Development of a more versatile approach to flexible and flexible composite pavement design

Prepared for Highways Agency

M Nunn

TRL Report TRL615

First Published 2004
ISSN 0968-4107

This report has been produced by TRL Limited, under/as part of a contract placed by the Highways Agency. Any views expressed in it are not necessarily those of the Agency.

TRL is committed to optimising energy efficiency, reducing waste and promoting recycling and re-use. In support of these environmental goals, this report has been printed on recycled paper, comprising 100% post-consumer waste, manufactured using a TCF (totally chlorine free) process.

CONTENTS

Executive Summary

The current UK pavement design standard deals with the use of a limited number of materials in a restricted range of design options. The versatility of this method needs to be increased to give the highway engineer a wider choice of materials and design configurations. The increased versatility will lead to more economic designs by allowing new materials, recycled materials and a wider range of secondary aggregates and binders to be used. It will also enable stronger foundations, incorporating hydraulically bound materials, to be used with reductions in the thickness of the more expensive surfacing layers.

Currently, separate design methods are used for flexible and flexible composite pavements. Many of the designs currently being submitted for road schemes reside in a 'grey' area. They involve pavement structures with material layers that are not covered directly by the current methods, and often there may be more than one way of estimating pavement life depending whether either fully flexible or flexible composite pavement design methodology is used. This can lead to ambiguities and therefore, a more versatile approach is required to deal with these difficult areas, and it will also help to deliver more sustainable and economic pavement construction by making it easier to adopt environmentally-friendly design solutions that make the best use of available materials.

This report describes the modifications to the current design method to improve its versatility. This modified method predicts pavement layer thicknesses that are compatible with existing methodology and methods of characterising materials. As a result, the modified method could be introduced with little disruption. The main features of the method are that:

1 A simpler calculation procedure has been developed for flexible composite designs that does not involve calculating thermal stresses.

2 The current analytical design method for fully flexible pavements has been modified and changes have been made to the flexible composite design method to make the methods more compatible with each other.

3 Foundation stiffness classes have been defined and the revised design methods allow for reductions in the more expensive surfacing layers if stiffer foundations are used.

4 The methods use 360 day material properties.

5 A wider range of hydraulically bound materials can be included, and adjustment factors (K_{Hyd} and K_{Safety}) can be determined that take into account material characteristics and risk.

6 A new design criterion has been defined for fully flexible pavements, which limits the amount of flexing of the asphalt base.

7 The adoption of the adjustment factors recommended in this report will give pavement designs using traditional materials that are in close agreement with the current designs given in the Design Manual for Roads and Bridges.

8 Initially, this more versatile approach could be adopted using dynamic elastic modulus and flexural strength to characterise hydraulically bound materials, and in the longer term static dynamic modulus and direct tensile strength should be introduced to be compatible with new European Standards.

9 A test programme is required to develop more robust knowledge on the curing behaviour of hydraulically bound materials and to enable indicative factors to be developed that link early life material properties to 360 day design values.

1 Introduction

The current UK pavement design standard deals with the use of a limited number of materials in a restricted range of design options. The versatility of this method needs to be increased to give the highway engineer a wider choice of materials and design configurations. The increased versatility will lead to more economic designs by allowing new materials, recycled materials and a wider range of secondary aggregates and binders to be used. It will also enable stronger foundations, incorporating hydraulically bound materials, to be used with reductions in the thickness of the more expensive surfacing layers.

Currently, separate design methods are used for flexible and flexible composite pavements. Many of the designs currently being submitted for road schemes reside in a 'grey' area. They involve pavement structures with material layers that are not covered directly by the current methods, and often there may be more than one way of estimating pavement life depending whether either fully flexible or flexible composite pavement design methodology is used. This can lead to ambiguities and therefore, a more versatile approach is required to deal with these difficult areas. A more versatile approach will also help to deliver more sustainable and economic pavement construction by making it easier to adopt environmentally-friendly design solutions that make the best use of available materials.

This report describes the modifications to the current design method to improve its versatility. This modified method predicts pavement layer thicknesses that are compatible with the existing methodology and methods of characterising materials. As a result, the modified method could be introduced with little disruption.

A further important aspect that has to be considered is that in the UK, cement bound materials have traditionally been characterised in terms of compressive strength. However, for pavement design dynamic elastic modulus and flexural strength are required and this either needs to be measured or a relationship has to be established between compressive strength and these properties. The new European Standard (British Standard EN 14227, 2004) allows two methods of material characterisation: materials can either be characterised by compressive strength or by static elastic modulus and direct tensile strength. The implications of harmonising with the material characteristics classification approach in the new European Standard are briefly examined in Appendix E.

In future, developments in mechanistic pavement models should be examined to assess their potential for either further refining this method or to ascertain whether they can form the basis for a more fundamental approach to design.

2 Pavement design

2.1 Current design methodology for fully flexible pavements

The design method for fully flexible pavements is a mechanistic-empirical method that was developed by Powell *et al.* (1984) and is described in TRL Laboratory Report LR1132. The method uses a linear, multi-layer elastic model of the road to calculate permissible levels of the horizontal tensile strain at the underside of the binder course and vertical compressive strain at the top of the subgrade, induced by a standard wheel load. These strains are used to predict the fatigue life and the deformation life of the road in terms of the cumulative number of 80 kN standard axles. The calculations are carried out at an equivalent pavement temperature of 20°C, which is a traffic and damage-weighted mean temperature. Performance data from 34 sections of experimental road were used to calibrate the model. Designs are considered to have an 85% probability of surviving 20 years before they require strengthening. The method is used to design roads that are predicted to carry up to 200 million standard axles (msa).

2.2 Current design methodology for flexible composite pavements

Designs for flexible composite pavements are based on the performance of experimental road sections, supplemented by structural analysis. Two design strategies are employed.

An empirical method is used to design roads for less than 20 msa. These roads are expected to deteriorate gradually under the influence of traffic. For higher traffic loading, the thickness of the cement-bound base is calculated to prevent longitudinal cracking due to the combined effect of restrained thermal warping stresses caused by temperature gradients in the road and the traffic-induced stress. This is achieved by ensuring that the flexural strength of the concrete base is somewhat greater than these combined stresses.

The life of pavements designed for traffic levels greater than 20 msa is considered to be long, but indeterminate. The ultimate structural life of these pavements is not known but, at present, their nominal design life is restricted to 200 msa.

2.3 Modifications to the current design method

In the development of a more versatile approach, the current design methodology has been followed as much as possible to minimise disruption that would be incurred in transferring to a radically new method. A linear elastic, multi-layer pavement response model has been used to determine the stress or strain at critical locations in the pavement. These criteria have been calibrated using information on pavement performance; the criteria are then used in a wider context. Experience has demonstrated that existing designs have performed well since the method developed by Powell *et al.* (1984) was introduced. Therefore, it was considered prudent to use these designs to calibrate the new method.

The new modified approach continues to rely upon current test methods and material properties. Complex mechanistic models are not considered to be sufficiently validated to be included. Also, their use would require the introduction of new and probably complex test methods to measure material properties for input data.

Design criteria have been developed for hydraulically-bound and bitumen-bound materials. If a pavement contains both types of material, the predicted performance of the most critical layer will be the determinant of pavement life.

In producing a more unified approach, the different traffic threshold levels of 20 msa for indeterminate designs for flexible composite roads and 80 msa for long life flexible pavement construction required harmonisation. To avoid conflicts, a common transition level of 80 msa is proposed.

2.3.1 Standard conditions

The standard conditions used for design calculations are given in Table 1. Generally, the material properties identified by Powell *et al*. (1984) are retained with the exception that the design values are those that are achieved after 360 days in service. This will enable slow curing materials to be more readily incorporated.

2.3.2 Analytical design

Design criteria

The UK design method uses a multi-layer, linear elastic response model to calculate the critical stresses or strains induced under a single standard wheel load (40 kN) that is represented by a circular patch (0.151 m radius) with a uniform vertical stress. The critical stress or strain is then compared with a permissible value to achieve the required design life. To give satisfactory performance, LR1132 (Powell *et al*., 1984) employs the following main structural criteria:

- The asphalt and hydraulically bound materials must not crack under the influence of traffic; this is controlled by the horizontal tensile stress or strain at the bottom of the base.
- The subgrade must be able to sustain traffic loading without excessive deformation; this is controlled by the vertical compressive strain at formation level.

The new approach will use foundation classes and the retention of a subgrade strain criterion will predict implausibly long lives for pavements incorporating stronger hydraulically bound sub-bases. A switch to using foundation classes, in which the foundation layers are represented by an equivalent half-space, will make the subgrade strain criterion redundant. Also, replacing this criterion with a criterion based on the predicted deflection at the top of the foundation will result in similar difficulties.

Therefore it is proposed to drop the subgrade strain criterion and rely on a single criterion that limits the flexural stress or strain at the underside of the base layer to a permissible level to achieve the required pavement life. This critical stress or strain will be that induced by a standard wheel load and it will be calculated using a linear elastic, multi-layer pavement response model.

For hydraulically bound materials, it is proposed that the strength ratio: i.e., the ratio of the traffic induced stress and the flexural strength, be retained. The calculated critical tensile stress at the underside of the base layer will then be

Table 1 Standard conditions for design

	LR1132 (Current method)	*Versatile approach*
Calculation conditions		
Response model	Linear elastic	Linear elastic
Layer interface conditions	Full bond for all layers.	Full bond for all layers.
Design criteria:		
Asphalt layer	Strain	Strain
Hydraulically bound layer	Stress	Stress
Subgrade	Strain	Not used
Load	40 kN	40 kN
Radius	0.151 m	0.151 m
Temperature:		
Flexible	Uniform 20°C	Uniform 20°C
Flexible composite	Temp. gradient	Uniform 20°C
Effective loading frequency	5 Hz	5 Hz
Structural properties of layers		
Asphalt	Poisson's ratio = 0.35 E values at 20°C and 5 Hz	Poisson's ratio = 0.35 E values at 20°C and 5 Hz
Hydraulically bound material	Poisson's ratio = 0.20 28 day values for dynamic modulus and flexural strength	Poisson's ratio = 0.20 360 day values for dynamic modulus and flexural strength
Foundation classes(half-space stiffness)	Poisson's ratio = 0.45 Discrete foundation layers are used in the calculation.	Poisson's ratio = 0.35 Class 1 ≥ 50 MPa Class 2 ≥ 100 MPa Class 3 ≥ 200 MPa Class 4 ≥ 400 MPa

multiplied by a material specific factor that takes into account thermal effects, curing, nature of transverse cracks, etc. If a fatigue calculation were adopted, fatigue characteristics would be required and this would necessitate the introduction of a new laboratory test for hydraulically bound materials.

For the determinate life zone (< 80msa) the value of the permissible strength ratio as a function of traffic is based on a back-analysis of the current standard design curves.

To overcome the difficulty of characterising slow curing materials, the long term structural properties of materials in the completed pavement will be used in design. For practical reasons these will be the one year or 360 day properties.

Wheel load configuration

Methods used elsewhere often use a dual wheel load to calculate the critical strains. Historically a dual wheel was the most common tyre configuration, but now the majority of 5 and 6 axle articulated vehicles use triple (in-line) wide-base tyres (COST Action 334, 1998). Therefore a single wheel load would now be more appropriate. However, provided the method is calibrated using a particular wheel load configuration, and that this configuration is maintained for all subsequent calculations, then the choice of a dual or a single wheel will only have a minor effect. If the pavement structure is changed, a similar relative change will occur in the calculated value of the critical strain for the same axle load configuration. For these reasons, the standard wheel load configuration will remain the same as that used in LR1132. However, this could be adjusted at a later stage if required.

Load frequency/effective loading time

The UK (Powell *et al.,* 1984), French (SETRA/LCPC, 1998) and the Asphalt Institute (1991) methods use an equivalent frequency whilst the Nottingham University (Brown *et al.*, 1985) and the Shell (Strickland, 2000) methods use an equivalent loading time. For a visco-elastic material, the equivalence between loading time (t_0 seconds) and frequency (f Hz) can be approximately related by, $t_0 = 1/(2\pi f)$, which suggests that the Nottingham and Shell methods are based on an effective load frequency of approximately 8 Hz. An increase in load frequency of 1 Hz approximately equates to an increase in speed of 10 km/h, which suggests that the various methods use design traffic speeds in the range 50 to 100 km/h (Nunn and Merrill, 1997).

For typical asphalts, a doubling of frequency will have the effect of increasing the elastic stiffness modulus by between about 8 and 25 per cent, depending on the stiffness of the bitumen.

For similar reasons as given for the choice of wheel load configuration, the reference frequency of 5 Hz is retained. A change in frequency would require that all standard stiffness values in the Highways Agency's Design Manual for Roads and Bridges (DMRB) and the Specification for Highway Works (SHW) to be amended.

Pavement foundation

Pavement design is typically carried out in two stages. The foundation is designed to carry construction traffic and act as a construction platform. Its long-term properties are then used in the analytical model to design the pavement structure.

In the proposed method, foundation stiffness classes will be adopted (see Appendix A). It should be borne in mind that the design value represents the long term stiffness which is considered to be the minimum value during the life of the pavement, and for unbound granular materials the confinement effect of the pavement overburden will affect their stiffness under the completed pavement. Values measured during pavement construction may therefore differ significantly from the long term values used for design.

The foundation stiffness classes are defined in terms of the equivalent half-space stiffness of the composite foundation. The following four divisions, described in more detail in Appendix A, are proposed for design:

- Class 1 ≥ 50 MPa
- Class 2 ≥ 100 MPa
- Class 3 ≥ 200 MPa
- Class 4 ≥ 400 MPa

The standard UK foundation (equivalent to 225 mm of Type 1 sub-base on a subgrade with a California Bearing Ratio, CBR, of 5%) will correspond to Class 2. The Class 1 construction platform is applicable to construction on a capping layer and Class 3 and 4 foundations will involve bound sub-bases.

A Poisson's ratio of 0.35 is proposed for all foundation classes. The value of Poisson's ratio makes only a small difference to the layer thickness calculations. A change from 0.45 to 0.20 generally results in only a few millimetres difference in the calculation of the base thickness.

Material properties

The standard properties used in the current method are retained for both asphalt and hydraulically bound materials (Powel *et al.*, 1984). These are stiffness modulus and fatigue resistance for asphalt, and dynamic stiffness modulus and flexural strength for hydraulically bound materials.

There are a variety of hydraulically bound materials available to the highway engineer that cure at varying rates. For design calculations, it is the value of the stiffness and strength attained in the long term that is important. For this reason the values after 360 days (or one year) are proposed for design purposes. It will be impractical, for site compliance purposes, to specify 360 day material properties. It is suggested that conservative indicative factors to link early life material properties to the 360 day values should be defined.

3 Flexible composite design

The existing design method only considers cement bound material (CBM). However, the general philosophy developed in this report will be applicable to pavements containing other hydraulically bound materials in the base, provided that there is empirical justification for the calibration of the design criterion.

The current method is empirical for designs up to 20 msa and, above this traffic level, the road is designed to have a long, but indeterminate life. To achieve indeterminate life, the strength ratio of the pavement must be greater than about 1.5 (there is some variation in the current design charts). The strength ratio is defined as the ratio between the flexural strength of the CBM (at 28 days) and the combined traffic and thermally induced stresses. In the new design method, 360 day rather than 28 day values for material properties will be used, and it is proposed to make the transition to indeterminate life at 80 msa. This will help bring flexible composite designs in line with fully flexible designs which have a transition to long life at this traffic level.

The existing design method does not include a fatigue criterion for hydraulically bound base material. The fatigue relationship for hydraulically bound materials (fatigue exponent of 12 to 16) is very sensitive to the traffic induced stress compared with that of asphalt (fatigue exponent of approximately 4). A high exponent will produce a design curve in which a relative small increase in thickness will result in a large increase in life. As a result, the indeterminate design has required that the combined value of the traffic and thermally induced stresses at the bottom of the base layer to be somewhat lower than the flexural strength of the material. If this criterion is met, the pavement is considered to have a long, but indeterminate life.

