FEHRL REPORT 2009/01

ELLPAG PHASE 2

A Guide to the use of Long-Life Semi-Rigid Pavements
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- Increase innovation in European road construction and road-using industries.
- Improve the energy efficiency of highway engineering and operations.
- Protect the environment and improve quality of life.
Making Best Use of Long-Life Pavements in Europe

ELLPAG PHASE 2  :

A Guide to the Use of Long-Life Semi-Rigid Pavements

Title: ELLPAG Phase 2 Report

Keywords: long-life pavements, semi-rigid pavements, economic benefits, deterioration mechanisms, design, maintenance

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Executive Summary

During 1998/99, the Western European Road Directors (WERD), now called the Conference of European Directors of Roads (CEDR), asked its members for topics of interest affecting the European road network, with the aims of identifying any knowledge gaps and initiating research. Long-life pavements (LLPs) was one of the topics suggested by the UK Highways Agency which was later endorsed by WERD as an appropriate area for a co-operative approach.

The European Long-Life Pavement Group (ELLPAG) was subsequently established in 1999 as a FEHRL and CEDR Working Group to act as the focal point for determining the way forward. Members of the Group comprise representatives of research institutes (FEHRL members) and the UK Highways Agency, representing CEDR.

The original objectives of ELLPAG can be stated chronologically as short-term, medium term and long-term objectives.

- The short-term objective of the Group was to produce, within 12 months of starting the formal project, a state-of-the-art review of current European knowledge on the design and maintenance of long-life fully-flexible pavements.

- The medium-term objectives are to produce similar state-of-the-art reviews for the other common pavement types.

- The long-term objective is to produce a user-friendly comprehensive Best Practice Guidance note on long-life pavement design and maintenance for all the common types of pavement construction used in Europe.

In 2002, a proposal for a first phase of work to be undertaken by ELLPAG was approved by FEHRL and CEDR. Funding was secured from each of the core member’s national road administrations to participate in this work. Phase 1 was completed in 2004 with the publication of ‘A guide to the use of long-life flexible pavements in Europe’. This document covers the second phase of the work and provides guidance on the use of long-life semi-rigid pavements in Europe. The next phase 3, which is already underway, will cover the use of long-life rigid pavements.

As for the first phase the work in this report has been sub-divided into six main areas which correspond to the main chapters of this Report: Traffic Assessment, Design and Construction, Assessment and Upgrading, Maintenance, Economics and Research Recommendations. The methodology for creating this report mainly involved each core ELLPAG member providing National Report covering each of the above subject areas. These National Reports have then been combined and summarised to provide the essence of each of the six chapters.

The majority of the chapters deal specifically with semi-rigid pavements but the chapters on traffic assessment and economics have more general applicability to both flexible and semi-rigid pavements.

During Phase 2 it was recognised that the definition of long-life pavements used during Phase 1 was not fully appropriate to semi-rigid and rigid pavements. Therefore the definition was revised, as follows, to encompass all types of long-life pavement.

A well designed and well constructed pavement where the structural elements last indefinitely provided that the designed maximum individual load and environmental conditions are not exceeded and that appropriate and timely surface maintenance is carried out.
This Phase 2 Report demonstrates that the technology available for long-life semi-rigid pavements is not as developed as that for fully-flexible pavement types. Whereas the concept of long-life fully-flexible pavements has been adopted in some European pavements and explicit design, assessment and methods exist, the equivalent long-life approach to semi-rigid pavements does not exist, except in the UK.

Two strategies for the management of long-life pavements covering construction, assessment and maintenance have been identified and these strategies are a theme in each of these parts of the review. It is important to bear in mind the type of strategy employed when considering all aspects of long-life semi-rigid pavements.

- A ‘prevention of cracks’ strategy is widely employed in Europe. Within this strategy, pavements are designed to prevent the onset of reflection cracking. Thereby long-life pavements assessment and maintenance procedures concentrate on the functional aspects of pavement condition.

- A ‘living with cracks’ strategy accepts, such as that operated by the French highway administration, accepts that the asphalt material can crack. However, the risk of structural degradation is controlled by the application of appropriate design, assessment and maintenance procedures.

The ‘living with cracks’ strategy is becoming less favoured but may still be a viable option in some specific cases. Each strategy should be chosen based on local economic analysis.

Only the UK currently operates pavements that are explicitly labelled as long-life semi-rigid pavements. Pavements, that are considered to have a long, but indeterminate life, are in operation in the UK; in other countries similar pavement constructions are available for heavy traffic.

No explicit methods for the assessment and upgrading of these types of pavement exist. In general, most countries use a measure of bearing capacity, such as the Deflectograph or Falling Weight Deflectometer, to assess the structural condition of semi-rigid pavements but it is equally important to take into account the thickness of the layers and the visual condition of the pavement. The nature of the cracking that is seen on the surface is a strong indicator of the condition of the hydraulically bound base.

Three potential options were identified for upgrading a semi-rigid pavement to long-life depending on the strategy to be employed and whether the hydraulically bound base was considered as suitable for upgrading. One option covers the transition of the existing pavement from a semi-rigid structure to a long-life fully-flexible type of structure.

The approach to the maintenance of long-life semi-rigid pavements is identical to fully-flexible pavements with the important exception of how to treat reflection cracks. This exception depends upon the strategy employed; for example within a ‘prevention of cracks’ strategy, a long-life semi-rigid pavement should be free from reflection cracking.

From the information collected in the review and the discussions at ELLPAG meetings, five research themes were identified. Research recommendations have been constructed to cover these themes; these are:

- investigation of the nature of reflection cracking in semi-rigid pavements;
- assessment of techniques for the control of reflection cracking;
- examination of the effect of traffic on semi-rigid pavements;
- development of economic analysis tools;
- optimisation of maintenance strategies for long-life semi-rigid pavements.

This review has identified best practice for the design, assessment, upgrading and maintenance of long-life semi-rigid pavements.
It is clear that there is a potential for long-life semi-rigid pavements. One European country has adopted an equivalence between fully-flexible and semi-rigid design and will soon be considering some semi-rigid pavements to be equivalent to long-life pavements. However, the more widespread adoption of long-life semi-rigid pavements is only likely to occur once the appropriate methods for quantifying the economic and environmental benefits of these pavements are more widely available.
1 Introduction

During 1998/99, the Western European Road Directors (WERD), now called the Conference of European Directors of Roads (CEDR), asked its members for topics of interest affecting the European road network, with the aims of identifying any knowledge gaps and initiating research. Long-life pavements (LLPs) was one of the topics suggested by the UK Highways Agency which was later endorsed by WERD as an appropriate area for a co-operative approach.

The European Long-Life Pavement Group (ELLPAG) was subsequently established as a FEHRL Working Group to act as the focal point for determining the way forward. Members of the Group comprise representatives of research institutes (FEHRL members) and the UK Highways Agency, representing CEDR.

Two levels of membership of the Group have been created. Core members are directly involved in the work of the Group and their representatives attend regular meetings as required. All core members are from either the FEHRL or CEDR organisations. Associate or affiliate members are kept informed of the work of the Group and contribute through supplying information as requested and commenting on draft outputs; these members do not need to be part of either FEHRL or CEDR.

The original objectives of ELLPAG can be stated chronologically as short-term, medium-term and long-term objectives.

- The short-term objective of the Group is to produce within 12 months of starting the formal project a state-of-the-art review of current European knowledge on the design and maintenance of long-life fully-flexible pavements. This report has been completed (FEHRL, 2004).
- The medium-term objectives are to produce similar state-of-the-art reviews for the other common pavement types.
- The long-term objective is to produce a user-friendly comprehensive Best Practice Guidance note on long-life pavement design and maintenance for all the common types of pavement construction used in Europe.

Since then, the Group has progressed to the second phase of the work as shown in Figure 1.1 and is considering semi-rigid pavements in addition to fully-flexible pavements. This report is a state-of-the-art review of semi-rigid long-life pavements.

The Phase 1 Report used the following definition of a long-life pavement. “A long-life pavement is a type of pavement where no significant deterioration will develop in the foundations or the road base layers provided that correct surface maintenance is carried out”. However, in Phase 2 (and Phase 3) it was recognised that an improved definition was required that was also appropriate to both semi-rigid (and rigid) pavements. The new definition, that encompasses all pavement types, is as follows.

