitive to errors in traffic estimates but, deterioration such as surface rutting and surface wear will be.

Asphalt in the structural layers of long-life roads will be expected to provide acceptable service for 40 years or more. Roadbase material progressively cures, albeit at a diminishing rate over time. The perception is that this material will become progressively more brittle and susceptible to cracking. There is little information available on the performance of very old (aged) roadbase. However, investigations carried out on 12 trunk roads and motorways aged up to 35 years (Leech and Nunn, 1997) found no evidence of roadbase cracking. In the oldest of these roads, the penetration of binder recovered from the lower roadbase of DBM was 6 dmm, the structural condition of the road was very good and no structural maintenance was considered necessary. Curing is strongly dependent on voids content (Chaddock and Pledge, 1994) and in this instance the reason for the low penetration is believed to be the higher voids content (5.1 per cent) of the lower roadbase compared to voids content of the upper roadbase (3.0 per cent). The penetration of the binder in the upper roadbase had reduced to 25 dmm. It is not known at what point, if at all, curing will be considered to be excessive and detrimental but good compaction and regular checks on condition will minimise any potential risk.

The risks affecting both present and future flexible pavements are summarised in Appendix F. The incidence of the various risks is not known precisely and the terms, low, medium and high, are only intended to rank the various risks.

Most other significant risks, not included in Appendix F, relate to HRA surfacing and reflect the complex and conflicting requirements for this material:

- deformation resistance - requires stiff binder, lower binder content, medium voids, angular aggregate and high percentage of coarse aggregate
- cracking resistance and chipping retention - both require a softer binder, higher binder content and a medium percentage of coarse aggregate
- durability - requires low voids to minimise ageing.

Compared to basecourse and roadbase asphalt layers, these are demanding requirements particularly because the wearing course is subject to the highest stresses. The design of HRA wearing course mixes optimises many factors. Although the design process is based on a version of the Marshall test procedure, it also involves two binder content adjustments, one mandatory and the other optional. Volumetric criteria are not necessarily considered in determining the target binder content. This is unusual as most mix design procedures, whether based on Marshall or gyratory compaction, do consider mix volumetrics and specify limits for air voids content or voids filled with binder. Daines (1995) has indicated that air voids criteria can be used for both mix design and for controlling the laying of hot rolled asphalt. The Highways Agency will include an air voids requirement in the Specification for Highway Works (MCHW) in 1997 under Clause 943 and Clause 929.

Age hardening of the binder in the wearing course increases the risk of surface cracking and fretting. Only binder hardening during manufacture and laying is controlled in the bitumen specification (by the rolling thin film oven test (RTFOT)). However, unless and until a rapid conditioning procedure can be established, which realistically simulates long-term ageing in the pavement, no further ageing criteria can be specified. A long-term ageing procedure (taking 120 hours) has been developed as part of the US Strategic Highway Research Program and incorporated in an AASHTO provisional standard (1994). However, test precision and its relation to field ageing have yet to been demonstrated.

Exceptionally hot summers during the last decade or so have led to excessive rutting on a number of major trunk roads and motorways, especially where traffic speeds are reduced due either to congestion, contraflow or climbing gradients. This problem has been aggravated by heavier flows of commercial traffic and the trend towards triple *super-single* wheels replacing tandem dual wheels. These demanding conditions will require improved wearing course and basecourse materials. To this end, trials of an end-product performance specification for HRA wearing course are being monitored by TRL on behalf of the Highways Agency. This specification includes wheel tracking criteria at both 45°C and 60°C (MCHW, Clause 943). Stone mastic asphalt and thin wearing course materials are also likely to provide suitable solutions, especially if they are laid on a deformation resistant substrate.

Super-singles are wider than the individual wheels of duals and have a lower maximum total axle weight. Concern over the damage they cause has been expressed (Bonnaquist, 1992 and Sebaaly, 1992). Accelerated testing of pavement in the TRL Pavement Test Facility, sponsored by the Highways Agency, produced 4 times greater wear after 0.5 msa using a super-single assembly than 1.8 msa applied through a dual wheel assembly (Blackman, Earland and Halliday, 1996). The greater damaging affect of these wide wheels, particularly rutting, is supported by theoretical studies, static load tests and accelerated trafficking of trial pavements but their effect on in-service pavements is difficult to quantify.

10.2 Observed pavement performance

There is very little information available on which to assess the risks associated with pavements expected to survive 40 years or more because pavements constructed to the higher standards introduced in the mid-1980s have not yet carried sufficient traffic. Most of the available performance information, obtained through routine major maintenance investigations, relates to pavements designed using Road Note 29 (Road Research Laboratory, 1970). Since then, better material specifications have evolved and typically HD 26/94 (DoT, 1994) requires the total thickness of the asphalt layer to be about 45 mm greater than the earlier designs.

The National Audit Office (NAO Report, 1990) has identified some risks after examining the records of the major problems relating to roads and bridges since 1975. The NAO found that of the 63 roads with reported problems, 34 related to either the granular foundation layers or to the asphalt materials of flexible pavements. Specific problems included wet-mix roadbase, poor performance of asphalt in resisting both summer rutting and winter cracking, poor performance of surfacing laid in cold weather, geotechnical problems and early strengthening due to higher than expected traffic loading. The report states that inadequate design and specifications were the more frequent causes of defects rather than supervision and workmanship. Most of the pavements examined by the NAO had been designed and constructed in accordance with Road Note 29 (Road Research Laboratory, 1970). Many of the problems should not recur because either the design methods or the material specifications have been improved or changed. Wet-mix roadbase is no longer specified and the difficulty of overlaying under overbridges has been eased by an increase of the mandatory headroom clearance to allow for this. The reasons for the several instances of traffic under-forecasts are not known but may have been the result of limited area traffic models. The pressure from Public Inquiries of road schemes to consider the heaviest possible traffic (representing the severest environmental impacts) as well as the lower, conservative traffic flows used for economic justification, should make under-forecasting less likely in future. For pavement strengthening purposes, the assessment of future traffic should be less susceptible to error provided that the trunk road network is not significantly altered.

The TRL experience of flexible pavement deterioration, obtained from detailed investigations and also from reports by the maintenance agents and their consultants, is summarised in Appendix A. The main findings are that, in practice, there is very little evidence of structural deterioration in the form of structural rutting or cracking at the bottom of the roadbase of thickly constructed flexible roads. This view has been confirmed by a survey of opinions of 15 County Council Materials Engineers. Except in cases where there were clear initial defects in the roadbase, all the deterioration originated at the surface. The distress typically took the form of wheel-track rutting and cracking, affecting the wearing course and sometimes the basecourse. It was also very clear that many of the pavements investigated had carried considerably more traffic than the cumulative traffic levels predicted for their designs using current methods. Asphalt pavements of inadequate thickness or with built-in defects required strengthening in order to achieve their design expectations.

A single local geotechnical problem has been identified on the M25 and also some instances of sub-base and capping deterioration. In the majority of investigations, the only unsatisfactory features found were surface defects and short residual lives predicted from deflectograph measurements.

Poor drainage is generally considered to be a significant factor in pavement performance but obtaining examples of this is difficult. Sir Owen Williams and Partners (1988), in a general review of drainage practice, have reported that there was little hard evidence to show that pavement failures had been caused by breakdowns in drainage. Examples were given of a public utility severing gully connections and of unsealed filter drains leading to voids formation in chalk. No cases of high water levels in the ground causing failures were reported but failures had occurred due to water trapped within the pavement layers, particularly in stage construction or reconstructed pavements where water in permeable layers could not find an outfall.

10.3 Comparison of risks of conventional and long-life designs

Long-life pavements differ from conventional pavements in two important respects:

- better quality control of construction will ensure that future problems are not built into the pavement. End-product performance specifications are seen as a means of eventually ensuring that all pavement material layers are designed to be fit for purpose.
- apart from periodic resurfacing, they are expected to last in excess of 40 years.

Improved quality control will significantly reduce the risk of unsatisfactory roadbase and foundation layers because the compaction, performance properties and layer thicknesses of the laid material will be measured. Long-life pavements with regular condition checks will lead to a better understanding of pavement deterioration with the identification of which layers are affected and the underlying cause. The role of deflection measurement will change from being the principal indicator of residual life to that of locating weak areas.

Evidence from many pavements that have carried considerably more traffic than predicted by HD 26/94, suggests that there is little risk of the roadbase of a properly constructed long-life pavement not performing satisfactorily. Assuming that HRA wearing course is used, the risks to the surfacing of conventional and long-life pavements will be the same.

Considering the rutting problems associated with some conventional HRA wearing courses during the recent hot summers, alternative surfacing material such as thin surfacing, stone mastic asphalt or HRA designed to be more stable appears attractive for both conventional and long-life pavements. However, the risks associated with alternative materials differ from those relating to HRA. For example, from experience in Germany, the life expectancy of stone mastic asphalt is between 15 and 18 years, about 3 years longer than HRA. In Germany there are no friction or texture standards to be complied with and, therefore the effective life may be shorter in the UK. Whilst there may be concern over the performance of this material later in its life, the risk of early life rutting is very low.

The eventual extensive use of end-product performance specifications will lead to all layers, including the wearing course, performing better. This form of specification encourages the development of new materials and is relatively adaptable. Performance criteria can be set at higher levels or new criteria introduced if problems persist.

10.4 Summary

1 Risks are associated with each stage in the development of a pavement; in the design process, construction and service life.

2 Hotter summers, increased traffic flows and the greater use of super-singles will impose greater demands on asphalt surfacing. Good quality control of construction and the introduction of end-product performance specifications for all pavement layers will reduce future risks.

3 Regular checks on pavement condition and timely and appropriate intervention to remedy surface distress before it begins to effect the structural integrity of the pavement will reduce the risk of poor pavement performance.

4 Inadequate design, poor materials and construction practice, and a lack of appreciation of the manner in which pavements deteriorate when carrying out condition assessments are seen as the greatest threats to satisfactory performance.

11 Implementation

This project has resulted in a number of significant developments. Some are firmly based, and could be implemented with relative ease, while others may either require further development or, because of their more radical nature, may require a gradual phased introduction. The more important developments are:

- Long-life design
- Innovative materials
- End-product performance specifications
- Pavement condition assessment.

The status of these developments and the implementation processes are discussed in this Section.

11.1 Long-life design

The recommended design thicknesses for long-life flexible pavements, using standard materials, could be incorporated into the current design standards.

This project has indicated that the elastic stiffness modulus, which is a measure of load spreading ability, is the most important structural property of the roadbase. The road is most vulnerable in its early life before the structural properties of the roadbase have fully developed by curing. At this time, the calculated level of stress and strain induced by a wheel load in the pavement structure is the main indicator of future performance. Therefore, design thicknesses of the asphalt layers should be related to the early life properties of the main structural layers. This assumption is implicit in the development of an end-product performance specification for the asphalt roadbase.

Most of the current information on the initial elastic stiffness modulus of asphalt, and its subsequent behaviour over the life of the road, comes from measurements using the indirect tensile test. The Highways Agency is currently sponsoring programmes of work to improve the precision of the test and compile a database of properties of newly laid materials from road construction contracts. This work, when it is sufficiently progressed, will enable design criteria to be established based on the early life structural properties of the asphalt roadbase. The way will then be clear for the introduction of a performance-based specification.

11.2 Innovative materials

The Highways Agency has had a procedure in place for some time for assessing new materials that involves desk and laboratory studies and pilot- and full-scale trials. If materials then show good potential, specification trials are carried out under contractual conditions before they are permitted to be used more widely. Specification trials are normally required for the following reasons:

a To ensure that contractual problems do not make the innovation unattractive.

b To test the specification to determine whether it requires changes.

c To assess variability and design parameters over a range of conditions.

d To assess future costs and benefits of the innovation.

This assessment process has recently been used for high modulus base (HMB) to demonstrate its potential in more cost effective long-life roads (Nunn and Smith, 1997).

To stimulate innovation and to anticipate developments in European Standardisation, the Highway Authorities in the UK have promoted a formal assessment framework devised to control proprietary products. This scheme is named the *Highway Authorities Products Approval Scheme* (HAPAS), administered by the British Board of Agrément (BBA), and it covers the interests of all highway authorities (Nicholls, 1997b).

Under HAPAS, the assessment is initially based on laboratory tests that are defined to cover the properties required of the material. In addition to this, the promoters have to demonstrate that their systems can be applied on site and, if there is insufficient evidence of successful performance, the application trial will be extended into a two year road test. Hence, accreditation will take at least two years for new products.

11.3 Performance-based specifications

The main obstacle to the introduction of end-product performance specifications has been the availability of practical and economic test equipment. A realistic performance specification will be a mixture of material performance properties that can be measured directly and surrogate properties for use when suitable tests are not available to measure the desired property directly.

The development of end-product performance specifications for all pavement layers is being sponsored separately by the Highways Agency but, as part of this study, the potential for adopting a performance specification for the road foundation was examined. The way forward from the present position to implementation, described in the following paragraphs, can be applied generally to any performance specification method.

The investigations described in Appendix C concluded that a performance specification for the road foundation would be potentially advantageous. A comprehensive range of tests are required to support a full performance specification. The Contractor also needs to be familiar with these tests and be experienced in controlling his material to achieve the specified properties. In order to specify values, the Client will require knowledge of the relationship between the measured values, using the proposed tests, and the performance of the foundation, both during construction and for the in-service pavement. In addition the specified sampling and testing regimes must be practicable and robust. The following steps will enable the necessary information and experience to be accrued so that the specification method can be assessed for feasibility of implementation:

1 A phased development of a practical method of specification is introduced in selected contracts. The present work will form the basis of the initial performance specification trial. The specification will be refined in a number of stages with each stage benefitting from feedback from the Contractor and Client.

2 At the same time, measurements using the end-product tests together with other supporting tests would be specified for all new construction and reconstruction sites. This will require Contractors to purchase and operate the test equipment and become familiar with its operation and gain insight into the behaviour of their materials. The systematic collection of this information will aid the specification of appropriate performance levels.

There are, at present, on-going trials to develop performance specifications for asphalt roadbase and HRA wearing course using the approach suggested above. These are further developed than the performance specification for the road foundation.

11.4 Pavement condition assessment strategy

This project has demonstrated that thick, well-constructed fully flexible pavements, that have performed well in their early life, are unlikely to exhibit structural weakening thereafter. However, it is likely that a significant proportion of the network will have been constructed to lesser standards and will have only achieved the threshold thickness for long-life following the application of strengthening overlays. It is quite likely that the underlying structural layers had deteriorated before the overlays were applied.

Long-life roads do not generally behave in the manner previously expected (eg. deflection can decrease instead of increase with time, cracking invariably starts at the surface rather than at the bottom of the asphalt roadbase and deformation is more likely to occur in the asphalt surfacing than deep in the pavement structure). Thus, provided that cracking is arrested before it is allowed to penetrate the main structural layers, the pavement will have a structural life that is far in excess of the design lives anticipated under current design relationships. These major changes in our understanding of how pavements behave make the correct diagnosis of pavement condition even more important.

The change in deterioration mechanisms requires a re-examination of the emphasis in the current pavement assessment procedures. Although deflection will still play a part in identifying structural problems, the residual life and overlay design elements of the deflection design system no longer apply to roads conforming to the criteria for long-life. For this reason the term *investigatory condition* has replaced *critical condition* (DMRB 7.3.3, HD 30).

There will also be major changes required in the way results from other surveys are used and interpreted. Modifications to the current assessment strategy are being considered in another on-going research project being carried out by TRL on behalf of the Highways Agency.

12 Future design

Deterioration of the pavement by cracking initiating at the surface and propagating downwards has received relatively little attention from researchers. However, a better understanding of the deterioration mechanisms in general will result in better designed materials and improved construction practices that prevent or delay the onset of problems. The ultimate goal of pavement design is to develop a mechanistic method that is based on a fundamental understanding of the behaviour of materials in the road. Although this goal will be difficult to achieve, it should be pursued. A fully developed and validated analytical design method will lead to better pavement design, will aid economic planning and will give insight into the consequences of future changes in materials or vehicle characteristics.

This paper provides the link between the design inputs and design thicknesses. The relationships developed will be dependent on historical evidence, therefore, there is a need to continuously monitor factors that may affect pavement performance. For the future, there are many changes afoot such as heavier lorries, air suspension, increasing proportions of super-single tyres and the development of innovative materials and construction practices, all of which will need careful monitoring so that designs can be adjusted, as necessary, to ensure that we continue to obtain value for money from road pavements.

13 Conclusions

1 A well-constructed, flexible pavement that is built above a defined threshold strength will have a very long structural service life provided that distress, in the form of cracks and ruts appearing at the surface, is detected and remedied before it begins to affect the structural integrity of the road.

2 No evidence of structural deterioration due to fatigue or cracking of the asphalt roadbase, or deformation originating

deep within the pavement structure, has been found in existing roads that conform with the criteria for long-life.

3 The concept of critical condition, based on pavement deflection, is not applicable to these thicker, well-constructed, flexible pavements. This has far-reaching implications for the design and condition assessment of fully flexible pavements. Many of the existing procedures for pavement monitoring and investigation can be applied to long-life roads, but there needs to be a change of emphasis and the introduction of some new techniques. The most important change will be the reduction of the influence of pavement deflections to predict the remaining life of the pavement. For this reason the more descriptive term *investigatory condition* has replaced the term *critical condition*. The strategy for condition assessment is currently under review by the Highways Agency.

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15 Glossary

AADF	Annual Average Daily Flow
CBM	Cement Bound Material
CBR	California Bearing Ratio
CHART	Computerised Highway Assessment of Rating and Treatment
COMPARE alternative	Whole life cost model to compare designs and maintenance strategies
CRF	Crushed Rock Fines
D2AP	Dual 2-lane All Purpose road
D3M	Dual 3-lane Motorway
DBFO	Design Build Finance and Operate
DCP	Dynamic Cone Penetrometer
DBM	Dense Bitumen Macadam (100 penetration grade binder)
DBM50	Dense Bitumen Macadam (50 penetration grade binder)
DMRB	Design Manual for Roads and Bridges
DSR	Dynamic Shear Rheometer
EME	Enrobé à Module Élevé
FWD	Falling Weight Deflectograph
HAPAS	Highway Authorities Products Approval Scheme
HAPMS	Highways Agency Pavement Management System
HDM	Heavy Duty Macadam
HMB	High Modulus Base
HMB35	High Modulus Base (35 penetration grade binder)
HMB25	High Modulus Base (25 penetration grade binder)
HMB15	High Modulus Base (15 penetration grade binder)
HRA	Hot Rolled Asphalt
HSTM	High Speed Texture Meter
ITSM	Indirect Tensile Stiffness Modulus
MTM	Mini-Texture Meter
MCHW	Manual of Contract documents for Highway Works
NDT	Nuclear Density Test
NAO	National Audit Office
PA	Porous Asphalt
PANDEF	Processing and ANalysis of DEFlections
PDPBT	Portable Dynamic Plate Bearing Test
PFI	Public Finance Initiative
PRD	Percentage of Refusal Density
PSV	Polished Stone Value
PTF	TRL Pavement Test Facility
QCRS	Quality Control Reporting System (DoT)
RMMS	Routine Maintenance Management System
RTFOT	Rolling Thin Film Oven Test
sa	Standard axle
SBR	Styrene-Butadiene Rubber
SBS	Styrene-Butadiene-Styrene block copolymer
SCRIM	Sideway force Coefficient Routine Investigation Machine
SMA	Stone Mastic Asphalt, Splittmastixasphalt (Germany), Stone Matrix Asphalt (USA)
TLA	Trinidad lake Asphalt
WLC	Whole Life Cost

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Appendix A: Pavement performance

The majority of the information presented in this Appendix comes from previous research funded by the Highways Agency. In this project the information was interpreted and limited studies carried out to confirm earlier findings.

A1 Introduction

Flexible pavement design methodologies have been developed to enable pavement engineers to design the pavement thickness to control the observed mechanisms of deterioration under the predicted level of traffic. The design method for fully flexible pavements developed by Powell et al (1984), in TRRL Laboratory Report LR 1132, was based on the assumption that asphalt roads deteriorate due to the accumulation of small amounts of damage caused by the passage of each commercial vehicle. The damage mechanisms are considered to be either a fatigue phenomenon that causes weakening and eventual cracking at the underside of the roadbase or structural deformation that originates in the subgrade. Over the last decade more information has been obtained that indicates that the predominant forms of deterioration in thick, well-constructed, fully-flexible pavements are not structural and only affect the uppermost layers of the pavement as cracking and rutting.

This Appendix draws together information on the performance of fully-flexible pavements that form part of the trunk road and motorway network in the United Kingdom. This information comes from full-scale experimental pavements, studies of deterioration mechanisms on the road network, long-term deflection monitoring of motorways and condition assessment reports prepared to aid the design of structural maintenance.

A2 Performance

A2.1 General

The current UK design method requires the road to be designed to reach a *critical condition* after about 20 years, at which stage it requires a strengthening overlay to extend its life for a further 20 years. The critical condition is considered to represent the latest opportunity when the existing strength of the road can be used to good effect in the design of a strengthening overlay, otherwise reconstruction will be necessary. Kennedy and Lister (1978) showed that the critical condition can be related to the transient deflection under a standard wheel load moving at creep speed. This deflection is predicted to increase gradually by traffic until the onset of the critical condition that, on roads constructed to a reasonable specification, was shown to correlate broadly with the development of rutting and cracking.

This method is based on the performance of experimental roads, which had carried up to 20 million standard axles (msa), interpreted in the light of structural

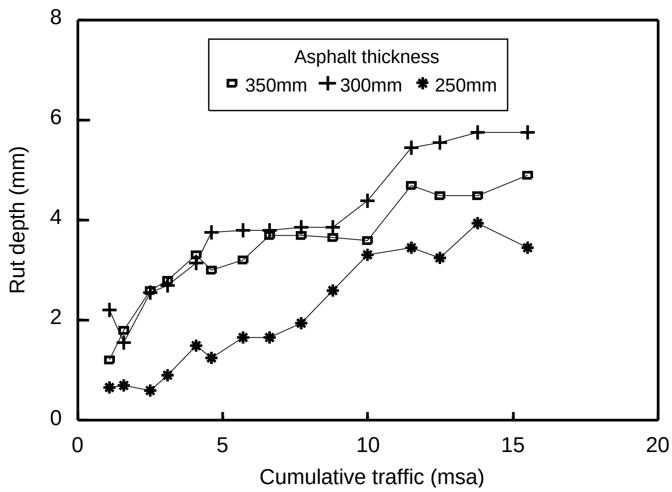

Figure A2 Rut development of experimental pavements

Figure A1 Surface rutting on a section of the M6

theory. It was therefore necessary to assume that the measured performance trends could be extrapolated to give realistic estimates of future performance, and that these trends were good indicators of structural deterioration.

The present position is that the heaviest trafficked motorways and trunk roads have carried in excess of 100 msa and that these roads provide an opportunity to confirm the validity of the extrapolation. In the following sections the observations of rutting, fatigue damage and long-term pavement strength are reviewed. The criteria for judging critical condition and the relevance of the concept of critical condition are also assessed.

A2.2 Rutting

Rutting is the result of deformation in one or more of the pavement layers. At one extreme the deformation is restricted to the uppermost asphalt layer or layers, termed surface rutting, and at the other extreme, the main component of deformation will arise in the subgrade and this is termed structural deformation. Deformation within the upper asphalt layers will begin to affect the structural integrity of the pavement when it becomes excessive. An example of this form of deterioration is given in Figure A1, which shows a cross-section of the M6 in which the rutting can be seen to be confined to the wearing course and basecourse. On the other hand, excessive structural deformation is a symptom of the load spreading ability of the asphalt and granular layers being insufficient to protect the subgrade from the effects of traffic and, if unchecked, it will lead eventually to a break-up of the pavement structure. Measurement of the rutting profile at the surface only does not identify the source of the rutting. The consequences for pavement design and maintenance depend substantially on whether the deformation originates solely in the surfacing or deep within the pavement structure.