The new method will be analytically based for all traffic levels, and for simplicity it will only consider the traffic-induced stress at the underside of the base layer and this will be multiplied by the following two factors:

K_{Hyd}: This material specific calibration factor includes temperature effects, curing behaviour and transverse cracking characteristics. This factor should be determined empirically if there is sufficient performance data from in-service pavements.

K_{Safety}: This factor can be used to control the inherent risk in pavement design. The value could be adjusted for very heavily trafficked roads or for roads constructed in sensitive areas or to give added conservatism to designs using new materials or new construction practices. The default value will be 1.0 and a lower value produces a more conservative design.

3.1 Material properties at 360 days

The adoption of 360 day design values will bring the characterisation of hydraulically bound material more in line with asphalt material. It is recognised that asphalt cures and the design stiffness of the asphalt base is the stiffness attained after some time in service. A move to 360 days for all materials will enable slow curing hydraulically bound materials, as well as slow curing asphalts, to be accommodated more easily.

To transfer the existing practices to 360 day values, knowledge of the curing relationship for CBM materials is required so that 360 day values can be estimated from the 28 day values that are used at present. The following curing relationship has been assumed for the compressive strength of CBM:

Curing Period	*Relative compressive strength of CBM*
7 day	0.67
28 day	0.80
360 day	1.00

Kolias and Williams (SR344, 1978) suggested a ratio between the 28 and 360 values of about 0.80 for compressive strength and 0.90 for dynamic modulus. It should be noted that the designs in LR1132 assumed that the 28 day compressive strength was 10% greater than the 7 day value; this appears to be conservative. It is also assumed that the modulus and strength of CBM can be related using the following relationships (Croney, 1977):

$$E = \frac{Log\left(f_f\right) + a}{b} \quad (1)$$

$$f_f = c.f_c \quad (2)$$

Where, E is the dynamic modulus in GPa; f_f is the flexural strength in MPa; f_c is the compressive strength in MPa; and *a, b* and *c* are material constants. These relationships were also assumed in LR1132.

Table 2 gives the values of these constants for standard CBM grades using typical gravel (G) and crushed granite or limestone aggregate (R).

Table 2 Constants to be used in equations 1 and 2

	Values of constants		
	a	*b*	*c*
Gravel (G)	0.773	0.0301	0.11
Crushed rock (R)	0.636	0.0295	0.16

The calculated 7, 28 and 360 day properties for CBM are given in Table 3.

For design, the crucial material properties are the elastic modulus and flexural or tensile strength rather than compressive strength. Therefore, it would be advisable to introduce a method of measuring tensile strength rather than compressive strength; compressive strength is routinely measured for compliance purposes and is currently used as a proxy to estimate flexural strength.

3.2 Design criterion for hydraulically bound layers

3.2.1 Heavy traffic designs (> 80 msa)

The design criterion for traffic levels greater than 80 msa is that the predicted critical tensile stress σ_r at the underside of the hydraulically bound base will have to satisfy the following relationship:

$$\sigma_r \leq \text{Flexural strength.}\, K_{Hyd}.K_{Safety} \quad (3)$$

Table 3 CBM properties

Age	*Compressive strength (MPa)*	*Flexural strength (MPa)*	*Dynamic modulus (GPa)*	*Compressive strength (MPa)*	*Flexural strength (MPa)*	*Dynamic modulus (GPa)*
		CBM3G			*CBM3R*	
7	10.0	1.10	27.0	10.0	1.6	28.5
28	12.5	1.38	30.3	12.5	2.0	31.8
360	15.0	1.65	32.9	15.0	2.4	34.5
		CBM4G			*CBM4R*	
7	15.0	1.65	32.9	15.0	2.4	24.5
28	18.8	2.06	36.1	18.8	3.0	37.7
360	22.5	2.48	38.8	22.5	3.6	40.4
		CBM5G			*CBM5R*	
7	20.0	2.20	37.1	20.0	3.2	38.7
28	25.0	2.75	40.3	25.0	4.0	42.0
360	30.0	3.30	42.9	30.0	4.8	44.7

The 360 day values for the structural properties given in Table 3 were used to calculate the critical stress, σ_r. The adjustment factors given in Table 4 were determined to give agreement with the designs in the Highways Agency's Design Manual for Roads and Bridges (7.3.3). The standard case assumes pre-cracking at 3 m intervals and that 180 mm of asphalt cover is sufficient to give adequate resistance to reflection cracking for this crack spacing.

Table 4 Adjustment factors

Material	K_{Hyd}	K_{Safety}
CBM3G	0.329	Default value of 1.0
CBM4G	0.308	
CBM5G	0.272	
CBM3R	0.305	
CBM4R	0.244	
CBM5R	0.234	

Empirical performance data from roads constructed using other hydraulically bound materials would be required to help establish adjustment factors for these materials.

3.2.2 Determinate designs (< 80 msa)

The current designs for less than 20 msa are empirical (see Appendix B). The design curve shown in Figure 1 was used to develop the following design criterion for pavements carrying less than 80 msa:

$$Log(N) = 1.23 \times \left(SR.K_{Hyd}.K_{Safety} + 0.1626\right)^2 + 0.2675 \qquad (4)$$

Where, the strength ratio (SR) is the ratio between flexural strength and σ_r, the tensile stress at the underside of the hydraulic base that has been determined using linear elastic theory.

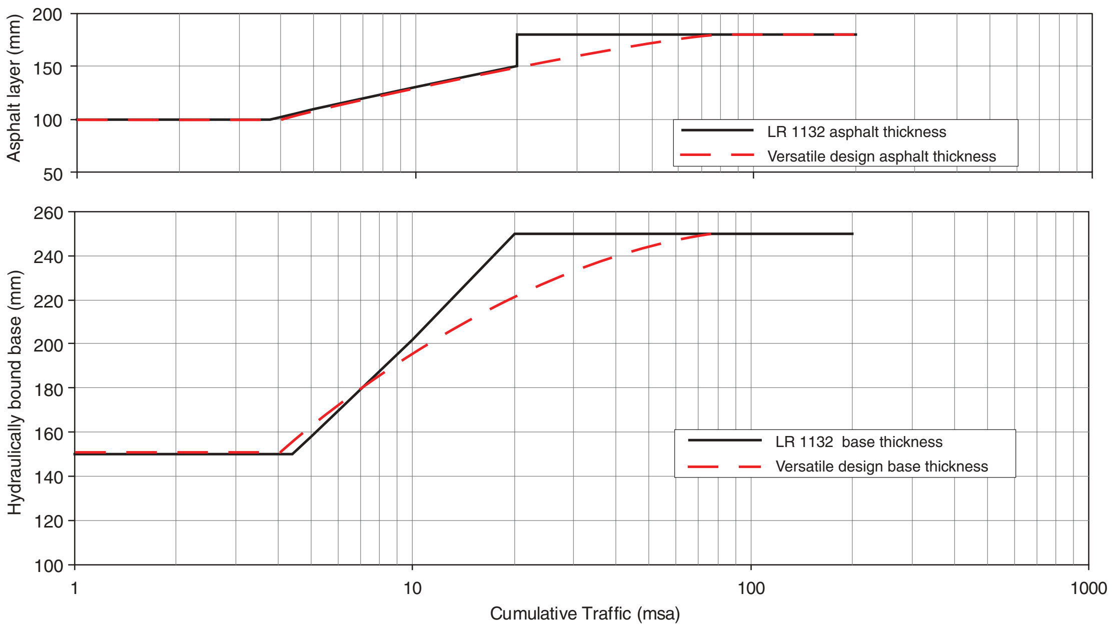

Figure 1 Comparison of CBM3G versatile design thickness and LR1132 design

It should be noted that the thickness of asphalt layer broadly complies with the recommendations given in the Design Manual for Roads and Bridges (7.3.3), except that the asphalt thickness ($H_{Asphalt}$), has been reduced by 10 mm to 180 mm for designs of 80 msa with proportionate reductions for lower traffic levels. This design curve incorporates experience gained from pre-cracking the CBM base at 3 m intervals. The thickness of asphalt cover can be determined using the following empirical equation:

$$H_{Asphalt} = -16.05 \times \left(Log(N)\right)^2 + 101 \times Log(N) + 45.8 \qquad (5)$$

The asphalt layer consists of 35 mm of thin surfacing with the remaining thickness consisting of dense bitumen macadam incorporating 100 penetration grade bitumen. The design stiffnesses of these layers are 2.0 GPa and 3.1 GPa respectively. The thickness of asphalt is required to reduce the risk of reflection cracking to an acceptable level. The thicknesses determined using Equation 5 should be adhered to even if stiffer grades of asphalt are used. However, the use of a stiffer asphalt layer will result in a small reduction in the thickness of the hydraulically bound base.

The base thicknesses, using the criterion given in Equation 4, are in excellent agreement with the existing designs and this criterion can be applied to other hydraulically bound materials using the K_{Hyd} factors that are applicable to those materials.

In Figure 1 the designs determined using these criteria are compared with existing designs from the Design Manual for Roads and Bridges (7.3.3). This comparison has been made assuming that the base was pre-cracked at 3 m intervals and the standard foundation or Class 2 foundation was used. Figure 2 gives design thicknesses for the current grades of CBM laid on a Class 2 foundation. CBM4G and CBM3R are shown to be structurally equivalent and so too are CBM5G and CBM4R.

The procedure to estimate the layer thicknesses for a flexible composite pavement that is designed to carry a given number of standard axles is illustrated in Appendix C.

3.3 Foundation classes

The thicknesses of the CBM base required for a design life greater than 80 msa are given in Table 5 for each of the foundation classes, rounded up to the next 5 mm. Pre-cracking at 3 m intervals is assumed with an asphalt thickness of 180 mm. The design curves for lower traffic levels for a CBM3G base are shown in Figure 3. The minimum permitted CBM thickness is 150 mm and the minimum permitted thickness of asphalt is 100 mm.

Table 5 Design thicknesses for CBM base for > 80 msa

	Foundation class			
CBM base	*Class 1 50 MPa*	*Class 2 100 MPa*	*Class 3 200 MPa*	*Class 4 400 MPa*
CBM3G	275	250	225	200
CBM4G	220	200	180	150
CBM5G	200	180	160	150
CBM3R	220	200	180	150
CBM4R	200	180	160	150
CBM5R	165	150	150	150

The thicknesses given in the Design Manual for Roads and Bridges (7.3.3), using a granular sub-base, are in agreement with the design thicknesses given in Table 5 for Foundation Class 2. Both Table 5 and Figure 3 illustrate the sensitivity of the base thickness to foundation strength. The current method allows no reduction in base thickness when stiffer bound foundations are utilised.

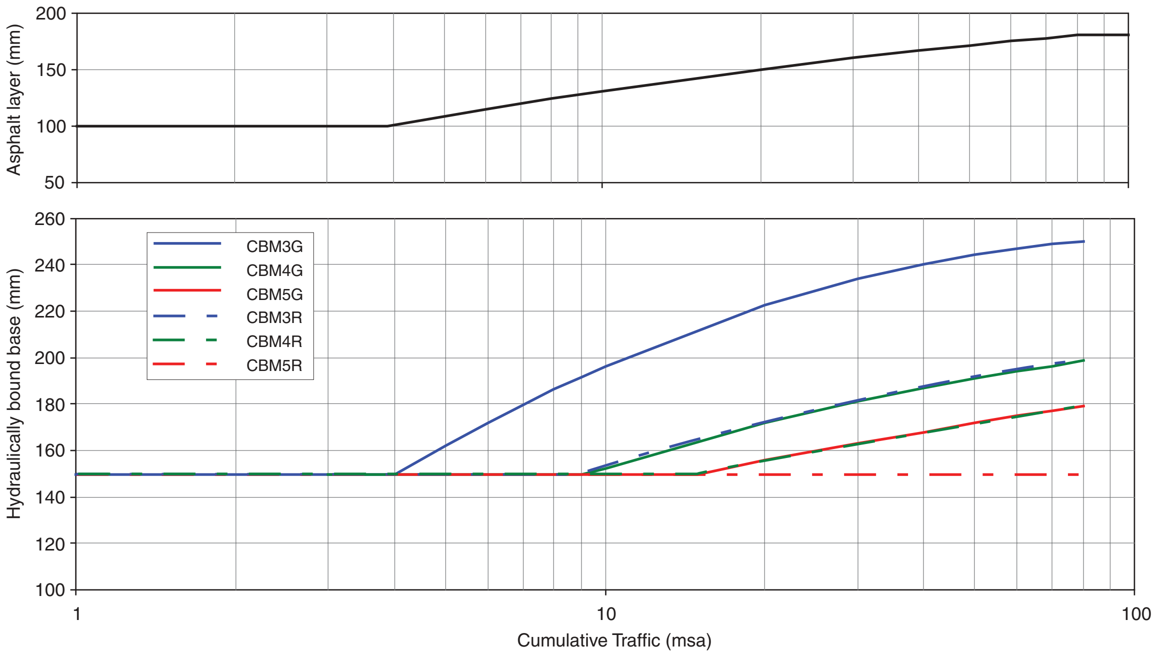

Figure 2 Design thickness for the different grades of CBM

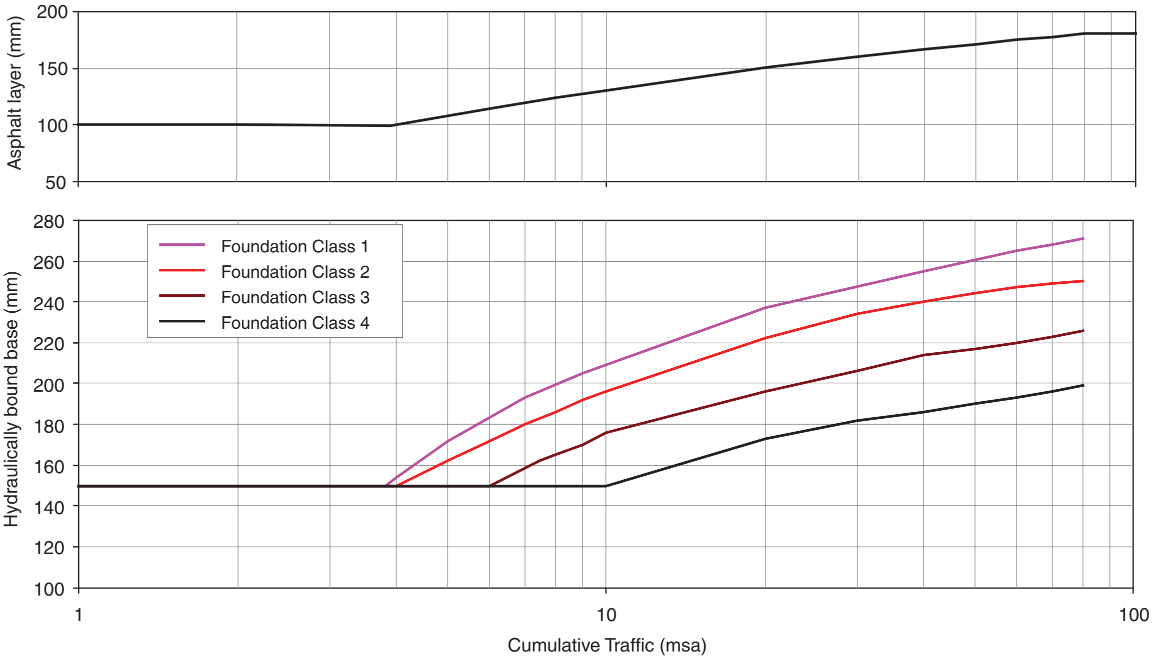

Figure 3 Thickness of CBM3G base for different foundation classes

3.4 Asphalt surfacing

The calculations assume that a certain thickness of asphalt is required to reduce the risk of reflection cracking to an acceptable level. For designs in excess of 80 msa, 180 mm of asphalt is deemed adequate, provided that the hydraulically bound base is pre-cracked at 3 m intervals. Below 80 msa a variable thickness of asphalt is specified. Without pre-cracking, an additional 20 mm of asphalt is required over the full design range.