A long-life pavement is a well designed and well constructed pavement where the structural elements last indefinitely provided that the designed maximum individual load and environmental conditions are not exceeded and that appropriate and timely surface maintenance is carried out.

Some organisations use the term indeterminate rather than long-life but the latter expression is that preferred by ELLPAG. In contrast, a determinate life pavement is the opposite of a long-life pavement, or more formally:

A determinate life pavement is a pavement that is designed, constructed and maintained to achieve a predefined design life.
Figure 1.1. Plan for the flow of the work of ELLPAG

Semi-rigid pavements comprise an asphalt layer laid onto a hydraulically bound pavement layer. This style of construction utilises different construction materials and some modes of deterioration are particular to this construction type.

Semi-rigid pavement construction gained popularity in Europe during the 1970's when, due to an escalation in the oil price, the unit cost of asphalt increased. Since semi-rigid pavements utilise less asphalt than their fully-flexible counterparts, the economic efficiency of this type of construction was increased. In subsequent years, their popularity waned because of problems associated with reflection cracks. However, with the introduction of pre-cracking techniques to reduce the adverse effects of reflection cracks, semi-rigid pavements have had a resurgence in recent years. The increased awareness of the environment and the quest for more sustainable pavement solutions has added to this popularity. Hydraulically bound material can utilise a wider range of aggregate than is suitable for asphalt materials, therefore semi-rigid pavements have been used to optimise the use of locally available aggregate resources. This can help to reduce the demand for primary aggregates and hence reduce environmental impact and improve the sustainability of pavement construction.
In addition to economic and environmental considerations, semi-rigid pavements function in a different manner to fully-flexible pavements. Although the surface characteristics of the two types of construction will be similar, the rigid formation of the base layer can reduce deformation in the pavement although there is a risk of cracking in the asphalt layer. Therefore, this type of construction may be attractive especially for use in hot climates where deformation is considered as a significant problem.

Although semi-rigid pavements are not the most common type of pavement construction, they can form a significant proportion of the network. COST 343 (FEHRL, 2003) reported that in Europe these type of pavements form up to 20% of the primary road network and up to 40% of a country’s total network. Data obtained by the ELLPAG members suggested that these proportions are still representative of the current position.

Some types of pavement construction can be difficult to classify as either a fully-flexible pavement, semi-rigid pavement or rigid pavement. Some hybrid pavement types, such as continuously reinforced concrete roads (CRCR), have similar properties to semi-rigid pavements concerning the asphalt layer while the main structural layers are similar to rigid pavement types. In order to focus the review, the ELLPAG members agreed to restrict the scope of the review to particular types of semi-rigid construction. Within this review, only long-life semi-rigid pavements have been considered with the following dimensional attributes: the asphalt layer will generally be between 80 mm and 200 mm, the hydraulically bound layer will be between 150 mm and 300 mm. In Spain, a soil cement base is commonly used and with this base material the asphalt layer can be up to 250 mm thick.

To assist in the understanding of the terminology as used in this report a glossary is presented in Appendix B.

The work undertaken during Phase 2 has been sub-divided into six main areas which correspond to the main chapters of this report: Traffic Assessment, Design and Construction, Assessment and Upgrading, Maintenance, Economics, and Research Recommendations. The methodology combined reviews of existing documentation and contributions for each area from the core member participants provided as National Reports and included in Appendix A. Table 1.1 lists the core ELLPAG members who participated in the tasks carried out in this Phase. A more detailed list of the organisations involved in the core group of ELLPAG is given in Appendix D.

ELLPAG gratefully acknowledges the support of CEDR and the National Highway Administrations of the participating countries in the completion of this work. ELLPAG also gratefully acknowledges the assistance of Maria de Lurdes Antunes (LNEC), Hans Jørgen Ertman Larsen (DRI) and Derek Carder (TRL) for checking the Phase 2 Report prior to publication.
Table 1.1. Phase 2 core ELLPAG member participants

<table>
<thead>
<tr>
<th>Organisation</th>
<th>Participant member</th>
</tr>
</thead>
<tbody>
<tr>
<td>ISTU – Austria</td>
<td>Michael Wistuba / Johann Litzka</td>
</tr>
<tr>
<td>BRRC – Belgium</td>
<td>Michel Gorski / Carl Van Geem</td>
</tr>
<tr>
<td>CDV – Czech Republic</td>
<td>Karel Pospisil / Josef Stryk</td>
</tr>
<tr>
<td>DRI - Denmark</td>
<td>Hans Ertmann Larsen</td>
</tr>
<tr>
<td>LCPC – France</td>
<td>Francois de Larrard</td>
</tr>
<tr>
<td>NTUA – Greece</td>
<td>Andreas Loizos</td>
</tr>
<tr>
<td>KTI – Hungary</td>
<td>Laszlo Gaspar / Robert Karoly</td>
</tr>
<tr>
<td>DWW (now DVS) – The Netherlands</td>
<td>Arthur van Dommelen / Arjan Venmans</td>
</tr>
<tr>
<td>CEDEX – Spain</td>
<td>Jose Baena</td>
</tr>
<tr>
<td>LAVOC – Switzerland</td>
<td>Mehdi Ould-Henia / Nicolas Bueche</td>
</tr>
<tr>
<td>IBDiM – Poland</td>
<td>Miroslaw Graczyk</td>
</tr>
<tr>
<td>TRL – United Kingdom</td>
<td>Brian Ferne (Chairman) / Khaled Hassan &amp; Guy Watts (Secretariat)</td>
</tr>
</tbody>
</table>
2 Traffic Assessment

The assessment of traffic for the design and maintenance of pavements is usually carried out using a load equivalence relationship of the form given in Equation 2.1; where \( L \) = load equivalence, \( P \) = axle load, \( Pr \) = reference axle load and ‘\( n \)’ is an exponent.

\[
L = \left( \frac{P}{Pr} \right)^n
\]  

Equation 2.1

A discussion was included in the Phase 1 report on the range of different exponents, the reference axle loads (\( Pr \)) used throughout Europe and the implications of these differences (FEHRL, 2004). Whereas for fully-flexible pavements in Europe, the exponent is usually taken to be either 4 or 5, a much wider range of exponents are used for semi-rigid pavements. In many countries, no differentiation is made for different pavement types and a uniform exponent is used; these countries include Germany, the Netherlands, the UK and Switzerland. In France, an exponent of 5 is used for traffic assessment on fully-flexible pavements while an exponent of 12 is used for other pavement types. In Belgium, an exponent of 4 is used for traffic assessment on fully-flexible pavements while an exponent of 33 is used for semi-rigid structures; the Belgian design method also uses a different reference load for traffic assessment on fully-flexible pavements (80 kN) than for semi-rigid pavements (130 kN). In Spain, an exponent of 4 is used for flexible pavements while an exponent of 8 is used for semi-rigid pavements.

This type of axle-load equivalence factors is no longer used in Austria. Instead, the damaging impact on the pavement is determined by a direct analytical approach, using fatigue equations, from the destructive effect due to the passage of different axle loads and axle configurations (relevant stresses and strains). In the framework of a comprehensive sensitivity analysis (Litzka et al., 1996) the key components of the damaging impact of single and twin tyres and different axle configurations on both asphalt and concrete pavements were examined. Then, in a next step, detailed data on the distribution of axle loads and vehicle types found in Austria’s federal road network were used as a basis for computation of mean equivalence factors for different vehicle types and characteristic collectives of heavy vehicles (see Table 2.1).