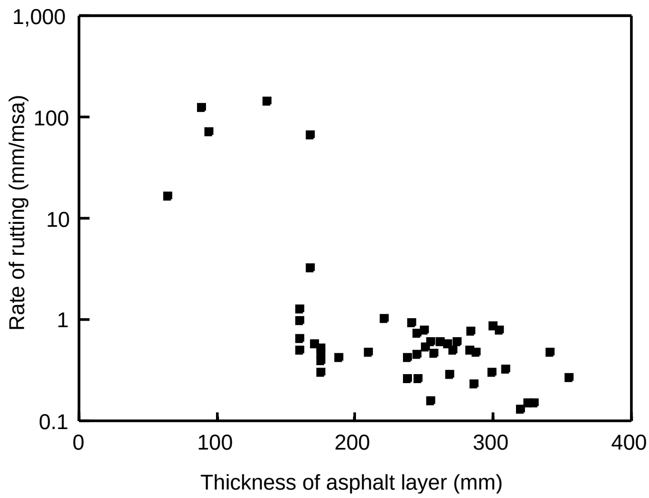

Figure A3 Measured rate of rutting of trunk roads with DBM roadbase

The measured rates of surface rutting of sections of experimental pavements, with DBM roadbase, are shown in Figure A2.

In this example of three pavements of different thickness from the same site, the thinnest pavement has the lowest rutting rate and the intermediate thickness, the greatest. The rates of rutting are not related to thickness and therefore are unlikely to indicate a structural origin for the ruts. The performance shown covers a period of more than 15 years and the final rut depths are less than 6 mm. This represents good performance and exceeds the usual life expectation for an asphalt surfacing. Structural rutting is unlikely to be a problem with pavements displaying this behaviour.

Characterising the behaviour of pavements by the mean rut rate is likely to lead to some pessimistic estimates of performance. In the example above, the rutting rates are reasonably constant with time and traffic but in other cases the rate reduces markedly with time. However, unless the pavement has been continuously monitored, the mean rut rate is the most convenient measure because it only involves measurement of the current level of rutting and an estimate of the cumulative traffic.

A summary of the mean rutting rates of a number of pavements with DBM roadbases is shown in Figure A3.

Table A1 Comparison of rates of rutting

	Rate of Rutting (mm/msa)			
Roadbase Type	*Mean*	*Standard Deviation*	*Sample Size*	*Life for 10 mm Rut (msa)*
HRA or DBM (subgrade CBR < 5%)	0.58	0.18	20	17
HRA or DBM (subgrade CBR > 5%)	0.36	0.12	21	28
Lean concrete	0.38	0.14	28	26

Rutting from both experimental pavements built into the trunk road network and normal pavements investigated by TRL are represented. For many of the sites the rutting behaviour was similar to that in Figure A2.

Figure A3 indicates a discontinuous relationship between the rate of rutting and pavement thickness, the data forming two clusters. Pavements with less than about 180 mm of asphalt deform at a high rate but thicker pavements deform at a rate about two orders of magnitude less; the sudden transition between the two types suggests a threshold effect. The measured rates of rutting of pavements with hot rolled asphalt (HRA) roadbase were similar to those of pavements with DBM roadbase.

Above the threshold thickness, in the range 180 to 360 mm, there is no correlation between the rate of rutting and pavement thickness. This was also illustrated in Figure A2, which shows a lack of correlation between thickness and surface rutting in three pavements even though they were constructed with different thicknesses of asphalt. The foundations and the materials used were nominally identical. The results suggest that for these thicker pavements nearly all the rutting is due to deformation within the upper layers and that the traffic induced strains in the subgrade are too low to cause structural deformation. It is apparent that for asphalt thicknesses less than about 180 mm, the much higher traffic induced subgrade strains have a substantially greater effect. However, the lack of data for the thinner pavements prevents a clear relationship between rate of rutting and thickness being defined.

The effect of the different subgrade strengths of the experimental roads has also been reviewed. It was found that, for roads constructed on soils with a subgrade equilibrium CBR less than 5 per cent, the rate of surface rutting was significantly greater than for roads on subgrades of CBR greater than 5 percent, irrespective of the thickness of the asphalt cover. The average rates of rutting are given in Table A1 below. The predicted lives to 10 mm rut depth, assuming a linear relationship with traffic, are also shown.

The use of capping layers in modern road design guarantees a foundation equivalent to at least 225 mm of sub-base over a CBR of 5 percent and, therefore, modern roads would be expected to rut at the lower rate of 0.36 mm/msa. This figure is almost identical to the rutting rate predicted by Jacobs (1981) for good HRA wearing course of 0.35 mm/msa for deformation within the asphalt layers. It therefore seems likely that the majority of the rutting, on roads with a strong foundation, occurs in the upper asphalt layers and that rutting in these cases is not likely to be an indicator of structural deficiency.

Further supporting evidence comes from experimental sections of flexible composite construction which were laid at the same time as the fully flexible sections, by the same contractors using the same sources of asphalt. The flexible composite pavements examined had predominantly 100 mm of asphalt surfacing, although the thicknesses for individual sections varied between 90 mm and 160 mm, and no longitudinal cracks were present in the cement bound roadbase, laid more than 150 mm thick. Therefore, all the deformation must have been in the asphalt and the majority of the rutting must have occurred in the wearing course and basecourse. The average rate of rutting of 0.38 mm/msa that occurred in the flexible composite sections was not significantly different from that for the fully flexible roads with a foundation at least as strong as that referred to above (i.e., built to a minimum equivalent to at least 225 mm of sub-base over a CBR of 5%).

A similar rutting phenomenon was observed at some bridges where rutting on the approaches persists across the bridge deck. The deck ruts can only be the result of deformation within the 100 mm or so of the bridge surfacing. Provided that the ruts on the approaches are of similar depth and character, these must also occur in the surface rather than in the structure. An example of this is given in Figure A4.

A feature that is apparent in the data from the experimental pavements, is that each site has a characteristic behaviour and consequently there is less variation in the performance of experimental sections within the same site than in nominally similar constructions at different sites. This may reflect variability in material quality and construction practice and possible short comings in the concepts of CBR and standard axle. However these sources of variability are reasonably

Figure A4 Surface rutting extending onto a bridge deck

consistent within a site. The pavements within each experimental site had significantly different thicknesses of asphalt but, as illustrated in Figure A2, there is no large and consistent effect of thickness on performance within the site.

A2.3 Fatigue

The roadbase is the most important structural layer of the road but, unlike the wearing course in which cracks are easy to observe, assessment of its structural condition is difficult because of its position in the road; consequently the mechanisms of roadbase deterioration are less clearly understood. All modern analytical design methods include a criterion, based on laboratory studies, to guard against the possibility of fatigue cracks initiating at the underside of the asphalt roadbase. These methods consider fatigue cracking, caused by repeated traffic loading, to be the major form of structural deterioration. Investigation of the roadbase fatigue mechanism in full-scale pavements is much more difficult than in the laboratory and Goddard and Powell (1987) and Thrower (1979) noted that, although surface cracking is often observed, there is little evidence of fatigue cracking in the roadbase of in-service asphalt pavements in the UK. Furthermore, it is known that the stiffness of asphalt roadbase increases with time and that this influences the fatigue resistance of the road. However, this effect has received little attention in the past in relation to roadbase performance.

The indicators of conventional roadbase fatigue are considered to be:

Table A2 Pavements investigated

Site	*Age (years)*	*Cumulative traffic (msa)*	*Roadbase type*	*Thickness of asphalt layer (mm)*
M4	11	22	HRA	230
M5	19	66	DBM	300
M1	23	71	DBM	350
M62	21	57	DBM	300

- the occurrence of cracks at the underside of the roadbase
- a lower residual fatigue life and stiffness of roadbase samples taken from heavily-trafficked sections of pavement compared to lightly-trafficked sections
- a lower residual fatigue life and stiffness of pavement samples taken from the lower part of the roadbase, where the tensile strains are high, compared to the top.

The absence of positive evidence for fatigue prompted TRL to initiate an investigation into the residual fatigue life of asphalt roadbases from heavily- and lightly-trafficked areas of road. In this study, roadbase materials were extracted from selected motorways for subsequent laboratory testing. The aim was to compare the structural properties of samples of roadbase measured in the laboratory with the overall condition of the pavements from which they were extracted. A literature review revealed that no other similar detailed study had been carried out elsewhere. This work, which is reported in more

Table A3 Comparison of fatigue life of roadbase

Site	*Lane*	*Number of tests (N)*	*Relative fatigue life of roadbase*
M4	1	80	1.0
	3	76	1.5
M5	1	35	1.0
	3	33	0.7
M1	1	19	1.0
	3	11	1.6
M62	1	28	1.0
	3	31	0.4
Mean	1	162	1.00
	3	151	1.05

detail by Wu (1992) and Nunn (1992), has helped to improve our basic understanding of the mechanisms of structural deterioration and the essential findings are summarised in the following section.

A2.3.1 Investigation of roadbase fatigue in UK motorways

Short sections of four motorways, representing a range of age and traffic loading, were selected for detailed investigation. The sections chosen were based on previous structural surveys and they were representative of the condition of significant lengths of carriageway. All the pavements examined had exceeded their nominal design life, as determined either by the standard in force at the time of construction or by the current design standard. Details of the sites are given in Table A2.

Cores were cut to enable the structural properties of materials that had been subjected to heavy commercial traffic in the wheel-path of lane 1 to be compared to the lightly trafficked material of the same age and nominal composition from between the wheel-paths of lane 3. Specimens for fatigue testing were cut separately from the upper and lower roadbase layers and tested using the TRL laboratory fatigue method described by Goddard, Powell and Applegate (1978). A comparison of the laboratory measured residual fatigue life of the roadbase from lanes 1 and 3 and the calculated fatigue life of the pavements is shown in Table A3.

If traffic had a weakening effect on the roadbase, then the residual fatigue life of roadbase material subjected to heavy traffic in lane 1 would be significantly lower than that of the untrafficked material extracted from between the wheel-paths of lane 3. Table A3 shows that there was no significant and consistent difference between the measurements on samples extracted from lanes 1 and 3. The differences at each site were mainly accounted for by differences in binder hardness and binder content in the samples extracted from the two lanes. When these factors were taken into account, none of the differences were significant at the 5 per cent level. It is also noteworthy, that the presence of a longitudinal crack in the wheel path of lane 1 of the M5 had only penetrated the wearing course and basecourse and that it had not yet had a measurable detrimental effect on the structural properties of the roadbase immediately below it.

A separate comparison between the top and bottom roadbase layers showed that, again, there were no significant and consistent differences in fatigue life and stiffness. The widely held understanding is that tensile strains induced by traffic loading cause maximum fatigue damage at the underside of the roadbase. It would therefore be expected that the samples from the bottom layer should have been in a worse condition if traffic loading was a major factor.

Both of the above comparisons indicate that the level of traffic loading was not the major factor determining the residual fatigue life of the roadbase layers. As the two lanes had carried different traffic loads, any effect of traffic load would be reflected in the differences between the samples.

Material composition is a very important determinant of the fatigue life of asphalt. A multiple regression analysis of the combined data from this study showed that over 90 per cent of the variation in the measured residual fatigue life was accounted for by composition variables, with volume of binder accounting for the largest amount of variability, followed by the penetration of the recovered binder. This illustrates that differences in the fatigue lives between materials from the four motorway sites are accounted for by variations in material composition rather than by differences in traffic loading.

While traffic loading was not found to be the major factor for the difference in fatigue life, age will have been an important factor because it is well known that binders age with time, resulting in increased binder stiffness. However, the gradual hardening of the roadbase materials with age, as demonstrated by a shorter residual fatigue life and higher elastic stiffness, is very unlikely to result in fatigue cracking of the pavement. Provided that the surfacing remains intact, it appears that the roadbase is able to perform its structural function for a considerable period of time. Calculations using the relationships developed in this study show that the increase in elastic stiffness with age produces a reduction in the traffic induced tensile strain at the underside of the roadbase that is responsible for fatigue. This reduction more than compensates for the lower laboratory fatigue life of the aged roadbase. The net effect is that the predicted fatigue life of the road increases with age. This is probably the reason why no positive evidence has been found of fatigue cracking in any of TRL’s experimental roads; the available evidence does not support the occurrence of classical fatigue cracking originating in the roadbase.

A2.3.2 Further supporting evidence

Structural assessments of asphalt roads carried out by TRL and Agent Authorities have failed to detect any evidence of roadbase fatigue damage (Leech & Nunn, 1997). There are no authoritative reports of cracks propagating upwards. In older roads, apart from the absence of roadbase cracks, the measured elastic stiffness modulus of roadbase material extracted is usually substantially higher than that expected for new construction. This is indicative that fatigue weakening is not occurring.

A literature search revealed that no other country had carried out an investigation of the residual fatigue life of the roadbase similar to that carried out by TRL. However, The Netherlands have recently carried out detailed investigations and have concluded that conventional roadbase fatigue is unlikely to play a significant role in the structural deterioration of thicker, well-constructed roads.

The Road and Hydraulic Engineering Division of the Dutch Ministry of Transport has examined 176 sections of flexible pavement (Schmorak and Dommelen, 1995) to verify their pavement design method. This study revealed that in pavements with a thickness of asphalt greater than 160 mm, cracks initiated at the surface and penetrated to a depth of approximately 100 mm. In thinner pavements, where the cracks penetrated the full thickness of the asphalt, a structural analysis, which examined the likelihood of roadbase fatigue, suggested that the cracks in these roads had also initiated at the surface and propagated downwards. The overall conclusion of this work was that, *conventional fatigue will rarely or never be the predominant failure mechanism, but surface cracking will be the main cause of structural distress.* It was also concluded that future research should concentrate on surface cracking rather than conventional fatigue.

Although the fatigue behaviour of asphalt can be measured in the laboratory, there is no field evidence for it occurring at the underside of the roadbase in thicker, well-constructed pavements. As pointed out by Thrower (1979), the laboratory conditions of loading, stress distribution and environment are very different from those occurring in real pavements.

A2.4 Surface cracking

Cracking of the surface of thick, mature, flexible pavements is relatively common. The most usual form is longitudinal, single or multiple cracking in one or both wheel tracks. This cracking has often been regarded as evidence of conventional fatigue in which cracks are assumed to have initiated at the bottom of the roadbase and then propagated to the surface. However, where such cracking has been investigated by the Maintenance Agent by cutting cores, it is invariably found that either the cracks partially penetrate the thickness of asphalt or if the crack is full depth, the propagation is downwards rather than upwards. Maintenance agents have reported longitudinal cracking with a definite surface origin in the following roads:

A50 Blythe Bridge Bypass	A449 Claines/Ombersley/ Oldfield Lane
M1 Northamptonshire	M5 Junction 14
M26 Junctions 5 to 2a	

A typical example of this type of cracking is shown in Figure A5. This longitudinal crack in the near-side wheel path of the M1 extended continuously for over 1 km. The crack was cored at several locations and only at the most seriously cracked locations, judged from a visual inspection, had the cracks progressed further than the top 100 mm of asphalt. None of the cracks had penetrated the full thickness of asphalt.

Sections of pavement with surface cracking are not necessarily weaker than intact sections. At the M1 Northamptonshire site, deflectograph measurements on cracked and uncracked sections of pavement showed no relative weakness in the former, with deflections in both areas generally being less than 0.20 mm. Penetration tests on the binder recovered from the HRA wearing course showed somewhat lower values (10 to 34 pen) in cracked areas compared to uncracked lengths (32 to 52 pen). Penetration tests on binder recovered from the DBM upper roadbase showed no clear differences between cracked and uncracked material. However, binder penetration is not always a good indicator of the susceptibility of the wearing course to surface cracking. In some cases the penetration of the recovered binder measured at 25°C from cracked sections has been found to be similar to that of the uncracked sections at the same site but, in other cases, harder binder has been extracted from the uncracked sections. It is evident that more detailed studies will be required to understand the loading, material and environmental factors that determine the initiation and propagation of cracks.

Longitudinal surface cracks have also been observed at a number of sites investigated by TRL on behalf of the Highways Agency:

A1(M) Durham Bypass	A3 Thursley bypass
A30 Blackbush	A38 Derby
A500 Stoke-on-Trent	M1 Leicester
M5 Junctions 10 to 11	M6 Junction 15 to 14
M23 North of M25	M25 Junctions 6 to 7
M25 Near Junction 8	M26 Near A225 overbridge
M32 Junction 1 to 2	

At these sites, provided the crack had not propagated into the roadbase, there was no observable or measurable damage to the roadbase directly beneath the longitudinal wheel-path cracks. The elastic stiffness of the roadbase was similar to material extracted from relatively untrafficked areas of the road. In the case of the M5, the measured residual fatigue life of the roadbase from the cracked near-side wheel-path was not significantly different to material extracted from the lightly-trafficked and uncracked area between the wheel-paths in lane 3.

Surface cracking is not always longitudinal. On other sites investigated by TRL, either transverse or block cracking occurred, it was not confined to wheel tracks and occurred in both lanes of each carriageway. Figure A6 gives an example of transverse cracks in the M32. This motorway had carried 30 msa since it was laid 24 years ago, and in that

(a) Longitudinal crack in the near-side wheel-path

(b) Core showing depth of cracking penetration

Figure A5 Longitudinal surface cracking on the M1

(a) Transverse crack in lane 1

(b) Core showing depth of crack penetration

Figure A6 Transverse surface cracking on the M32 time it had not been resurfaced or overlaid. The penetration of recovered binder from the surfacing was 15 and it was concluded that the cracking was not traffic related but due to brittle surfacing resulting from aged binder. As with longitudinal cracking, transverse cracks generally only penetrated up to 100 mm into the surfacing.

The phenomenon of surface cracking has not received much attention in the UK. However, there are a considerable number of observations of surface cracking of flexible pavements from other parts of Europe and further afield, for example, in France (Dauzats et al, 1982 and 1987; Maia et al, 1985), Netherlands (Gerritsen et al, 1987; Pronk and Buiter, 1982; de Wit and Cortenraad, 1990; Schmorak and Van Dommelen, 1995), Japan (Himeno et al, 1987) and South Africa (Hugo and Kennedy, 1985).

The mechanism of surface cracking is complex and there is no satisfactory explanation of this phenomenon. Surface cracking is not explained by the conventional

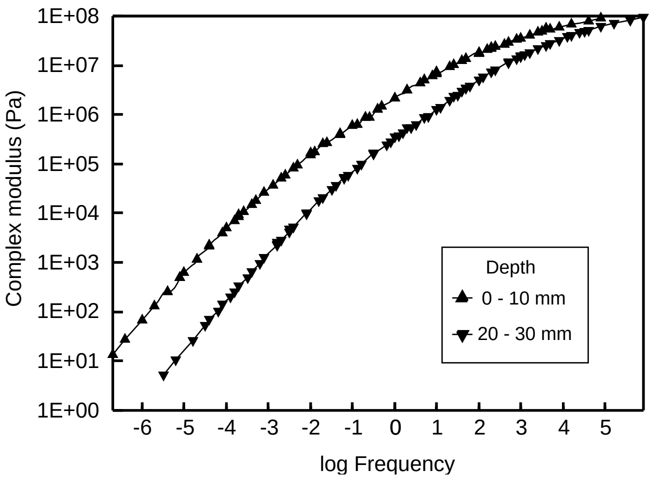

Figure A7 DSR master curves at 25ºC illustrating increased binder ageing near surface

fatigue theory, which models wheel loading as a static, uniform, normal stress, approximately equal to the tyre inflation pressure, applied over a circular area and considers the tensile strain only at the underside of the bound layers, where it is assumed to have the maximum value for all circumstances.

Calculation of the traffic induced stresses at the road surface in contact with the tyre is much more complicated due to the vertical contact stress being non-uniform over the tyre contact area and the presence of radial horizontal forces (Johansen and Senstad, 1992). The consequences are that significant horizontal tensile stresses can be generated at the surface of the asphalt layers (Collop, 1995). The induced tensile surface stresses under the new wide base tyres (super-singles) are predicted, under certain circumstances, to be *much larger* than those at the underside of the roadbase (Jacobs, 1995). The assumption of linear elasticity to represent the behaviour of the asphalt layers, which ignores time dependent viscous effects, may also be a misleading over-simplification. Strain observations of moving wheel-loads on trial pavements show several effects not explained by simple elasticity, such as accumulating transverse strain and different pavement stiffnesses in the longitudinal and transverse directions (Huhtala et al, 1990).

Thermally generated stresses will also contribute towards the initiation and propagation of surface cracks. This is especially so for transverse cracking where thermal stresses are likely to be the principal cause of the tensile condition required for crack initiation. Age hardening of the binder in the wearing course will also play a part, with hardening over time progressively reducing the ability of the wearing course to withstand the thermal and traffic-generated stresses at the surface.

Recent binder rheological studies reported by Leech and Nunn (1997), as part of a Highways Agency funded research programme to examine deterioration mechanisms in thick pavement, have shown that the binder in the top few millimetres of the wearing course becomes particularly hard with age. Figure A7 gives an example of results obtained, using the dynamic shear rheometer (DSR), from testing binder extracted from the top 10 mm of the wearing course and from between 20 mm and 30 mm below the surface.

This Figure shows that the top 10 mm of the wearing course is substantially harder than that of the lower layer. Van der Poel (1954) showed that the penetration test relates well to the stiffness modulus of the binder at 25°C and at a loading frequency of 0.4 Hz. The investigations of 4 sites

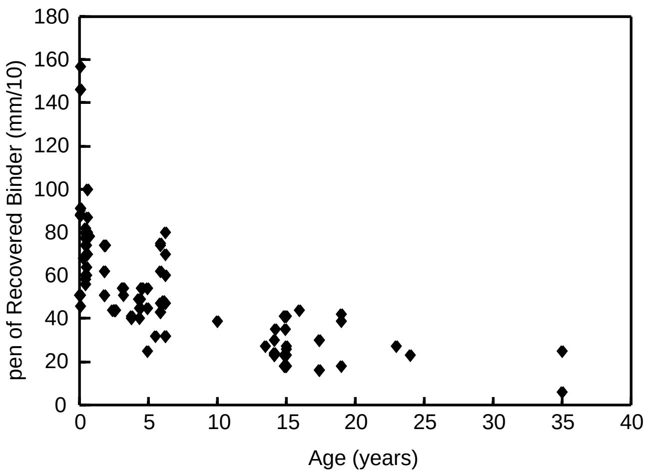

Figure A8 Change in pen of recovered binder with time

carried out by TRL indicated that the penetration of binder recovered from the top 10 mm of wearing course, which were from 18 to 24 years old, was between 39 and 68 per cent of that obtained from the lower layer. This aged, and hence brittle, surface skin of the wearing course is likely to be a major factor for the initiation of surface cracks.

A2.5 Curing of asphalt

It has long been known that the bitumen in pavement layers stiffens with time. Whereas a gradual hardening of the main structural layers appears to be beneficial and is most accurately described as *curing*, excessive ageing of the

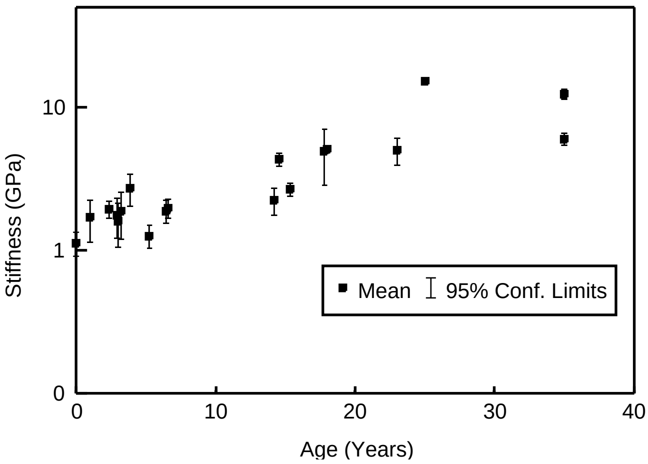

Figure A9 Change in roadbase stiffness of in-service roads over time

wearing course can lead to cracks initiating at the surface.

During the mixing and laying process, the penetration of the bitumen in standard dense bitumen macadam typically drops from an initial nominal value of 100 to about 70. In subsequent service, a further variable reduction takes place resulting in values often as low as 20 after 20 years. This hardening of the roadbase and basecourse has been studied at TRL using small-scale, untrafficked, test pavements replicating standard flexible pavements (Chaddock and Pledge, 1994). These pavements, with a 40 mm thick hot rolled asphalt wearing course, were constructed using full-scale plant, on a granular foundation. Stiffness and binder penetration measurements were carried out on cores taken from the test pavements over a period of 2 years. Core stiffnesses were measured using the indirect tensile test described in the British Standard Draft for Development, BS DD 213 (British Standards Institution, 1993).