Knowledge of the relationship between crack spacing and the thickness of asphalt surfacing to prevent the occurrence of reflection cracking for a specified number of years is required before the thickness of asphalt can be determined more precisely. For this to be determined analytically, the mechanism of reflection cracking needs to be fully understood, or alternatively the thickness could be established empirically provided sufficient performance data exists.

With some slow-curing hydraulically bound materials it is generally recognised that well-defined, regular transverse cracks do not occur and it is assumed that a large number of micro-cracks are induced throughout the material. Diffuse cracking in hydraulically bound material could reduce the risk of reflection cracking, and this introduces another design possibility. A practical means of dealing with these materials would be to categorise hydraulically bound materials in terms of the nature of cracking in the base layer and specify the maximum asphalt thickness accordingly; as shown in Table 6.

Table 6 Possible cracking categories

Hydraulic base cracking characteristics	*Maximum asphalt surfacing thickness*
1. Natural well defined cracks	200 mm
2. Pre-cracked at 3 m intervals	180 mm
3. Diffuse cracking	150 mm

4 Fully flexible pavements

Design calculations using foundation classes rely on limiting the tensile strain at the underside of the base layer, which is traditionally considered to be a fatigue criterion. The subgrade strain criterion will become inoperative with the introduction of foundation classes. Whereas in the existing design method (LR1132), subgrade strain is the critical design criterion for asphalt bases that are stiffer than DBM100, the proposed new method will be based solely on limiting the tensile strain at the underside of the asphalt base layer (for more details see Appendix D). To reflect the more general nature of this criterion, it is suggested that the term flexural criterion be adopted and that a material specific flexural factor, K_{Flex}, be introduced into the design criterion. The calculated tensile strain induced at the underside of the asphalt base (ε_r) to achieve a design life of N msa would then become:

$$\varepsilon_r = K_{Flex}.K_{Safety}.\varepsilon_6 \times \left(N/10^6\right)^{-0.24} \tag{6}$$

Where, ε_6 is the strain for a fatigue life of 1 msa which is derived from the fatigue criterion given in LR1132 (Powell *et al.*, 1984). Substituting for ε_6 gives:

$$\varepsilon_r = K_{Flex}.K_{Safety}.201\times10^{-6}\left(N/10^6\right)^{-0.24} \tag{7}$$

or,

$$N/10^6 = \left(\varepsilon_r / \left(K_{Flex}.K_{Safety}.201\times10^{-6}\right)\right)^{-(1/0.24)} \tag{8}$$

K_{Flex} will depend upon the design stiffness (E) of the asphalt base and the following relationship gives a good estimate of K_{Flex}:

$$K_{Flex} = 1.089 \times E^{-0.172} \tag{9}$$

The use of this criterion will ensure that pavements with stiff asphalt bases do not flex as much as pavements with a less stiff asphalt base. Equation 9 results in the values of K_{Flex} given in Table 7, and these can be used to calculate design thicknesses with a reasonable trade off between a stiffer foundation and a reduction in asphalt thickness. This suggests that, depending on the grade of asphalt, the base thickness can be reduced by up to 50 mm by moving to a superior foundation class.

Table 7 Values of K_{Flex}

Material	*Design stiffness (GPa)*	K_{Flex}
DBM125	2.5	0.929
DBM100	3.1	0.897
DBM50	4.7	0.835
HDM	6.2	0.796
HMB35	7.0	0.778

Figure 4 shows the design curves using this new criterion for a Foundation Class 2 compared to the design curves derived from the criteria given in LR1132, using the standard foundation (225 mm of Type 1 sub-base and a 5% CBR subgrade).

In accordance with the original designs given in LR1132, the designs shown in Figure 4 include 40 mm of HRA surface course. The new design criterion has been calibrated to agree with the LR1132 designs at 80 msa, which has resulted in slightly thicker designs for low traffic levels with less stiff grades of asphalt.

4.1 Foundation classes

The design thickness of the asphalt layer for the different foundation classes are given in Figures 5 to 9. These designs incorporate a 35 mm of thin wearing course which is assigned a design stiffness of 2.0 GPa. For this reason they tend to be slightly thicker than the standard designs shown in Figure 4. These figures illustrate the sensitivity of design thickness to foundation class for the various grades of asphalt. These designs are considered to be long life designs at 80 msa and their thickness has been truncated in accordance with the design principle developed in TRL Report 250 (Nunn *et al*., 1997).

5 Comparison between flexible composite and fully flexible designs

Flexible composite designs using standard cement bound materials are compared with fully flexible designs using conventional asphalt base with 35 mm of thin surfacing in Table 8. In this table, the thicknesses of the layers have been rounded up to the next 5 mm.

Table 8 can be used to determine the relative thickness of a CBM base compared with an asphalt base. For example, 165 mm of asphalt and 170 mm of CBM4G is required to carry 40 msa with a Class 3 foundation; the corresponding fully flexible design requires 265 mm of asphalt with HDM. This implies that the economic balance between the two designs is determined approximately by the cost of 170 mm of CBM4G compared with 100 mm of HDM. The economic and structural balance is distorted for superior foundation classes and stronger cement bound materials where, for practical reasons, minimum thickness requirements of layers come into play.

Flexible composite pavements constructed on strong foundations are required to have a minimum base