Table 2.1 Equivalence factors for heavy traffic used in Austria (RVS 03.08.63, 2008)

<table>
<thead>
<tr>
<th>Mean equivalence factors for different heavy vehicles in Austria</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>lorry, single</td>
<td>0.70</td>
</tr>
<tr>
<td>lorry plus trailer, articulated lorry</td>
<td>1.20</td>
</tr>
<tr>
<td>bus, coach</td>
<td>0.60</td>
</tr>
<tr>
<td>bus, public transport</td>
<td>0.80</td>
</tr>
<tr>
<td>articulated bus, public transport</td>
<td>1.40</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Mean equivalence factors of heavy vehicle fleet (AADTcv) for different road categories in Austria</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>motorways</td>
<td>1.00</td>
</tr>
<tr>
<td>other roads</td>
<td>0.90</td>
</tr>
</tbody>
</table>

For the computation of design loading, traffic aggressivity factors are often used. The load equivalence concept is used to compute average traffic aggressivity. Equation 2.2 illustrates this
type of approach where \( T = \) design traffic, \( N = \) number of loads and \( C = \) aggressivity factor; other factors can be used in this calculation such as lateral wander of traffic.

\[
T = N \cdot C
\]  
(2.2)

In Belgium, the average aggressivity factor for semi-rigid pavements (3.1) is almost four times that used for fully-flexible pavements (0.82). In France, a low traffic aggressivity factor (0.8) is assigned to thick, fully flexible pavements; however the factor is increased when the pavement layers contain hydraulic binders (1.3). In Spain, the situation is similar but the traffic aggressivity factors are even lower, 0.6 for flexible pavements and 0.8 for semi-rigid pavements. In the UK a single set of pavement aggressivity factors, called wear factors, are presented in HD 24 (DMRB 7.2.1) for all pavement types.

The reason for the difference in the exponents of the load equivalence relationships and average ‘aggressivity’ of vehicles is connected to the anticipated deterioration modes in asphalt and hydraulically bound materials. The onset of deterioration in asphalt, such as deformation or fatigue, is assumed to be progressive and related to the axle loads on the structure. The main mode of deterioration in hydraulically bound materials is a tensile failure caused when the stress in the layer exceeds the strength. Provided that the stress levels are low, failures rarely, if ever, occur; however if these stress levels are high, fracture within the material can occur and a sudden onset of deterioration can be seen. Consequently, the onset of deterioration in hydraulically bound layers is likely to be more sensitive to axle loads than asphalt layers, hence the use of greater exponents.

To illustrate the effect of estimated wear on different pavement types, two heavy goods vehicles have been defined: Vehicle A has a mass of 37 tonnes carried over five axles; Vehicle B has a mass of 22 tonnes carried over four axles. The relative aggressivity of these two vehicles has been calculated using the methods from Belgium, France, Spain and the UK; these are summarised in Table 2.2.

Table 2.2 shows that the traffic loading estimation for semi-rigid pavements is predicted to be more sensitive to the load than fully-flexible pavements. However there is a difference in how that sensitivity changes; the UK shows no change while the change in aggressivity is large in France and Spain and it is especially large in the Belgian method.

Some national methods consider the different effect of traffic on asphalt and hydraulically bound layers. In order to realise the long-life concept for semi-rigid pavements, an understanding is required of the relative methods of assessing traffic and the assumptions made about the deterioration modes. Although no difference between fully-flexible and semi-rigid structures is often assumed, the presence of a range of methods to account for the difference between pavement types suggests that further study in this area would be of value.
Table 2.2. Relative aggressivity for the above vehicles

<table>
<thead>
<tr>
<th>Country</th>
<th>Pavement Type</th>
<th>Vehicle A: Vehicle B</th>
</tr>
</thead>
<tbody>
<tr>
<td>France</td>
<td>Fully-flexible</td>
<td>4.82</td>
</tr>
<tr>
<td>France</td>
<td>Semi-rigid</td>
<td>23.20</td>
</tr>
<tr>
<td>Spain</td>
<td>Fully-flexible</td>
<td>3.90</td>
</tr>
<tr>
<td>Spain</td>
<td>Semi-rigid</td>
<td>14.46</td>
</tr>
<tr>
<td>UK</td>
<td>Fully-Flexible</td>
<td>3.78</td>
</tr>
<tr>
<td>UK</td>
<td>Semi-rigid</td>
<td>3.78</td>
</tr>
<tr>
<td>Belgium</td>
<td>Fully-flexible</td>
<td>3.78</td>
</tr>
<tr>
<td>Belgium</td>
<td>Semi-rigid</td>
<td>&gt; 100.00</td>
</tr>
</tbody>
</table>
3 Design and Construction

The following chapter is concerned with the design and construction issues specific to long-life semi-rigid pavements. Many aspects of design and construction are similar to those of fully-flexible pavements, therefore the review of requirements for long-life, fully-flexible pavements (FEHRL, 2004) should also be considered.

3.1 The semi-rigid pavement structure

Typical European semi-rigid pavements are made of 150 - 300 mm of hydraulically bound material (up to 480 mm in two layers for semi-rigid pavements in France) and covered with 80 - 200 mm of asphalt (down to 60 mm in France). In the Netherlands, sand-cement can be also used provided that durability to the effects of frost and moisture is demonstrated. In Spain, a type of soil-cement (or sand cement) is often used as base layer and in this case the asphalt layer can be as thick as 250 mm for long-life pavements.

The role of each of the semi-rigid pavement layers is as follows:

- The asphalt layer provides most of the functional characteristics of the pavement. The layer ensures a smooth riding surface and its surfacing will have suitable friction and noise properties. The asphalt layer also acts to preserve the structural integrity of the structure. It insulates the pavement so that the foundation is free from frost and the thermal movements in the hydraulically bound layer are controlled. Asphalt is a relatively impervious material which, prior to reflection cracking, prevents moisture from entering the main structure of the pavement.

- The hydraulically bound base layer is the primary structural layer. The base layer is formed of materials that are durable and have good load spreading properties. Cracks in this layer can form naturally; however, the design of the pavement and the materials should ensure that the effects of these cracks are small by limiting the possible movement around the crack.
The foundation provides support to the pavement. It can have an important role for the limitation of traffic stresses in the base layer and it can also limit the effects of degradation due to cracks in the hydraulically bound layer. For this reason, the foundation layers of a semi-rigid pavement are often formed of hydraulically bound material.

### 3.2 Design concepts in semi-rigid pavements

Semi-rigid pavements are designed to resist the onset of specific modes of deterioration.

- The hydraulically bound layer is designed to resist the effect of stress due to traffic and temperature changes. The stress is limited by the strength of the hydraulically bound material so that degradation of the base, which would result in a loss of bearing capacity, is prevented.
- The asphalt layer is designed to provide the functional characteristics of the pavement. The thickness of the asphalt layer plays an important role. A thicker asphalt layer will:
  - affect the stress in the underlying hydraulically bound base; therefore, the thickness can be chosen to assist in limiting the stress in the main structural layer;
  - slow down the development of reflective cracking;
  - thermally insulate the hydraulically bound base and thereby reduce thermal movements of the crack;
  - reduce thermal gradients and the resultant restrained thermal warping stresses in the base.
- The foundation is designed to be a construction platform and to provide adequate support to the pavement layers. The design of the pavement layers may be adjusted for the properties of the foundation.

Within the ELLPAG group, two design strategies have been identified which affect the role of the asphalt layer: 'living with cracks' and 'prevention of cracks'. Such strategies have implications for the design, assessment, upgrading and maintenance operations of long-life semi-rigid pavements.

Under the ‘living with cracks strategy’, the asphalt layer is designed to provide the functional role but it provides little assistance to the structural layers of the pavement. For such a strategy, the asphalt layer can be relatively thin. However, the pavement should be designed to make it less sensitive to the ingress of water and weakening by ‘pumping’. This can be achieved with thicker hydraulically bound structural layers and bound foundation layers that make them less moisture susceptible. These pavements may require more regular assessment and maintenance operations.

Under the ‘prevention of cracks’ strategy, the asphalt layer has a significant role in the assurance of the structural integrity of the pavement in addition to the functional characteristics. Long-life semi-rigid pavements designed according to this strategy will tend to have relatively thick asphalt layers. The thick asphalt layer will serve to reduce the traffic stresses and thermal movements in the structural layers; it will also inhibit the onset of reflection cracking from the base layer.

Both strategies can be accommodated in the long-life semi-rigid design concept provided that the appropriate provision is made in the future assessment and maintenance procedures.