The test pavements demonstrated that, although the curing behaviour was variable, the stiffness modulus of DBM roadbase could change by over 100 per cent over the first 12 months in service. The majority of these changes are due to binder hardening. The average penetration values of the binder decreased from 69 just after laying, to 43 and 35, one and two years later, respectively.

In addition to the test pavements, data on the curing of roadbase material from a large number of in-service roads has been collected by TRL. This database contains the measured properties of the recovered binder and stiffnesses of materials from pavements of different ages. Figure A8 shows the variation of penetration of the recovered binder, with time, for DBM roadbase manufactured with a nominal 100 penetration grade binder.

This Figure clearly illustrates that the penetration has reduced from about 70, shortly after laying, to a value in the range 20 to 50 after 15 years. The binder softening point also changes with time, in step with penetration. The corresponding changes in the elastic stiffness modulus over time, are shown in Figure A9.

The stiffness values given in Figure A9 were measured

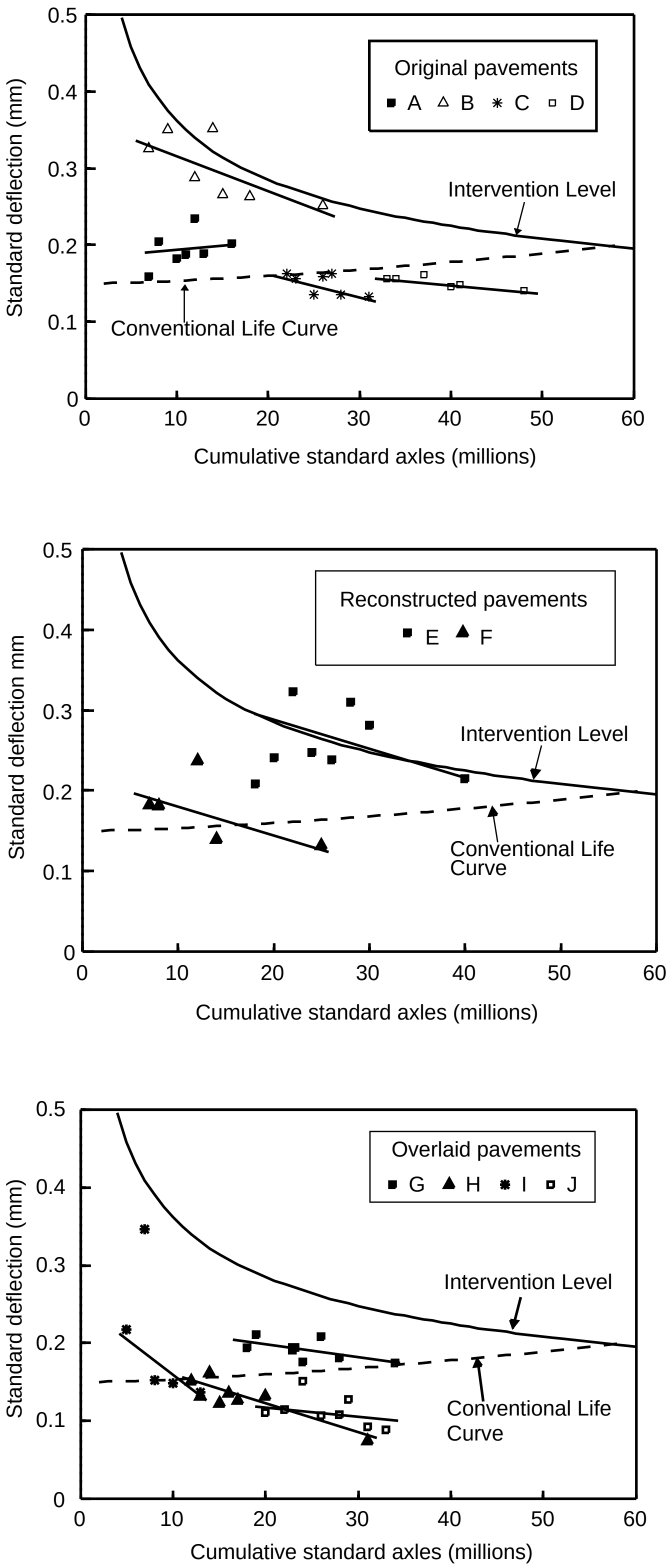

Figure A10 Deflection histories of in-service trunk roads and motorways

using the TRL 3 point bending test, at a loading frequency of 5 Hz and a temperature of 20°C. Additional stiffness data is included from several more recent pavement condition assessment studies carried out by TRL. In these later studies, the stiffnesses were measured using the indirect tensile test, which applies a pulsed load of lower equivalent frequency than the 3 point bending test. Therefore the roadbase stiffnesses were adjusted to a loading frequency of 5 Hz. Figure A9 shows a definite upward trend in stiffness of the DBM with time. Stiffnesses are in the range 3 to 15 GPa after 20 years compared with initial values in the range 1.25 to 3 Gpa (Nunn, 1996).

The increase in stiffness of asphalt has major implications for pavement design and explains some of the unexpected features of performance described in previous sections of this Appendix.

A2.6 Long-term pavement strength

The fact that the elastic stiffness and hence the load spreading ability of asphalt roadbase in thick, well-constructed roads, increases steadily over time shows that traffic associated deterioration of the roadbase does not occur. This improvement in load spreading ability should manifest itself as a reduction in deflection over the life of the road.

The measurement of pavement deflections under a slowly-moving, standard wheel-load is the normal method of routine pavement structural assessment in the UK (Kennedy and Lister, 1978) and is also widely used elsewhere. The deflections are expected to increase with the passage of traffic, reflecting a weakening of the structure. Increased deflections imply increases of the traffic induced strains in the roadbase and subgrade, which are considered to control pavement deterioration. The deflection method was based, predominantly, on observations of in-service roads which had carried only moderate traffic up to 12 msa.

The deflection histories of ten, heavily-trafficked sections of motorway were examined to determine whether the strength of thick, fully-flexible pavements reduces with time and traffic. These sections consisted of 4 original constructions, 2 reconstructions and 4 overlaid motorways. The total thicknesses of asphalt ranged from 270 to 615 mm and the cumulative traffic from 18 to 56 million standard axles. Figure A10 shows the deflection trends against cumulative standard axles for these groups of pavements. The 85 percentile values of the standard deflection of the whole of each site (nominally one kilometre long) have been used.

Apart from one site, all the sections show decreasing deflection with age and traffic. These decreases in deflection imply that the overall stiffnesses of the pavements are increasing over time and that any traffic related damage is more than offset by hardening or curing of the roadbase, or strengthening of the foundation due to drying or further consolidation. For whatever reason, the roads are becoming stiffer with time and the traffic induced stresses and strains in the roadbase and the subgrade that are considered to be responsible for structural deterioration, are reducing.

The deflections of the majority of the sites also show considerable fluctuations, which may be partly due to the difficulty of applying a temperature correction. Cracking and rutting were present on some of the sections but not to a serious degree. On one site the penetration of the recovered binder from cracked sections was considerably lower (harder) than from uncracked sections. This, together with the fact that there was no significant difference in deflection between cracked and uncracked sections, strongly suggests that the cracking is restricted to the surfacing layers.

The conclusions from these deflection studies of motorways are:

- The original, overlaid and reconstructed pavements have similar deflection/traffic trends.
- The deflection data, with one exception, indicate that the pavements are becoming stiffer with time.
- Three of the pavements, which have substantially exceeded their design life, are still maintaining a decreasing deflection trend.

A3 Conclusions

The overall conclusion of this review is that well-constructed roads will have very long lives, provided that they are built strong enough to resist structural damage in their early life.

This review has provided the opportunity to examine the validity of the extrapolations involved in the formulation of the current design method in 1984 (Powell et al, 1984). No evidence of conventional roadbase fatigue or structural deformation has been found in well-constructed, flexible pavements. These mechanisms of deterioration are the basis of the current design method. The observed rutting and cracking were found to originate in the surface layers and, provided these deteriorations are treated in a timely manner, structural deterioration is unlikely to occur.

The evidence suggests that roadbase fatigue and structural deformation are far less prevalent than surface initiated deterioration in thick, flexible pavements. It also shows that changes that occur in asphalt over the life of the road are crucial in understanding its behaviour. These changes, which are referred to as *curing*, help to explain why the conventionally accepted mechanisms of deterioration do not occur. Provided roads are constructed above a minimum threshold strength, they should have very long, but indeterminate, structural lives.

The improvement in the bearing capacity of the road, as determined by deflection measurements, provides confirmation that the load spreading ability of the main structural layers increases significantly with time.

Investigations carried out on roads, where deflection surveys have shown them to be in a critical condition, have generally failed to detect other symptoms of structural deterioration. This raises concerns about relying on the deflectograph method of assessing pavement condition for roads designed for heavy traffic. The mechanisms of deterioration for thick, well-constructed roads are likely to differ from those designed to carry relatively low levels of traffic.

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Appendix B: Pavement foundations

Apart from where indicated, the work described in this Appendix was sponsored by the Highways Agency, British Aggregate Construction Materials Industries and the Refined Bitumen Association.

B1 Introduction

There is an interdependence between all layers of a flexible road construction. Structural layers, designed for long-life, will be ineffective without good foundation design. Stiffer road foundations will contribute to smaller pavement deflections under a loaded axle and reduced traffic induced strains in the pavement structure. One method of ensuring adequately stiff foundation is to specify its surface stiffness as an end-product requirement for the completed foundation. Although the advantages of specification by end-product have been generally recognised, the problem has always been to find a practicable method of measuring the end-product.

As part of this project, test methods for measuring two properties that can be related to foundation performance, and have good potential to be used in an end-product specification, were assessed. The most appropriate properties identified for specification purposes were the compacted density of the foundation layers and the resulting surface stiffness. The test methods selected to measure these properties were the nuclear density test (NDT) (British Standard BS 1377: Part 9, 1990a) and the Dynamisches Plattendrückgerät, referred to in this report as the portable dynamic plate bearing test (PDPBT), which was developed in Germany (Kudla et al, 1991). Both these methods are fast, cost-effective and practicable.

Experience in the UK, summarised in Appendix A, has demonstrated that foundations constructed to the current Specifications (MCHW 1) are generally adequate, although they do not offer the Contractor the flexibility of an end-product specification.

Elsewhere in the developed world, the options for foundation construction are more wide ranging. This has been accomplished by making the Contractor more responsible for the work. For example, Germany and France specify minimum elastic stiffness values at the top of the subgrade and on the surface of the completed foundation, thus allowing a greater freedom of choice of construction materials and methods. These practices are described in Section B3.

This Appendix details the current design procedure, describes trials carried out to investigate the potential of surface stiffness measurement in an end-product performance specification and discusses the practical implications of adopting this form of specification.

B2 Current UK practice

Foundations are currently designed primarily to withstand the loading from construction traffic and to act as a construction platform for the laying and compaction of subsequent layers. In addition, the foundation protects the subgrade, should adverse weather occur during construction, and contributes to the structural strength of the completed pavement. This design is empirically based on the performance of experimental roads (Powell et. al. 1984) and was confirmed by results obtained from trafficking experiments on unsurfaced roads by Potter and Currer (1981) and Ruddock, Potter and McAvoy (1982). All UK foundations are designed to HD25/94 (DMRB Vol.7).

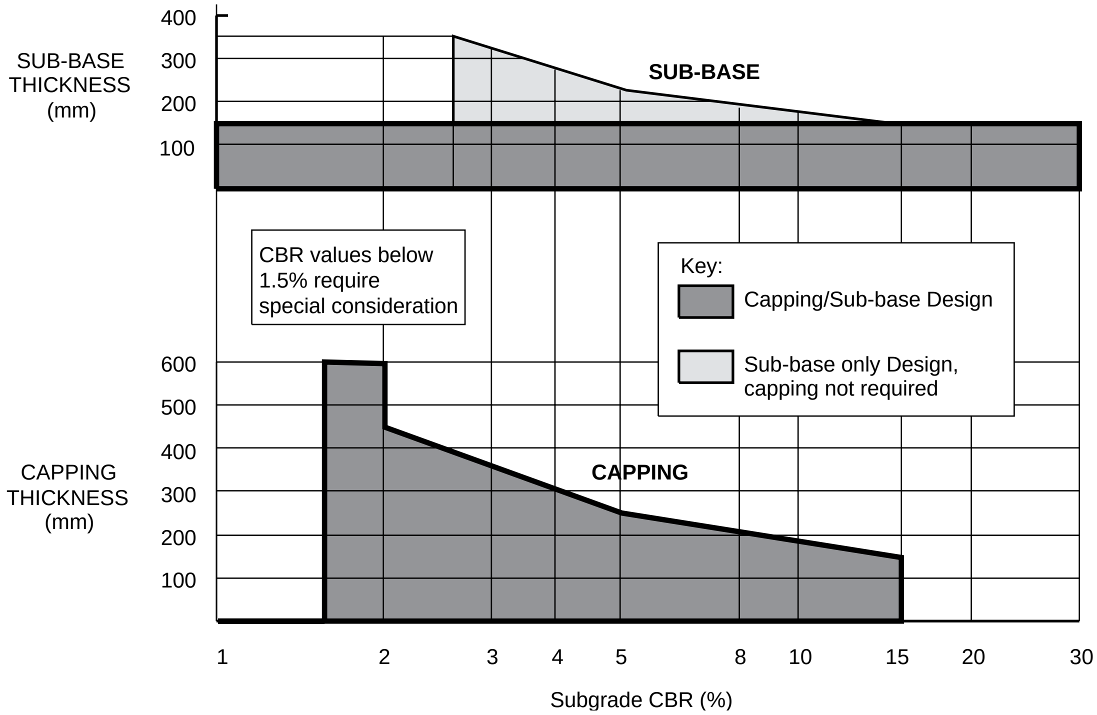

Figure B1 Capping and sub-base thickness design

The thickness of capping and sub-base is obtained from Figure B1. The subgrade design CBR used is the lower value of either the equilibrium CBR or the construction CBR. The method of subgrade assessment for the design of long-life pavements is described in HD25/95 (DMRB Vol.7). For subgrades with a CBR of greater than 15% a sub-base thickness of 150 mm is required as this is considered to be the minimum thickness to ensure satisfactory compaction. When the CBR is between 2.5 and 15% there are two options available:

- 150 mm of sub-base can be used on a thickness of capping that depends on the subgrade CBR value, or
- an increased thickness of sub-base can be used depending on the subgrade CBR value.

The first of these options must be used with all pavements constructed on a subgrade CBR below 2.5%. For subgrade CBRs of less than 2% there are a number of subgrade treatment options available prior to the construction of the foundation. These are described in more detail in HD25/94 (DMRB Vol.7).

The curves shown in Figure B1 are used to design foundations to limit deformation caused by construction traffic to a maximum of 40 mm for 1,000 passes of a standard axle. This has been defined previously as the maximum that can be tolerated if the sub-base surface is to be effectively reshaped and recompacted.

A section of the pilot-scale trial constructed to the current design produced a 30 mm rut after 1000 axles. Thinner foundations exhibited greater rutting and alternative, stronger, designs resulted in as little as 2 mm rutting after 1000 axles.

B3 European practice

Many countries, such as Germany and France, specify minimum elastic stiffness values, in addition to minimum density requirements, at the top of the subgrade and on the surface of the completed foundation. This gives the Contractor greater freedom in the choice of construction materials and methods, and ensures a foundation having satisfactory structural properties.

B3.1 Germany

Foundations in Germany are constructed according to the *Guideline for the Standardisation of the Structure of Traffic Bearing Surfaces (RStO 86)* (1989). Subgrades are required to have a minimum surface modulus (E_v^2) of 45 MPa at formation level when tested with a 300 mm diameter static plate bearing test. The E_v^2 value is calculated from the amount of deflection under the second loading of the plate. If this value is not attained, soil consolidation or soil improvement will be required in order to meet the criterion. The way in which the required level of surface modulus is obtained is the responsibility of the Contractor.

The thickness of the sub-base required above formation level is determined by the amount of frost resistance required. A surface modulus (E_v^2) requirement using the plate bearing test is also mandatory at the top of the sub-base layer, as shown below.

Light traffic		*Heavy traffic*	
Standard asphalt design	*Reduction in asphalt thickness of 20 mm*	*Standard design*	*Reduction in thickness of 20 mm*
120 MPa	159 MPa	150 MPa	180 MPa

The use of this performance-based specification allows the Contractor more freedom in his choice of materials and method of construction.

B3.2 France

A performance-based specification for the completed road foundation is also used in France. There is both a long and short term requirement for the foundation. In the short term the foundation must comply with one of the following to ensure that it is robust enough to support the construction traffic:

- a deflection of less than 2 mm under an axle load of 13 tonnes
- a surface modulus (E_v^2) of greater than 50 MPa
- a coefficient of restitution of greater than 0.5 from the Dynaplaque

The long-term requirement for the foundation, in the completed road, is the strength of the subgrade. This leads to a number of classes of pavement foundation

- PF X over 200 MPa
- PF 3 120 to 200 MPa
- PF 2 50 to 120 MPa
- PF 1 20 to 50 MPa

The construction of earthworks for road contracts is covered by the *Technical Guide for the Construction of Embankments and Capping Layers* (1992). This document classifies all the materials used as capping and identifies the way in which they may be put to the most economic use together with treatments to improve their properties. The guide also specifies the recommended number of compaction passes needed for each type of capping.

For the design of the upper pavement layers, tables of various construction options and thickness are given which are dependant on the surface modulus of the foundation and the traffic classification.

B4 End-product testing

The road foundation is required to act as a construction platform and then as a structural element in the completed pavement. During the construction phase it is most vulnerable and any inability of the foundation materials to support construction traffic or asphalt layer construction equipment will be due to either inadequate elastic stiffness, or insufficient shear strength in one or more of the foundation layers.

A loss of elastic stiffness will mean that the layer will no longer be capable of withstanding the stresses induced by the traffic, thus causing excessive strains within the pavement structure. Whereas, an insufficient shear strength will lead to an excessive amount of shear occurring under the action of the wheel load. This shear will result in permanent deformation and result in the foundation requiring remedial work before the roadbase can be laid, Leech and Nunn (1991).

During the service life of the road, the traffic induced stresses in the foundation layers will be very much reduced by the load spreading ability of the asphalt layers. However, the large number of load repetitions can ultimately lead to deformation of the subgrade.

From the review of current UK practice and of practice elsewhere, it seems likely that the next development in the construction of road foundations will be a move toward performance-based specification. The adoption of a performance-based specification would have two important effects:

- Allow the use of materials currently outside specification, if they can be shown to produce the necessary in-situ performance.
- Shift the responsibility for the quality of the foundation construction to the Contractor.

An end-product testing regime at both formation level and top of sub-base would improve the consistency of the layer and ensure that design stiffness values are achieved. Specifying foundation materials by their mechanical properties after laying will reduce variability, improve the quality and ensure that pavement design requirements are met.

In order to adopt an end-product specification, suitable tests to assess the performance of the material in-situ must be identified. The CBR test, which is often used to quantify the quality of foundation materials, is recognised to be unsuitable for crushed rock granular materials. The ideal test should combine both the shear strength and the elastic stiffness of the material tested. The CBR test does not correlate consistently with either parameter because the relative influence of the two properties varies from material to material.

Laboratory tests can provide values for the elastic stiffness of a material and its resistance to permanent deformation. However, there is currently no satisfactory in-situ method of directly assessing the foundations resistance to permanent deformation. It is therefore proposed that the state of compaction and elastic stiffness at formation level and on the completed foundation should be measured to ensure consistent construction quality.

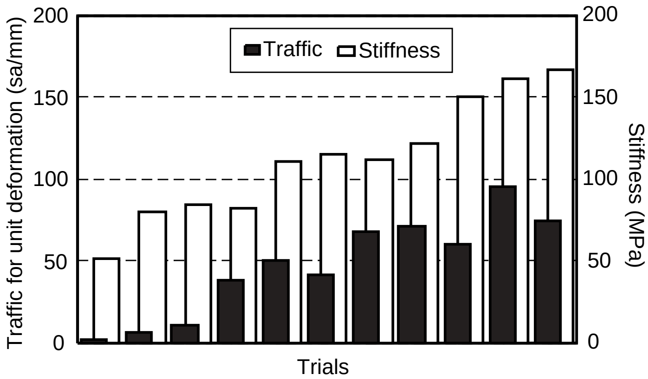

Figure B2 Performance of previous full scale trials

B4.1 State of compaction

Construction of any layer of the pavement requires the

Table B1 Methods of assessing foundation performance

Device	*Loaded Area*	*Type of test*	*Device cost £ k*	*Advantages*	*Disadvantages*
Clegg hammer	50 mm	Impact	< 2	Quick and easy to operate.	Unrealistic stress. Load area too small.
Dynamic cone penetrometer	N/A	Impact	< 2	Assess layers beneath the surface individually.	Slow. Semi-destructive.
Falling weight deflectometer	300 or 450 mm	Dynamic	~ 100	Reliable and repeatable. Well recognised.	Very expensive.
Portable dynamic plate bearing test	300 mm	Dynamic	< 5	Quick and easy to operate. Used extensively in Germany.	Little experience in the UK to date.
Loadman	132 or 200 mm	Dynamic	< 5		Loaded area too small. Erratic results.
Static plate bearing test	300 mm	Static	< 2	Lancs CC report good experience.	Very slow.

constituent materials to be sufficiently compacted to provide good performance. The dry density of compacted in-situ foundation materials expressed as a percentage of the maximum dry density achieved in a standard laboratory test (BS1377: Part 4, 1990) will indicate the state of compaction.

The in-situ bulk density can be measured using two methods:

- *Nuclear Density Gauge* (NDG) - A small radioactive source, located in a probe, is placed in the foundation material with a detector positioned on the surface. The intensity of radiation reaching the detector can be related to the bulk density of the material. This is a quick and effective way of measuring in-situ bulk density from which the dry density can be calculated. Further details of the testing procedure can be found in BS1377: Part 9 (1990).
- *Replacement methods* - This requires the removal of a sample of compacted material which is then replaced by a material of known density (commonly sand), as described in BS1377: Part 4 (1990). The mass of the material needed to fill the hole is known and, hence, the volume can be calculated. The mass of the sample removed can be measured and its density calculated. The method is relatively slow and labour intensive and is therefore not recommended for routine use.

B4.2 Elastic stiffness

In a series of foundation trials carried out as part of earlier research on behalf of the Highways Agency, Chaddock and Brown (1995) obtained a broad relationship between the measured elastic stiffness of a foundation prior to trafficking and performance. This relationship is shown in Figure B2 together with the permanent deformation

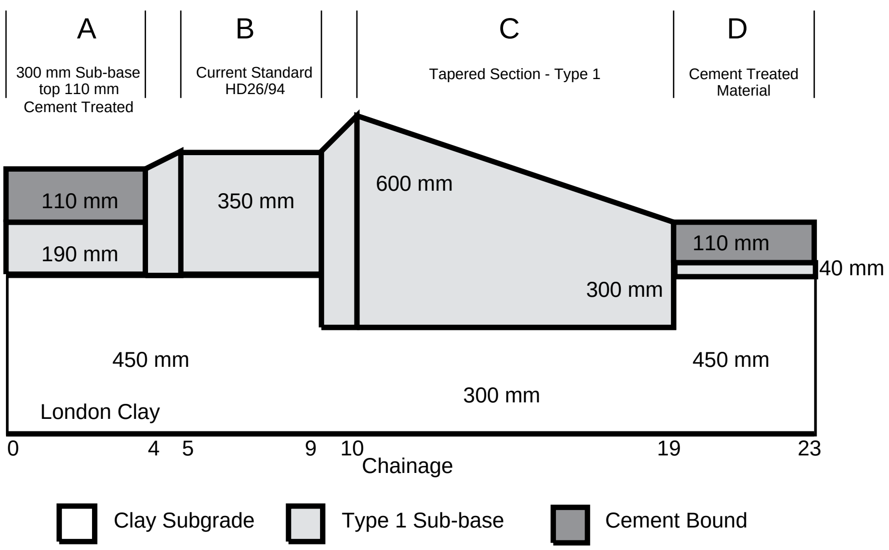

Figure B3 Layout of pilot-scale trial expressed as a rate of deformation. Both these quantities follow the same trend which would imply that the measure of elastic stiffness is a reasonable indicator of the pavement foundation strength.