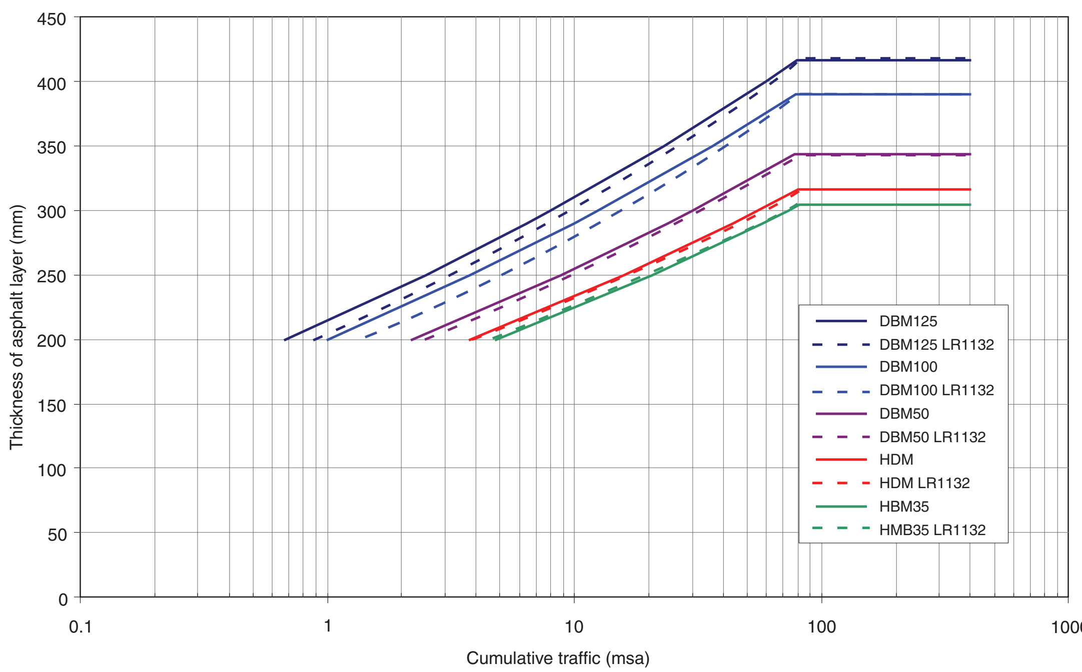

Figure 4 Comparison with LR1132 design curves on a Class 2 foundation

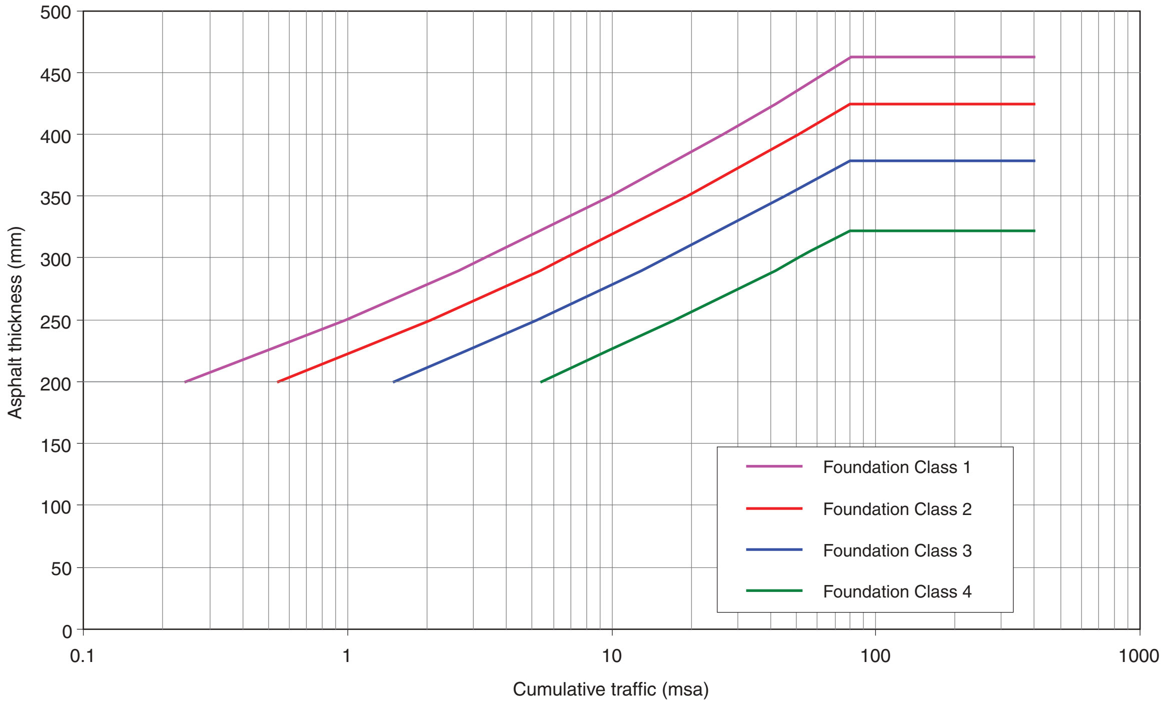

Figure 5 DBM125 design thicknesses for different foundation classes

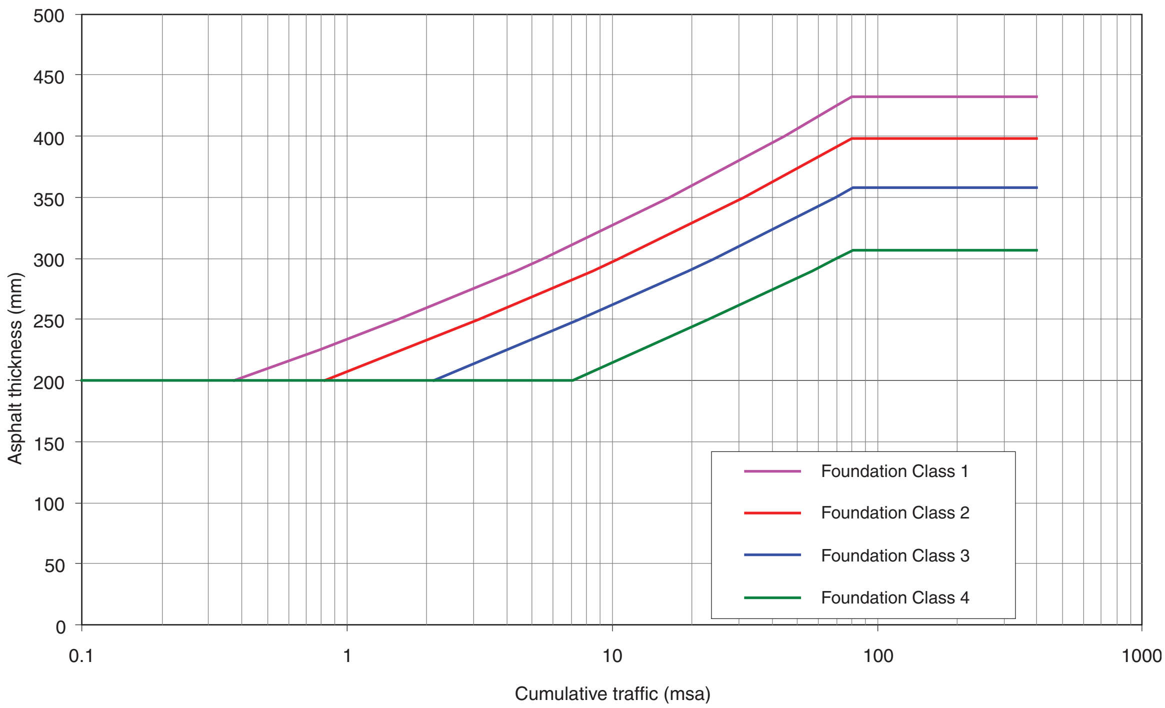

Figure 6 DBM100 design thicknesses for different foundation classes

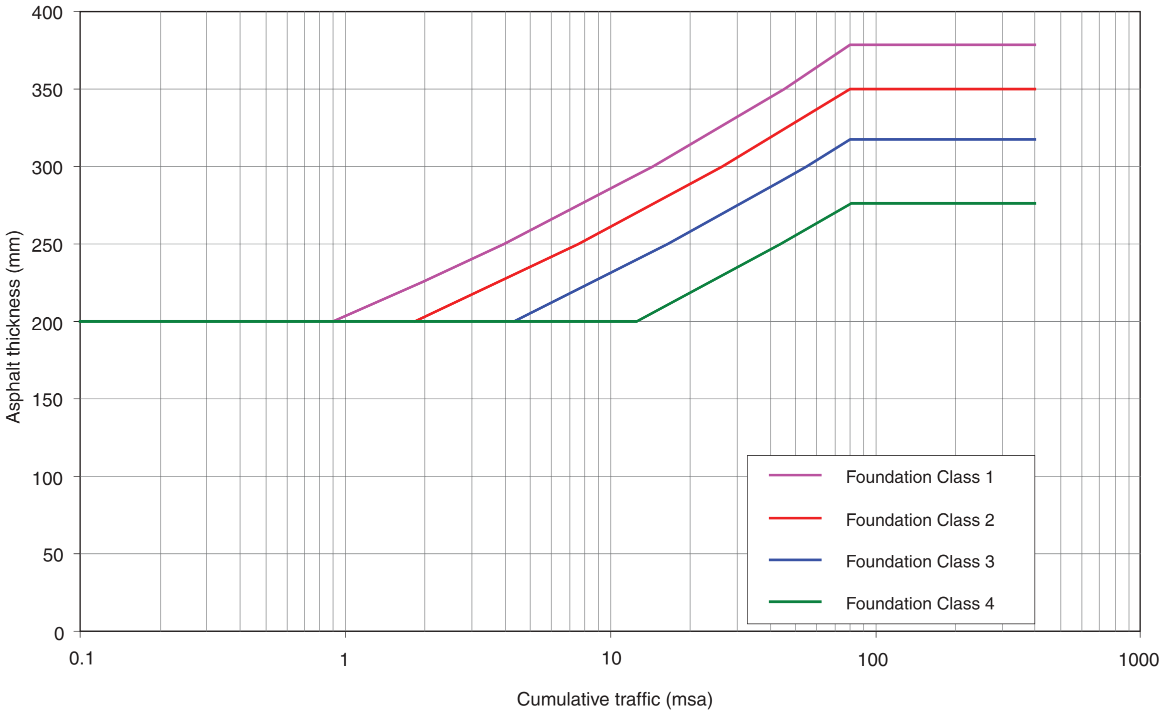

Figure 7 DBM50 design thicknesses for different foundation classes

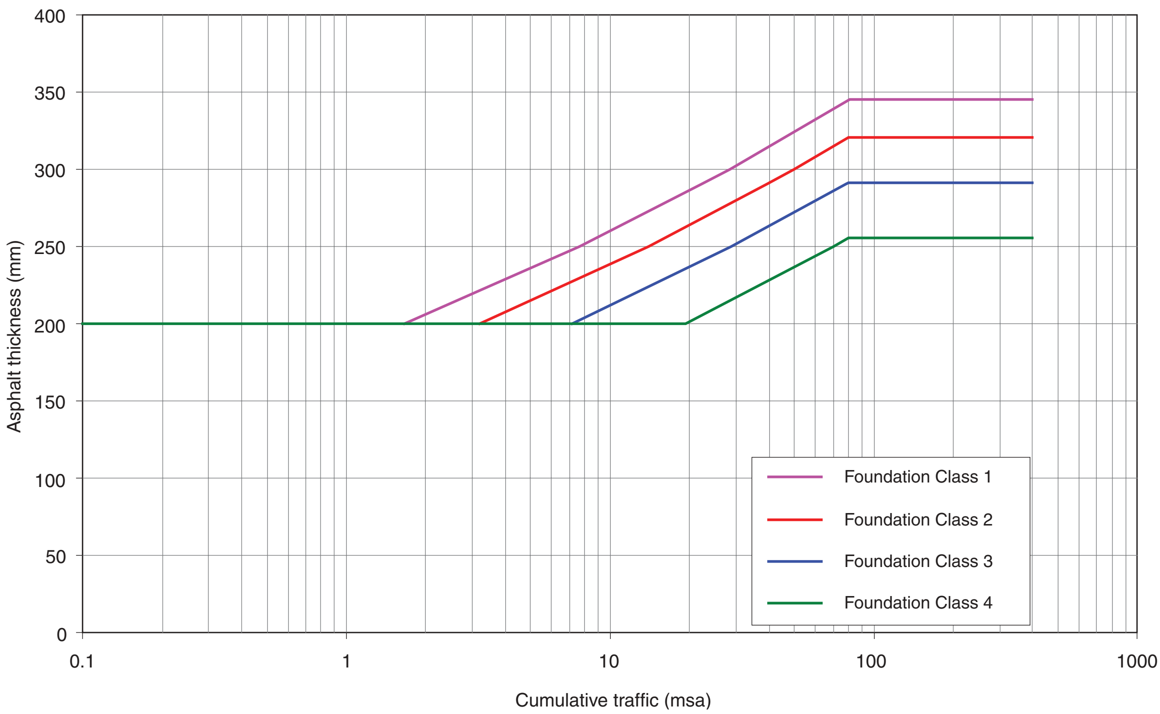

Figure 8 HDM design thicknesses for different foundation classes

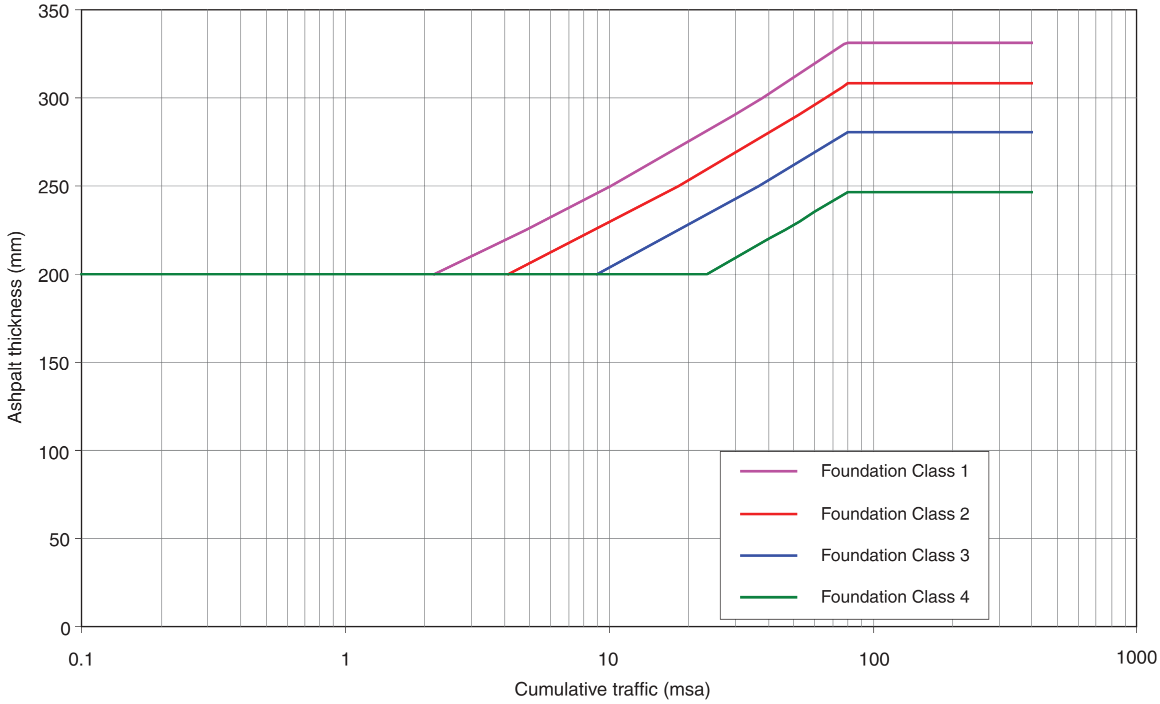

Figure 9 HMB35 design thicknesses for different foundation classes

thickness of 150 mm irrespective of traffic level and, for design traffic greater than 80 msa, 180 mm of asphalt is required. With very stiff foundations it may be possible to relax these minimum requirements. For example, the minimum thickness of asphalt was set to delay the onset of reflection cracking and to prevent these cracks propagating rapidly through the asphalt layer. However, the concern at the time was for asphalt laid on an unbound granular sub-base corresponding to Foundation Class 2. It could be argued that when a surface crack initiates, the remaining thickness of asphalt will not be so highly stressed with a stronger and more uniform foundation support and this may allow a reduction in the thickness of asphalt. For example, in France heavily trafficked flexible composite roads laid on a strong foundation require only 145 mm of asphalt (SETRA/LCPC, 1998). There is a possible case for reviewing the minimum thickness requirements for asphalt surfacing in the context stronger foundations.

Table 8 Flexible composite and fully flexible designs

Foundation Class 1 (50 MPa)
Flexible composite designs

Base	10 msa	20 msa	40 msa	80 msa
Asphalt	130	150	165	180
CBM3G	210	240	255	275
Asphalt	130	150	165	180
CBM4G	170	190	205	220
Asphalt	130	150	165	180
CBM5G	155	170	185	200
Asphalt	130	150	165	180
CBM3R	170	190	205	220
Asphalt	130	150	165	180
CBM4R	155	170	185	200
Asphalt	130	150	165	180
CBM5R	150	150	155	165

Fully flexible designs (Asphalt thickness)

Base	10 msa	20 msa	40 msa	80 msa
DBM125	350	390	425	470
DBM100	330	365	400	435
DBM50	285	315	345	380
HDM	260	290	315	345
HMB35	250	275	305	335

Foundation Class 2 (100 MPa)
Flexible composite designs

Base	10 msa	20 msa	40 msa	80 msa
Asphalt	130	150	165	180
CBM3G	195	225	240	250
Asphalt	130	150	165	180
CBM4G	155	175	190	200
Asphalt	130	150	165	180
CBM5G	150	160	170	180
Asphalt	130	150	165	180
CBM3R	155	175	190	200
Asphalt	130	150	165	180
CBM4R	150	160	170	180
Asphalt	130	150	165	180
CBM5R	150	150	150	150

Fully flexible designs (Asphalt thickness)

Base	10 msa	20 msa	40 msa	80 msa
DBM125	320	355	390	430
DBM100	300	330	365	400
DBM50	265	290	320	355
HDM	240	265	295	320
HMB35	230	255	280	310

Foundation Class 3 (200 MPa)
Flexible composite designs

Base	10 msa	20 msa	40 msa	80 msa
Asphalt	130	150	165	180
CBM3G	180	200	215	225
Asphalt	130	150	165	180
CBM4G	150	155	170	180
Asphalt	130	150	165	180
CBM5G	150	150	150	160
Asphalt	130	150	165	180
CBM3R	150	155	170	180
Asphalt	130	150	165	180
CBM4R	150	150	150	160
Asphalt	130	150	165	180
CBM5R	150	150	150	150

Fully flexible designs (Asphalt thickness)

Base	10 msa	20 msa	40 msa	80 msa
DBM125	280	315	350	385
DBM100	265	295	325	360
DBM50	235	260	285	320
HDM	215	240	265	295
HMB35	205	230	250	270

Foundation Class 4 (400 MPa)
Flexible composite designs

Base	10 msa	20 msa	40 msa	80 msa
Asphalt	130	150	165	180
CBM3G	150	175	190	200
Asphalt	130	150	165	180
CBM4G	150	150	150	150
Asphalt	130	150	165	180
CBM5G	150	150	150	150
Asphalt	130	150	165	180
CBM3R	150	150	150	150
Asphalt	130	150	165	180
CBM4R	150	150	150	150
Asphalt	130	150	165	180
CBM5R	150	150	150	150

Fully flexible designs (Asphalt thickness)

Base	10 msa	20 msa	40 msa	80 msa
DBM125	230	260	290	325
DBM100	215	245	280	310
DBM50	200	220	250	280
HDM	200	205	230	255
HMB35	200	200	220	245

6 Non-standard materials

6.1 Hydraulically bound materials

6.1.1 Determination of K_{Hyd}

The K_{Hyd} factors can be related to the structural properties of the hydraulic base material, a reasonable approximation of this relationship can be determined using the data given in Table 9.

Table 9 Adjustment factors for CBM base (360 day values)

Material	K_{Hyd}	*Dynamic modulus (GPa)*	*Flexural strength (MPa)*
CBM3G	0.329	32.9	1.7
CBM4G	0.308	38.8	2.5
CBM5G	0.272	42.9	3.3
CBM3R	0.305	34.5	2.4
CBM4R	0.244	40.4	3.6
CBM5R	0.234	44.7	4.8

A regression analysis relating dynamic elastic modulus (E in GPa) and flexural strength (f_f in MPa) to the adjustment factor (K_{Hyd}) gives the following relationship:

$$K_{Hyd} = 0.368 + 5.27 \times 10^{-5} E - 0.0351 f_f \qquad (10)$$

The quality of this prediction ($R^2 = 0.96$) is illustrated in Figure 10.

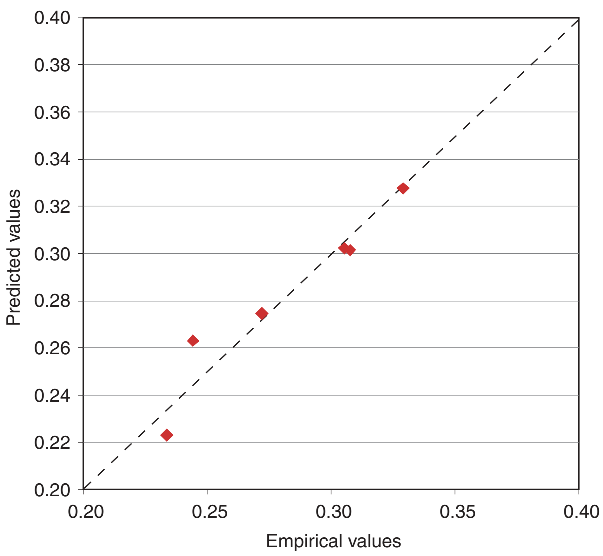

Figure 10 Comparison between empirical and predicted values for K_{Hyd}

Long term performance data from a representative sample of in-service *slow* or *quick* curing hydraulically bound pavements would be preferred to calibrate the adjustment factor K_{Hyd}. However, a representative sample is rarely available, and generally data will only be available from a few sites that are part way through their life. Therefore it is proposed to use the relationship given in Equation 10, to generate indicative values for K_{Hyd} and then use performance data available to help justify the reasonableness of this choice.

6.1.2 Classification of hydraulically bound materials

The combination of stiffness and strength is crucial for the design of a hydraulically bound base. Two different hydraulically bound materials can have the same base thicknesses for a given level of traffic, provided their flexural strengths compensate for any differences in their levels of stiffness. If stiffness is increased, the traffic induced tensile stress in the base that influences performance will also increase; therefore the strength of the base would need to be higher to achieve the same performance. Relationships between dynamic elastic modulus and flexural strength have been developed for equivalent performance and grouped into nine zones of hydraulically bound base (H1 to H9). These are shown for materials of stiffness between 5 GPa and 60 GPa in Figure 11.

These relationships were determined by carrying out design calculations for a flexible composite pavement using different combinations of stiffness and strength. The example, shown by a square symbol, illustrates that a material with a dynamic elastic modulus of 20 GPa and a flexural strength of 1.3 MPa would fall into material Zone H5.

Each zone shown in Figure 11 is bounded by combinations of modulus and strength that will produce the same design thicknesses. The structural properties corresponding to any point on this lower bound curve will result in the same pavement designs. It is suggested that the flexural strength corresponding to a dynamic modulus of 20 GPa be used as standard design values for materials lying in each Zone. This would result in the standard design inputs given in Table 10.

With this classification, CBM5R would be a Zone H9 material, CBM4R a Zone H8 material, CBM3R a Zone H7 material and CBM3G a Zone H5 material. Materials CBM4G and CBM5G may require a marginal improvement in properties to fall into Zone H8 and Zone H7 respectively. The bounding curves for the zones have been selected to give a reasonable change in design thickness. The resulting designs are given in Figures 12 to 15 for pavements constructed on Foundation Classes 1 to 4, respectively.

6.2 Bitumen bound materials

End-product, performance specifications for asphalt base layers will enable new hot mix asphalt base materials to be used provided they fulfil the requirements of the performance specification. For asphalt base, Clauses 929 and 944 (Specification for Highway Works) define the volumetric requirements of the mixture and the initial elastic stiffness modulus (load-spreading) requirements of asphalt base.

Cold mixed asphalt materials cannot be so readily accommodated into the proposed new method. These materials include foamed bitumen stabilised materials and emulsions. They may also involve the use of recycled aggregates, secondary aggregates or primary aggregates, as well as blends of bitumen and hydraulic binders.