Only the UK currently uses long-life semi-rigid pavement designs. They have recently developed a new design procedure that links the fully-flexible and semi-rigid pavement design method (Nunn, 2004). Above 80 msas80 long-life semi-rigid designs are now offered as equivalent options to long-life fully-flexible pavements.
3.3 Anti-reflection cracking techniques

One of the features of semi-rigid pavements is that due to thermal shrinkage, naturally occurring transverse cracks can form in the hydraulically bound layers soon after construction. These cracks are exacerbated in materials with a high coefficient of thermal expansion and with high stiffness compared to strength and where the aggregates are more susceptible to changes in temperature. The onset of reflection cracking is likely to occur sooner and be more severe in eastern European countries where the annual and diurnal temperature range is higher than in the temperate regions of western Europe.

Directly above the cracks in the hydraulically bound layer, cracks can form in the asphalt layer and these are called reflection cracks. The onset of reflection cracking depends on the crack opening displacement in the stabilised material and the thickness of the overlying asphalt layer.

Where cracks have penetrated the full depth of the asphalt layer, sealing of cracks with bituminous mastic is often required to prevent water ingress and de-bonding between the asphalt and hydraulically bound layer.

Measures have been developed in order to reduce the risk of reflection cracking. One of the main methods of restricting the likelihood of reflection cracking in the asphalt layer is pre-cracking the hydraulically bound layer during construction.

3.3.1 Pre-cracking

The onset of reflection cracking can be delayed by creating transverse cracks in the HBM at short distances (usually between 2 and 4 m) at the construction stage before the material has set. The surface of the HBM is then compacted before placing the asphalt layer.

There are a range of pre-cracking treatments available. In France, the following techniques that involve cutting a groove in the fresh concrete and placing a vertical discontinuity in the layer have been developed:

- **CRAFT** (CFTR, 2004)
  The Craft process (CRéation Automatique de Fissures Transversales - Automatic creation of transverse cracks) is applied before compacting. It consists of creating a discontinuity in the layer of hydraulically treated material by making a transverse groove in which a bitumen emulsion is injected.

- **Olivia** (CFTR, 2004)
  A discontinuity is created in the HBM layer by placing a simple flexible plastic film vertically in the thickness. It is placed by means of a plough, which cuts a transverse groove to the mid-depth of the layer and through which the film is unreeled.

- **Joint actif** (CFTR, 2003)
  The joint-active process consists of placing a rigid insert of corrugated PVC within the layer of hydraulic material.

Spain also uses the CRAFT technique, but other systems are being developed in order to make treatment more competitive.
Figure 3.2. Pre-cracking with the CRAFT technique

Figure 3.3. Pre-cracking with the Olivia technique
3.3.2 Other techniques

In addition to pre-cracking, other techniques have been employed to delay or inhibit the onset of reflection cracking. The following techniques have been identified in the review:

- **Low heat cements** have two features: they are weak in early-life (30% less strength than mixtures with Portland Cement at 7 days); they have a high mechanical strength (50% greater than mixtures with Portland Cement).

- **Micro-cracking.** Using a vibratory roller to compact the mixture, will induce micro-cracks in the material thereby relieving stress.

- **Stress absorbing layers.** The use of a *sand asphalt* layer or a *stress absorbing membrane interface* (SAMI) at the interface of the asphalt and hydraulically bound layers will inhibit the transmission of strains, generated by cracking, from one layer to the other.

- **Fast emulsion waterproofing** protects the hydraulically bound layer from the rapid loss of moisture in early life. Excessive loss of moisture in early life can lead to weak materials and high volumetric shrinkage.

- **Limiting the strength of hydraulically bound layers at early life** can result in much finer cracks in the hydraulically bound layer which will reduce or mitigate reflective cracking.

- **The use of inverse pavements**, where a relatively thin granular layer (around 160 - 210 mm) is interposed between the asphalt layer and the hydraulically bound materials, is another technique used in some countries to mitigate the reflective cracking.

All these techniques can be used in combination with pre-cracking to reinforce its effect in delaying the onset of reflection cracking.

3.4 Theoretical service life according to thickness

The design of semi-rigid pavements is often based on the fatigue effect due to traffic. As discussed in Chapter 2, the higher exponent for fatigue damage feeds through into the exponent for the load equivalence law for semi-rigid pavements which is often higher than for fully-flexible pavements; this increase makes the thickness design of the hydraulically bound layer relatively insensitive to traffic compared to fully-flexible pavements.

To illustrate this sensitivity, the effect on bearing capacity for additional thickness of hydraulically bound layer has been computed according to the French analytical design standard (LCPC, 1997).

- An increase of 10 mm in the hydraulically bound base layer increases the service life from 30 years to almost 50 years.
- An increase of 20 mm in the hydraulically bound base layer increases the service life from 30 years to almost 100 years.

In theory, very long service lives can be achieved with only a marginal increase in base thickness.
3.5 Long-life semi-rigid designs

3.5.1 Comparison of semi-rigid designs

Figure 3.4 provides a summary of semi-rigid pavement designs for heavily trafficked conditions in Europe; only the UK currently operates specific long-life semi-rigid pavement designs. A wide range of solutions for heavy traffic conditions are available using semi-rigid pavements (Figure 3.4); reference should be made to Appendix A for the design assumptions made for each of these solutions. A hydraulically bound material (HBM), as shown in Figure 3.4, may be bound with cement or another hydraulic binder.

![Diagram showing suggested semi-rigid designs for heavy traffic conditions](image)

* Semi-rigid 'living with cracks' pavement (see Section 3.5.1)

*Figure 3.4. Suggested semi-rigid designs for heavy traffic conditions (soil-cement sub-base layer required under the hydraulically bound base)*

The designs shown in Figure 3.4 include pavements operating ‘living with cracks’ and ‘prevention of crack’ strategies. Most of these pavements could be considered to be designed according to the ‘prevention of cracks’ approach.

The French catalogue of pavement structures also includes a type of pavement called ‘Chaussée mixte’ (asphalt surfacing and hydraulically bound base of comparable thickness) (FR) as well as ‘Chaussée semi-rigide’ (asphalt surfacing of up to 80 mm thick over a thick hydraulically bound base material) (FR*); both these types of pavement may be considered within the scope of this review. ‘Chaussée mixte’ would be covered by the ‘prevention of cracks’ strategy since these pavements have a minimal risk of reflection cracking. This first type of section is also used in Spain. ‘Chaussée semi-rigide’ would be covered by a 'living with cracks' strategy; this design includes the thickest hydraulically bound layer and the thinnest asphalt layer of the group. Greece has adopted the French practice of using a thick cement bound layer and a relatively thin asphalt layer.
3.5.2 Climatic considerations

Climatic conditions will influence the performance of a semi-rigid pavement. Temperature changes will cause thermal movements of any transverse cracks in the hydraulically bound base layer, greatly influencing the formation of reflection cracks in the asphalt layer. In addition, temperature gradients will induce warping stresses in the hydraulically bound base layer.

There is a wide variation in climate throughout Europe. The temperate regions of western Europe, typified by cool summer and mild winters, can be contrasted with the extreme climate of eastern Europe countries where the summers are hot and the winters are cold. These climatic effects are likely to make maintenance of reflection cracks more difficult in countries like Poland and the Czech Republic. Future European research on long-life semi-rigid pavements should also contain a climatic aspect.

3.6 Best practice

Prior to defining the design of long-life semi-rigid pavements, a strategy for operation of the pavements must be selected. This strategy will be chosen according to the outcomes of life-cycle cost evaluation (see Chapter 6).

The base materials must be selected so that they provide the necessary structural requirements of the pavement. Long-term stiffness and strength should be considered, as well as durability.

Consideration must be given to the nature of naturally forming cracks in the base layer; in some cases frequent but narrow cracks are preferred to the wider cracks at greater longitudinal spacing that can occur in some high stiffness materials. Larger thermal movements are likely to occur with wider spaced cracks. Where the nature of cracking is a concern for the integrity of the structure over a long-life, there are techniques available to mitigate the negative effects of these cracks, for example pre-cracking.

Within a ‘prevention of cracks’ strategy the asphalt layer thickness can be chosen to delay the onset of reflection cracking. A threshold thickness of 200mm is used in the UK for heavily trafficked roads; modest reductions in this threshold thickness can also be made through the application of techniques to prevent reflection cracking.

Good engineering practices and quality control are essential requirements for pavements designed for long-life to ensure that the construction does not contain potential problems. Good practices will result in pavements being constructed to a more uniform standard without weak areas that may be the source of localised poor performance. Most of the more common material problems could be minimised by specifications based on the relevant performance properties and rigorously applied quality control measures.