There are a number of devices available which are capable of measuring the in-situ stiffness properties of the foundation and they are listed in Table B1. These devices either load the surface through a rigid plate of various diameters or force a probe into the foundation material. The results from these devices cannot be compared directly because of the different loading conditions and the non-linear behaviour of the foundation materials.

Due to its use within national specifications elsewhere in Europe and its quick and easy operation, the PDPBT (Dynamisches Plattendrückgerät) from Germany (Kudla et al, 1991) was selected as a candidate end-product foundation test.

B5 Pilot-scale trial

An end-product specification simply based on the elastic stiffness at the surface of the completed foundation would

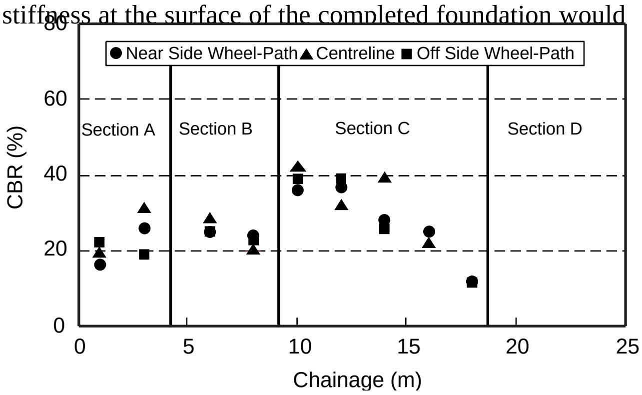

Figure B4 CBR results derived from DCP testing

suggest that the respective layer thicknesses of sub-base and capping are not critical as long as the end-product criteria are achieved. In order to assess the importance of thickness, a pilot-scale foundation trial was constructed.

The trial, illustrated in Figure B3, was designed to cover a range of stiffnesses and to relate them to their performance under trafficking. The trial was divided into four test sections with the following constructions:

Section A 300 mm crushed granite sub-base of which the top 110 mm consisted of sub-base with an additional 3% cement (by mass).

Section B 350 mm crushed granite sub-base as per the current design standard HD26/94.

Section C A section of crushed granite sub-base, tapering from 600 mm thick down to 300 mm thick over a length of 9 metres.

Section D 150 mm crushed granite sub-base, with 3% cement (by mass) added to the top 110 mm of the layer.

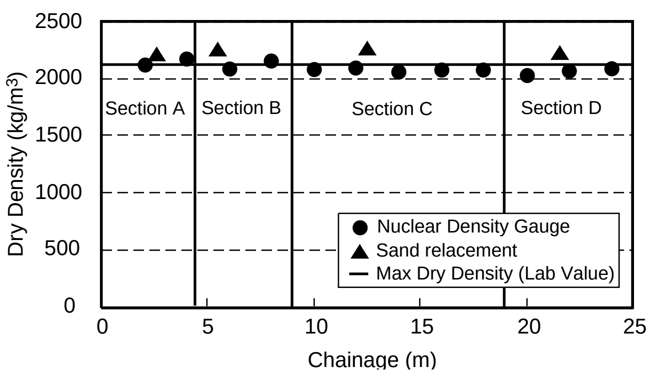

Figure B5 In-situ densities using NDG and sand replacement techniques

Table B2 Density values from cubes compared with in-situ wet density measurements for cement bound material

Section	*Cube sample*	*Cube sample density (Mg/m³)*	*In-situ wet density (NDG) (Mg/m³)*
	A1	2.379	
	A2	2.342	2.320
A	A3	2.339	
	A4	2.310	2.227
	A5	2.352	
	D1	2.309	
	D2	2.320	2.186
D	D3	2.310	
	D4	2.350	2.110
	D5	2.286	

B5.1 Field testing methods and results

Table B3 Compression strength testing results from cubes

Cube sample	*Compressive strength (N/mm²)*	*Mean compressive strength @ 7 days (N/mm²)*
A1	7.6	
A2	10.7	
A3	7.2*	9.1
A4	8.9	
A5	9.2	
D1	8.7*	
D2	9.6	
D3	10.2	10.6
D4	13.7	
D5	8.7	

*Tests at 7 days except where indicated * (3 days)*

Following completion of the foundation layers, in-situ density measurements were made using the NDG in direct transmission mode and the sand replacement method described in BS 1377: Part 9 (1990). Samples of the Type 1 sub-base were taken in order to establish the maximum dry density according to BS 1377: Part 4 (1990).

The elastic stiffness was measured, using the FWD, the PDPBT, and the Static Plate Bearing test, along the centre line of the site and along each wheel-path to be trafficked. These tests were carried out at 1 metre intervals. Dynamic Cone Penetrometer (DCP) testing was also undertaken according to the method described in TRL Overseas Unit Information Note (1986) at 2 metre intervals along the site in order to determine the CBR values of the foundation and assess any variations in the subgrade condition.

A re-assessment of elastic stiffness was made 7 days after the cement bound layer was laid using the methods described above. Repeat DCP measurement were also undertaken where possible.

B5.1.1 CBR

CBR was assessed using the dynamic cone penetrometer at 2 m intervals along the site in both wheel tracks and along the centre line prior to the removal, cement treatment and replacement of the foundation material in Sections A and D. The sub-base layer of Section D was too thin to provide a realistic testing situation. The results of these tests are shown in Figure B4. The introduction of cement in Sections A and D increased the measured CBR value to over 100%.

B5.1.2 Density measurements

Figure B5 shows the comparison of in-situ dry densities obtained using the NDG and sand replacement techniques. They are compared directly with the maximum dry density achieved in the laboratory. This shows that acceptable compaction was generally obtained in all the test sections. The refusal densities from cube manufacture and in-situ wet densities, measured using the NDG after the addition of the cement bound layers, are shown in Table B2.

The mean in-situ wet densities measured in Sections A and D were about 97% and 93% of the mean cube densities, respectively. The lower value in Section D may have been due to the drainage problem during construction which led to difficulties in achieving the same level of compaction as those in Section A.

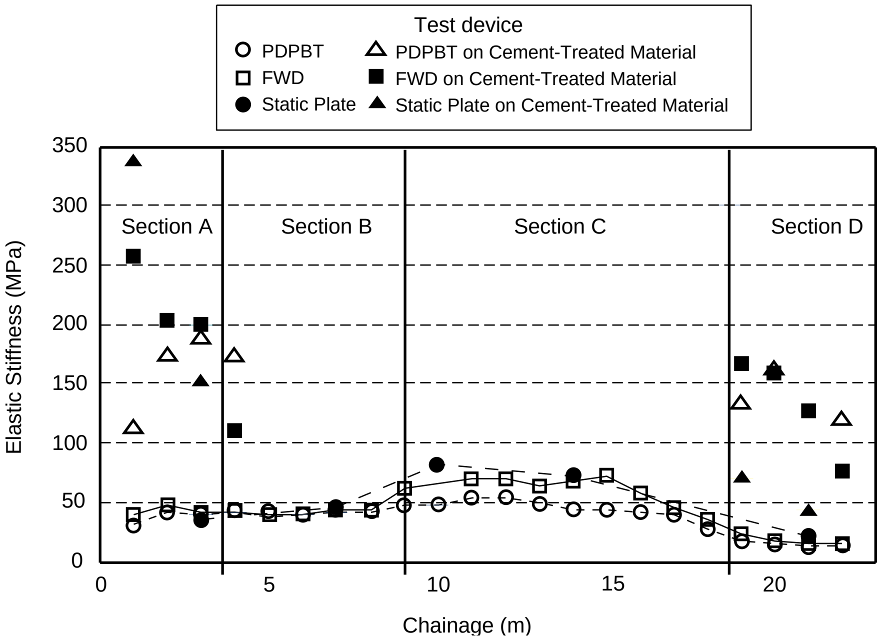

Figure B6 Measured stiffness values along the centre-line of the trial

Unconfined compression strength tests were carried out on 1 sample from Sections A and D after 3 days and the remaining 4 samples were tested after 7 days. These results, recorded in Table B3, show that the cube strengths easily meet the 7 day strength of 6 N/mm^2 specified in Germany for cement bound foundation layers.

Table B4 Mean elastic stiffness values by section and test method

	Mean elastic stiffness (MPa)		
Section	*PDPBT* (*Ef_{PDPBT}*)	*FWD* (*Ef_{FWD}*)	*Static plate* (*Ef_{SP}*)
A	36.0 (9)	39.4 (9)	33.3 (2)
A-CBM	159.5 (6)	204.7 (9)	243.5 (2)
B	41.1 (12)	42.6 (12)	43.0 (2)
C-600	49.7 (3)	67.6 (3)	79.5 (2)
C-450	46.4 (3)	62.6 (3)	63.4 (2)
C-300	33.8 (6)	41.0 (6)	-
D	12.2 (9)	13.5 (9)	25.6 (2)
D-CBM	132.6 (5)	127.7 (9)	43.6 (1)

No of results shown in brackets. Test points on borderlines between sections were not used.
FWD applied stress = 95 - 100 kPa
C-600 = section C, 600 mm thick sub-base layer

B5.1.3 Elastic stiffness

The FWD, the PDPBT and the Static Plate Bearing test were used on the full depth of Type 1 sub-base, prior to the addition of the cement bound layers, to evaluate the elastic stiffness of the foundation. After the cement treated material was laid the tests were repeated. As the PDPBT uses a nominal applied stress of 100 kPa, the FWD was loaded to give a similar applied stress from its lowest drop height position. The static plate technique, which has a 450 mm diameter plate, uses a method which is based

Figure B7 Lorry used for trafficking the trial foundations

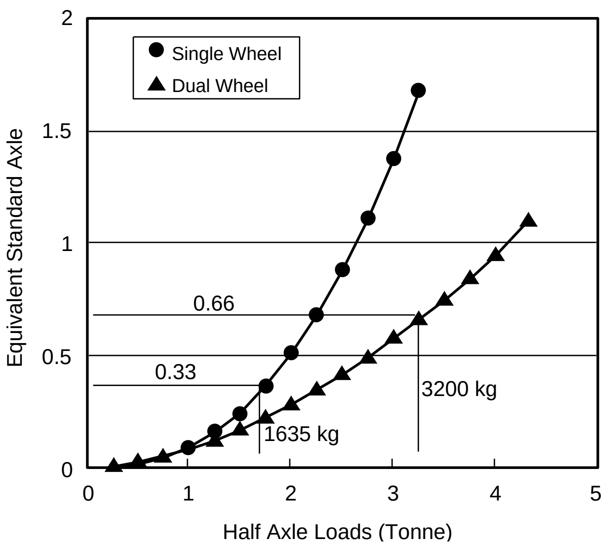

Figure B8 Equivalence factors for different wheel loads

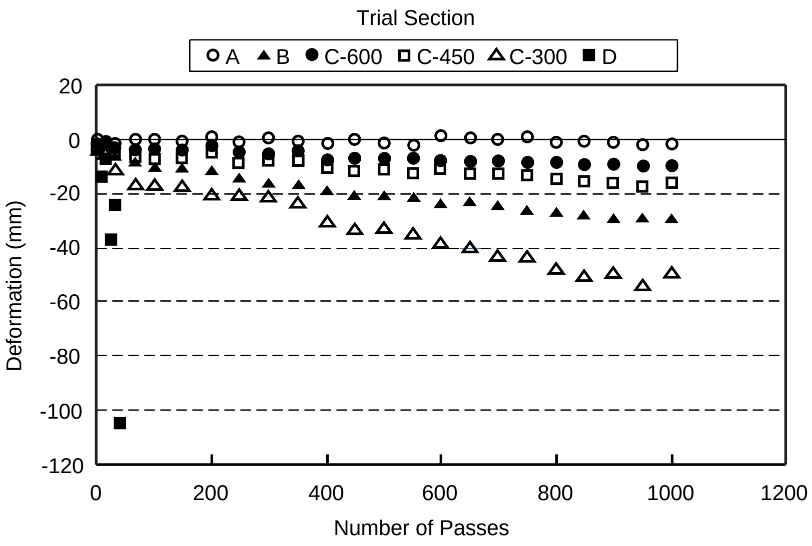

Figure B9 Cumulative foundation deformation under trafficking

Figure B10 Failure of Section D after 32 passes

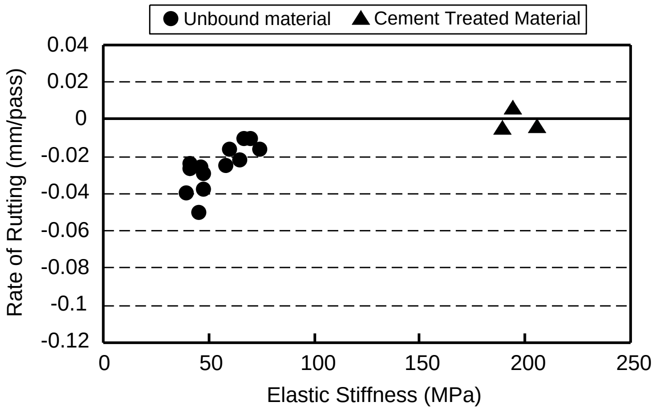

Figure B11 Rate of deformation under trafficking in comparison to foundation stiffness

upon a recovered deflection after initially loading the plate to give a nominal 1 mm deflection. The plate is loaded by using a lorry to provide the reaction.

Figure B6 shows the comparison of all three methods along the centre line of the site before and after the addition of the cement treated material. Similar patterns in results were obtained from test points located on each wheeltrack. Mean elastic stiffness values by section and test method are shown in Table B4. These tabulated values also include tests undertaken in each of the wheel paths; the test values close to the boundaries between sections were excluded. Prior to the addition of the cement, reasonable agreement between the devices is seen throughout the length of the site. After its addition the scatter in results is much greater. Although clearly this is mainly due to the substantial improvement in the measured elastic stiffness values which produced a corresponding reduction in deflection of Sections A and D.

Table B5 Deformation in the subgrade layer after 1000 passes

Section	*Sub-base thickness (mm)*	*Estimate of rutting in subgrade due to trafficking (mm)*
A	300	0
B	350	10-15
C	600	0
C	450	0
C	300	15
D	150	40*

** after only 42 passes*

B5.2 Trafficking

The vehicle shown in Figure B7 was used to traffic the pilot-scale trial. The front and rear (twin wheel) axle weights were 3270 kg and 6400 kg respectively. Research carried out by Ruddock et al (1982) to compare relative damage factors of axle loads of both single and dual wheeled axles produced the relationships shown in Figure B8. Using these relationships, one pass of the loaded lorry was shown to be equivalent to approximately one standard axle.

The current specification for foundation design states that the rutting after a loading of 1000 standard axles should not exceed 40 mm. Rutting in each of the test sections was, therefore, monitored until 1000 passes were completed. Figure B9 shows the increase in rut depth during this trafficking. The figure clearly shows the collapse of section D after relatively few passes and this is also illustrated in Figure B10. Despite the addition of the cement treated material, the foundation thickness was insufficient to support construction traffic, although a drainage problem at this end of the site may have contributed to its early failure.

After 1000 passes the maximum deformation in the sections constructed with Type 1 was in excess of 40 mm and the average for Section B was about 30 mm. In Section A, the rutting was only about 5 mm after 1000 passes although the start of the section showed signs of heave and the end of the section produced ruts in excess of 10 mm.

Figure B11 shows the rate of deformation over 1000 passes compared with the initial elastic stiffness determined from FWD testing using an applied stress of between 95 and 100 kPa. This Figure shows that the deformation is reduced as the elastic stiffness increases. However Section D, where the elastic stiffness was also high after the addition of the cement treated material failed after relatively few passes and is not included in the Figure. In this case the thickness of the foundation layer (150 mm) was insufficient to support the loading from traffic and cracking occurred which led to the rapid failure of the Section. A cement-treated layer needs to maintain its structural integrity to function as a load-spreading layer. The FWD or PDPBT will not induce stresses high enough to cause cracks but the construction traffic will. A high surface stiffness alone will not be an adequate requirement if the cement treated layer is not strong enough to resist cracking by construction traffic.

Table B6 Initial end-product acceptance criteria

		Dry density		
Formation	% of vibrating hammer test result		Average > 95 %	Individual > 90 %
Foundation	% of vibrating hammer test result		Average > 97 %	Individual > 92 %
		Elastic stiffness		
Formation	Test method	PDPBT	FWD	Static plate
	Stiffness (MPa)	30	40	50
Foundation	Test method	PDPBT	FWD	Static plate
	Stiffness (MPa)	50	65	80

Following completion of trafficking, each foundation layer was excavated across half of its width in order to provide an estimate of the amount of deformation in the subgrade due to trafficking. These results are shown in Table B5.

B5.3 Conclusions from pilot-scale trial

1 Section B, constructed to the current UK Specification,

performed as expected and is suitable for use in long-life pavement design.

2 Use of bound foundation materials can reduce the risk of foundation failure due to variability in material quality, construction practices or adverse environmental conditions.

3 End-product testing cannot replace foundation thickness design. A weak subgrade overlaid with a very stiff, thin layer (Section D) may achieve the required stiffness but fail disastrously when subjected to the higher stresses induced by construction traffic.

4 The trials indicate that a reduction in foundation thickness is possible when stiffer bound materials are used.

B6 Full-scale trial

Following the successful outcome of the pilot-scale trial, the end-product method was evaluated under full-scale

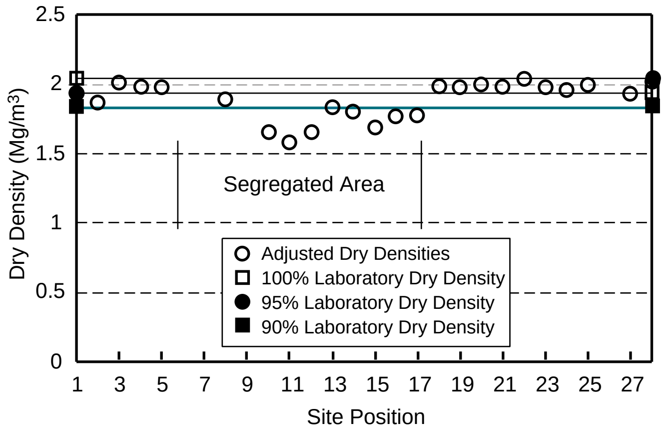

Figure B12 In-situ dry density at the formation

Table B7 In-situ dry density values at formation

	Site values			Laboratory values (vibrating hammer)		
Dry density (Mg/m^3)	*Average*	*Lowest*	*Standard deviation*	*100%*	*95%*	*90%*
Capping	1.971	1.864	0.043	2.030	1.929	1.827
Capping (poorly graded)	1.740	1.588	0.095			

conditions as part of a motorway construction contract. An end-product testing regime at both formation level and at the top of sub-base was proposed in addition to specifying the material properties, which were identical with the current Specification (MCHW1).

In the full-scale trial, the Contractor was required to construct the foundation in the conventional manner and the normal acceptance tests were carried out. Access was allowed for end-product testing to be carried out by TRL staff. The Contractor was responsible for satisfying the Specification, but, not for achieving the proposed end-product criteria for either layer. The objectives of the trial were to:

- provide information for the further development of acceptance criteria
- examine the practicality of an end-product system in contractual conditions.

From the pilot-scale foundation trial and results obtained from testing on other foundation constructions, the acceptance criteria in Table B6 were suggested for the full-scale trial. The results of the full-scale trial would be used to judge the performance of the formation and

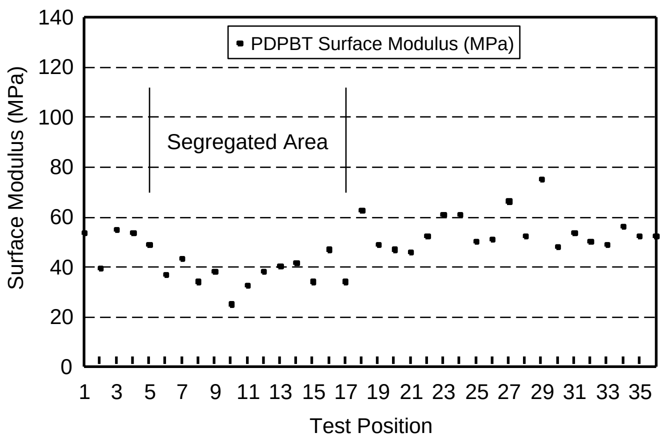

Figure B13 Stiffness at the formation (PDPBT results)

Table B8 Average values of stiffness at the formation

Surface modulus (MPa)	*Average*	*Lowest*	*Highest*	*Standard deviation*
Capping	51.2	39.5	75.0	7.2
Capping (poorly graded)	37.1	25.0	46.9	5.5

completed foundation against these criteria.

B6.1 Formation

As with the majority of capping layers in the UK, the capping laid on the M65 at the trial location was used as a haul road for construction traffic. The capping layer on the eastbound side of the carriageway had been laid and trafficked for almost twelve months before the site was made available. This resulted in a material that had consolidated, increasing the stiffness and was considered to be unrepresentative. Therefore, tests were carried out on nominally similar material that had recently been placed and compacted on the westbound carriageway in order to obtain values of newly laid material.

The area chosen for testing showed variable grading in the material along approximately a third of its length. It was unclear whether this was due to segregation of the material on site, or delivery of material not complying with the Specification. End-product testing would identify possible deficiencies in the quality of the construction.

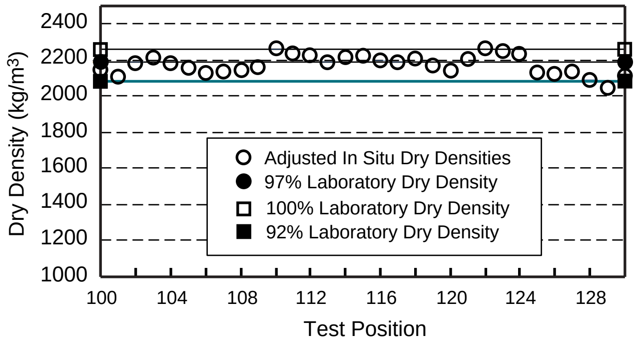

Figure B14 In-situ dry density on the completed foundation

Table B9 Average values of in-situ dry density values on the completed foundation

Dry density (Mg/m^3)	*Site values*			*Laboratory values (vibrating hammer)*		
	Average	*Lowest*	*Standard deviation*	*100%*	*97%*	*92%*
Sub-base	2.175	2.047	0.053	2.260	2.192	2.079

Both surface modulus and in-situ dry density testing were carried out on the capping using the PDPBT and the NDG, respectively. The PDPBT was used according to the manufacturers instructions (ELE, 1995). Dry density was measured using the NDG and calibrated using the sand replacement method, as described in BS1377: Part 9 (1990).

B6.1.1 In-situ density

The dry density results, shown in Figure B12, have been adjusted using in-situ sand replacement measurements to calibrate the NDG for the specific material. A summary of the results is given in Table B7.

The minimum value suggested for adequate performance of 95% of the vibrating hammer test (BS1377: Part 4, 1990) was not achieved in the poorly graded area of the capping. However, based on this limited information it is suggested that the criteria should be an average of 95% with no individual value of less than 90% to ensure adequate performance for long-life pavements.

Table B10 PDPBT results on the completed foundation

	Number of test positions	*Average surface modules (MPa)*	*Lowest value (MPa)*
Test length 1	31	55.8	33.6
Test length 2	31	49.3	31.7
Test length 3	31	53.0	35.7

Table B11 FWD results on the completed foundation

	Number of test positions	*Average surface modules (MPa)*	*Lowest value (MPa)*
Test length 1	31	83.0	53.5
Test length 2	31	68.7	48.6
Test length 3	31	55.8	44.3

B6.1.2 Surface modulus

Figure B13 shows the surface modulus at the top of the capping layer at 36 pre-defined positions. Tests on the poorly graded area, between positions 6 and 17, clearly show a lower average value of surface modulus that correspond to the low dry densities shown in Figure B12. A summary of the results is given in Table B8.

These results show that the PDPBT was capable of identifying the poorly graded material. However, this material still achieved the previously defined minimum value of 30 MPa and is thought to be suitable for long-life construction. The high stiffness value achieved is thought to be due to the good quality of the capping material, which was a crushed sandstone, obtained elsewhere on the site.

B6.2 Foundation

A 150 mm thick layer of sub-base formed the completed foundation across both the Control Section and the Trial Area. Three representative 30 m test lengths were chosen, two in the Control Area and one in the Trial Area. The three test lengths were assessed using both the PDPBT and the FWD. In-situ density testing with the NDG was carried out on the third test length, together with the calibration tests using the sand replacement method.