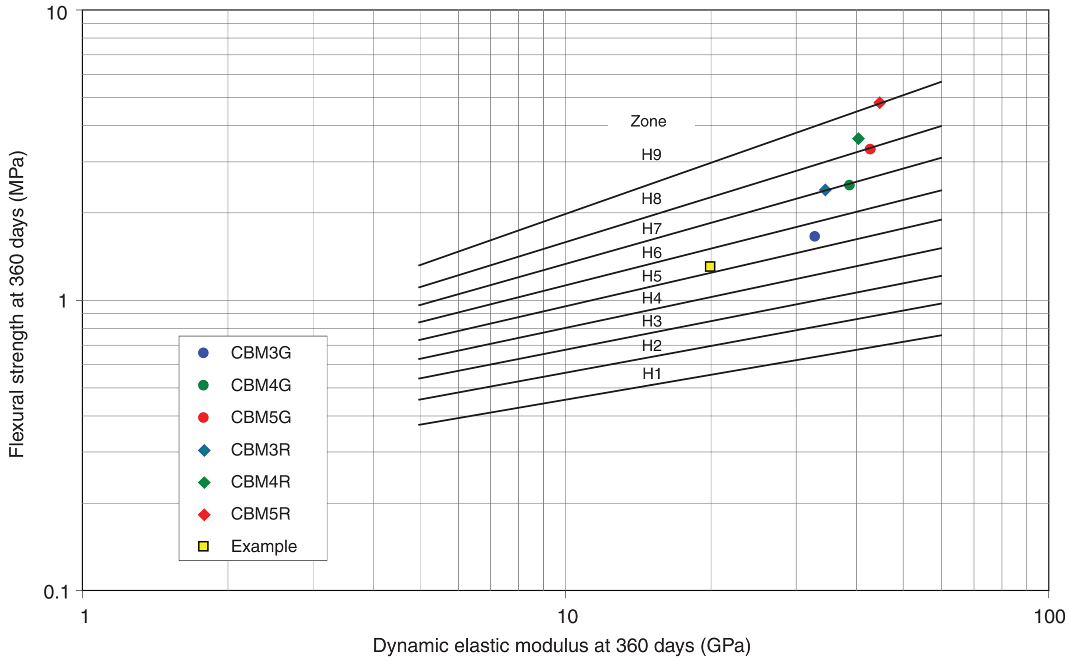

Figure 11 Relationship between stiffness and strength for equal performance

Table 10 Standard design values for materials in each zone

Zone	*Stiffness modulus (GPa)*	*Flexural strength (MPa)*
H1		0.55
H2		0.69
H3		0.85
H4		1.03
H5	20	1.25
H6		1.51
H7		1.85
H8		2.27
H9		2.96

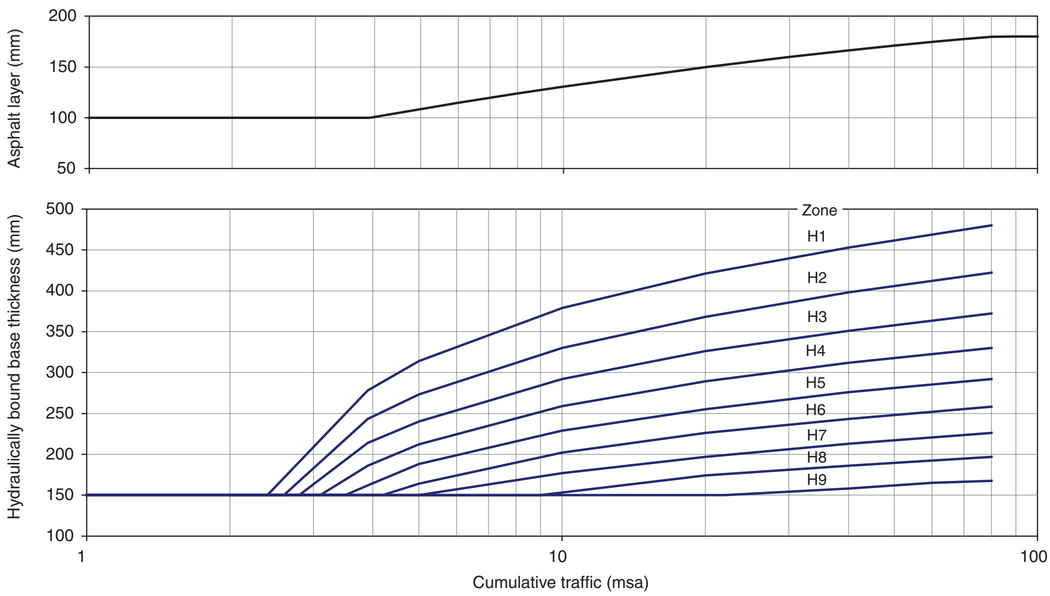

Figure 12 Flexible composite design with a Class 1 foundation

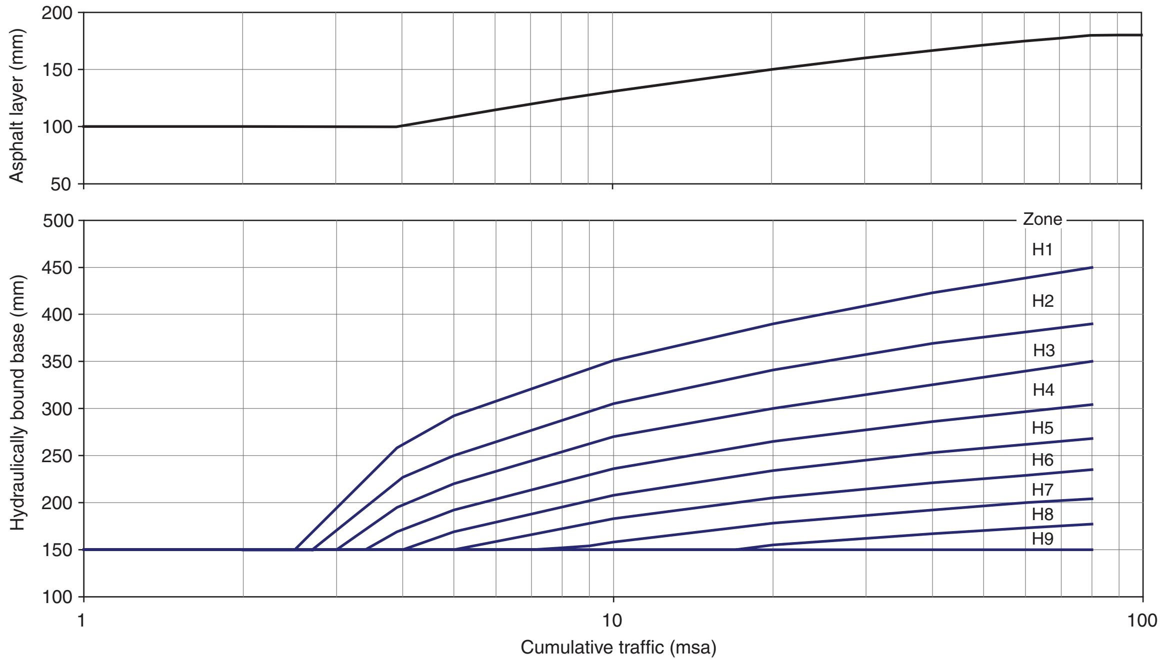

Figure 13 Flexible composite design with a Class 2 foundation

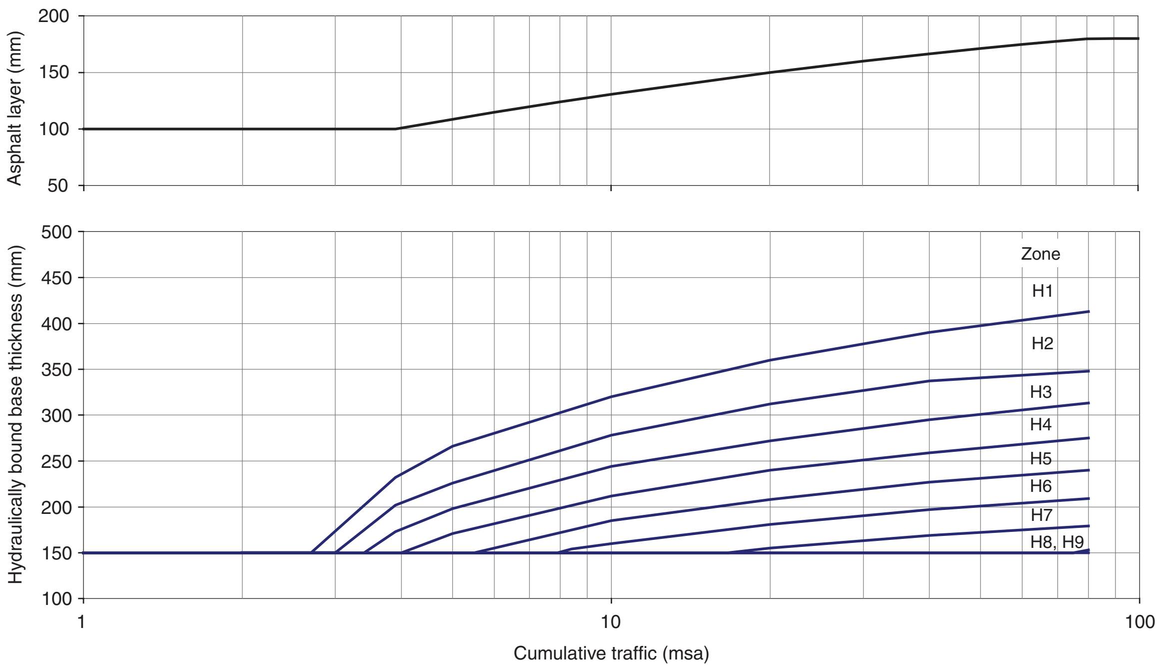

Figure 14 Flexible composite design with a Class 3 foundation

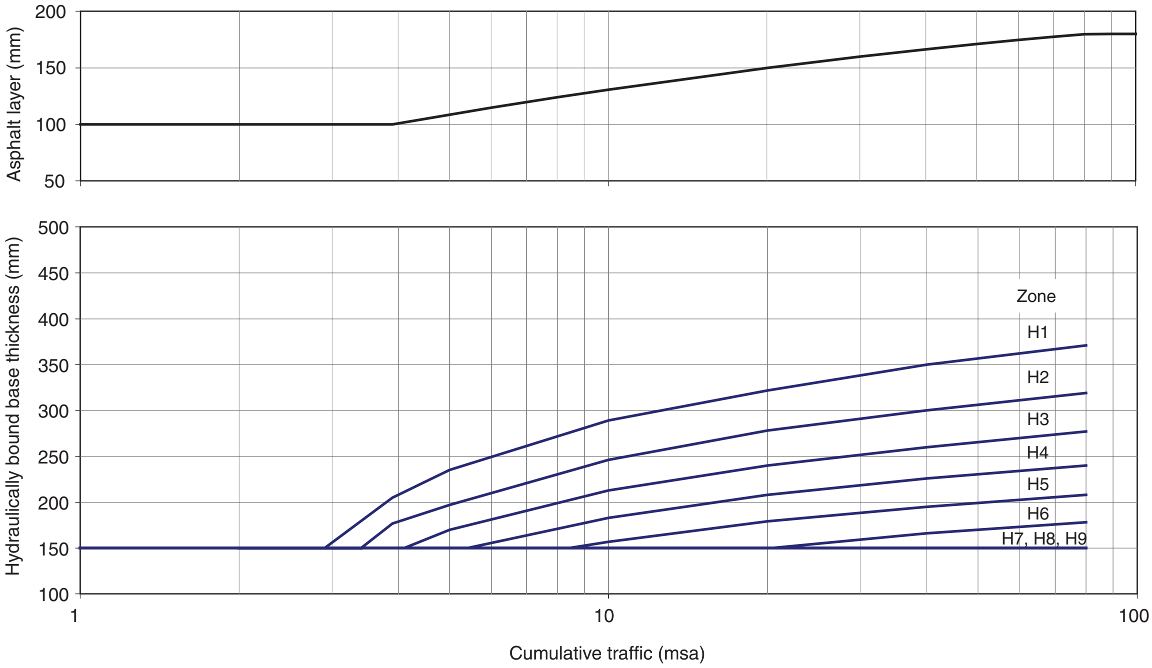

Figure 15 Flexible composite design with a Class 4 foundation

Materials such as foamed bitumen stabilised material behave in a complex manner, and it is generally produced as a hybrid material that contains both bitumen and hydraulic binder. It can be mixed *in situ* or mixed in plant off site (*ex situ*). The *ex situ* mixed material is generally more consistent than the *in situ* mixed material. Work by Milton and Earland (1999) and Nunn and Thom (2002) have demonstrated that cold recycled material using foamed bitumen stabilisation can be incorporated into moderately heavily trafficked roads. For these materials, a specification procedure has been developed that involves the submission of a quality plan that covers a pre-contract laboratory study, mixing, laying, compaction and material sampling and testing. This was developed as part of the SMART (Sustainable MAintenance of roads using cold Recycling Techniques) project (Merrill *et al.*, 2004). SMART will also cover slow curing hydraulically bound materials.

A move to 360 day properties for all roadbase materials will enable slow curing materials to be judged on a similar basis as hot mix materials using primary aggregates. It is already recognised that the initial properties of hot mix macadams, specified in Clause 944 of the SHW, and the values used for design are considerably different and that the time taken for dense bitumen macadam to reach its design stiffness is considered to be between one and two years.

7 Recommendations

The following recommendations are made for the implementation of this modified design method:

7.1 Foundation layers

The concept of foundation classes should be adopted and analytical design calculations carried out using the following equivalent half-space stiffnesses of the four foundation classes (that is, specific foundation layer properties shall not be used in design calculations):

- Class 1 ≥ 50 MPa
- Class 2 ≥ 100 MPa
- Class 3 ≥ 200 MPa
- Class 4 ≥ 400 MPa

A Poisson's ratio of 0.35 shall be used for all foundation classes.

It will be mandatory to design a foundation that falls into one of these classes (see Appendix A for further information relating to foundation design).

7.2 Hydraulically bound materials

360 day values for dynamic modulus and flexural strength shall be used for design. These values shall be the characteristic values; that is, the lower 10 percentile values.

Designs for greater than 80 msa shall use the following factors:

Material	K_{Hyd}	K_{Safety}
CBM3G	0.329	To be decided by the specifying authority (default value = 1.0)
CBM4G	0.308	
CBM5G	0.272	
CBM3R	0.305	
CBM4R	0.244	
CBM5R	0.234	

K_{Safety} can be used to develop conservative designs to ease the introduction of new materials. It can also be used for the design of roads in sensitive areas or to take into account any additional risk. For example, it could be set lower for a new material for which little performance information was available.

The design criterion for long but indeterminate life (>80 msa) shall be:

$$\sigma_r \leq \text{Flexural strength}.\,K_{Hyd}.K_{Safety} \qquad (11)$$

For this case the thickness of the asphalt surfacing shall be 180 mm, with pre-cracking at 3 m intervals unless advised otherwise.

The criterion for design traffic of less than 80 msa shall be:

$$Log(N) = 1.23 \times \left(SR.K_{Hyd}.K_{Safety} + 0.1626\right)^2 + 0.2675 \qquad (12)$$

The thickness of asphalt surfacing will be given by:

$$H_{Asphalt} = -16.05 \times \left(Log\left(N\right)\right)^2 + 101 \times Log\left(N\right) + 45.8 \qquad (13)$$

7.3 Bitumen bound materials

The design criterion shall be:

$$\varepsilon_r = K_{Flex}.K_{Safety}.\varepsilon_6 \times \left(N/10^6\right)^{-0.24} \qquad (14)$$

Where, K_{Flex} is given by:

$$K_{Flex} = 1.089 \times E^{-0.172} \qquad (15)$$

7.4 Other materials

The material classification system described in Section 6 for hydraulically bound materials is a practical method for adopting new materials. This approach has been adopted by Merrill *et al.* (2004) to develop guidance for pavement design and specification for cold recycled materials. Guidance is aimed at *end performance* and sets out design guidelines and specifications applicable to both *in situ* and *ex situ* recycling techniques. Cold recycled materials have been divided into families based upon the binder or the binder blend being used in the construction.

Although this work is targeted at cold recycled materials, the design approach is based on the versatile method and the general principles can be applied to materials irrespective of whether or not they are recycled. Structural design is given for pavements with cumulative traffic levels up to 80 msa, although it is stressed that this is an extrapolation of current knowledge of these materials and designs for heavier trafficked roads need to be applied with caution until further knowledge can be gained on their performance. Use of these materials and designs will result in more prudent use of natural resources and protection of the environment in line with Government Policy. The report by Merrill *et al.* (2004) should be consulted for further details.

7.5 Material properties

An important aspect that has to be considered is that in the UK, cement bound materials have been traditionally characterised in terms of dynamic elastic modulus and flexural strength for pavement design purposes. However, the new European Standard (British Standard EN 14227, 2004) will characterise hydraulically bound materials using one of two mutually exclusive systems:

System 1: Classification by compressive strength.

System 2: Classification by static elastic modulus and direct tensile strength.

The design advice contained in this report relates to the material designations given in the traditional UK national specifications (MCHW 1) in place prior to the publication of this report. Within the context of European specifications, the traditional designations are no longer directly applicable and different ways of describing material have been used. At the time of the publication of this report, the European specifications for hydraulically bound materials have just been published; therefore the use of the traditional materials described in this report should be related to material covered by the European specification. The implications of harmonising with the material characteristics classification approach in the new European Standard are examined in Appendix E.

Also, there is only limited data available on 360 day values of dynamic modulus and flexural strength of hydraulically bound materials and even less on static modulus and direct tensile strength. It is therefore recommended that a laboratory study be carried out to develop more robust material characterisation values and indicative factors to link 7, 28, 60 or 90 day properties to 360 day values.

8 Conclusions

A more versatile approach to design has been developed to enable a wider range of materials to be incorporated more easily into the road construction. The main features of this study are that:

1 A simpler calculation procedure has been developed for flexible composite designs that does not involve calculating thermal stresses.