Deterioration in long-life pavements is inevitable. However in properly designed and well constructed long-life pavement this deterioration will be restricted to loss of surface characteristics and surface rutting that can be remedied by replacement of the upper asphalt layers. Generally, reflection cracks will appear at some time during the life of the pavement. In long-life design, either thick asphalt layers will prevent or delay their onset, or the strength and thickness of the hydraulically bound layers coupled possibly with stabilised or non-moisture susceptible foundation layers will reduce their impact. Reflection cracks will require treatment at some stage after their appearance. Strategies for dealing with reflection cracking are considered in later sections of this report.
4 Assessment and Upgrading

Specific assessment and upgrading techniques are necessary in order to exploit the opportunities offered by long-life pavements on the existing road network. In this context, assessment relates only to the measurement of the structural quality of the pavement; assessment of the functional aspects of pavement condition is covered in Chapter 5 under Maintenance.

The assessment procedure is carried out to indicate whether or not the pavement can be considered as a long-life pavement. When the assessment procedure has not indicated that the pavement can be considered as a long-life pavement, it may be considered for upgrading to a long-life pavement provided that such a solution is economically justified.

The objectives of this review are:

- to determine the methods employed throughout Europe for the structural assessment of long-life semi-rigid pavements,
- to explore the methods of upgrading existing semi-rigid pavements to form a long-life pavement.

4.1 Assessment

In order to effectively manage a road network asset, it is necessary to conduct routine assessments of the structural condition of the pavement; these form a network level assessment programme. Where the condition of a pavement exceeds an investigatory level, more detailed assessment can be carried out, often called the project level assessment.

The determination of whether a pavement is a potentially long-life pavement usually requires some appreciation of the structural properties of the pavement. While network level pavement condition data can indicate whether structural deterioration is evident, long-life assessment cannot be achieved unless some form of detailed structural assessment is carried out. It is therefore more likely that such detailed structural assessment will be carried out at a project level rather than a network level.

Since the structural layers of a semi-rigid pavement are formed from layers with very different attributes and which provide different contributions to the pavement structure, it is necessary to consider the contribution or condition of these layers separately as this will have a direct bearing on the nature of the upgrading measures available.

In a hydraulically bound base layer, structural deterioration is likely to be in the form of cracking. Cracking in itself cannot constitute deterioration since the design of semi-rigid pavements accommodates naturally occurring cracks in the base layer; however, structural deterioration can occur in the form of either:

- degradation of the integrity of the hydraulically bound layer as a whole;
- degradation at the interface of naturally occurring cracks that were formed in early life; resulting in poor load transfer across the crack; or
- secondary cracking occurring as a result of loss of foundation support in the vicinity of the crack caused by water entering the crack and a pumping action, that results from the wheel load passing over the crack, washing fines from the pavement foundation layers.
The asphalt layer can experience similar forms of deterioration to long-life fully-flexible pavements: functional deterioration such as loss of friction, deformation within the asphalt layer etc. In addition, the naturally forming cracks in the hydraulically bound layers can produce cracking apparent on the surface over the top of these cracks, called reflection cracking. Reflection cracking is most commonly observed as transverse cracks which tend to develop at regular intervals along the length of the pavement.

Once cracks have propagated through the asphalt surfacing, water can enter the road and a ‘pumping’ effect, caused by vertical movements as traffic passes from one slab to the other over the crack, can wash fines from the foundation layers and weaken the pavement in the region of the crack. This undermining of the pavement, if allowed to progress, will lead to the formation of secondary cracks around the reflection crack and accelerated deterioration. This situation needs to be avoided in long-life pavements.

4.1.1 Network Level Assessment

In addition to the functional measures that are carried out on all pavement types, the structural assessment of semi-rigid pavements can be performed by deflection measurements at a network level. While these deflection measurements alone may indicate areas with severe structural problems, they are unlikely to provide a detailed knowledge on the nature and cause of deterioration; they are therefore unlikely to be used to determine whether a semi-rigid pavement is a potentially long-life pavement.

However, one of the key characteristics of long-life pavements is that they are not expected to weaken with time. Regular network level deflection measurements on mature, heavily-trafficked semi-rigid pavements can be compiled over a number of years to produce a deflection history for the pavement. A strong indicator of a long-life semi-rigid pavement is deflection not increasing or even reducing with time; records of visual condition indicators should be used to support any such trends.

4.1.2 Project Level Assessment

The project level assessment is aimed at determining the extent and origins of deterioration in the pavement structure so that an appropriate treatment can be designed. More detailed testing can be performed to extract information about the condition of each layer in the pavement structure.

For a project level assessment of semi-rigid pavements, the main objectives are:

- to determine whether there is any deterioration in the structural layers. If detected, the pavement is not suitable for consideration as a long-life semi-rigid pavement.
- To determine the extent of deterioration in the surfacing. The nature of deterioration in the surfacing will affect the type and timing of the operations required for upgrading semi-rigid pavements but also for maintenance of those pavements that can be considered as long-life semi-rigid structures.
- To determine whether degradation has occurred in the vicinity of any reflection cracks that have formed. The severity of the cracks will determine the type of treatment required.

Structural rutting is not expected in a semi-rigid pavement; if serious deformation of the surfacing is detected, care should be taken to identify whether this is due to flow within the asphalt material or degradation in the base.
4.1.3 General Assessment Procedures

A common type of assessment procedure encountered in this review, particularly for project level assessment, is to take a measure of the bearing capacity of the pavement possibly combined with an assessment of the visual condition of the pavement. Such assessments are conducted in Austria, Belgium, France, Hungary, Poland, Spain, the Netherlands and the UK.

The devices that are used to provide a measure of bearing capacity include the Benkelman Beam, Deflectograph, the Curviameter and the Falling Weight Deflectometer. Non-destructive deflection testing can be performed to assess the total stiffness of the pavement layers. The stiffness of the asphalt and the hydraulically bound base layer contribute to the overall stiffness of the pavement layer, therefore by considering these stiffnesses against some form of criterion, detection of deterioration can be achieved. Adequate material properties in the base and the foundation are also essential; the strength of the hydraulically bound materials in the base and in the foundation should be checked.

Back-analysis procedures are used in Spain, the Netherlands and the UK to obtain the stiffness of the pavement layers. Care must be taken when utilising this approach to avoid anomalies which can often occur when analysing semi-rigid structures; unrealistic stiffnesses can result due to the sensitivity of the system caused by the stiffness of the hydraulically bound layer.

The Falling Weight Deflectometer can also be used to compare the bearing capacity in the wheel tracks and between the wheel tracks; such a method was proposed by the Netherlands. If the measurement in the wheel tracks is significantly lower than between the wheel tracks, this is an indication of traffic-related structural deterioration in the roadbase layer. Where low bearing capacity is found in unloaded areas of the pavement with no serious reflection cracking, structural deterioration due to environmental factors such as frost and/or moisture is a possibility.

In France, indicators of pavement condition, mainly for network level assessment, are determined from high-yield road monitoring equipment together with knowledge of the pavement's history, construction, age and traffic. The condition of the reflection cracks is regarded as the most important indicator of the condition of semi-rigid pavements and a series of treatments can be applied to limit the development of cracks, prevent water ingress and to limit deterioration at the interface between the asphalt and hydraulically bound layer.

The thickness of the pavement layers plays an important role in the determination of bearing capacity. The thickness can be obtained from construction records, but coring of the layer is preferable and should be done to determine the actual construction. Radar surveys (calibrated by coring) can provide a detailed profile of the construction of the pavement; the layers of a semi-rigid pavement should be more readily distinguishable than for fully-flexible pavements due to the larger disparity between material properties.

The visual condition of the pavement is commonly used to augment the bearing capacity assessment and may be used to diagnose the origins of structural deterioration so that structural maintenance measures can be appropriately designed. For heavily trafficked pavements, a key indicator of deterioration is the type and extent of visual deterioration. A major mode of structural deterioration in a semi-rigid pavement is reflection cracks that originate from the base. Since these reflection cracks are considered to originate from the base layer, they can also be used to interpret the condition of the base. Three conditions exist (Figure 4.1):

A. the pavement surface is either free from cracks or has minor cracking;
B. the pavement surface contains regularly spaced reflection cracks;
C. the pavement surface contains extensive cracking.