Table B12 Initial end-product acceptance criteria

	Dry density				
Formation	% of vibrating hammer test result	Average > 95 %			Individual > 90 %
Foundation	% of vibrating hammer test result	Average > 97 %			Individual > 92 %
	Elastic stiffness				
Formation	Test method		PDPBT	FWD	Static plate
	Stiffness (MPa)		30	40	50
Foundation	Test method		PDPBT	FWD	Static plate
	Stiffness (MPa)		50	65	80

B6.2.1 In-situ density

The dry density results from the NDG are shown in Figure B14. The results have been adjusted using the in-situ sand replacement measurements to calibrate the NDG for the material. A summary of the results is given in Table B9.

These dry density results are slightly below the target values set for in-situ density, namely, an average of 97% with no single value below 92%. The average dry density was just over 96% with only one single test location falling below the 92% value.

B6.2.2 Surface modulus

Summaries of the results from the two devices are given in Tables B10 and B11 for the PDPBT and FWD respectively.

Average stiffnesses measured with the PDPBT and the FWD are approximately the suggested levels indicating suitable long-life foundations of 50 and 65 MPa respectively. The material appeared well compacted and complied to the current Specification for Type 1 sub-base.

B6.3 Conclusions from full-scale trial

1 Dry density and elastic stiffness could be used as the basis of an end-product specification for foundations.

2 Elastic stiffness is both site and material dependent and dry density is material dependent.

3 Improved quality of foundation design reduces risk but does not provide a case for reducing the thickness of asphalt layers for long-life roads.

B7 Implementation

Road foundations constructed to the current Specification are generally adequate for long-life pavements. Trials have demonstrated that surface stiffness and density can be used as the basis of an end-product performance specification for the road foundation. The portable dynamic plate bearing test (PDPBT) and the nuclear density test (NDT) are practicable and economic methods that have potential to be used in an end-product specification.

Information on the results of end-product tests would need to be gathered from sites constructed to the current Specification before these methods could be incorporated into the Specification. This could be achieved with the introduction of a Specification Clause requiring the test data to be collected, thus enabling a database of results for various foundation constructions to be established.

These data would then enable reliable acceptance criteria to be developed, which may prove to be dependent on both material type and foundation thickness design. This research has suggested the following acceptance criteria as a guide (Table B12).

Dry density requirements are based upon the percentage of density achieved in-situ when compared to the value obtained in the laboratory using the vibrating hammer test (BS1377: Part 4, 1990). Throughout the research, elastic stiffness results have been shown to be dependent on the test device used. Therefore, a value has been suggested for each piece of equipment. Any other test devices would require correlation with one of these before use as an alternative test method. The use of an end-product system would require site specific adjustments to be made to both stiffness and density criteria based on good construction practice.

B8 Practical implications

The use of stronger foundation materials, such as CBM1 & CBM2 and the strengthening of Type 1 with hydraulic binders or substituting asphalt for all or part of a granular sub-base are worthy of consideration for a long-life pavement foundation. The use of such materials would allow a greater amount of variability, but with a lower risk of failure whilst still providing a long-life design. Bound foundation materials would also suffer less damage from construction traffic and tolerate a much larger amount of additional moisture without degradation.

Laying and compaction of Type 1 is currently carried out to a method specification (MCHW 1), with a roller of known mass providing a specified minimum number of passes to a maximum thickness of laid material. Unbound materials laid outside the current Specification and compaction in over-dry conditions can result in an inadequate long-life pavement foundation.

The overall thickness of the foundation layers should be monitored because, as the trials showed, a 15% reduction in the unbound material thickness resulted in an increase in rutting from 29.9 mm to 49.8 mm after 1000 axles. However, if the foundation is to be completely constructed from bound materials a saving in material thickness may be achieved (Chaddock and Atkinson, 1997).

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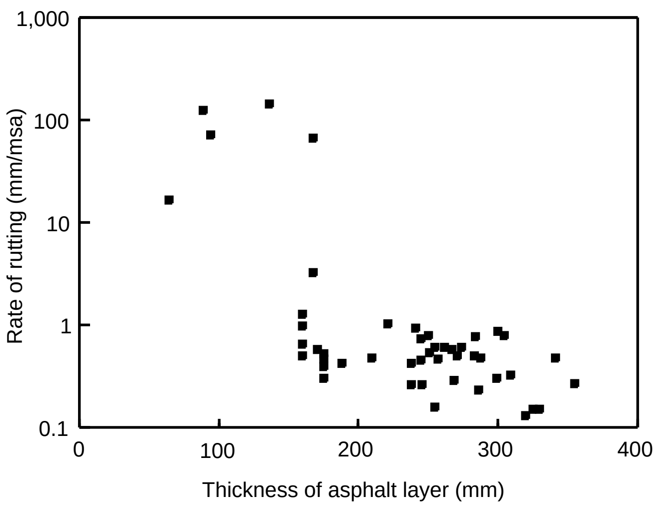

Figure C1 Rate of rutting versus thickness of asphalt layer

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Appendix C: Design of long-life roads

C1 Factors influencing pavement performance

Appendix A concludes that, above a threshold strength, the road will remain structurally serviceable for a considerable period, provided that non-structural deterioration in the form of surface initiated cracks and deformation are detected and remedied before they have a serious impact on the integrity of the road. To achieve a long life it is necessary for the road to be well constructed with good quality asphalt and a strong foundation so that deterioration does not result from construction or material inadequacies.

Figure C1 suggests that, for roads with low traffic flows, 180 mm of asphalt is sufficient to prevent structural deformation. However, the risk of structural deterioration must be kept very much lower in roads that are more strategically important. Although this risk will be reduced as the road becomes thicker, it is not possible to quantify the risk and reach an optimum solution, in which the reduction in risk by adding further thickness is balanced by economic considerations. Therefore, the analysis in this Appendix uses a conservative approach. It may become possible to reduce further the threshold thickness as confidence in the designs is increased as more knowledge is accumulated.

The threshold strength of the pavement required to achieve a long but indeterminate pavement life is examined in this Appendix. The effects of curing, the initial traffic flow, surface cracking and changes in vehicle loading characteristics are considered.

Figure C2 Typical curing curve for macadam roadbase

C1.1 Curing and traffic flow

This analysis considers pavement deterioration due to structural deformation. Fatigue cracking or weakening of the lower roadbase can be treated in a similar manner. The damage law for structural deformation, used in the current pavement design method, is implicit in the relation given by Powell et al (1984), namely:

$$N = K_c \times \varepsilon_z^{-3.95}$$

Where the cumulative traffic in standard axles, N, required to produce 10 mm deformation is related to the compressive strain, ε_z, induced at the top of the subgrade by a standard wheel load. This equation was derived using performance data from full-scale experimental pavements which cured naturally. Therefore, the effects of natural curing, together with factors such as the traffic characteristics and the pavement temperature spectrum, are contained in the empirical calibration factor, K_c, which was found to be 6.17×10^{-8}.

This equation was established from the performance of roads with relatively light traffic flows of between 0.1 and 0.5 msa per year compared to present day traffic flow rates that can be 5 msa per year or more for the busier motorways. A modern road, subjected to these higher traffic levels, is at greater risk of incurring structural damage in its early life before its structural properties have fully developed. Therefore, had the full-scale experimental roads been subjected to higher traffic flow rates their structural lives may have been reduced and a higher threshold strength would have been required to achieve long-life.

The constant, K_c, was established by assuming that the main structural layers had an effective uniform elastic stiffness modulus of 3.1 GPa (at 5 Hz, 20°C) throughout the life of the pavement. This level of stiffness will normally be attained in the first year or so after construction. Another constant, K_o, can be calculated which allows for the curing in a full-scale experimental road. This can be achieved by approximating the curing to incremental increases in stiffness, and summing the damage over each incremental change using Miner's hypothesis. Both calculation methods produce the same pavement life. The relationship between K_c and K_o, which is dependent on the curing behaviour of the material, can be calculated using the method given in Section C4. The value K_o is required to estimate how increased traffic flows affect the threshold strength necessary for a long-life road.

Measurements carried out on materials sampled from roads, at various stages in their life, have shown that the stiffness modulus of macadam roadbase containing 100 penetration grade binder can increase by a factor of four or more over 20 years, and that the rate of increase declines steadily with time (Chaddock and Pledge, 1994; Nunn, 1996). Although there is insufficient information available to establish the typical natural curing behaviour that occurred in the experimental pavements, measurements indicate a behaviour similar to that illustrated in Figure C2. Using this curing behaviour, and the calculation method given in Section C4, the value 3.63×10^{-8} was derived for K_o.

C1.1.1 Threshold strength

The measured rate of surface rutting in the TRL experimental roads, shown in Fig C1, reduced to a low level for pavements built above a minimum threshold strength. This threshold was equivalent to a well constructed road with about 180 mm of asphalt laid on a foundation consisting of 225 mm of granular sub-base on a subgrade with a CBR of at least 5 per cent. The surface rutting of these thicker pavements was independent of the thickness of the asphalt, which is consistent with the deformation being non-structural and confined primarily to the upper part of the asphalt layer.

The transition to a lower rate of rutting strongly suggests the existence of a threshold pavement strength and that, if the road is designed so that the traffic induced stresses and strains in the road structure are below the threshold level, significant structural rutting will not occur. The experimental pavements were laid more than 20 years ago and the measured rutting rates were caused by the relatively low traffic flows, which gave the main structural layers ample opportunity to cure. Roads constructed to meet the demands of present-day traffic levels, which may be 10 or 20 times higher than those encountered in the experimental roads, will need to be stronger to avoid excessive deterioration in their early life.

For a 180 mm thick pavement that cures according to Figure C2, the critical vertical compressive strain at the top of the subgrade, induced by a standard wheel load, would be typically about 750 μstrain when the road is new, falling to about 380 μstrain after 20 years. There can be a four-fold difference across the range of stiffnesses for nominally similar roadbase asphalts, and many of the experimental pavements were incorporated into major road schemes and often not opened to traffic until they were several months old. Bearing this in mind, some curing of the asphalt roadbase must be allowed for. To gain some insight on how traffic flow rates will affect the threshold strength, the following conservative structural deformation criteria have been assumed:

For $\varepsilon_z > 380$ μstrain,
$N = 3.63 \times 10^{-8} \varepsilon_z^{-3.95}$
For $\varepsilon_z < 380$ μstrain,
Structural deformation is negligible.

Calculations suggest that a road constructed with a thickness of more than 260 mm of asphalt would have a long but indeterminate life for traffic flows of up to 5 msa per year and that 270 mm would be sufficient for any traffic flow. For a road with the latter thickness, the traffic induced compressive strain at the top of the subgrade would be less than 380 μstrain when first opened to traffic. This thickness would ensure long-life even if curing did not take place, provided that the effective thickness of the asphalt layer was not reduced by deterioration due to cracking.

C1.2 Surface Cracking

Cracks can initiate at the surface in the wheel paths of fully flexible pavements that are about ten or more years old and then propagate downwards. These cracks are believed to be caused by the thermal and traffic induced stresses exceeding the strength of the wearing course, which has become progressively more brittle with age.

Surface cracking in a road that is built just above the minimum strength required for long-life, may weaken the road and produce an acceleration in deterioration. To guard against this possibility, it will be necessary to increase the thickness of the road to enable it to withstand some penetration of surface cracks. Timely remedial action should be taken before these cracks can have a severe structural impact on the road, however, cracks may have propagated up to 100 mm into road before this action is taken.

The most conservative estimate is to assume that the material down to the depth of the crack penetration does not contribute to load-spreading. This would imply that a road constructed with a 370 mm thick layer of asphalt would be able to tolerate a surface initiated crack propagating 100 mm into the road even if curing occurred at a minimum level.

C1.3 Provision for increase in maximum legal axle load

The current legal maximum axle load is 10.5 tonnes and this will be increased to 11.5 tonnes in 1999. After that, further increases are possible.

An increase in the thickness of the bound layer of 20 mm would be more than sufficient to allow for an increase in the legal maximum axle load from 10.5 to 11.5 tonnes.

C2 Risk of premature failure

The emphasis has been placed on structural deformation in this analysis but fatigue cracking initiated at the underside of the roadbase can be considered in a similar manner. The curing of the roadbase material increases its susceptibility to fatigue cracking and improves its stiffness modulus or load spreading ability. Nunn and Bowskill (1992) have shown that this improvement in load spreading ability results in a reduction in the traffic-induced tensile strain considered to be responsible for fatigue. The net effect is that curing in thick pavements improves the overall fatigue resistance of the pavement.

The conservative estimates of thickness to allow for increased traffic flow, surface cracking and the planned increase in the legal maximum axle load suggest that a pavement consisting of about 390 mm of asphalt is more than sufficient for a long-life road. This summation will introduce further conservatism, because, if surface cracks occur, they will normally appear at least 10 years after the road is laid. By this time the roadbase will have cured considerably and the remaining thickness after cracks have propagated 100 mm will still be well above the

Table C1 Design thicknesses for long-life roads using standard roadbase materials

Roadbase material	*stiffness modulus relative to DBM (M)*	*Design thickness of asphalt layer (mm)*
DBM	1.0	390
DBM50	1.5	350
HDM	2.0	320

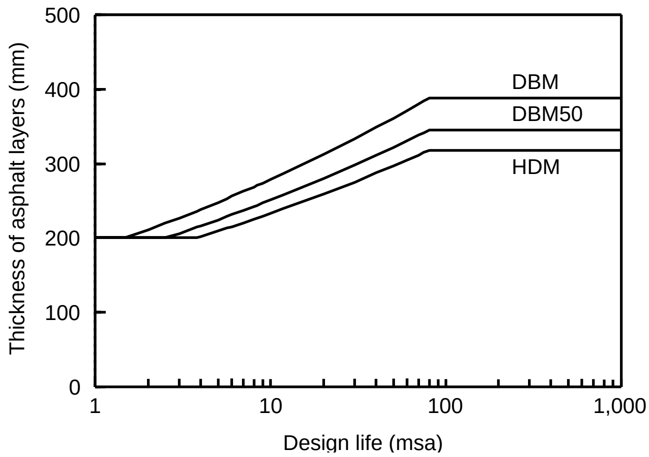

Figure C3 Design curve for roads with asphalt roadbase

threshold strength for a long-life road. A pavement with 390 mm of asphalt, constructed with a roadbase macadam using 100 penetration grade binder and laid on a good foundation, will be able to tolerate opening traffic well in excess of 5 msa per year even if the binder in the main structural layers does not cure. A measure of the in-built conservatism of the newly laid road can be gauged by comparing the calculated, vertical compressive strain at the top of the subgrade of 216 μstrain with the structural deformation criterion given earlier. This construction will also be able to tolerate a surface initiated crack penetrating 100 mm into the asphalt layer in the unlikely event of surface cracking initiating soon after the road is constructed, before curing can occur. In any event, many roads will not experience surface cracking.

Constructing roads with a stronger foundation will further reduce any risk of premature failure. Currently, a method specification is used and foundations generally have a surface stiffness of 70 MPa or more measured using a static plate bearing test (Goddard, 1990). It is proposed that an end-product specification for the road foundation should be introduced to ensure a minimum surface stiffness of the foundation. This will provide added assurance against the risk of early structural failure.

A practical way forward would be to use the existing design curve for traffic up to 80 msa (390 mm thickness of asphalt using 100 pen macadam for the structural layers) and then regard the road as long-life, with no additional thickness required to extend life.

C3 Design thicknessess

The elastic stiffness modulus or load spreading ability is the most important structural property of the roadbase. The elastic stiffnesses of standard macadams relative to the roadbase macadam incorporating 100 penetration grade binder were determined by Nunn, Rant and Schoepe (1987). These values can be used to determine their design thicknesses for traffic levels up to 80 msa using the following relationship:

$$H = \frac{0.077}{M^{0.65}}(\mathrm{Log}_{10}N + 8.5M^{0.12})^{3.64}$$

Where H is the design thickness of the asphalt layer in mm, N is cumulative traffic to be carried in msa and M is the elastic stiffness modulus of the material relative to that of DBM. These design thicknesses assume that the same type of material is used for the basecourse and road base and the wearing course consists of 40 mm of hot rolled asphalt.

This equation can be reduced to the following form, to calculate the threshold thickness for a long-life pavement:

$$H = \frac{0.80}{M^{0.65}}(1 + 4.47M^{0.12})^{3.64}$$

For standard macadams these thicknesses, rounded up to a whole number of centimetres, are summarized in Table C1 and illustrated in Figure C3.

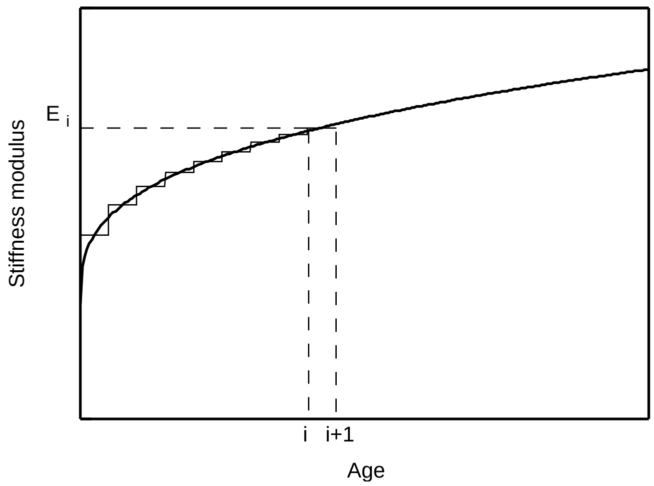

Figure C4 Determination of K_o

C4 Determination of the constant, K_o

The constant K_0 which is used in the determination of the effect of curing on the stiffness modulus is determined in the following way:

1 Divide the curing curve into k equal time intervals as illustrated in Figure C4.

2 Determine the elastic stiffness modulus of the roadbase, E_i, for time interval i to i+1.

3 Calculate the critical compressive subgrade strain ε_i using a multi-layer elastic model and the value E_i for this time interval.

4 Determine the pavement life, N_i, for a constant stiffness of E_i, that is:

$$N_i = K_O X \varepsilon_i^{-3.95}$$

5 Determine the fraction of life, f_i, expired in the time interval i to i+1:

$$f_i = \frac{n_i}{N_i}$$

Where n_i is the traffic the road carries in the interval i to i+1.

6 The sum of these life fractions to the end of the design life will equal 1:

$$\sum_{i=1}^{k} f_i = \sum_{i=1}^{k} \frac{n_i}{N_i} = 1$$

and if the road is subjected to an equal amount of traffic in each time interval, then n_i will equal:

$$n_i = \frac{N}{k}$$

Where N is the life of the road given by the LR1132 criterion:

$$N = K_c \varepsilon^{-3.95}$$

and ε is the vertical compressive strain in the subgrade determined using the effective elastic stiffness of 3.1 GPa for a standard roadbase macadam.

7 Substituting in the above equations gives:

$$K_o = K_c \frac{\varepsilon^{-3.95}}{k} \sum_{i=1}^{k} \varepsilon_i^{3.95}$$

This method gives a value for K_o of 3.63x10^{-8} for the curing behaviour illustrated in Figure C1. This can be compared to the value K_c of 6.17x10^{-8} derived in LR 1132 by assuming a constant stiffness of 3.1 GPa.

C5 Input data for standard designs up to 80 msa

The structural properties of the pavement materials and subgrade used to calculate critical strains in the standard designs are given by Powell et al (1984) and reproduced below.

Asphalt material

Loading frequency	5 Hz
Equivalent temperature	20°C

Elastic stiffness modulus of:

Dense bitumen macadam, DBM (100 pen)	3.1 GPa
Dense bitumen macadam, DBM50 (50 pen)	4.7 GPa
Heavy duty macadam, HDM (50 pen + extra filler)	6.2 GPa
Hot rolled asphalt, HRA	3.5 GPa
Poisson's ratio	0.35

Fatigue criterion:

For DBM, DBM50, HDM and HMB	$Log_{10}N_f = -9.38 - 4.16\ Log_{10}\ \varepsilon_r$
For HRA	$Log_{10}N_f = -9.78 - 4.32\ Log_{10}\ \varepsilon_r$

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

Deformation criterion: $Log_{10}N_d = -7.21 - 3.95\ Log_{10}\ \varepsilon_z$

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

Sub-base (Type 1)

Elastic stiffness modulus that separately	The modulus of each layer has been compacted is given by:
	$E_n = 3\,E_{n+1}$ for $E_{n+1} < 50$ MPa $E_n = 150$ MPa for $E_{n+1} > 50$ MPa
	Where E_{n+1} is the stiffness modulus of the underlying layer and the upper limit for the thickness of a compacted layer is 225 mm.
Poisson's ratio	0.45

Capping layer

Elastic stiffness modulus	Between 50 and 100 MPa
Poisson's ratio	0.45

Subgrade (cohesive soil)

Elastic stiffness modulus	$E = 17.6\,(CBR)^{0.64}$ MPa
Poisson's ratio	0.45

Standard wheel load

Load	40 kN
Contact radius	151 mm

C6 References

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Appendix D: Wearing course trials using modified binders

This Appendix summarises roads trials involving bitumen modifiers used in hot rolled and porous asphalt. A bitumen modifier is a polymer that is blended with the bitumen to change the rheological properties of the combined binder from those of the bitumen; other additives, such as fibres, that are added to a mixture to alter the properties of the mixture without affecting the rheology of the bitumen binder will be referred to here as additives.

The majority of this information presented in this Appendix is from previous research funded by the Highways Agency. This project was responsible for assembling and interpreting the information.

D1 Hot rolled asphalt

Most road trials involving modified materials have focused on hot rolled asphalt wearing courses because binder viscosity is one of the most important properties governing the resistance to deformation of this material. Modifiers and modification have been assessed for their ability to improve the deformation resistance, and for some modifiers, the flexibility of hot rolled asphalt at higher in-service temperatures without compromising its properties at low road temperatures and in the mixing/ compacting temperature range.

D1.1 M4, Avon

Thirteen experimental sections of wearing course were laid on the eastbound carriageway of the M4 motorway, east of Junction 20, in September 1980. The traffic over the duration of the trial averaged between 3,000 and 4,000 commercial vehicles per day. The objective was to compare the following methods of increasing the deformation resistance of hot rolled asphalt wearing courses, using standard hot rolled asphalt as a control:

- the use of a heavy duty bitumen with a softening point of 63 ± 5°C
- the addition of 5 per cent of ethylene vinyl acetate (EVA grade 19 ± 2 per cent vinyl acetate, 150 ± 10 melt flow index) to a 50 pen grade bitumen
- the use of crushed rock fine aggregate to replace sand
- the use of sulphur to replace about 25 per cent by volume of the 50 pen grade bitumen.

The following programme of measurements was carried out at regular intervals until the wearing courses were replaced in 1987:

1 Texture measurements

2 Mean summer SCRIM coefficients (MSSC)

3 Rut depth measurements

4 Visual inspections.

Over the seven year period, all sections showed signs of rutting, albeit in the case of the EVA-modified surfacings the extent was slight. As expected, the lowest binder content surfacings showed little evidence of rutting but the texture and visual appearance of these sections indicated that the surfacings were less durable. The sulphur-modified material had a lower bitumen content (4.3 per cent) than that specified (5.0 per cent) and performed rather poorly in terms of both deformation resistance and durability.

The higher binder content materials (optimum plus 0.6 per cent and optimum plus 1.2 per cent for the crushed rock fines) performed relatively well, apart from problems at the longitudinal joint, whereas the other materials exhibited varying degrees of chipping loss and fretting. Most of the sections were deemed to be in need of replacement in early 1987, and the whole experiment was terminated later in 1987.

The more important observations from this trial are summarised as follows:

- Designing a mixture based purely on the optimum binder content tends towards a *lean* material.
- An increase in the stability of a mixture alone cannot guarantee a deformation resistant surfacing. Other factors, such as ease of compaction and initial chipping embedment, remain important considerations.
- Binder stiffness undoubtedly affects the performance of the resulting material; too high a viscosity is likely to lead to increased mixing, laying and compaction temperatures, with the risk of poorly compacted material. This is accentuated in the case where the asphalts are laid 40 mm thick.
- Sulphur-modified asphalts, though potentially promising materials for ease of compaction and deformation resistance, are severely limited by the critical mixing temperature of 150°C, above which hydrogen sulphide is emitted. Furthermore, this experiment demonstrated that, where the bitumen content requirement is not fully met, the material will 'weather' at an excessively high rate, reducing the durability of the surfacing.
- Where modifiers such as EVA are employed, or crushed rock fine aggregate is used, a softer grade of bitumen (70 pen for EVA and 100 pen for crushed rock) may produce a more durable material.