2 The current analytical design method for fully flexible pavements has been modified and changes have been made to the flexible composite design method to make both methods more compatible.

3 Foundation classes have been defined and the revised design methods allow for reductions in the surfacing layers if superior foundations are used.

4 The methods use 360 day material properties.

5 A wider range of hydraulic materials can be included, and adjustment factors (K_{Hyd} and K_{Safety}) can be determined that take into account both material characteristics and risk.

6 A revised design criterion has been defined for fully flexible pavements which limits the amount of flexure in the asphalt base.

7 Adoption of the adjustment factors recommended in this report will give pavement designs that are in close agreement with the current designs given in the Design Manual for Roads and Bridges for traditional materials.

8 Initially this more versatile approach could be adopted using dynamic elastic modulus and flexural strength to characterise hydraulically bound materials. In the longer term, static elastic modulus and direct tensile strength could be introduced to be compatible with new European Standards.

9 A laboratory study is required to develop more robust knowledge on the curing behaviour of hydraulically bound materials and to enable indicative factors to be developed that link early life material properties to 360 day design values.

9 Acknowledgements

The work described in this paper was carried out in the Infrastructure Division of TRL (Director: Mike Head), as part of the Highways Agency programme of research, with Project Partners Scott Wilson Pavement Engineering and Sitebatch Technologies Ltd. Dr Darren Merrill was the Quality Audit and Review Officer.

The project was led by a steering committee with the following membership:

Robert Armitage	Scott Wilson Pavement Engineering
Ian Carswell	TRL
John Kennedy	Independent Consultant
Wyn Lloyd	Highways Agency
Mike Nunn	TRL
Nick Thom	Scott Wilson Pavement Engineering
John Williams	Highways Agency
David York	Sitebatch Technologies Ltd.

10 References

Asphalt Institute (1991). *Thickness design. Asphalt pavements for highways and streets*. MS-1.

British Standards Institution (2004). *Unbound and hydraulically bound mixtures- Specifications*. BS EN 14227. London: British Standards Institution.

Brown S F, Brunton J M and Stock A F (1985). *The analytical design of bituminous pavements*. Proceedings of the Institute of Civil Engineers.

COST Action 334 (1998). Effects of wide single tyres and dual tyres. Retrieved: July 2004, from http://www.minvenw.nl/rws/dww/home/cost334tyres. European Commission, Directorate General Transport. Brussels

Croney D (1977). *The design and performance of road pavements*. London: The Stationery Office.

Department of Transport, Scottish Office Industry Department, the Welsh Office and the Department of the Environment for Northern Ireland (1994). Design Manual for Roads and Bridges. Volume 7 Pavement Design and Maintenance. London: The Stationery Office.

HD 25/94, Section 2: Part 2 – Foundations.

HD 26/01, Section 2: Part 3 – Pavement design

Highways Agency, Scottish Office Development Department, the Welsh Office and the Department of the Environment for Northern Ireland (2001). *Manual of Contract Documents for Highway Works. Volume 1, Specification for Highway Works.* London: The Stationery Office.

Kolias S and Williams R I T (1978). *Cement bound road materials: Strength and elastic properties measured in the laboratory*. Supplementary Report SR344. Crowthorne: TRL Limited.

Leech D and Nunn M E (1990). *Asphalt substitution: road trials and cost considerations*. Research Report RR303. Crowthorne: TRL Limited.

Milton L and Earland M (1999). *Design guide and specification for structural maintenance of highway pavements by cold in-situ recycling*. TRL Report TRL386. Crowthorne: TRL Limited.

Merrill D, Nunn M E and Carswell I (2004). A guide to the use and specification of cold recycled materials for the maintenance of road pavements. TRL Report TRL611. Crowthorne: TRL Limited.

Nunn M E, Brown A, Weston D and Nicholls J C (1997). *Design of long life pavements for heavy traffic*. TRL Report TRL250. Crowthorne: TRL Limited.

Nunn M E and Merrill D (1997). *Review of flexible and composite pavement design methods*. Papers and Articles PA3298. Crowthorne: TRL Limited.

Nunn M E and Thom N (2002). *Foamix: Road trials and design considerations*. Viridis Report VR1. Crowthorne: TRL Limited.

Powell W D, Potter J F, Mayhew H C and Nunn M E (1984). *Design of bituminous roads*. Laboratory Report LR1132. Crowthorne: TRL Limited.

Strickland D (2000). *Shell pavement design software for Windows, 2000.* Shell International Petroleum Company Limited.

SETRA-LCPC (1998). *Reseau Routier National: Catalogue des structures types de chausses neuves (National road network: Catalogue of structures for new roads)*. Paris.

Appendix A: Examples of designs for foundation classes

The four proposed foundation classes for flexible and flexible composite pavements can be specified as follows:

- Foundation Class 1 is a capping only design that is permissible for the construction of the base of the pavement provided the capping material has adequate shear strength. The application of this class for high traffic roads may need to be limited.
- For all traffic categories, Foundation Class 2 is a sub-base only or sub-base on capping design that is considered as equivalent to the current standard unbound granular foundation (HD 25/94).
- Foundation Classes 3 and 4 are designs incorporating hydraulically bound materials that provide a range of foundations of superior quality to current standard unbound granular foundations. These classes could permit thinner overlying pavements than those currently in the Design Manual for Roads and Bridges (HD 26/01).

The Foundation Classes 1, 2, 3 and 4 increase in quality with increasing class number and are required to have a long-term minimum design stiffness of 50, 100, 200 and 400 MPa respectively. For Foundation Class 1, great care will be necessary to avoid damage under construction traffic and to ensure adequate compaction of the pavement layers above. Examples of the classification of foundations comprised of various standard sub-base materials are shown in Figure A1. The foundations shown in these examples are designed for a subgrade strength of 5% CBR using the thickness designs in HD 25/94 (DMRB). Designs

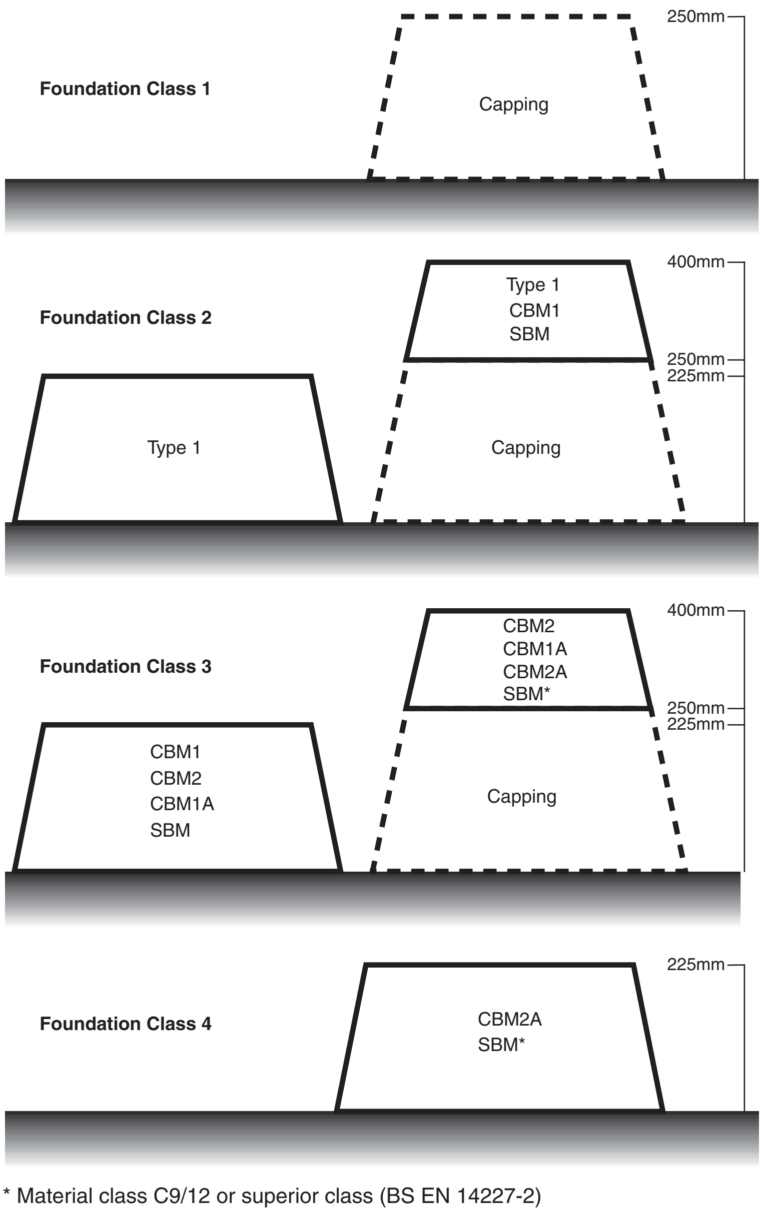

Figure A1 Example designs for foundation classes 1 to 4 on 5% CBR subgrade
This example uses thicknesses from HD 25, other options are possible

for other subgrade strengths can be obtained using the thickness designs in HD 25 whilst maintaining the classification of sub-base materials shown in Figure A1.

The classification examples are neither comprehensive nor definitive and many other options are possible. The examples show mainly well-used materials but other approved materials can be assessed in an identical fashion to those shown in Figure A1. The examples presented are simply an illustration of how certain materials fit into the foundation class system using the following methodology and generally minimum strength values have been assumed. It is possible that the material assignments and the thickness designs may be revised as experience is gathered with the foundation classification system or where specific knowledge of the long-term behaviour of certain materials is collated. It is intended that the classification process will be refined and improved as experience is gained.

Method of classifying foundations

The following steps have been followed in order to classify the foundations in Figure A1:

1 The nature of the sub-base and its structural properties at one year after construction are assessed.

2 These material properties are used to define a value for the sub-base layer stiffness, where, for hydraulically bound materials, the degrading effects of naturally occurring, or induced, cracks are included. The degree of degradation is assumed to reduce as the strength of the sub-base increases. Capping, if used, is considered as unbound and its layer stiffness and the stiffness of the underlying subgrade soil are based on the analysis in LR1132 (Powell *et al*. 1984).

3 The elastic response of the complete foundation built on a subgrade of strength CBR 5% is simulated with a multi-layer elastic model using the previously derived value of sub-base layer stiffness and values of the sub-base and, if used, capping thickness given in DMRB (HD25).

4 The analysis is repeated for a range of subgrade strengths and the minimum derived foundation stiffness is then used to classify all foundations built with this sub-base.

A.1 References

British Standards Institution (2004). *Unbound and hydraulically bound mixtures- Specifications – Part 2: Slag bound mixtures – Definitions, composition, classification.* BS EN 14227-2. London: British Standards Institution.

Department of Transport, Scottish Office Industry Department, the Welsh Office and the Department of the Environment for Northern Ireland (1994). *Design Manual for Roads and Bridges. Volume 7, Pavement Design and Maintenance*. London: The Stationery Office.

HD 25/94, Section 2: Part 2 – Foundations.

HD 26/01, Section 2: Part 3 – Pavement design.

Powell W D, Potter J F, Hyden H C and Nunn M E (1984). *Design of bituminous roads,* Laboratory Report LR1132. Crowthorne: TRL Limited.

Appendix B: Development of composite design curves for traffic less than 80 msa

Parry (1997) reviewed the performance of flexible composite pavements in the UK that had carried up to 100 msa and proposed a new design curve for determinate life pavements of less than 60 msa. He also pointed out that the evidence suggested that there was scope for relaxing the current requirement for 200 mm of asphalt for indeterminate life.

A multiple regression analysis of pavement life to the first significant maintenance was carried out using six pavement variables. This produced to following best fit equation:

$$Log(N) = 5.0 \times (\text{CBM thickness}) + 5.8 \times (\text{asphalt thickness}) + 0.65 \qquad \text{(B1)}$$

It was also noted that most of the pavements that had been strengthened could have continued to carry traffic in their former condition.

Based on this approach Parry (1997) suggested the design given in Table B1.

The observation that many of the pavements in this study did not appear to require the strengthening overlay introduced a degree of conservatism into the analysis. More recently, pre-cracking has become standard and this allows the thickness of the asphalt layer to be reduced by 20 mm. If this is taken into account, the designs in Figure 1 of this report are comparable to those suggested by Parry (1997).

B.1 References

Parry A (1997). *Recommendations for the design of flexible composite pavements*. Project Report PR/CE/27/97. Crowthorne: TRL Limited. *(Unpublished report available on direct personal application only).*

Table B1 Design thickness

	LR1132 Design			*Proposed by Parry et al. (1997)*			*Proposed in this report (see Figure 1)*		
N (msa)	*Asphalt*	*CBM*	*Total*	*Asphalt*	*CBM*	*Total*	*Asphalt*	*CBM*	*Total*
	Layer thickness (mm)								
10	130	200	330	110	200	310	130	195	325
20	150	250	400	140	220	360	150	225	375
30	200	250	450	170	230	400	160	235	395
40	200	250	450	180	240	420	165	240	405
50	200	250	450	190	250	440	170	245	415
60	200	250	450	200	250	450	175	250	425
80	200	250	450	200	250	450	180	250	430

Appendix C: Procedure for designing flexible composite pavements

1 Establish the 360 day properties for dynamic elastic modulus and flexural strength.

2 Determine which category the hydraulically bound material falls into by using Figure 11.

3 Determine the standardised design parameters from Table 10.

4 Use either these design values to determine the thicknesses of pavement layers by following the procedure given in Figure C1, or use the design charts given in Figures 12 to 15; the value of K_{Hyd}, given by Equation C1, was used to produce the design charts. For a new material, information to support this value is required.

5 If the procedure given in Figure C1 is used, it will be necessary to determine the value of K_{Hyd}. This can be determined empirically by back-analysing a representative set of designs of known performance. Alternatively, if there is insufficient data, a provisional value can be determined using the following relationship, and experience and data that does exist used to support this value:

$$K_{Hyd} = 0.368 + 5.27 \times 10^{-5} E - 0.0351 f_f \qquad \text{(C1)}$$

Client co-operation would be required if the material was a radical departure from current or traditional practice. In these cases it may be possible to introduce the material on a trial basis with construction details and material properties being collected and future performance monitored to provide feedback on the material.

The general principles that are proposed for cold recycled materials could be applied to materials irrespective of whether or not they are recycled; the report by Merrill *et al.* (2004) should be consulted for details.

C.1 References

Merrill D, Nunn M and Carswell I (2004). *A guide to the use and specification of cold recycled materials for the maintenance of road pavements*. TRL Report TRL611. Crowthorne: TRL Limited.