The severity of transverse reflection cracks, rather than their longitudinal spacing, is more important for long term performance of the pavement. Reflection cracks can be relatively benign as is the case for ‘living with cracks’ philosophy. In these cases, transverse cracks in the asphalt layer are expected to be narrow in width and stable as a result of strong underlying layers of hydraulically
bound material often in combination with hydraulically bound stabilised foundation and subgrade layers or non-moisture susceptible foundation layers. With these pavements, water ingress is unlikely to lead to rapid local deterioration.

If these conditions are not fulfilled, once the full thickness of asphalt has cracked moisture can enter the road foundation and with moisture susceptible foundation materials, ‘pumping’ under the action of traffic will undermine the pavement in the locality of the crack. The resulting increase in the relative vertical movements across the crack, as the wheel load transfers from one slab to the next, will also erode the load transfer across the crack. Without appropriate and timely intervention, serious maintenance will be required and the integrity of the long-life pavement is likely to be compromised. The load transfer across a crack can be investigated using a Falling Weight Deflectometer, as described in UK documentation HD29 (DMRB 7.3.2).

Semi-rigid pavements with severe transverse cracks that have been under-mined by ‘pumping’ may be restored to long-life by deep excavation and replacing the cracked material, provided the pavement between the cracks is structurally sound. The economic viability of carrying out this treatment will depend on the longitudinal spacing between the cracks.

It is now generally accepted that longitudinal cracking in the wheel path of thick fully flexible pavements is a top-down phenomenon (ISAP, 2004). A similar mechanism can occur in semi-rigid pavements. A core survey will resolve whether longitudinal cracks are the result of a failure in the hydraulically bound base or top-down cracking of the asphalt surfacing.

Generally speaking, regularly spaced transverse reflection cracks, that have penetrated the entire thickness of the asphalt layer, are not necessarily indicative of a structural problem in the hydraulically bound base layer; provided that good load transfer has been maintained across the crack. This is normally indicative that the foundation is not moisture susceptible and prone to ‘pumping’. The structural integrity of these pavements can be preserved by the replacement of the cracked asphalt material.

Extensive cracking that has occurred on the surface of the pavement is more likely to indicate a structural problem.

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![Figure 4.1. Illustration of different cracking conditions on semi-rigid pavements](image)

A variation of the combined structural and visual condition assessment is used in Belgium. A detailed radar survey is performed in order to detect relatively weak zones; these zones are then investigated further using the Falling Weight Deflectometer (FWD).
4.1.4 Assessment of Long-Life Pavements

The assessment of whether a pavement is a long-life pavement depends upon the strategy employed towards semi-rigid pavements. Two strategies have been identified by the ELLPAG group: the ‘living with cracks’ strategy accepts that reflection cracks occur and the maintenance of these cracks are managed so that they do not adversely affect the structure of the pavement; the ‘prevention of cracks’ strategy is to construct the pavement to minimise the occurrence of reflection cracking. For both strategies the condition of the base layer is important as this is the main structural element in the pavement.

No country currently operates a long-life assessment procedure for semi-rigid pavements. However the UK has an approach to the assessment of semi-rigid pavements to identify pavements that could be upgraded to potentially long-life fully-flexible pavements.

The UK method for the assessment of semi-rigid pavements (called flexible-composite pavements in the UK) is described in HD 30 (DMRB 7.3.3). The method involves the categorisation of structural condition using a suite of tests. Class A is the only category for which the pavement is free from structural deterioration and is thus suitable for consideration as a potentially long-life pavement or for upgrading. A pavement that satisfies a Class A condition will have the following characteristics.

- **Visual condition**: widely spaced cracks that are of a fine width, no evidence of foundation degradation.
- **CBM strength**: compressive cube strength of hydraulically bound base is greater than 10 N/mm².
- **Pavement structure**: Using the FWD, an assessment of individual layer stiffness can also be made HD 30/08 (Highways Agency 2008). Good integrity is sought in each layer; therefore the asphalt layer is expected to be greater than 7 GPa, the base layer greater than 15 GPa and the foundation layer greater than 100 MPa; the combined asphalt and CBM layer should have a stiffness in excess of 10 GPa.

A similar type of assessment procedure based on categories of condition is operated in Poland. Here, Category ‘A’ pavements are those that are free from structural deterioration although these may have widely-spaced, fine cracks.

4.2 Upgrading

The review has determined that no country provides a specific method for upgrading semi-rigid pavements to potentially long-life pavements, except the UK. Instead, the possibilities for the development of an upgrading approach have been reviewed.

Three options exist for upgrading the structure of a semi-rigid pavement to form a potentially long-life pavement depending on the condition of the structural layers of the pavement and the type of long-life strategy employed.

1. **Provided that the base layer has been assessed as suitable for upgrading**, an asphalt overlay can be placed so that future structural deterioration is prevented and the asphalt layer resists reflection cracking. A potentially long-life semi-rigid pavement can be produced according to a ‘prevention of cracks’ strategy.

2. Where the base layer is not suitable for upgrading, it cannot be upgraded to form a long-life semi-rigid pavement. Instead, it may be upgraded to form a long-life fully-flexible
pavement; the existing base of the semi-rigid pavement will form part of the foundation of a potentially long-life fully-flexible pavement.

3. When upgrading according to a ‘living with cracks’ strategy, the hydraulically bound base is assessed to be free from structural deterioration. The pavement is designated as a potentially long-life semi-rigid pavement according to a ‘living with cracks’ strategy without performing any structural maintenance treatment; appropriate provision is made within this strategy for suitable assessment and maintenance measures.

More details on how to upgrade a pavement according to Options 1 and 2 are discussed in this Chapter. Option 3 is essentially a designation change and as such, the resulting changes required to the assessment and maintenance strategy are covered in Chapter 5 of this report on “Maintenance”.

4.2.1 Option 1: Existing base is suitable for upgrading

The process of upgrading a semi-rigid pavement to form a potentially long-life pavement, where the base layer has been assessed to be suitable for upgrading, is the determination of a suitable asphalt thickness that will prevent future structural deterioration in the base layer and will resist the penetration of surface initiated cracking through the asphalt layer.

The thickness of the asphalt surfacing influences the thermal gradients in the pavement, the stress distribution in the pavement under traffic loading and restrains the thermal movements in the hydraulically bound base. As the asphalt thickness increases, the stresses in the base due to traffic loading reduce resulting in a lower risk of overstressing (and therefore degrading) the base layer. A greater thickness of asphalt reduces the thermal variation at the asphalt/base interface and exerts a greater static load on the base layer; these effects coalesce to reduce the movement around naturally forming cracks and lower the risk of reflection cracking originating from the base layer.

A primary consideration for the determination of this thickness is the consideration of the risk of structural deterioration that can be effected by cracking that has penetrated all or most of the thickness of the asphalt layer. For some types of semi-rigid pavement, there is a low risk of structural deterioration even when cracks expose the base layer to greater movement or the environment. In such cases, cracking can be accommodated within a long-life pavement structure.

However, in many cases, cracking that penetrates beyond the surface course and binder course layers could be considered a serious risk to the integrity of the semi-rigid pavement structure. For such pavements, the thickness of the asphalt layer should be chosen and/or a maintenance strategy such that this risk is minimised; this has been termed a ‘prevention of cracks’ long-life strategy.

The UK operates a strategy akin to a ‘prevention of cracks’ strategy and has developed a threshold approach to the determination of the thickness of asphalt, at which the risk of reflection cracking is low. As well as this, the selection of the base layer thickness ensures that the risk of the cement bound base degrading under traffic is negligible. This construction is called an indeterminate life design. Cracks may still originate from the surfacing, but provided these are maintained in a timely fashion, such cracks are not permitted to jeopardise the structure of the pavement.

Indeterminate life designs for flexible-composite (or semi-rigid) roads exist; these could potentially be long-life designs. Provided that the hydraulically bound base layer is comparable to the current standard designs, an overlay of the asphalt layer can be undertaken to increase its thickness to 200 mm. This upgrading methodology does not take into account any existing deterioration within the asphalt layer. The upgraded structure should be free from deterioration.