D1.2 A303, Wiltshire

The deformation resistance of hot rolled asphalt can be improved, without resorting to modifiers, by replacing part (or all) of the sand fraction by crushed rock fines (CRF). The resultant mixtures are usually much stiffer because of the fines being more continuously graded than the sand.

To assess the performance of crushed-rock fines mixtures (where the crushed rock replaces the sand portion), a trial pavement was constructed on the A303, Wiltshire in 1982. Previous trials (Jacobs, 1974) had shown that mixtures containing crushed-rock fines performed better than their sand fines counterparts. The drawback often associated with crushed rock fines mixtures is that they often have a very high stability, which may lead to poor workability and consequent under-compaction and are, generally, less tolerant of variations in binder content.

The trial on the A303 examined a number of crushed-rock fines mixtures using different binder grades (50, 100 and 200 pen bitumen) at different binder contents around the optimum binder content (Daines, 1992a). The trial sections were monitored for a period of 6.5 years.

It was concluded from this trial that:

- Whilst the crushed-rock fines mixtures using 200 pen were durable, they were prone to deformation; however, the crushed-rock fines mixtures containing 50 pen gave good deformation resistance but at the expense of durability. This latter aspect was almost certainly due to the high voids content resulting from difficulties with compaction.

Table D1 Modified Hot Rolled Asphalt Trials A38, Staffordshire

Binder	*Fine aggregate*	*Stability (based on base bitumen)*	*Binder Content (per cent)*	*Initial properties* density* (Mg/m^3)	*Initial properties* W/T rate @ 45°C* (mm/h)
50 Pen (control 2)	Dorrington/CRF	8 ± 2 kN	6.9 ± 0.6	2.419	2.2
Cariphalte DM	Dorrington/CRF	8 ± 2 kN	6.9 ± 0.6	2.383	1.1
50 Pen (control 1)	Stourport/Croxden	6 ± 2 kN	7.2 ± 0.6	2.354	4.5
50 Pen + SR	Stourport/Croxden	6 ± 2 kN	7.2 ± 0.6	2.333	2.1
Multiphalte	Stourport/Croxden	6 ± 2 kN	7.2 ± 0.6	2.339	3.1
Cariphalte DM	Stourport/Croxden	6 ± 2 kN	7.2 ± 0.6	2.337	1.0
70 Pen (control 3)	Dorrington	3.5 ± 2 kN	8.1 ± 0.6	2.326	11
70 Pen + 5% EVA†	Dorrington	3.5 ± 2 kN	8.1 ± 0.6	2.309	6.1

** Densities and W/T rates determined from cores taken from the road after 2 months*

† EVA grade 18/150 with 18 to 19 per cent vinyl acetate content and 150 melt flow index

Table D2 Performance results after 7 years A38, Staffordshire

Binder	Skidding resistance (sfc)	Texture Scrimtex (mm)	Rut depth (since 1988) (mm)	Visual condition*
	Performance results after 7 years (nearside lane)			
50 Pen (control 2)	47	1.3	2.0	G
Cariphalte DM	48	1.5	1.0	G/M
50 Pen (control 1)	46	1.2	2.9	G
50 Pen + SR	44	1.5	2.0	G
Multiphalte	47	1.5	0.6	G
Cariphalte DM	45	1.3	1.7	G
70 Pen (control 3)	47	0.9	8.6	M/A
70 Pen + 5% EVA	48	1.3	1.3	G

** Visual assessment marking system as developed by TRL (Nicholls, 1997b)*

- There was a statistically significant correlation between voids content at the design stage, durability and binder hardening in the surfacings. A design voids content of between 3 and 4 per cent gave the best overall performance.
- The range of binder contents to produce acceptable performance was more limited than for hot rolled asphalts using sand fine aggregates, confirming earlier work.
- A balance needs to be struck between mixture stability, binder content, voids content and deformation resistance in order to provide a durable material.

D1.3 A38, Staffordshire

A major trial was carried out on the A38, Staffordshire to compare the performance of a number of modified hot rolled asphalt materials. The trial sections were laid in 1987 and formed part of a larger trial, which included porous asphalt materials (Daines, 1992b). The hot rolled asphalt materials laid that are still available commercially, together with initial results are shown in Table D1.

The reason for including a low stability asphalt was to establish whether the addition of polymers would help in up-grading low stability sands, which are currently deemed unsuitable for heavily-trafficked conditions (Carswell, 1987).

The skidding resistance, texture and deformation of the rolled asphalt sections have been monitored together with regular visual assessments by an Inspection Panel. The performance after 7 years of trafficking is shown in Table D2.

The modified binders are all performing better than Control 1, with the Multiphalte binder showing the greatest deformation resistance. In terms of visual assessment, at the present time all materials, including the control, are in a good condition. The Cariphalte DM section, when compared with the Control 2, is deforming at half the rate of the control material. The EVA-modified material, nominally comparable with Control 3, is performing significantly better and is comparable with the modified asphalts based on Control 1.

D1.4 M6, Staffordshire

The nearside lane (and hard-shoulder) of the southbound carriageway of the M6 near Hilton Park, Staffordshire was chosen to assess the performance of three different materials; a control 50 pen asphalt, an 18/150 grade EVA-modified asphalt and an organo-metallic modified (Chemcrete) asphalt. This section of the M6 is one of the most heavily trafficked in the UK with in excess of 6,000 commercial vehicles per day. The materials were all laid, without any problems, in 1985.

Over a four year period the trials were monitored and the following conclusions were drawn:

- All the materials were performing satisfactorily with both skidding resistance and texture being maintained;
- All the materials exhibited low rut depths (less than 4 mm after 4 years) with the rate of change being marginally greater for the control section than for the other two (70 pen + 5 per cent EVA and 70 pen + Chemcrete).
- After four years, cracking at regularly spaced intervals was noticed at the edge of the hard-shoulder in the Chemcrete section though not in the nearside lane. This effect, which has been noted elsewhere for this material by Nicholls (1990), is probably linked to the 'curing' of the modifier.

All the wearing course materials are still in place (after 12 years); the Chemcrete binder was withdrawn from the UK market in December 1990 because of its variable performance in other trials.

D1.5 Assessment of other modifiers used in hot rolled asphalt

D1.5.1 Assessment Procedure

Many modified binders have been subjected to rigorous laboratory testing to assess their potential before carrying out road trials to confirm it under in-service conditions. For a number of years, the Department of Transport and subsequently the Highways Agency have promoted, with the assistance of TRL, a 5-stage assessment procedure for such materials. The 5-stages in the *Highways Agency Procedure for Evaluating New Materials* are:

- Stage 1 Desk Study: Assess and evaluate existing information on the material.
- Stage 2 Laboratory Study: Test the mechanical properties of materials to allow theoretical predictions to be made of their performance.
- Stage 3 Pilot-Scale Trials: Evaluation of construction and performance of materials in small scale trials.
- Stage 4 Full-Scale Trials: Full-scale trial on a trunk road to establish whether the previous assessments obtained from Stages 2 and 3 are realised.
- Stage 5 Highways Agency Specification Trials: This stage is necessary to carry out further

evaluation of the material and to test the specification under contract conditions.

This methodology is now being developed with specific procedures for various groups of products by the British Board of Agrément on behalf of Highway Authorities, including the Highways Agency, other Overseeing Organisations and the County Surveyors' Society. The procedure, known as the *Highway Authorities Product Approval Scheme* (HAPAS), will provide a framework for assessing innovative developments and will initially cover:

- High-Friction Systems
- OverBanding
- Thin Surfacings
- Modified Binders for:
  - ° Hot mixed materials,
  - ° Surface dressings, and
  - ° Slurry surfacings and cold mixtures.

D1.5.2 Novophalt

A proprietary binder containing polyethylene was assessed as a potential polymer-modified binder for use in hot rolled asphalt wearing courses under the *Highways Agency Procedure for Evaluating New Materials*. Various stages of assessment were carried out, including binder testing, laboratory mixture studies, pilot-scale mixing and laying trials before proceeding to full-scale road trials.

The binder test programme showed that the Novophalt was an extremely stiff binder requiring much higher mixing and laying temperatures than a 50 pen bitumen (Denning and Carswell, 1983). The laboratory and pilot-scale studies both showed that, subject to satisfactory compaction being achieved, the polyethylene-modified material was more deformation resistant than the equivalent control (50 pen) material.

D1.5.3 Evatech H

A more recent assessment has been carried out on EVATECH H (Nicholls, 1994a). This assessment was limited in that only binder testing and laboratory evaluation of mixtures was carried out. The binder test programme included the rheological characterisation of the binder using a controlled stress rheometer. The potential of modified binders can be assessed by their rheological properties as measured by a controlled-stress rheometer (Gershkoff et al, 1997) using artificially aged samples to model the potential performance of the binder after mixing with the mineral aggregate. The recent outputs from the United States Strategic Highway Research Programme (SHRP) also place a high degree of importance on rheological characterisation using dynamic shear rheometers and the SUPERPAVE (SHRP, 1993) mixture design programme relies heavily on binder parameters determined from this type of testing.

These studies showed that the wheel-tracking rate was between a fifth and a third of that obtained with 50 pen bitumen and they indicated that EVATECH H should perform better than conventional unmodified materials in service.

D1.5.4 Shell Multiphalte

A high penetration index (PI) bitumen has been developed using the Shell QUALAGON test regime (van Gooswilligen et al, 1989). The advantage of a high PI for certain bitumens is improved resistance to permanent deformation but sometimes the disadvantages such as premature binder hardening leading to premature cracking or fretting of the road surface outweigh the advantages. The latter were attributed to an incompatible blend of blown bitumens, generally used in the manufacture of high PI bitumens, which gave poor results in several of the QUALAGON tests. By closely specifying the manufacturing process, a binder with considerably improved QUALAGON properties was achieved.

The resultant properties of the bitumen developed, known as Shell Multiphalte, are designed to have improved deformation resistance and adhesion properties when compared with conventional paving grade bitumens. Multiphalte is not intended to replace polymer-modified binders, which can have other beneficial properties. The ability of Shell Multiphalte bitumen to improve the deformation resistance of mixtures has been confirmed in both laboratory tests and numerous road trials.

An assessment in which mixtures made using Shell multigrade binder were compared with control materials using conventional binders (Nicholls, 1994b) found that they were more rut-resistant and as durable as conventional materials after 6 years of trafficking. However, they require higher mixing and laying temperatures.

D2 Porous asphalt

The work carried out on porous asphalt materials in the UK has been widely reported (Colwill & Daines, 1989; Colwill et al, 1993; Nicholls, 1997a).

Modifiers have been tried in both 10 mm and 20 mm gradings of porous asphalt surfacings. Their performance in early trials was generally disappointing because the modifiers were used with the same binder content as unmodified binders. It was only when the binder drainage test was developed (Daines, 1986) that the influence of polymers and additives on the maximum 'effective' binder content could be evaluated. The higher binder contents that can be obtained from the use of some polymers and additives have led to improved durability (Daines, 1992b) because the thicker binder film takes longer to age harden to the critical value. The rate of hardening does not appear to be influenced by the type of polymer or additive.

The porous asphalt specification (MCHW 1) includes the binder drainage test (now a British Standard Draft for Development (British Standards Institution, 1996)) and also a binder storage stability test to identify modified binders which remain stable prior to mixing without the need for procedures such as repeated stirring; this also minimises any risk of binder drainage.

The following modifiers have been used in porous asphalt:

Ethylene vinyl acetate (EVA): EVA co-polymers can be used to produce materials with a variety of properties, a more 'elastic' grade of EVA is currently performing satisfactorily in the 1987 trials at Burton after 9 years and, on the Northbound carriageway, a section of porous asphalt containing EVA laid in 1983 lasted for 13 years. However, some of the grades employed in the 1984 trials on the A38 at Burton appear to have given little or no improvement over the unmodified (control) bitumen.

A certain amount of segregation of polymer is normal with most grades of EVA such that EVA-bitumen blends do not always pass the binder storage stability test (MCHW 1). Such failure does not indicate any shortcomings in road performance, but identifies binders for which suitable procedures are required to prevent segregation in storage.

Styrene-butadiene-styrene (SBS): SBS-block-copolymer-modified binders have produced variable results. After 5 years service in the A40 Hillingdon mini-trial, an excellent result was obtained for 10 mm friction course. In the more comprehensive A38 Burton Trials, two types of SBS were used; one was compatible with the binder and the other required recirculation in the binder tank at the plant to maintain a homogenous blend. Both, which were used at 4.2 per cent by weight of binder, produced similar road performances but these were not significantly better than that of the control section containing 100 pen unmodified bitumen. However, the more stable binder could have been used at a higher binder content of 4.5 per cent (Daines, 1986), and it is likely that improved durability would have resulted. The stable SBS blends pass the binder storage stability test.

Natural rubber: Natural rubber, used in latex form and added directly to the mixture at the mixing stage, has given good results in terms of durability. The binder drainage test indicates that the use of natural rubber can allow a relatively high binder content to be used. The natural rubber is added directly at the mixing stage, so storage stability is not a problem.

Other Polymer Modifiers: Polymers, such as polyethylene, butyl rubber, ethylene propylene co-polymer, styrene-butadiene-rubber (SBR) and silicone fluid, have been restricted to small-scale trials which have not shown any obvious benefits. However, reassessment of some of these polymers using the binder drainage test could indicate potential to increase binder film thickness and hence increased durability. For example, SBR emulsion appears to behave similar to natural rubber.

Hard bitumens: Binders such as 70 pen and higher PI bitumen have not shown any improvement when compared with 100 pen bitumen; rather the reverse. The incorporation of Trinidad lake asphalt (TLA) did not lead to an increase in retained binder content as assessed by the binder drainage test; its high viscosity and weathering property indicate that it is probably unsuitable for use in porous asphalts. Nevertheless, successful use of TLA has been achieved in other countries. Other naturally occurring asphaltic bitumens would be expected to behave similarly. Generally, bitumen modifiers that promote hardening are not considered to be suitable.

Fibres: Fibres (mineral or organic) reduce the risk binder drainage and road trials are proving that increased durability is attainable (Daines, 1992b). These additives can be added into a drum mixer or a continuous plant.

Thermosetting Materials: Epoxy asphalt binder can give excellent results but the performance in the A38 Burton 1984 trials was disappointing, being not significantly better than the control material. However, used at the higher binder content of 4.5 per cent in the 1987 trials it is performing well after 9 years. Problems associated with short "pot-life" and adequate curing after laying, together

Table D3 Generic classes of bitumen modifier and additive

Generic Class	*Common Examples*	*Comments*
Chemical set	Epoxy resins, metallic compounds	May be expensive, rate of cure needs to be critically controlled
Plastomers	Ethylene vinyl-acetate co-polymers (EVA), polyethylene, polypropylene	Generally improved viscous characteristics at high road temperatures (reduced plastic flow), some improvement in elasticity. May not form stable dispersion in bitumen.
Elastomers	Natural rubber, synthetic rubber (including SBS and SBR), networked and cross-linked rubber	Generally highly elastic materials improving low temperature performance. Also reduce viscous flow at higher temperatures
Fillers	Cellulose and mineral fibres, sulphur, scrap rubber crumb	Generally stiffens the mixtures. Fibres allow greater film thickness in porous asphalt mixtures
Others	Blown bitumens	Altered rheological characterisation of bitumens, particularly over in-service temperature range

with cost, will probably preclude its general use and restrict laying to the warmer months. However, it may be useful in high stress situations and other specialist applications.

Hydrated Lime: The addition of 2 per cent of hydrated lime has consistently produced better porous asphalts. In the 1984 trials using 100 pen bitumen with hydrated lime gave a life of 8 years, but only 7 years where it was omitted. The same trend is evident for the 1987 trials. Hydrated lime enables a small increase in binder content to be obtained in the binder drainage test and it appears to retard the hardening of the binder marginally (Colwill & Daines, 1989); however, its main effect is to improve resistance to stripping.

D3 Types of bitumen modifier and of additive for asphalt surfacing

Bitumen modifiers and additives which have been investigated for use in wearing course materials are shown in Table D3 in terms of generic type. A more extensive review has been provided by other researchers (Whiteoak, 1990; Wardlow & Shuler, 1992; Terrel & Epps, 1989). However, not all modifiers or additives within a generic group modify the asphalt mixture to the same degree. Furthermore, there are different grades (or types) of many of the common examples listed (for example, the ethylene vinyl acetates - EVA). Some are used in proprietary processes, some are non-proprietary materials, some involve pre-blending with the bitumen and some involve addition at the asphalt mixing stage.

The main reason why most polymer modifiers improve the performance of hot rolled asphalts is that the polymer alters the visco-elastic response of the binder and hence, that of the mixture. This is particularly so at the extremes of temperature, below 0°C and above 30°C, with the polymer conferring a degree of flexibility at low temperatures and, an increased flow resistance at high temperatures. However, not all polymers work in the same way but the fundamental rheological characteristics of the modified binder can potentially be used to predict road performance (Gershkoff et al, 1997). Already, this approach has been applied to assess the effectiveness of currently available modifiers in the lower layers (Carswell & Gershkoff, 1993).

Often, modifiers are used in conjunction with a softer binder to obtain a more *workable* and hence more easily compactable material without compromising deformation resistance. However, it has never been proved conclusively whether the greater *workability* was due to the modifier or the softer grade bitumen.

In terms of cost, polymer modifiers are all more expensive than bitumen and normally incur additional blending and/or production costs. However, because only relatively small proportions are used, typically less than 8 percent of the binder, the overall cost addition is not considered to be excessive when related to the potential improvement in pavement performance.

For many of the bitumen modifiers listed, the more traditional binder tests or methods used to predict viscous parameters or in-service performance (for example, the Shell bitumen test data chart (Heukelom, 1969) and ring and ball softening point) are not appropriate as they assume normal (Newtonian) and/or equi-viscous behaviour. When polymers are added to bitumens, even in relatively small quantities, the rheological behaviour of the resulting binder is often dramatically different from that of the unmodified bitumen. Empirical test methods, such as penetration and softening point, do show some evidence of the presence of a modifier but cannot be relied upon to quantify the changes that have been brought about. Thus, it is becoming more common to measure the rheological properties of modified binders (and binders generally) by dynamic testing regimes (frequency and temperature sweeps) which characterise the viscous and elastic components (loss and storage moduli), the complex shear modulus (G*) and the associated phase angle, σ.

Binders, modified and unmodified, can be generally considered near-Newtonian in behaviour at temperatures greater than 150°C. Simply, this means that the viscosity is independent of rate of shear (or loading time). However, over the in-service road temperature range (-10°C to 60°C) the binders, and particularly modified binders, exhibit visco-elastic behaviour, which means that the measurement of viscosity will be dependent on the rate of shear or loading time. Therefore, unless the loading time of any measurement of viscosity is known, it will be difficult to relate viscosity values from different measuring systems.

Researchers are divided in their opinions as to the most appropriate loading time to choose for measuring the rheological properties of the binder so that close correlation can be obtained with observed practice. For binders exhibiting near-Newtonian behaviour, the selected loading time is not critical because the value of the rheological parameter would not change unduly. For heavily modified binders (especially those exhibiting pronounced elastic behaviour), the case is not so straightforward.

Recent research carried out in the USA as part of the Strategic Highway Research Program (SHRP) has concluded that it is the response at short loading times that governs the resistance to deformation (SHRP, 1993). SHRP have developed specifications based around the behaviour at 10 rad.s^{-1}, equivalent to a loading time of 0.1 seconds. Rather than use G*, the quantity (G*/sin σ) is used which, it is claimed, takes into account the increased elastic nature (lower σ values) of modified binders.

Recent laboratory work carried out on modified binders for wearing courses using the SHRP criterion indicates that this shorter loading time will generally apply to modified binders (Bahia, 1995). However, for those binders which exhibit very strong elastic behaviour the SHRP deformation criterion will greatly underestimate their potential to resist permanent deformation (Gershkoff et al, 1997). This is confirmed in practice, where these same modified binders have been shown to be more deformation resistant than conventional 50 pen bitumen in hot rolled asphalt wearing course.

Further, if the criterion of (G*/sin σ) at the longer loading time of 1000 seconds were chosen, then the same ‘highly elastic’ binder would rank closer to that observed in practice. The other modified binders investigated (together with the unmodified binders) generally remain in the same ranking order.

Thus, it appears that the response of the binder at long loading times may predict deformation resistance more accurately; this is favoured by some researchers (Shell, 1978; Nunn, 1986). The shorter loading times proposed by SHRP will require further investigation, particularly for modified binders, before changing from the longer loading time predictive models.

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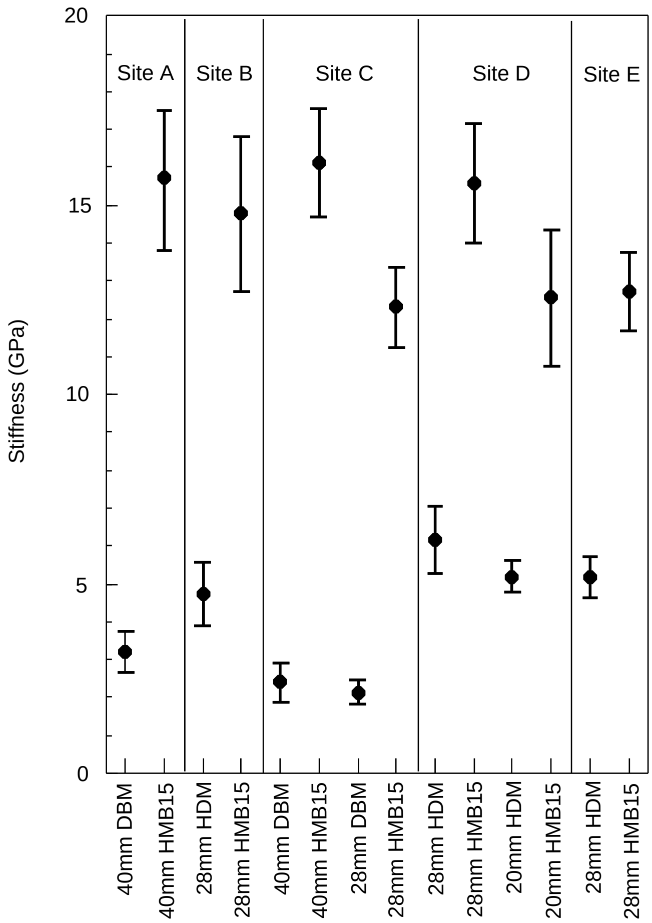

Figure E1 Mean ITSM and standard deviation of HMB15 and control materials

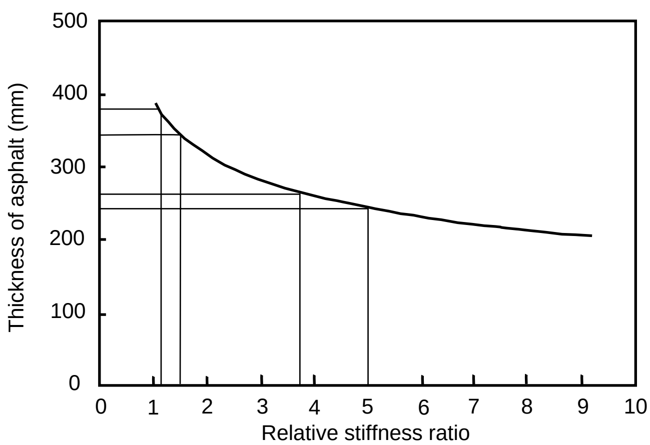

Figure E2 Relative stiffness ratio versus asphalt thickness

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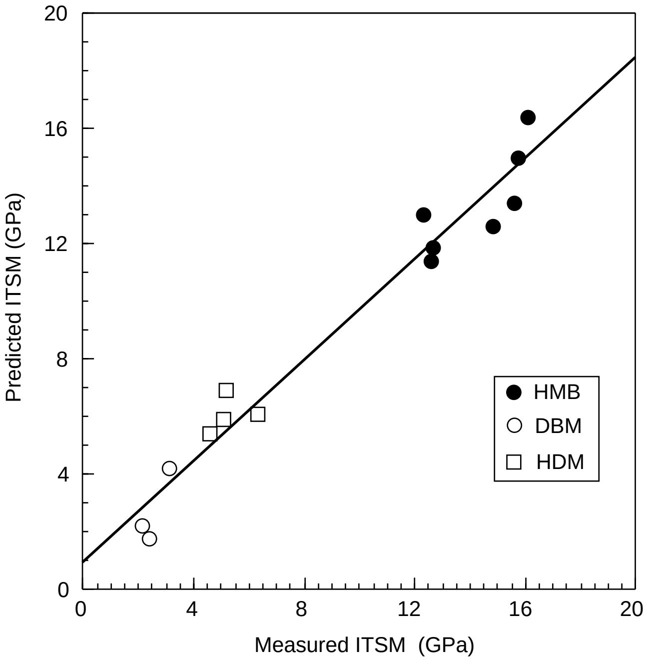

Figure E3 Regression of stiffness against mix variables

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Appendix E: Adapting designs for high modulus asphalt base (HMB)

The assessment of High Modulus Roadbase under contractual conditions in a road trial was sponsored by the Highways Agency, British Aggregate Construction Materials Industries and the Refined Bitumen Association. The work is reported in more detail by Nunn and Smith (1997).