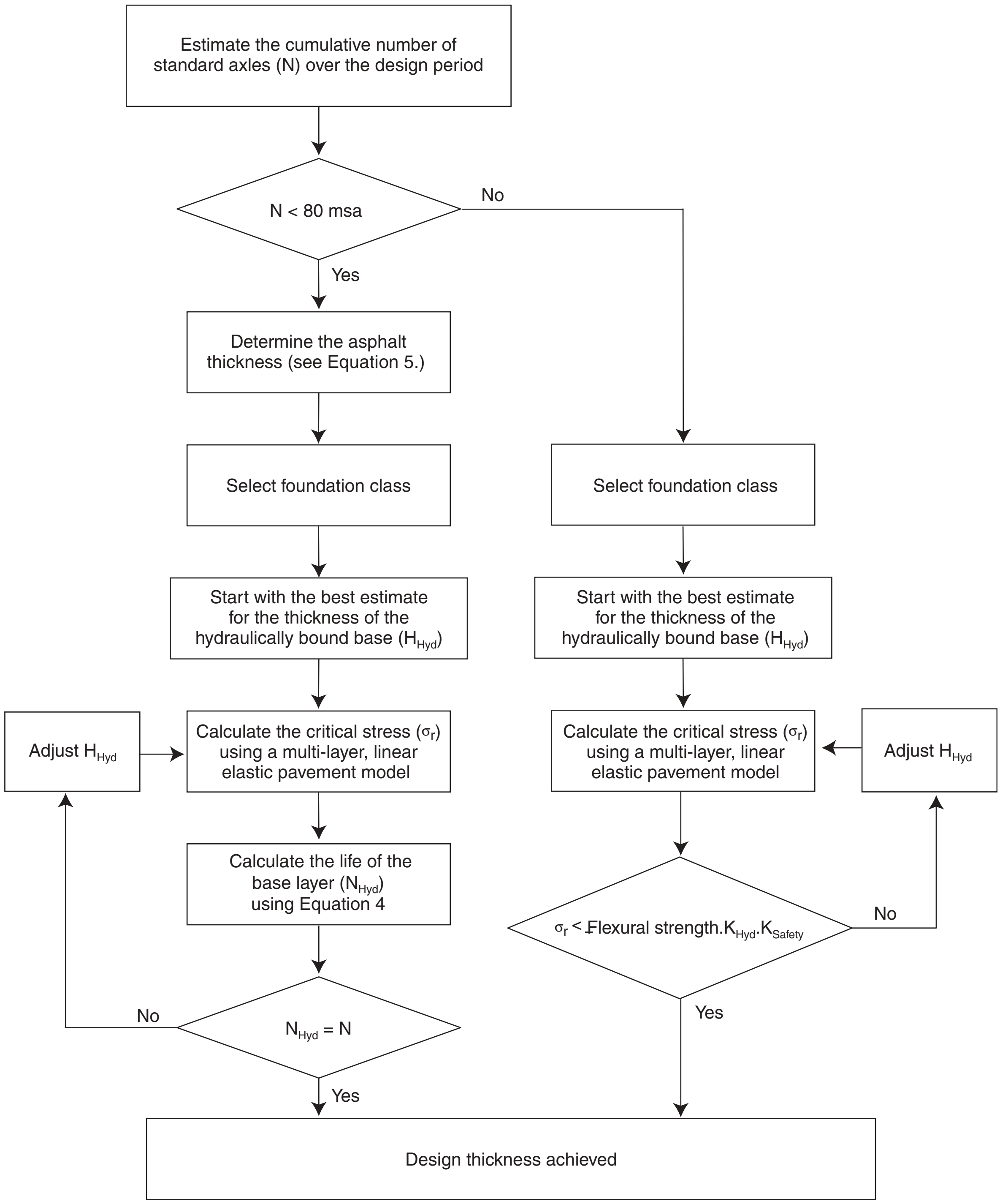

Figure C1 Design flow chart

Appendix D: Asphalt fatigue considerations

The base layer is the most important structural layer of the road. Unlike the wearing course where cracks are easy to observe, assessment of structural condition of the base is difficult because of its position in the road; consequently the mechanisms of base 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 base (Nunn and Merrill, 1997). These methods consider fatigue cracking, caused by repeated traffic loading, to be the major form of structural deterioration. Investigation of the fatigue mechanism in full-scale pavements is much more difficult than in the laboratory and Nunn *et al.* (1997) and Goddard and Powell (1987) noted that, although surface cracking is often observed, there is little evidence of fatigue cracking in the base of inservice asphalt pavements in the UK. Furthermore, it is known that the stiffness of asphalt base 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 base performance.

The fatigue law can be generally represented by:

$$\varepsilon_r = \varepsilon_6 \times \left(N/10^6\right)^n \qquad \text{(D1)}$$

where, ε_r is the strain induced at the underside of the asphalt base to achieve a fatigue life of N million standard axles (msa), ε_6 is the strain for a fatigue life of 1 msa and n is the fatigue exponent.

The fatigue criterion in LR1132 is expressed as:

$$Log\left(N\right) = -9.38 - 4.16 Log\left(\varepsilon_r\right) \qquad \text{(D2)}$$

which can also be written as:

$$N = 4.169 \times 10^{-10} . \varepsilon_r^{-4.16} \qquad \text{(D3)}$$

Therefore:

$$\varepsilon_r = \left(2.4 \times 10^9 \times N\right)^{-0.2404} \qquad \text{(D4)}$$

and, the tensile strain for a life of 1 msa will be given by:

$$\varepsilon_6 = \left(2.4 \times 10^{15}\right)^{-.2404} = 0.000201 \text{ or } 201 \text{ micro-strain} \qquad \text{(D5)}$$

In the new method, the strain criterion is given by:

$$\varepsilon_r = K_{Flex} . K_{Safety} 201 \times 10^{-6} . \left(N/10^6\right)^{-0.2404} \qquad \text{(D6)}$$

Substituting for ε_6 gives:

$$N/10^6 = \left(\varepsilon_r / \left(K_{Flex} . K_{Safety} . 201 \times 10^{-6}\right)\right)^{-4.16} \qquad \text{(D7)}$$

The fatigue criterion given in Equation D4 and used in LR1132 (Powell *et al.*, 1984) was based on the fatigue characteristics of DBM containing 100 penetration grade bitumen measured using the TRL uniaxial fatigue test (Goddard *et al.,* 1978). Subsequent investigations indicated that the volume of binder was the most crucial material parameter that influenced the fatigue life and that the fatigue life of macadams containing a similar volume of a harder binder was not significantly different (Nunn *et al.*, 1986, Nunn and Smith, 1994); therefore the same fatigue characterisation was adopted for DBM50, HDM and HMB35.

In LR1132 it was recognised that predictions of fatigue life based on laboratory tests grossly underestimated the observed life in the pavement but the functional form of the best fit curve of the data from full-scale pavements (see Figure D1 of LR1132) was in close agreement with that derived from laboratory fatigue tests reported by Goddard *et al.* (1978); large shift factors are required to bring the two curves into coincidence. The use of a fatigue criterion calibrated in this manner ensures that the risk of fatigue cracking in newly constructed roads will be no greater than that of roads constructed in the past. What it does not necessarily do is quantify this risk; it may well be that the risk is minimal.

The most conservative approach would be to ensure that the base material had fatigue characteristics that were as good, or even better than conventional macadam base. However, if this were adopted universally it would preclude the use of materials like foamed bitumen bound base material. This material has generally a low laboratory fatigue life and yet experience has shown that fatigue of the base layer has not emerged as an issue for its performance. In France, a pure foamed bitumen material without the addition of a hydraulic binder is assumed to be self-healing (Goacolou, 1999).

In Foamix trials reported by Nunn and Thom (2002) cores could be extracted reliably, provided that the air flushed coring was progressed slowly. If the coring operation was hurried, it was observed that the material broke up along defined planes, producing fragments that resembled pieces from a 3-dimensional jigsaw puzzle. This suggested that the material has multiple weak fissures and these may relieve traffic-induced strains in the Foamix and prevent further fragmentation of the material. A mechanism similar to this may account for the widespread experience that this material does not suffer from conventional fatigue problems (Maccarrone *et al.,* 1994) under confined conditions in the pavement. The use of secondary hydraulic binders improves the coherence of the material, however the balance between and the role of the two forms of binder requires further study to gain a good understanding the overall behaviour of Foamix.

Experience with Foamix demonstrates that practical knowledge of how different base materials behave in-service is necessary to ensure that material is not judged unfairly in terms of fatigue. The inherent assumptions in using laboratory fatigue data to predict in-service pavement performance is a complex issue that requires more discussion. Thrower (1979) pointed out that the simple laboratory conditions of loading, stress distribution and environment are very different from those occurring in real pavements and Nunn *et al.* (1997) noted that large changes that occur in material properties, especially stiffness, during the life of the road has received little attention in the past in relation to base performance.

Bearing in mind the above reservations, it is possible to generalise the design criterion to take into account the fatigue performance of different asphalt base materials. Many different forms of fatigue test exist (for example, trapezoidal bending, uniaxial push-pull, indirect tensile, 4-point bending, etc). Furthermore, tests can be carried out in controlled strain, controlled stress or controlled load mode, and results from one test cannot be easily be replicated by another. The fatigue criterion in LR1132 was determined using data from the TRL fatigue test. This test is not widely available and results using other laboratory fatigue tests will differ from those of the TRL fatigue test. If results from another fatigue test where to be used in the design method, it would be best to treat the results in a relative manner.

The proposed criterion relates to a DBM base material measured using the TRL fatigue push-pull test (fatigue test A). This test applied a continuous, sinusoidal tension/compression stress loading under controlled, uniaxial stress conditions (constant stress amplitude) at a frequency of 25 Hz and at a temperature of 20°C ± 0.5°C.

If a new material is being considered and its fatigue life measured using another fatigue test (fatigue test B), it would be best to consider fatigue life relative to that of a DBM base material to minimise error. If data from fatigue test B is normalised to that of the fatigue characteristics of DBM base using the same test, then Equation D7 could be modified as follows:

$$N/10^6 = \left(\varepsilon_r / \left(K_{Flex}.K_{Safety}.b.\varepsilon'_{6(New)}\right)\right)^n \qquad \text{(D8)}$$

Where:

$b = (201 \times 10^{-6})/\varepsilon'_{6(DBM)};$

$\varepsilon'_{6(DBM)}$ is the strain for the life of 1 million cycles of standard DBM measured in fatigue test B;

$\varepsilon'_{r(New)}$ is the strain for the life of 1 million cycles of the new base material measured in fatigue test B;

n is the exponent of the fatigue characteristic determined in fatigue test B, and is obtained from the fatigue characterisation equation, that is:

$N = K.\varepsilon_{r(New)}^{\;n}$ (where, K and n are constants).

This assumes that fatigue test B produces the same fatigue exponent (n) for conventional DBM as that in the TRL test A (that is, 4.16). The pavement design curve for DBM should be the same irrespective of the fatigue test used.

It should also be appreciated that fatigue test results are very variable and testing several nominally identical mixtures may be necessary to determine the representative fatigue life of say, DBM.

D.1 References

Goacolou H (1999). *Innovative pavement structure for high-traffic roads and motorways*. Revue General Routiers at Aerodromes No. 754, September 1997.

Goddard R, Powell D and Applegate M (1978). *Fatigue resistance of dense bitumen: the effect of mixture variables and temperature*. Supplementary Report SR410. Crowthorne: TRL Limited.

Goddard R T N and Powell W D (1987). Assessing the condition of bituminous roads. *Journal of the Institution of Highways and Transportation (London)*. No.5, Vol 34.

Maccarrone S, Holleran G, Leonard D J and Hey S (1994). *Pavement recycling using foamed bitumen*. 17th ARRB Conference. Proceedings, Volume 17 Part 3 Queensland, Australia.

Nunn M E and Smith T (1997). *Road trials of high modulus base (HMB)*. TRL Report TRL231. Crowthorne: TRL Limited.

Nunn M E and Thom N (2002). *Foamix: Road trials and design considerations*. Viridis Report VR1. Crowthorne: TRL Limited.

Nunn M E, Brown A, Weston D and Nicholls J C (1997). *Design of long life pavements for heavy traffic*. TRL Report TRL250. Crowthorne: TRL Limited.

Nunn M E and Merrill D (1997). *Review of flexible and composite pavement design methods.* Papers and Articles PA3298. Crowthorne: TRL Limited.

Nunn M E, Rant C and Schoepe B (1986). *Improved roadbase macadams: road trials and design considerations*. Research Report RR132. Crowthorne: TRL Limited.

Powell W D, Potter J F, Mayhew H C and Nunn M E (1984). *Design of bituminous roads.* Laboratory Report LR1132. Crowthorne: TRL Limited.

Thrower E N (1979). *A parametric study of a fatigue prediction model for bituminous road pavements*. Laboratory Report LR892. Crowthorne: TRL Limited.

Appendix E: Material characterisation with the introduction of the new European Standards

E.1 Introduction

The European Standard for hydraullically bound material (EN 14227: 2004) gives two mutually exclusive classification systems for hydraulically bound mixtures:

System 1: Classification by compressive strength.

System 2: Classification by tensile strength and modulus of elasticity.

System 2 uses static elastic modulus and direct tensile strength to characterise hydraulically bound materials and this characterisation will eventually be adopted in the UK, whereas the UK has traditionally characterised cement bound materials in terms of dynamic elastic modulus and flexural strength for the purposes of pavement design. The dynamic elastic modulus is measured at a loading frequency of several kHz and at low stress amplitude. This differs from the conditions induced by a rolling wheel load, which has an effective loading frequency of a few Hz and generally induces a much higher stress.

In this Appendix, the implications of harmonising with the material characteristics used to classify materials in the European Standard are examined. System 1 is closer to the traditional method of characterising cement bound materials in the UK than System 2.

System 2 requires knowledge of the static modulus and direct tensile strength for which there are few measurements available for the standard cement bound materials used in the UK. Furthermore, there is even less information on other hydraulically bound materials. It is also recognised that the relationship between these parameters and those that have been used traditionally in the UK is material specific. A programme of material testing would be required to establish authoritative relationships. Uncertainty is compounded by the fact that many of the values available were determined after 7, 28, 60 or 90 days of curing rather than after 360 days as required for design purposes. Therefore the calculations and values contained in this Appendix should be regarded with caution and the purpose of this Appendix is to explore the design implications of moving to the European characterisation.

E.2 System 1 classification

The design advice contained in this report relates to the material designations given in the traditional UK national specifications (MCHW 1) in place prior to the publication of this report. Within the context of European specifications, the traditional designations are no longer directly applicable and different ways of describing material have been used. At the time of the publication of this report, the European specifications for hydraulically bound materials have just been published; therefore the use of the traditional materials described in this report should be related to material covered within the European specification.

In this report, designs for cement bound material bases (CBM grades : CBM3, CBM4 and CBM5) have been presented. The use of these types of material will, in future, be covered by BS EN 14227-1 and will be referred to as Cement Bound Granular Material (CBGM).

Traditionally, CBM3, CBM4 and CBM5 materials have been specified in the Specification for Highways Works (MCHW 1) according to two grading envelopes based on either a 40 mm or 20 mm maximum grain size. BS EN 14227-1 no longer permits the use of the 40 mm grading envelope. The existing envelope for a CBM with maximum grain size of 20 mm is most closely related to Category G1 of a 0/20 mixture according to BS EN 14227-1. The grading envelope for CBGM comprises more fine material than has been traditionally used for CBM.

Compressive strength remains an option for the classification of CBGM utilising a characteristic strength according to System 1. However, 28 day strength is used rather than 7 day strength which has been traditionally used in the UK. According to Section 3.1 of this report '*Material properties at 360 days*' a factor of 1.2 can be used to relate 7 day and 28 day strength requirements. The relationship between the CBM grades and the CBGM grades are given in Table E1.

Table E1 Strength classification

Traditional material class	*7 day average compressive strength (MPa)*	*28 day average compressive strength (MPa)*	*CBGM strength class*
CBM3	10.0	12.0	$C_{5/6}$
CBM4	15.0	18.0	$C_{8/10}$
CBM5	20.0	24.0	$C_{12/15}$

The CBGM material classes permit material of lower strength than has traditionally been used; for example a $C_{5/6}$ class permits an average 28 day strength of 9 MPa whereas as a strength of 12 MPa would be expected for a CBM3. In order to ensure that the designs provided in this report are sufficient, the CBGM classifications should be promoted one class. The European specifications make no distinction between the thermal properties of the aggregates used in CBGM, however these remain an important part of the design process. Therefore, for a given strength class, the type of aggregate used should still be declared for design as shown in Table E2. It should be remembered that the designations given in Tables E1 and E2 are non-commutative since binders other than Portland cement are permitted for use in CBGM.

Table E2 Design classifications

Traditional material class	*CBGM strength class*
CBM3G	$C_{8/10}$(G)
CBM3R	$C_{8/10}$ (R)
CBM4G	$C_{12/15}$(G)
CBM4R	$C_{12/15}$(R)
CBM5G	$C_{16/20}$(G)
CBM5R	$C_{16/20}$(R)

E.3 System 2 classification

Existing data was reviewed to obtain provisional relationships between the dynamic and static elastic modulus and the flexural and direct tensile strength. A test programme would need to be devised to establish more robust static modulus and direct tensile strength values for cement bound and other hydraulically bound materials.