In France, there are no specific techniques to upgrade semi-rigid pavements to long-life. Different categories of maintenance treatments can be applied involving different thicknesses of overlay. The
highest category is reinforcement with 140 mm or more of asphalt. These treatments return the structural condition and surface characteristics of the pavement to that of a new pavement. Another unproven treatment option is the use of a High-Performance Concrete (HPC) surface layer (De Larrard, 2005) comprising a thin, unbonded, reinforced high-performance hydraulic flexible wearing course over a cracked hydraulic base has been used. Ingress of water into the pavement is prevented by laying a thick polyethylene sheet under the HPC, and owing to the high durability of HPC as a material, it is likely that such a concept will allow existing rigid or semi-rigid pavements to be upgraded into long-life pavements.

In Poland, a mechanistic method (based on controlling stress and strains) is used to design upgrade treatments for heavily trafficked semi-rigid pavements.

The bearing capacity of a material degrades with distress; therefore, given the existing condition of a pavement, it is theoretically possible to design an upgrade treatment (usually an overlay) such that the structural number exceeds some long-life criterion. At the current time, the criterion for long-life in terms of structural number has yet to be defined, but this may be a possible future method of upgrading should such a criterion exist.

### 4.2.2 Option 2: Existing base is unsuitable for upgrading

Where the base of a semi-rigid pavement has been assessed to be unsuitable to form part of a long-life semi-rigid structure, it can be adopted as the sub-base of the fully-flexible pavement. If severely deteriorated, the adopted sub-base can only be considered as equivalent to an unbound sub-base layer. If the base layer maintained significant structural capacity, it may be considered as part of some form of superior foundation within a long-life pavement structure.

The Phase 1 report (FEHRL, 2004) contained a review of methods for upgrading to form a long-life fully-flexible pavement. It is unlikely that such methodologies will be suitable for upgrading semi-rigid pavements in their explicit form since the asphalt layers of semi-rigid pavements are usually not greater than 200 mm. However, provided that the asphalt layer can be shown to be in a good condition and free from deterioration, the strong support that has been provided by the hydraulically bound base could theoretically ensure that the asphalt surfacing (albeit thinner than 200 mm) will be suitable to form part of an upgraded structure.

The principal method of upgrading to a long-life fully-flexible pavement was to ensure that the deflections of the pavement were low and the existing asphalt layer was free from deterioration, and to increase the asphalt thickness so that it exceeded 300 mm. This method could also be adopted for semi-rigid structures with deteriorated bases.

Alternatively, the existing asphalt layer could be sacrificed and the entirely new pavement structure reinstated using an adopted hydraulically bound sub-base layer. In this case, a long-life fully-flexible or semi-rigid pavement design could be produced.

Another option would be to remove the asphalt layer and ‘crack and seat’ a deteriorated hydraulically bound base and then an asphalt layer added (Langdale et al, 2003). Transverse cracks are induced every 2 to 4 metres using a drop-hammer guillotine blade. The slabs are then ‘seated’ using a heavy pneumatic tyre roller (PTR). This technique could be used to achieve a long-life, fully-flexible pavement or it could be classified as a long-life semi-rigid pavement that has been post-cracked rather than pre-cracked. This approach would make use of the existing foundation and base and may be a more sustainable solution compared to total reconstruction.
4.2.3 Opportunities for recycling

The operation of upgrading a pavement to form a potentially long-life pavement can involve the placement (or replacement) of a significant amount of bituminous material. It is therefore prudent to consider the opportunities available for recycled materials in this way.

For upgrading to potentially long-life semi-rigid pavements, recycling will be restricted to the use of hot-mixed or cold-mixed materials comprising recycled asphalt planings (RAP). Cold-mixed materials have traditionally been used on lightly trafficked roads although recent experience with these materials is being used to promote their use on more heavily trafficked pavements (Merrill et al., 2004).

Where existing asphalt material must be removed for the purposes of upgrading, the milled material can be removed to stockpile before re-mixing in plant and returning to the site. There will not be sufficient quantity of RAP to perform the upgrade treatment without virgin aggregate or aggregate from external sources. However, depending on the programme of work, the following options exist: use recycled asphalt for a proportion of the total treatment length or for one layer of a multiple layer upgrade; the recycled material is likely to form a blend of RAP and a proportion of virgin aggregate. An upper limit of 50% of RAP is normally permitted to be used in dense asphalt materials (UK, NL) no other design rated restrictions are made.

In Austria, up to 70 % of RAP can be used for the base layers of pavements provided that the traffic level is less than 1.3 msa100. It has been observed that some beneficial bonding effects are observed when recycled aggregates are used due to the higher total binder content.

In recent years, a rational strengthening approach has been undertaken in Greece, which is comparable to upgrading to a long-life pavement. More specifically, in areas where the bound base material was significantly damaged the original structure was rehabilitated using in-depth cold recycling with foamed asphalt stabilisation. All of the asphalt concrete layer and part of the CBM was recycled and the pavement was additionally overlaid with asphalt concrete in order to assure a long pavement life.

Where shown to be durable and economically viable, upgrading to long-life pavements using recycled asphalt materials further increases the sustainability of long-life semi-rigid pavements.

4.3 Best practice

Given that the foundation is providing good support to the pavement, the most important aim of the assessment should be to determine the structural condition of the hydraulically bound base. This should be the first step to assessing whether the pavement is suitable for consideration as a potentially long-life pavement and will determine the available routes for upgrading.

The visual condition of the pavement has been highlighted by many countries as an important indicator of the condition of the pavement layers. Non-destructive deflection tests can also be carried out to assess the condition of the lower layers.

Where structural deterioration has been detected in the hydraulically bound base, the pavement will not be suitable as part of a long-life semi-rigid pavement although it may be suitable to be upgraded as a long-life fully-flexible pavement. Careful consideration of the condition of the asphalt layers should be made before upgrading is performed.

Provided that the hydraulically bound base is suitable for consideration as a base within a long-life semi-rigid pavement, the most appropriate type of long-life strategy should be selected. It is likely
that such strategies will be selected on the basis of economic analysis. Depending on the strategy employed, both the pavement structure and/or pavement management strategy should be revised accordingly.
5 Maintenance

Pavement maintenance activities are carried out to restore the original condition and/or to delay a rehabilitation or reconstruction need. Proper and effective pavement maintenance techniques can significantly contribute to the actual long-life of pavements (COST 343: FEHRL, 2003)).

The maintenance requirements for semi-rigid (or flexible-composite) pavements are similar to fully-flexible pavements. Transversal reflection cracking has a major role in the degradation of semi-rigid pavements. If this crack type initiates at the surface of asphalt and is confined to the surfacing, only surface maintenance treatment is needed. However, the reflection cracking starting from the bottom of asphalt courses and appearing at the surface necessitates, sooner or later, upgrading; for this, the precondition of long-life is not fulfilled.

5.1 Condition evaluation of functional pavement surface characteristics

The asphalt wearing course of a new semi-rigid pavement structure should have excellent functional surface characteristics (evenness, favourable skid resistance, lack of surface defects and ruts, surface light reflectivity, low noise). These features gradually deteriorate as a consequence of traffic and environmental effects. If a condition parameter reaches the intervention level, an appropriate maintenance method is selected.

To choose the most suitable maintenance techniques, the present levels, and possibly the historical data of various condition parameters, should be known. Systematic pavement condition assessment can provide the necessary historical condition data. Maintenance assessment can be performed at network and project levels (FEHRL, 2004).

The network level condition assessment is usually done by high-performance measuring techniques and visual inspection. If the network level assessment identifies some pavement sections as candidates for maintenance-rehabilitation activities, detailed and/or more localized condition project level evaluation is required (FEHRL, 2004). The actual performance prediction models can be used to determine the frequency of monitoring rather than some regular system (FORMAT, 2004).

5.2 Criteria, prediction models and needs

The reason for establishing criteria at the network level is to provide an objective basis for identifying current needs and estimating future needs. At the project level, criteria have usually been in terms of specifications.

Development of good models for predicting performance has been a major challenge for pavement engineers. The prediction models can be deterministic or probabilistic (FEHRL, 2004).