E1 Design stiffness for HMB

High modulus base (HMB) is the generic name for dense roadbase macadams manufactured using binders with a nominal penetration of between 15 dmm and 50 dmm. Nunn and Smith (1997) demonstrated in a series of road trials carried out under contractual conditions that a dense macadam (British Standards Institution, 1992) incorporating 15 penetration grade binder had mechanical properties superior to conventional macadams. This material, known as high modulus base (HMB15), was stiffer and more deformation resistant and its resistance to fatigue was at least as good as standard materials. These trials support the conclusions from the earlier pilot scale trial of high modulus roadbase (Nunn and Smith, 1994) and confirm the potential of HMB15 for use in flexible pavement construction.

The binder used in the all test sections was 15 penetration grade and therefore the improved properties of HMB15 relative to the control materials can be obtained relatively easily. The indirect tensile test was used to determine the indirect tensile stiffness modulus (ITSM), which is a measure of load-spreading ability, of the control and test materials. Figure E1 gives the mean and the standard deviation of the ITSM measurements for each test and control material from each site.

The standard deviation of HMB15 is greater than that of the control material, however the variability is similar to the control when it is expressed as a proportion of the mean value. Any increase in material variability is unlikely to result in roads constructed using HMB having a more variable performance for the following reasons:

- A test sample is very small compared with the slab of material under the wheel that is responsible for load spreading. A large proportion of the specimen variability is due to its small size and this will be averaged out in the larger volume of material supporting the wheel load.
- The relationship between material stiffness, layer thickness and load-spreading ability is non-linear. Figure E2 illustrates the relationship between the stiffness of the roadbase relative to that of DBM and the thickness of asphalt required for a design life of 80 msa. This Figure illustrates that a proportional change in stiffness of high stiffness material will have less effect on thickness than a low stiffness material.

These road trials, which involved test and control materials that used nominal 100, 50 and 15 penetration grade binders, enabled the structural properties of roadbase macadam to be related to the penetration of the recovered binder. The regression between the mean ITSM and mixture variables is illustrated in Figure E3.

$$\mathrm{Log}_{10}(S_m) = 1.86 - 0.0138P - 0.144B \text{ .... (E1)}$$

Table E1 Predicted stiffness of roadbase macadam

Roadbase Macadam	*Nominal pen of binder*	*Pen of recovered binder*	*Target binder content (%M/M)*	*ITSM (GPa)*
DBM 28mm	100	70	4.0	2.1
DBM 40mm	100	70	3.5	2.4
HDM 28mm	50	35	4.0	6.3
HDM 40mm	50	35	3.5	7.4
HMB35 28mm	35	25	4.0	8.6
HMB35 40mm	35	25	3.5	10.2
HMB25 28mm	25	17.5	4.0	11.0
HMB25 40mm	25	17.5	3.5	13.0
HMB15 28mm	15	12	4.0	13.0
HMB15 40mm	15	12	3.5	15.4

Table E2 Design stiffnesses for roadbase macadams

Material	Design stiffness *(GPa)*	Relative stiffness
DBM	3.1	1.0
DBM50	4.7	1.5
HDM	6.2	2.0
HMB35	8.0	2.6
HMB25	10.3	3.3
HMB15	12.4	4.0

($R^2 = 0.93$)

Where: S_m = Indirect tensile stiffness modulus (ITSM),
P = Penetration of recovered binder, and
B = Percentage by mass of binder.

The regression analysis only identified penetration of the recovered binder and binder content as significant variables that influenced material stiffness and that 93 per cent of the variance is due to these variables. Equation E1 can be used to estimate the stiffness modulus of HMB using binders harder than a nominal 50 penetration grade that conform to the middle of the specification range. The properties of HMB incorporating either 35 or 25 penetration grade binder can also be estimated by interpolation of the results.

Bitumen hardens during mixing and laying to typically 70 per cent of its initial level. The average penetration of the recovered 15 pen binder was 12 and if 25 penetration grade binder was used the penetration of the recovered binder would be expected to be about 17.5 while that of a nominal 35 penetration grade binder would be expected to be about 25. Table E1 gives the ITSM of macadams using different binder grades conforming to the middle of the specification (BS4987 Part 1), predicted using Equation E1.

The lower stiffnesses of the macadams manufactured using smaller nominal sized aggregates is due to the higher binder content and not the smaller stone size.

Table E1 shows that HMB15 is about twice as stiff as HDM and about six times as stiff as DBM and that HDM is about 3 times stiffer than DBM. This can be contrasted with earlier trials reported by Nunn et al (1987) which indicated that typical HDM was twice as stiff as typical DBM. However, in these earlier trials, the stiffnesses were compared at a test frequency of 5 Hz and 20°C, recommended for pavement design purposes, using the TRL 3-point bending test. This is a higher frequency than the 2.5 Hz associated with the ITSM test (Nunn & Bowskill, 1992). HMB has a lower frequency sensitivity than conventional DBM. Measurement by Nunn and Smith (1997) showed that the stiffness of DBM increases by approximately 25 per cent for a change in test frequency from 2.5 Hz to 5.0 Hz compared to a change of only 7.5 per cent for HMB15. This would reduce the stiffness of HMB15 relative to DBM to just over five if the stiffnesses are compared at the 5.0 Hz used for design.

In this series of trials reported by Nunn and Smith (1997), the range of measured stiffness of the HMB15 macadams (12.3 - 16.1 GPa) were consistently higher than the range found in an earlier pilot-scale trial (7.2 - 9.3 GPa), reported by Nunn and Smith, (1994). Analysis by Nunn (1996), demonstrated that the stiffness of typical newly laid DBM is expected to be in the range 1.2 - 2.1 GPa, which is somewhat lower than the values expressed in Table E1. These earlier results suggest that the stiffnesses measured in the HMB road trials were towards the upper end of the expected distribution.

In view of the uncertainties expressed above, the novelty of HMB, and the lack of long-term experience of the material's performance, it is prudent to be conservative in selecting a design stiffness for HMB. The results of this trial indicate that the stiffness modulus of HMB15 at 20°C and 5 Hz is 12.4 GPa at the 5 per cent level of significance. Therefore, it is suggested that a design stiffness of 4 times that of DBM is adopted for HMB15. The same criteria will give 2.6 times that of DBM for HMB35 and 3.3 for HMB25. This will give design elastic stiffness moduli of 12.4 GPa, 10.3 GPa and 8.0 GPa for HMB15, HMB25 and HMB35 respectively at the reference condition of 20°C and 5 Hz used for pavement design. These design stiffnesses are reproduced in Table E2.

E2 Early-life stiffness, curing and design

The stiffness of DBM given in Table E2 is considerably higher than the stiffness of newly laid DBM. In LR1132 it was recognised that the stiffness of DBM can change dramatically over its life and that the design stiffness given above should be considered to be an *effective in-service modulus*. This change will improve the load spreading ability of the main structural layers of a flexible road during its service life and is better known as curing. The stiffness of DBM, measured using the indirect tensile stiffness modulus test, typically increases from about 1.5 GPa after it is first laid to 6 GPa or more after 20 years in service (Leech and Nunn, 1997).

Appendix A of this report reviews information that has

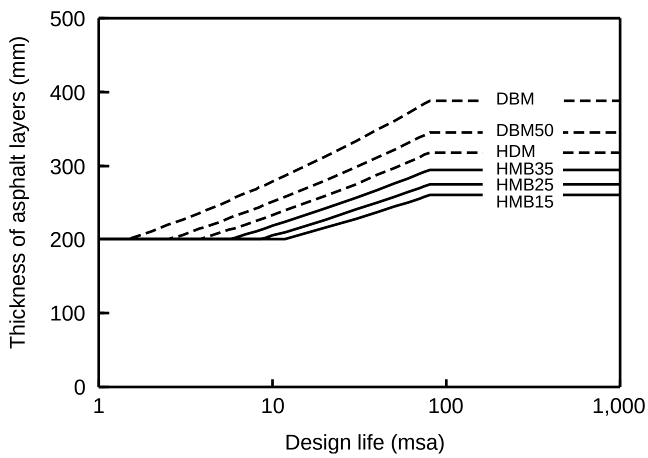

Figure E4 Design curve for high modulus roadbase (HMB) accrued since LR1132 was developed in 1984 and demonstrates that provided flexible roads are well constructed and built above a threshold strength they will have a very long structural service life of 40 years or more provided that distress, in the form of cracks and ruts appearing at the surface, is treated before it begins to affect the structural integrity of the road. These *long-life* roads maintain their strength or become stronger over time, rather than gradually weakening with traffic. It also demonstrates that a road is more vulnerable when it is first opened to traffic before the full load-spreading ability of the roadbase has developed through curing.

Roadbase materials with improved structural properties are usually produced using lower penetration grade binders. Materials that use a 50 penetration grade binder have only been in wide-spread use in the UK since 1987 (Nunn et al, 1987) and consequently less is known about their long-term curing behaviour than macadam using 100 penetration grade binder. Roadbase manufactured with harder bitumen can be viewed as a material which, when first laid, already contains some of the benefits gained during curing of a conventional material. However, as the threshold thickness for long-life pavements was estimated conservatively in Appendix C for the case where curing does not take place, the long-life designs for the improved macadams will still be adequate even if curing takes place at a reduced rate compared with DBM.

Site investigations have shown that the binder in DBM can reduce from its initial value of 70 soon after laying to between 20 and 50 after 20 years in service. A corresponding reduction for a nominal 15 pen bitumen would result in a penetration in the range 3 to 9. Although the curing behaviour of HMB is not known, it probably cures more slowly than a 100 pen bitumen. Furthermore that rate of curing depends on the voids content (Chaddock and Pledge, 1994) and well compacted materials will cure more slowly. Nevertheless, it is not known at what level curing becomes detrimental, if at all, to the performance of the roadbase and there is a need to address this uncertainty.

The improvement in stiffness will result in a reduction in the vertical compressive strain in the subgrade as well as a reduction in the horizontal strain at the bottom of the bound layer. Powell et al (1984) described a method of adapting proven pavement designs to take advantage of new materials by calculating strains at critical locations within the pavement. For a given level of traffic, measured in standard 80 kN axles, the thickness of new material appropriate to an 85% probability of survival can be calculated for a vertical subgrade strain criterion and for a fatigue criterion based on the strain at the bottom of the roadbase. For example, for a pavement with a design life of 80 million standard axles (msa) constructed on a foundation consisting of a subgrade of 5% CBR and 225 mm of Type 1 sub-base the calculated thickness of HMB15 roadbase and basecourse required is 220 mm plus a Hot Rolled Asphalt wearing course 40 mm thick. The thickness of DBM roadbase and basecourse required to meet the same criteria is 350 mm plus 40 mm of HRA wearing course.

Appendix C of this report concludes that roads do not need to be built thicker than that required by the current standard for an 80 msa design life in order to achieve a very long structural life of 40 years or more and that a thickness of less than 200 mm for the asphalt layers is not recommended for even lightly trafficked roads that are required to last for 40 years. Thin roads will be at risk of structural deformation and the rapid propagation of any surface initiated cracks through the full thickness of asphalt (Schmorak and van Dommelen, 1995). The effect of the improved stiffness of HMB15, HMB25 and HMB35 on thickness over a range of design lives is shown in Figure E4.

These design curves assume that the same type of material is used for the roadbase and basecourse and that the wearing course consists of 40mm of HRA. These designs are based on the design stiffnesses given in Table E2, and pavements designed using HMB are expected to perform as well as corresponding pavements constructed using conventional macadams.

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Appendix F: Whole-life cost

F1 Introduction

The whole-life costs of 40 year designs based on existing relationships and the new long-life designs have been examined to determine the economic advantages of the new long-life designs. In this process the user costs during maintenance and the costs of the maintenance itself are discounted to present values and totalled, together with the initial construction costs. This is a complex process involving the prediction of the timings of the maintenance treatments for different classes of road carrying different quantities of traffic, and includes the calculation of the durations of maintenance works and the costs of construction. The computer program, COMPARE, developed by the Highways Agency to determine the whole-life costs of road pavements, has been used in this examination of the whole-life costs of alternative pavement designs.

A three-lane motorway (D3M) and a two-lane all purpose dual carriageway (D2AP) have been considered at two traffic levels appropriate for these road types. In addition to the conventional dense bitumen macadam roadbase (DBM), heavy duty macadam (HDM) and high modulus base using 25 penetration grade binder (HMB25) of equivalent thicknesses have also been considered. With these roadbases, conventional hot rolled asphalt and alternative surfacing materials such as stone mastic asphalt (SMA) and thin surfacings may be used. However all the costs in this study were determined for normal hot rolled asphalt surfacing. It is known that in Germany, SMA can achieve a service life in excess of 20 years but it is not known if it would still satisfy UK performance criteria, particularly for skidding resistance, over this period. The effect of these long surfacing lives would be worthy of consideration in future.

F2 Pavement scenarios

Six scenarios for long-life (40 year designs) have been examined. Structural maintenance will not be required for these scenarios but different surfacing treatments have

Table F1 Whole-life costs scenarios: D2AP 70000 AADF: design life 40 years: 200 msa

Scenario	*Design*	*Roadbase type*	*Roadbase thickness (mm)*	*Maintenance profile (years in which works are carried out)*				
				Surface dress	*Resurface*	*Resurface*	*Resurface*	*Resurface*
				1	*2*	*3*	*4*	*5*
A	Standard	DBM	450	10	15	26	35	-
B	Long-life	DBM	340	10	15	26	35	-
C	Long-life	HDM	270	10	15	26	35	-
D	Long-life	HDM	270	9	14	23	31	39
E	Long-life	HDM	270	11	17	29	39	-
F	Long-life	HMB	230	10	15	26	35	-

Table F2 Whole-life costs scenarios: D2AP 25000 AADF: design life 40 years: 69 msa

Scenario	*Design*	*Roadbase type*	*Roadbase thickness (mm)*	*Maintenance profile (years in which works are carried out)*				
				Surface dress	*Surface dress*	*Surface dress*	*Resurface*	*Surface dress*
				1	*2*	*3*	*4*	*5*
A	Standard	DBM	340	10	20	28	34	-
B	Long-life	DBM	340	10	20	28	34	-
C	Long-life	HDM	270	10	20	28	34	-
D	Long-life	HDM	270	9	17	25	30	38
E	Long-life	HDM	270	11	21	31	39	-
F	Long-life	HMB	230	10	20	28	34	-

Table F3 Whole-life costs scenarios: D3M 90000 AADF: design life 40 years: 269 msa

Scenario	*Design*	*Roadbase type*	*Roadbase thickness (mm)*	*Maintenance profile (years in which works are carried out)*				
				Resurface				
				1	*2*	*3*	*4*	*5*
A	Standard	DBM	420	11	20	28	35	-
B	Long-life	DBM	340	11	20	28	35	-
C	Long-life	HDM	270	11	20	28	35	-
D	Long-life	HDM	270	10	18	25	31	37
E	Long-life	HDM	270	12	22	31	39	-
F	Long-life	HMB	230	11	20	28	35	-

Table F4 Whole-life costs scenarios: D3M 45000 AADF: design life 40 years: 160 msa

Scenario	*Design*	*Roadbase type*	*Roadbase thickness (mm)*	*Maintenance profile (years in which works are carried out)*			
				Resurface			
				1	*2*	*3*	*4*
A	Standard	DBM	320	12	23	33	-
B	Long-life	DBM	340	12	23	33	-
C	Long-life	HDM	270	12	23	33	-
D	Long-life	HDM	270	11	21	30	38
E	Long-life	HDM	270	13	25	36	-
F	Long-life	HMB	230	12	23	33	-