Elastic modulus

The static elastic modulus is defined as the secant modulus at 30% of the load to failure whereas the dynamic elastic modulus is measured at the resonant frequency of a prismatic specimen (BS 1881, 1990). This frequency is typically between 2 and 5 kHz. During the vibration, negligible stress is applied. Therefore, the dynamic modulus refers to an almost purely elastic effect and is unaffected by creep. For this reason the dynamic modulus is approximately equal to the initial tangent modulus determined in the static test and it is higher than the static (secant) modulus.

The following relationship between static (E_s) and dynamic modulus (E_d) was derived from the measurements on cement bound materials reported by Kolias and Williams, 1978 (Figure E1).

$$E_s = 1.08E_d - 9.07 \quad \left(R^2 = 0.97\right) \tag{E1}$$

The relationship between static and dynamic modulus is material dependent and at low modulus values the difference between the two measurements becomes greater. For example, Figure E2 shows the relationship between compressive strength and dynamic and static modulus of slag bound material (SBM) determined by Megan and Earland (1999). This illustrates that the two curves diverge for lower modulus values.

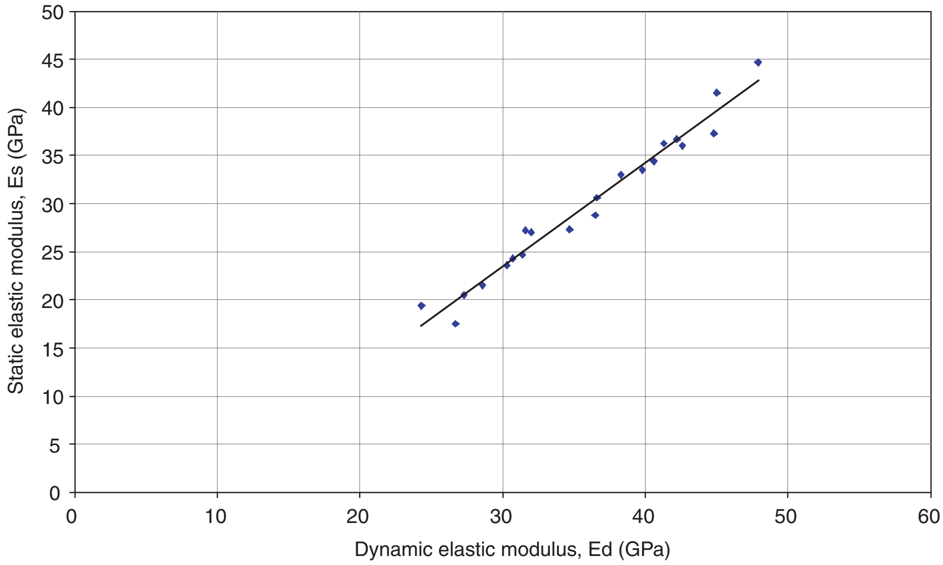

Figure E1 Relationship between dynamic and static modulus for CBM

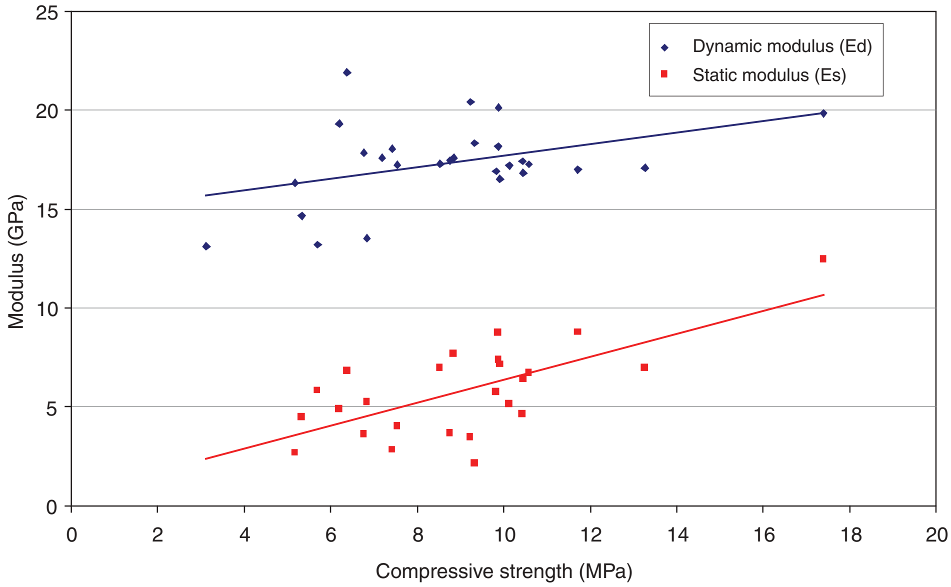

Figure E2 Relationship between compressive strength and modulus for SBM

Material strength

In the French design guide (LCPC/SETRA, 1998) the direct tensile strength of hydraulically bound is taken to be 80% of the indirect tensile strength. Also the indirect tensile strength is approximately 75% of the flexural strength (Raphael, 1984). These two relationships imply that the direct tensile strength is approximately 60% of the flexural strength.

Design based on static modulus and tensile strength

Equation E1 and the relationship between the strength parameters were used to estimate the 360 day structural properties for static elastic modulus and direct tensile strength of standard CBMs. These are given in Table E3 together with the corresponding values for the dynamic modulus and flexural strength.

Table E3 Design values

CBM base	*Dynamic modulus (GPa)*	*Static modulus (GPa)*	*Flexural strength (MPa)*	*Direct tensile strength (MPa)*
CBM3G	32.9	26.5	1.65	0.99
CBM4G	38.8	32.8	2.48	1.49
CBM5G	42.9	37.3	3.30	1.98
CBM3R	34.5	28.2	2.40	1.44
CBM4R	40.4	34.6	3.60	2.16
CBM5R	44.7	39.2	4.80	2.88

Heavy traffic designs (> 80 msa)

The design methodology using static modulus and direct tensile strength remains essentially unchanged. The basic design criterion for indeterminate life (> 80 msa) is that the predicted critical tensile stress σ_r at the underside of the hydraulically bound base induced by a standard wheel load will have to satisfy the following relationship:

$$\sigma_r \leq \text{ Direct tensile strength.} K'_{Hyd} . K_{Safety} \qquad \text{(E2)}$$

The 360 day values for the structural properties given in Table E3 have been used to calculate the critical stress (σ_r), and new adjustment factors (K'_{Hyd}) were determined to give agreement with the designs in the Highways Agency's Design Manual for Roads and Bridges (DMRB, Vol 7); these values are given in Table E4. The standard case assumes pre-cracking at 3 m intervals and that 180 mm of asphalt cover is required for this crack spacing to resist reflection cracking.

Table E4 Adjustment factors

Material	K'_{Hyd}	K_{Safety}
CBM3G	0.509	Default value of 1.0
CBM4G	0.474	
CBM5G	0.436	
CBM3R	0.469	
CBM4R	0.414	
CBM5R	0.354	

Traffic < 80msa

The design curve given in Figure 1 of the main report was used to develop the following design criterion for pavements carrying less than 80 msa, using material properties given in Table E3:

$$Log(N) = 1.25 \times \left(SR \times K'_{Hyd} . K_{Safety} + 0.15 \right)^2 + 0.25 \qquad \text{(E3)}$$

Where, the strength ratio (SR) is now the ratio between direct tensile strength and σ_r determined using a linear elastic pavement model.

The base thicknesses using the criterion given in Equation E3 are in excellent agreement with LR1132 designs and this criterion can be applied to other hydraulically bound materials using the K'_{Hyd} factors applicable to those materials.

Generally there will be insufficient long term performance data from pavements constructed with non-standard, hydraulically bound materials to calibrate the design criterion empirically. However, K'_{Hyd} factors can be related to the structural properties of the hydraulic base material. The data given in Table E3 were used to carry out a regression analysis to relate static elastic modulus (E_S in GPa) and direct tensile strength (D_T in MPa) to the adjustment factor (K'_{Hyd}). The following relationship was established:

$$K_{Hyd} = 0.55 + 2.0 \times 10^{-3} E_s - 0.095 D_T \quad \left(R^2 = 0.96 \right) \qquad \text{(E4)}$$

As mentioned in the main body of the report, the combination of stiffness and strength is crucial to the design of a hydraulically bound base and that two different hydraulically bound materials can have the same base thicknesses for a given level of traffic, provided that their strength compensates for any differences in their levels of stiffness. Relationships between elastic stiffness modulus and flexural strength required for equal performance are defined in Figure E3.

The points for slag bound material (SBM) plotted in Figure E3 used the static modulus implied by the relationship illustrated in Figure E1 and the assumption that the direct tensile strength is 60% of the flexural strength.

The static modulus is always less than the dynamic modulus and for less stiff hydraulically bound materials the difference between the static and dynamic modulus will be greater. On the other hand, the relationship between the strength parameters is more constant. The material classification system illustrated in Figure E3 shows that a disproportionate reduction in stiffness compared with strength can move hydraulically bound material into a higher classification. With the assumption used, Figure E3 shows that SBM is now comparable with CBM3G from the design viewpoint. Whereas it was two or three grades lower when characterised using dynamic modulus and flexural strength.

E.4 Summarising remarks

1 System 1 classification can be introduced relatively easily for cement bound granular materials (CBGM). The equivalences given in Tables E1 and E2 can be used

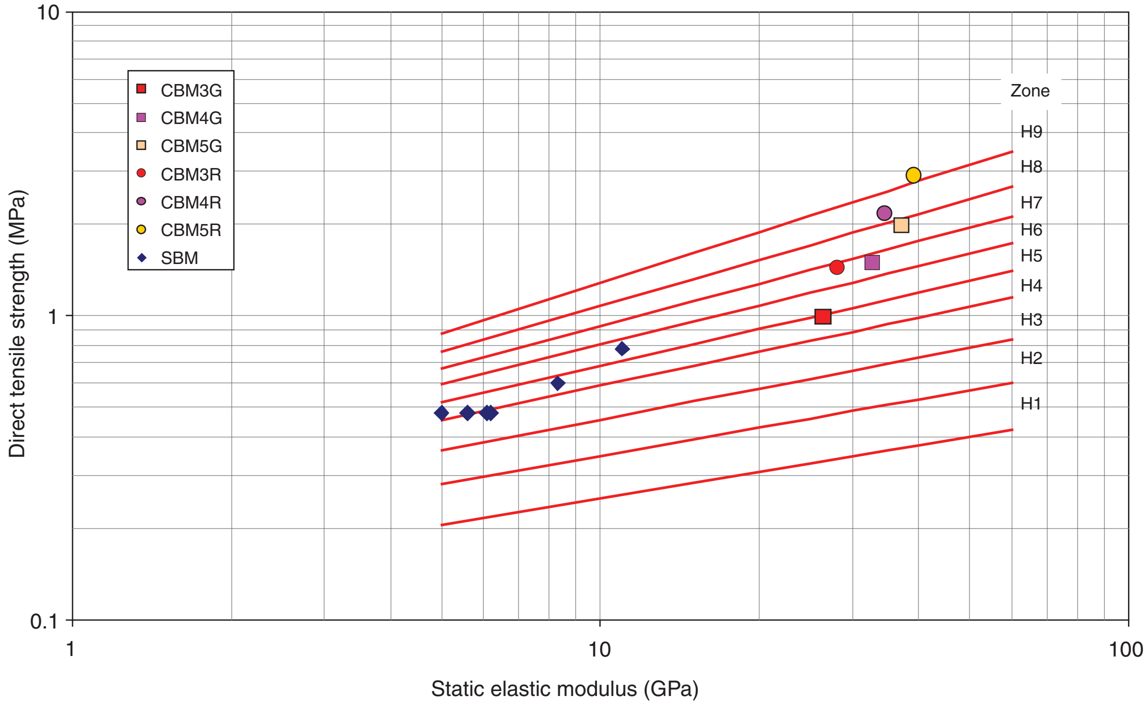

Figure E3 Relationship between static modulus and tensile strength for equal performance

together with the relationships given in the main report can be used to calculated 360 day values for dynamic modulus and flexural strength.

2 A move to material characterisation based on static modulus and direct tensile strength will require the design method to be recalibrated in the manner illustrated in this Appendix.

3 At present there is only limited data available on 360 day values of dynamic modulus and flexural strength of hydraulically bound materials in general and even less information available on static modulus and direct tensile strength. It is therefore recommended that further testing be carried out to develop more robust material characterisation values.

E.5 References

British Standards Institution (2004). *Hydraulically bound mixtures - Specifications*. BS EN 14227. London: British Standards Institution.

British Standards Institution (1990). *Concrete testing: Part 209 – Recommendations for the measurement of dynamic modulus of elasticity*. BS 1881. London: British Standards Institution.

Highways Agency, Scottish Office Development Department, the Welsh Office and the Department of the Environment for Northern Ireland (1998). *Manual of Contract Documents for Highway Works. Volume 1, Specification for Highway Works.* London: The Stationery Office.

Kolias S and Williams R (1978). *Cement bound road materials: Strength and elastic properties measured in the laboratory*. Supplementary Report SR344. Crowthorne: TRL Limited.

LCPC/SETRA (1997). *French pavement design manual. Translation of the December 1994 French version of the technical guide*. Published by Laboratoire Central des Ponts et Chaussees (LCPC) and Service d,Etudes Techniques Routes des Autoroutes (SETRA).

Megan M A and Earland M G (1999). *Construction of full-scale road trials to evaluate the performance of industrial by-product in roadbases*. Volume 2.4 – Slag bound material. Unpublished Project Report PR/CE/145/99. Crowthorne: TRL Limited. *(Unpublished report available on direct personal application only)*

Raphael J M (1984). Tensile strength of concrete. *ACI Journal*, vol. 80, no. 2, pp. 158-165.

Abstract

The current UK pavement design standard deals with the use of a limited number of materials in a restricted range of design options. The versatility of this method needs to be increased to give the highway engineer a wider choice of materials and design configurations. The increased versatility will lead to more economic designs by allowing new materials, recycled materials and a wider range of secondary aggregates and binders to be used. It will also enable stronger foundations, incorporating hydraulically bound materials, to be used with reductions in the thickness of the more expensive surfacing layers. In addition, separate design methods are currently used for flexible and flexible composite pavements and many designs currently being submitted for road schemes reside in a 'grey' area that involve pavement structures with material layers that are not covered directly by the current methods. Often there may be more than one way of estimating pavement life depending whether either fully flexible or flexible composite pavement design methodology is used. This can lead to ambiguities and therefore, a more versatile approach is required to deal with these difficult areas, and at the same time help to deliver more sustainable and economic pavement construction. This report describes the modifications to the current design method to improve its versatility. This modified method predicts pavement layer thicknesses that are compatible with existing methodology and methods of characterizing materials. As a result, the modified method could be introduced with little disruption. The method will facilitate more prudent use of natural resources and assist in the protection of the environment in line with Government Policy.

Related publications

TRL611 A guide to the use and specification of cold recycled materials for the maintenance of road pavements by D Merrill, M Nunn and I Carswell. 2004 (price £40, code HX)

TRL386 *Design guide and specification for structural maintenance of highway pavements by cold in-situ recycling* by L Milton and M Earland. 1999 (price £50, code L)

TRL250 *Design of long life pavements for heavy traffic* by M E Nunn, A Brown, D Weston and J C Nicholls. 1997 (price £50, code L)

RR303 *Asphalt substitution: road trials and cost considerations* by D Leech and M E Nunn. 1990 (price £20, code B)

SR344 *Cement bound road materials: Strength and elastic properties measured in the laboratory* by S Kolias and R I T Williams. 1978 (price £20)

LR1132 *Design of bituminous roads* by W D Powell, P F Potter, H C Mayhew and M E Nunn. 1984 (price £20, code A)

CT36.3 Recycling of road materials update (2001-2003) *Current Topics in Transport: selected abstracts from TRL Library's database* (price £20)

VR1 *Foamix: Road trials and design considerations* by M E Nunn and N Thom. 2002 (price £40, code HX)

Prices current at October 2004

For further details of these and all other TRL publications, telephone Publication Sales on 01344 770783, or visit TRL on the Internet at www.trl.co.uk.