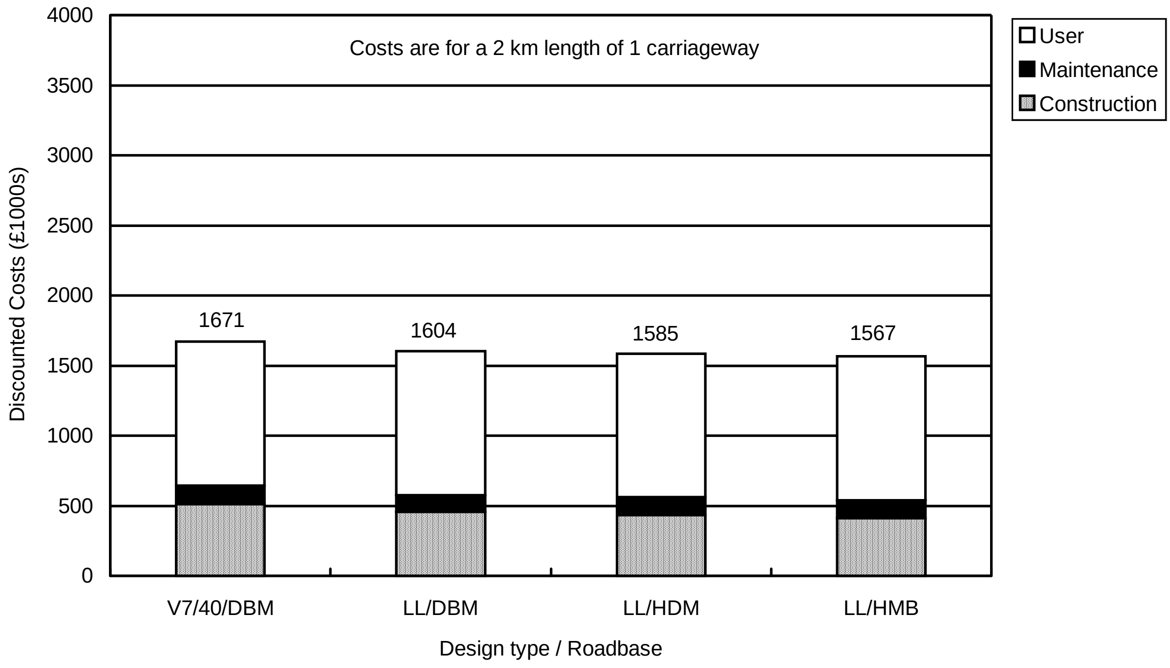

Figure F1 Whole-life costs: D2AP / 70000 AADF / Normal working

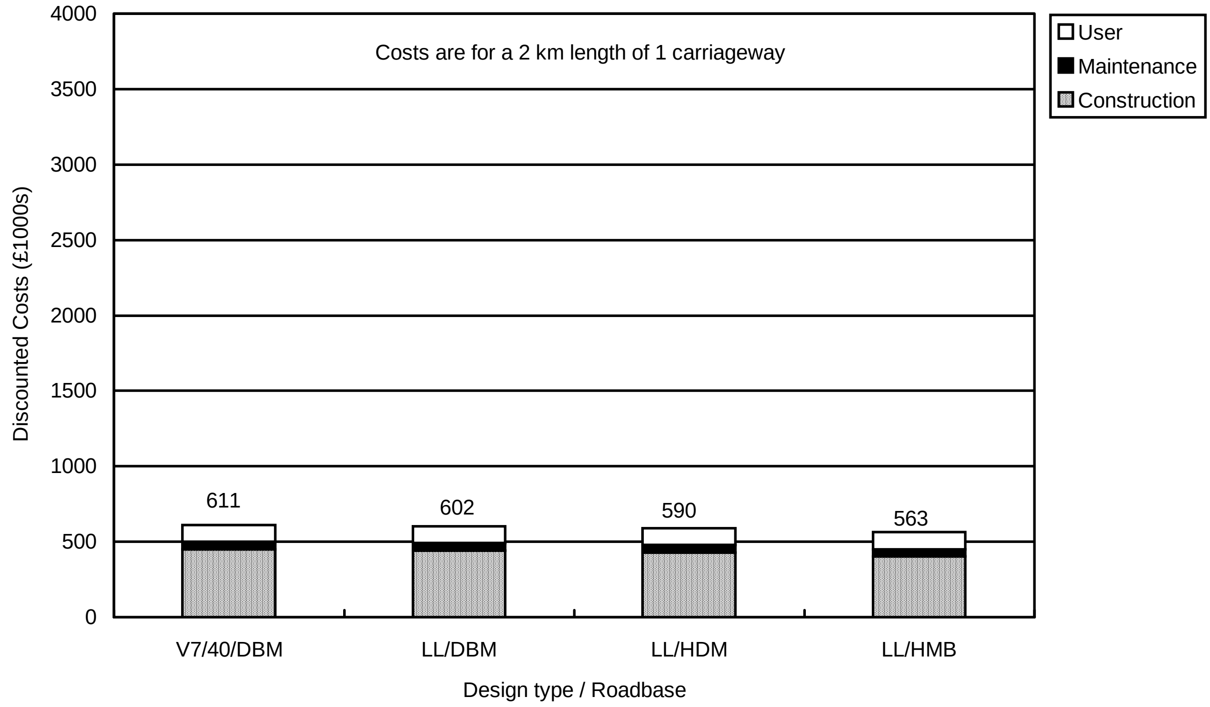

Figure F2 Whole-life costs: D2AP / 25000 AADF / Normal working

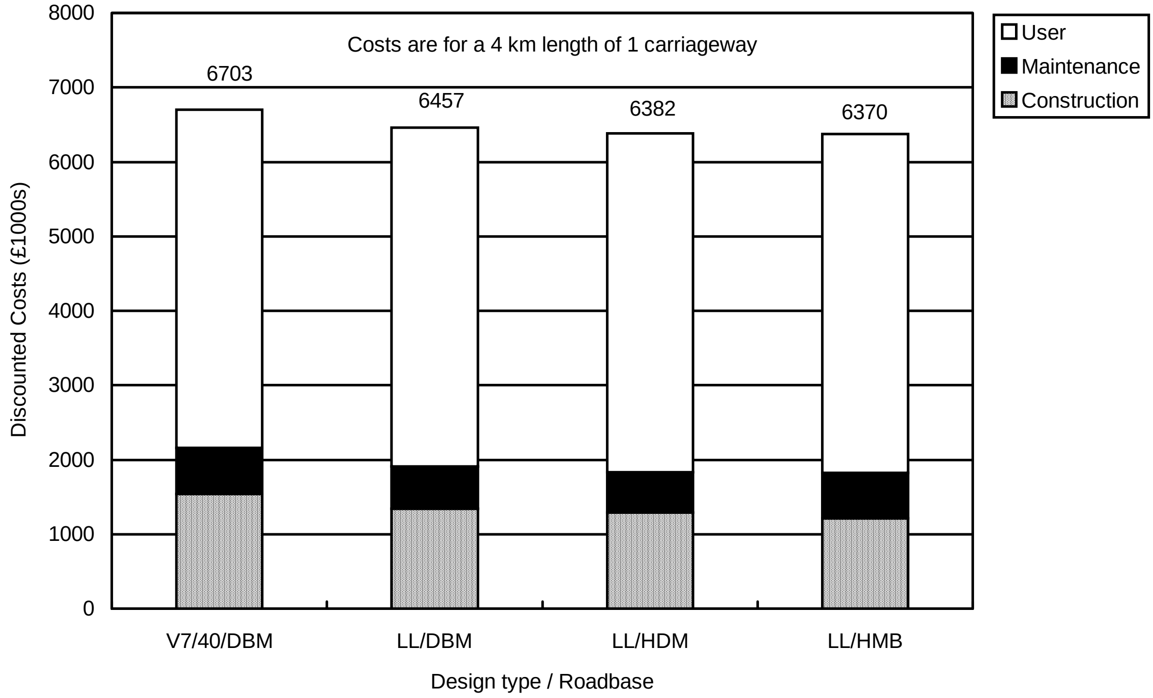

Figure F3 Whole-life costs: D3M / 90000 AADF / Normal working

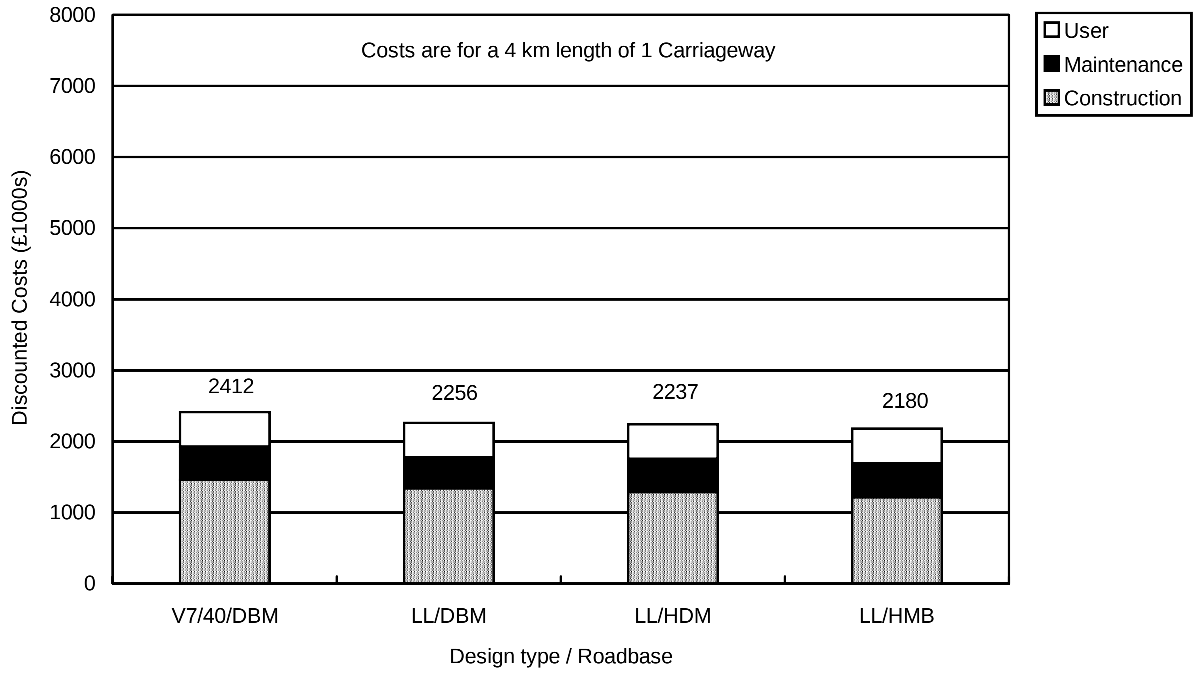

Figure F4 Whole-life costs: D3M / 45000 AADF / Normal working been considered. Details of the scenarios are set out in Tables F1 to F4 which give the level of traffic, pavement materials, thicknesses and maintenance intervals. Tables F1 and F2 deal with D2AP and Tables F3 and F4 deal with D3M. Scenario A in each of these tables is a pavement designed in accordance with HD 24/96 (DoT, 1996). The maintenance profile for this conventional pavement has been generated using COMPARE. It has been assumed that there is a minimum interval of 5 years between consecutive interventions. Scenarios B, C and F are for long-life pavements with DBM, HDM and HMB roadbases, respectively. However, there are no deterioration relationships for long-life pavements and in this study it has been assumed that the future surfacing maintenance requirements are the same as those for the conventional pavements. Whole-life costs for these scenarios have been calculated by adopting the maintenance profile generated using COMPARE for the conventional pavements.

The effect on whole-life costs of achieving different surfacing lives has been investigated by carrying out sensitivity tests on one of the scenarios being examined. The HDM roadbase was selected for the tests as this is approximately the average of the three long-life pavements in terms of thickness and construction cost. In scenario D each interval between consecutive treatments was reduced by one year relative to scenario C, and in scenario E, the intervals have been increased by one year.

In this analysis it has been assumed that surface dressing is not a treatment option for motorways and resurfacing has therefore been used both to restore skid resistance and to repair surface rutting. This does not imply any policy against using surface dressing on motorways but merely reflects the infrequent choice of that treatment for motorway sites. The overlay design has been determined using relationships developed for PANDEF (DoT, 1994). On the D2AP pavements, surface dressing has been used to restore skidding resistance and resurfacing has been carried out to remove surface ruts.

The total thicknesses of asphalt roadbase and surfacing required to achieve a long-life pavement are:

390 mm DBM   320 mm HDM   280 mm HMB25

The roadbase thicknesses given in these tables assume 50mm of surfacing. In the case of the D2AP / 25000 AADF pavement (Table 2), the HD 26/96 design thickness is just below the long-life thicknesses, so the long-life thicknesses have been used.

F3 Whole-life costs

The results of the COMPARE analyses are shown in Figures F1 and F2 for the dual carriageway (D2AP). The costs are for a 2 km length of one carriageway, which is the typical length of a maintenance scheme for this class of road. The sensitivity scenarios D and E are not shown. Figures F3 and F4 give the costs for the motorway (D3M) but in this case the costs are for a typical maintenance length of 4 km of one carriageway. The maintenance costs associated with the treatments and the traffic management costs assume that the maintenance works are carried out with continuous night and day closures. User costs represent the costs to road users resulting from delays and accidents at the roadworks sites. It has been assumed that the delay costs result mainly from queuing at the approaches to the roadworks and that the delays due to reduced speed as vehicles travel through the roadworks site can be ignored. The COMPARE analyses have been carried out using default data for works costs and durations from the HA study of existing schemes carried out during the development of COMPARE.

The total discounted costs reduce slightly in all cases for the long-life designs compared to the standard designs for 40 years traffic. Of the long life-pavements the thinner HMB scenarios have marginally lower costs. For the heavily trafficked situations (Figures F1 and F3) the costs are dominated by those of the road users, representing costs incurred by disruption due to roadworks. Traditionally, these costs do not influence the paving contractor but in current contracts involving lane rental charges an element of the user costs will be reflected in the contract and may affect the cost of the work. For design, build, finance and operate road schemes, user costs are not used directly, but lane charges represent the importance given to minimising the delays to road users.

Additional analyses have been carried out to determine the extent to which the user costs might be reduced if resurfacing was carried out overnight and the road re-opened each day. Surface dressing and overlays are assumed to be day-time operations. No changes in the performance of the maintenance treatments are assumed for those works carried out at night. Costs for the higher traffic option for each road type have been produced for two alternative maintenance procedures:

i maintenance with full night and day closures for the first 20 years and with only night closures thereafter

ii maintenance with only night closures for the full 40 years.

It has been assumed that at night, productivity would be lower and the costs higher compared to those for normal working. A value of two thirds of the day time output rates and doubled labour and plant costs have been used for night working in the analysis. The costs for the D3M high traffic scenario for different working methods are shown in detail in Figure F5 and in summary in Figure F6. It can be clearly seen that although the maintenance costs increase slightly there is a large reduction in user costs and the total costs are substantially reduced. The summary costs for different working methods for the D2AP high traffic scenario are shown in Figure F7 and also indicate that substantial savings can be made compared to closures of 24 hours each day.

The effect of one year shorter or longer intervals between resurfacing on total costs is illustrated in Figure F8 for all four pavement/traffic combinations, assuming normal day and night continuous closures for maintenance. The increases (13 to 17 per cent) and savings (3 to 4 per cent) are similar for all four cases and are principally due to changes in user costs. The maintenance costs are slightly changed as a result of discounting the same costs over

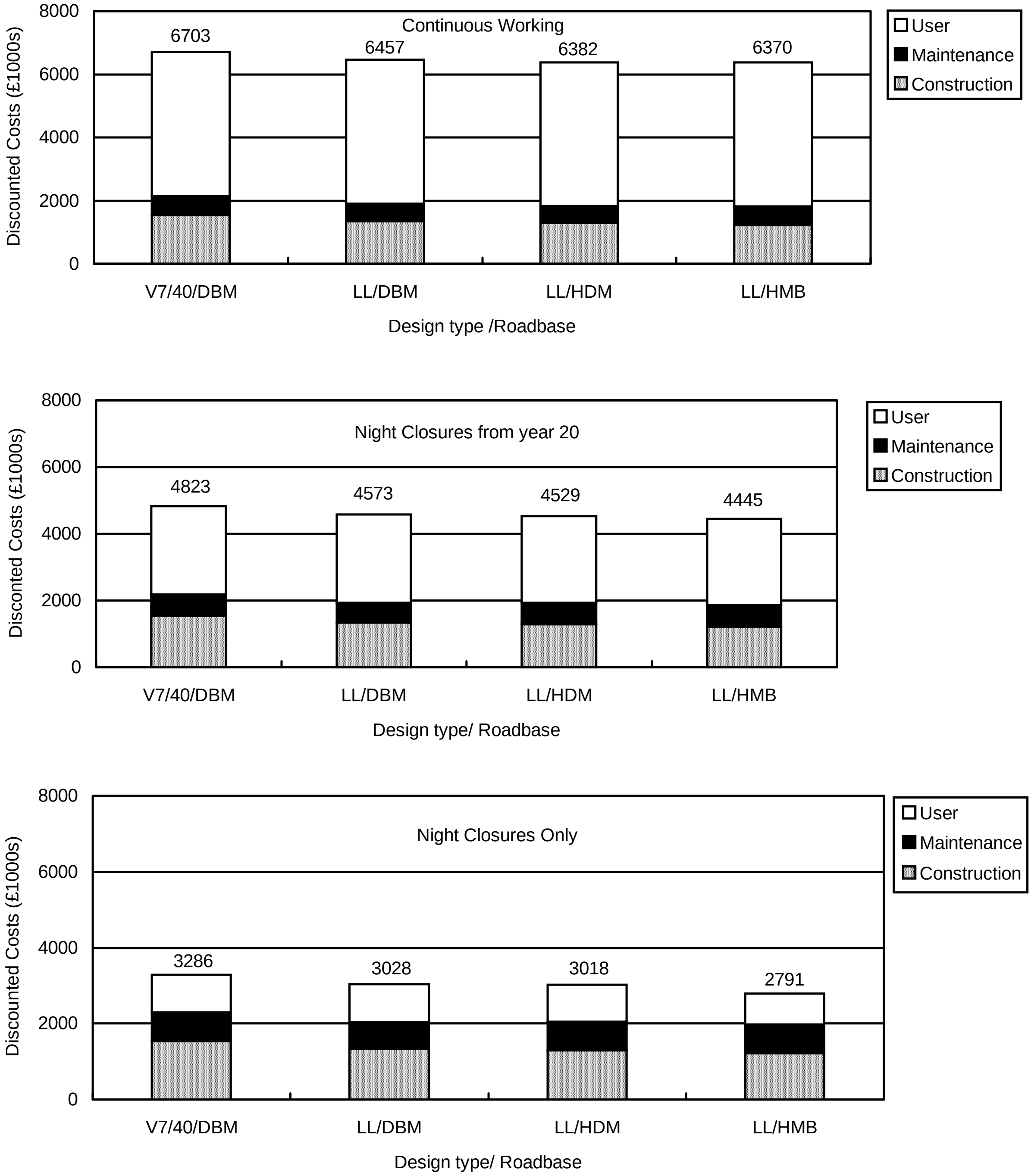

Figure F5 Whole-life costs: D3M / 90000 AADF / Different maintenance methods

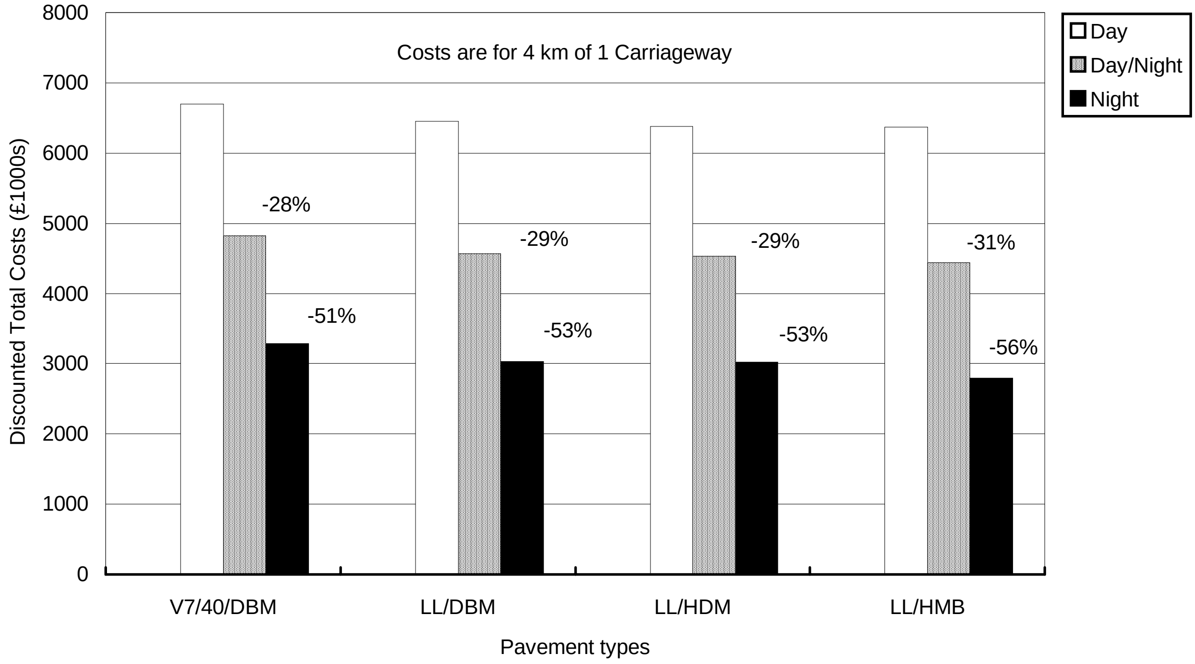

Figure F6 Whole-life costs: D3M / 90000 AADF / Different maintenance methods

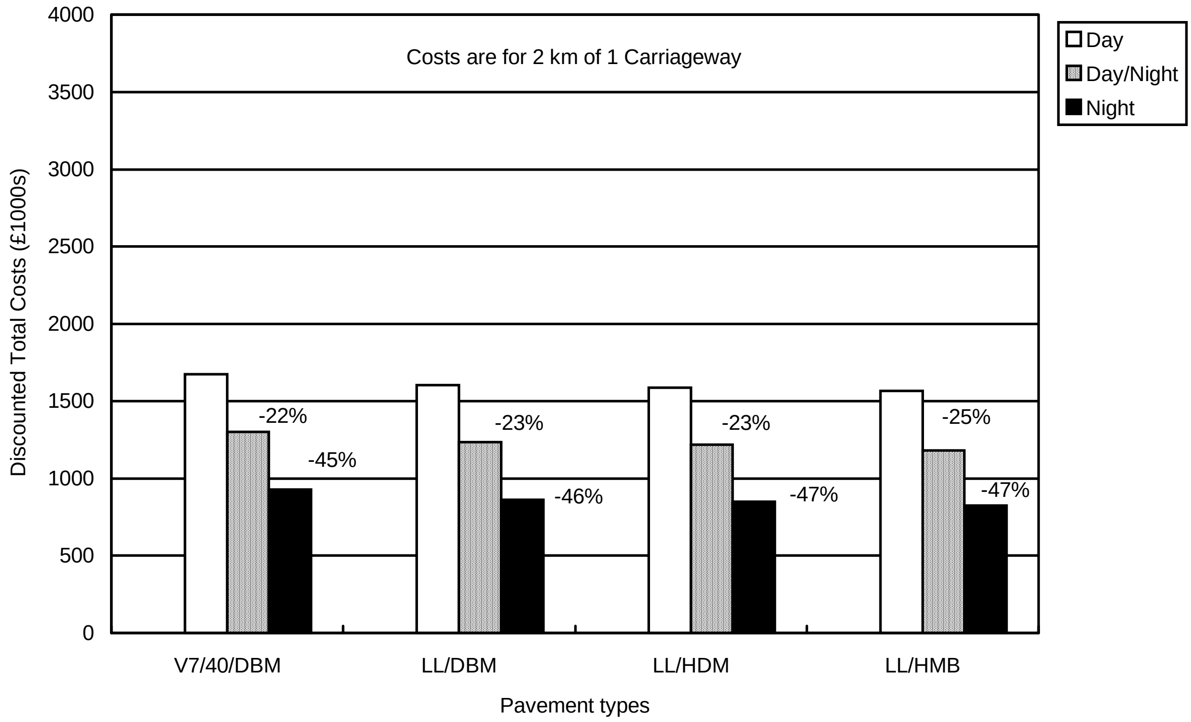

Figure F7 Whole-life costs: D2AP / 70000 AADF / Different maintenance methods

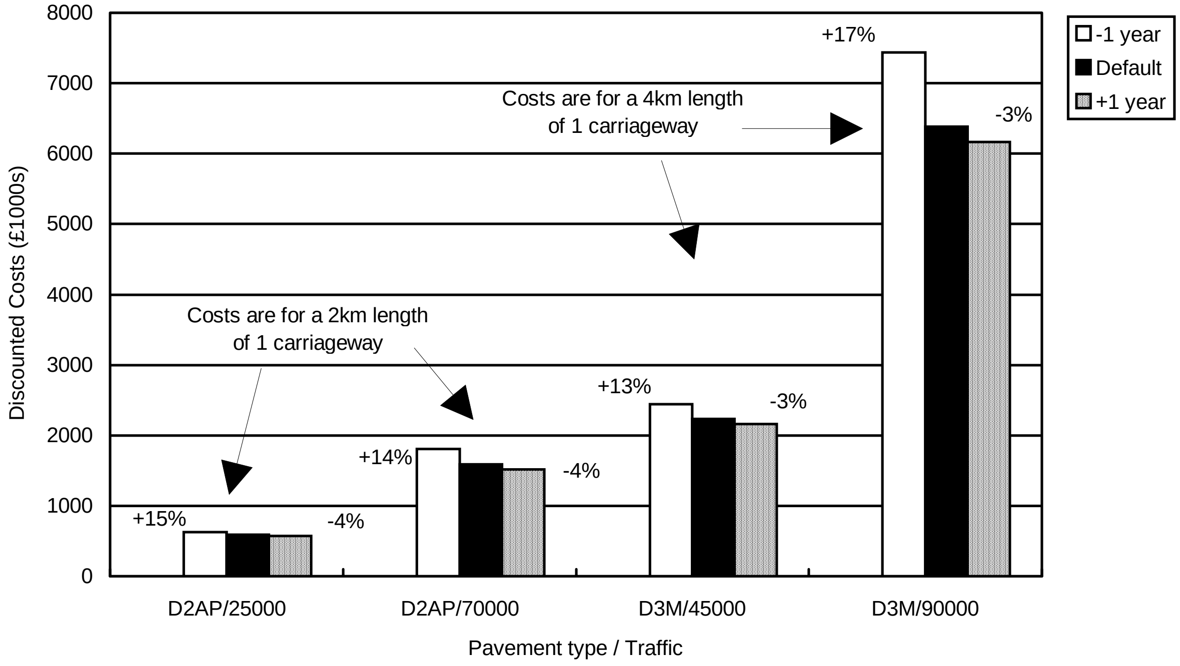

Figure F8 Whole-life costs: Sensitivity to resurfacing life: Normal working

Appendix G: Summary of possible risks

Risk	*Probable Causes*	*Level*	*Means of Reducing Risk*	*Remarks*
Axle loading underestimated	Commercial vehicle annual growth greater than standard value.	Low	Use scheme specific growth rate including a risk factor.	Standard Traffic Assessment Method (HD 24/94) conservative as it assumes high OGV2 percentages.
	Increases due to unexpected local development or diverted traffic.	Medium	Use larger traffic forecasting models. Use a risk factor of, say, +20%	Restricted area models cannot forecast traffic diverting from outside the area.
	Upward revision of vehicle and axle weights (EC harmonization).	Low	Use a risk factor of, say, +10%.	Damage increase of 10% based on strain at the bottom of the roadbase.
	Increasing incidence of *super-singles* will cause more pavement damage than duals.	High	Use a risk factor of, say, +20%.	Theoretical and experimental studies indicate that rates of rutting and wear are both higher for *super singles*.
Premature rutting (arising in the surfacing and basecourse)	Excess binder or low voids High temperature in early life Low traffic speeds	Medium Medium High	Use performance based specifications for asphalt materials. Clause 929 and 943 Use higher performance materials on climbing sections or where congestion is likely.	Particularly affects HRA. When these two causes combine can happen on resurfacing or reconstruction contracts some rutting may occur.
Surface cracking	Excessive ageing	Medium	Limit voids content.	
Loss of chippings	Excessive application rate Lack of adhesion Cold weather working	Low Low Medium	Improve quality control. Improve quality control or use adhesion agent. Use 50mm thick wearing course. Work at high end of temperature range.	
Loss of skidding resistance	Aggregate polishing	Low	No practical measures available.	Near-maximum PSV of natural aggregates already specified. Effect of re-texturing short-lived.
	Loss of surface texture	Low	Ensure wearing course is deformation resistant.	Can occur in association with rutting.
Cracking or disintegration of roadbase	Fatigue cracking	Low	Increase thickness of asphalt layer. Increase binder content of roadbase.	Fatigue cracking theoretically possible but not observed in thick pavements.
	Segregated material	Low	Use 28mm rather than 40mm aggregate. Improve quality control.	Since adoption of Clause 929 the risk has been reduced. HRA less susceptible.
	Stripping	Low	Increase binder content and/or compaction. Avoid use of susceptible aggregates. Prevent moisture content with the roadbase.	Can be a problem with some aggregate types.
Deformation of foundation layers (causing structural rutting)	Inadequate thickness or stiffness of overlying asphalt material.	Low	Improve quality control. Ensure minimum asphalt stiffness material.	Design thicknesses are adequate but constructed thickness may be deficient.
	Inadequate thickness or strength of foundation material.	Medium	Improve quality control. Introduce in-situ strength or stiffness tests.	End-product testing based on either conventional or novel tests is desirable.
	Loss of foundation strength due to inadequate drainage	Low	Use separate surface and sub-surface systems. Increase formation crossfall. Use permeable capping or a blanket drain. Improve quality control to ensure permeable layers are uncontaminated during construction. Improve inspection and maintenance of the drainage system.	Local occurrence, most likely in cut areas. Problems can arise either from groundwater, leaking or severed drains or services or surface water entering through cracks.

Abstract

The pavement design method for fully flexible pavements that has been used in the United Kingdom since the mid 1980s was established by considering the performance of a wide range of experimental pavements which formed part of the trunk road network. Performance trends from these roads were extrapolated to provide a design life of 40 years, based on staged construction in which major strengthening in the form of an overlay is normally applied after 20 years to carry the traffic predicted over the next 20 years. Since this method was developed in 1984, traffic levels have increased, with the consequent increase in traffic disruption at roadworks. Recently the option of considering a 40 year design life for very heavily trafficked locations was introduced that would not require major structural strengthening during the design life. This option was developed by further extrapolating the design curves.

Since these design curves were established, the most heavily trafficked roads have carried in excess of 100 million standard axles and these now provide the opportunity to confirm the validity of the initial extrapolations. Also, more information has become available on the performance of heavily trafficked roads and changes that occur in asphalt over the life of the road. This has indicated that deterioration, as either cracking or deformation, is far more likely to be found in the surfacing than deeper in the pavement structure, as assumed by the current design method. Also, it was found that the great majority of the thick pavements examined have maintained their strength or become stronger over time, rather than gradually weakening with trafficking. The overall conclusion of this project is that a well constructed pavement, built above a threshold strength, will have a very long structural service life provided that distress, in the form of cracks and ruts appearing at the surface, is treated before it begins to affect the structural integrity of the road. These roads are referred to as *long-life* roads.

Related publications

TRL264 *Review of UK porous asphalt trials* by J C Nicholls. 1997 (price code H £30)

TRL231 *Road trials of high modulus base for heavily trafficked roads* by M E Nunn and T Smith. 1997 (price code E £20)

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