The PARIS project (PARIS, 1999), financed partly by the European Union, created performance models of certain condition parameters for pavements, that included semi-rigid pavement
structures, in Northern, Central and Southern Europe, but models of reflective cracking were not covered which remains a knowledge gap.

The year in which a pavement section deteriorates to the minimum acceptable level would also be the action year for maintenance if sufficient funds are available. However, under conditions of limited resource, the action year may have to be deferred.

### 5.3 Maintenance management

The contribution of pavement surfacing maintenance to the long-life of the semi-rigid structure will only be fully effective if the maintenance activities are standardised and scientifically planned.

The management of cyclical (scheduled) and reactive works are treated separately. See details in the Phase 1 report (FEHRL, 2004).

Two maintenance management strategies can be differentiated for long-life semi-rigid pavements:
- ‘Prevention of cracks’ where, for example, special crack retarding techniques are applied to the pavement structure to delay pavement transverse cracking (at least for a longer time period),
- ‘Living with cracks’ where there is an acceptance of transverse cracks, and that they are considered a normal phenomenon that can be controlled by, for example, timely overbanding of cracks.

### 5.4 Maintenance treatment selection

Potential maintenance requirements for semi-rigid pavements, for the treatment of surface defects, are generally similar to those for fully-flexible pavements (Table 5.1). However, for semi-rigid pavements an obvious emphasis is given to crack sealing. Transverse reflection cracking is usually treated by overbanding. Crack sealing is essential to prevent the ingress of water and thereby maintain the bearing capacity of this kind of pavement.

The use of standard rules for treatment selection ensures that a consistent approach is taken to planning and specifying works throughout the road administration. This helps to ensure that funds are spent to greatest effect. Both scheduled and condition-responsive rules are available.

COST Action 343 (FEHRL, 2003) developed a flowchart procedure, Figure 5.1, to guide the engineer through the process of selecting the most appropriate pavement maintenance options for a particular project so that the selected maintenance treatment options have sufficient life time expectancy, while minimizing the disruption of traffic. It can help road practitioners to generate a list of possible treatment options suitable for maintaining a given road pavement taking into account the pavement condition and the prioritised performance properties for the road. Apart from these criteria, which give the best options from the point of view of efficiency and longevity, other criteria are also considered in the selection procedure to reduce road user delays during the application of maintenance treatments. The future performance requirements are also prioritised, taking into account the type of network, the traffic levels and other aspects related to the specific road site.
Figure 5.1. Flowchart procedure for maintenance treatment selection (after Cost 343 (FEHRL 2003))
The retarding of reflection cracking is an important consideration for maintenance of semi-rigid pavements. Several crack retarding treatments are in common use:

- overbanding of cracks (possibly following earlier v-shape crack replacement)
- bituminous membranes,
- non-woven interfaces (textiles),
- geogrids (synthetics, glass),
- steel meshes,

Such crack retarding treatments may not be necessary where pre-cracking is carried out during construction, as discussed in Section 3.

It needs to be born in mind that the success of a crack treatment will depend on the severity of the crack. For badly deteriorated cracks in which large vertical movements occur under traffic loading, deep excavation and reconstruction in the vicinity of the crack may be the best option.

Although the manner in which reflection cracks form in as-laid semi-rigid pavements is debatable. There is a large consensus view that they propagate upwards from the crack in the hydraulically bound base. On the other hand, there is strong evidence from the UK that reflection cracking in as-laid semi-rigid pavements, initiates at the surface immediately above the transverse crack in the cement bound base and propagate downwards to meet the crack in the concrete base. In as-laid pavements, these cracks initiate and propagate as a result of thermal movements at the crack and reflection cracks will form in the absence of traffic.

Once these cracks have completely penetrated through the asphalt layers, water can enter the crack and weaken the foundation layers. The vertical movements caused by wheel loads passing from one slab to another over the crack can cause a pumping action that will wash fines from the unbound foundation layers and undermine the cement bound layer near the crack. The lack of support can lead to secondary cracking close to the crack. These vertical movements will also erode interlock and load transfer across the crack.

Because of the traffic damage, reflection cracks will be more severe in the heavily trafficked lanes where deterioration will progress at a faster rate.

Once the reflection crack has penetrated through the full depth of the pavement, some deterioration of the nature described above will have occurred and the traffic induced vertical crack movements are likely to become more dominant compared to thermal movements. This will make crack maintenance more difficult and the success of any treatment is likely to depend on the severity of the crack to be treated. For this reason a method of rating crack severity and knowledge of the limitation of any treatment is required.
Table 5.1. Maintenance options for semi-rigid pavement structures  
(after COST 343 (FEHRL, 2003))

a) Strengthening treatments

<table>
<thead>
<tr>
<th>Treatment</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bituminous overlay ( &gt; 40 mm), hot mix</td>
<td>The rehabilitation of an existing pavement (generally through one or several additional layers) to increase its mechanical strength (bearing capacity) (hot mix).</td>
</tr>
<tr>
<td>Bituminous overlay (&gt; 40 mm), cold mix</td>
<td>The rehabilitation of an existing pavement (generally through one or several additional layers) to increase its mechanical strength (bearing capacity) (cold mix).</td>
</tr>
<tr>
<td>Bituminous overlay incorporating geogrids, geotextiles, SAMIs</td>
<td>The rehabilitation of an existing pavement (generally through one or several additional layers) to increase its mechanical strength (bearing capacity).</td>
</tr>
<tr>
<td>Deep cold in-situ recycling, plus new asphalt wearing course or overlay</td>
<td>The process of mixing in place the existing pavement with new binder – cement and/or bitumen – to improve the strength of the pavement.</td>
</tr>
<tr>
<td>Plant mix recycling</td>
<td>The process of planing/excavating the materials, processing them on site in a mixing plant and relaying through a paver.</td>
</tr>
<tr>
<td>Concrete overlay (&gt; 50 mm)</td>
<td>An overlay consisting of a cement concrete layer bonded to an existing pavement.</td>
</tr>
</tbody>
</table>

Note: Certain treatments may not be applicable in certain countries

Contd.
Table 5.1. Contd.

b) Treatments for surface deterioration

<table>
<thead>
<tr>
<th>Treatment</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Surface dressing</td>
<td>A surface treatment consisting of the successive laying of at least one layer of binder and at least one layer of chippings.</td>
</tr>
<tr>
<td>Crack sealing</td>
<td>The filling of cracks with a binder to prevent ingress of water.</td>
</tr>
<tr>
<td>Overbanding of cracks</td>
<td>The covering of cracks with a band of bituminous binder to prevent ingress of water.</td>
</tr>
<tr>
<td>Patching</td>
<td>The removal of localised areas of failed, poorly compacted or unsatisfactory materials from a road pavement, which is then replaced with selected compacted materials.</td>
</tr>
<tr>
<td>Pothole filling</td>
<td>The filling of a bowl-shaped hole of variable size in the pavement.</td>
</tr>
<tr>
<td>Thin bituminous overlay (&lt; 40 mm), hot mix</td>
<td>The rehabilitation of an existing pavement by adding a wearing course (hot mix).</td>
</tr>
<tr>
<td>Thin bituminous overlay (&lt; 40 mm), cold mix</td>
<td>The rehabilitation of an existing pavement by adding a wearing course (cold mix).</td>
</tr>
<tr>
<td>Bituminous inlay</td>
<td>The removal of the existing surface layer and replacing it with a new layer.</td>
</tr>
<tr>
<td>Slurry seal</td>
<td>A surface treatment consisting of a mixture of mineral aggregates, bitumen emulsion and additives, which is mixed and poured in place.</td>
</tr>
<tr>
<td>Recycling in-situ</td>
<td>Remix in place the existing wearing course and adding if necessary binder and aggregate to restore its properties.</td>
</tr>
<tr>
<td>Repaving</td>
<td>A process consisting in heating and scarifying a pavement, reshaping if necessary, adding a new mix and compacting.</td>
</tr>
<tr>
<td>Reshaping</td>
<td>An operation aimed at restoring the initial longitudinal or, more often, transverse profile of a pavement either by planing off or adding materials.</td>
</tr>
<tr>
<td>Rut filling</td>
<td>Filling of worn or deformed wheel paths.</td>
</tr>
<tr>
<td>Re-treading</td>
<td>A process consisting of scarifying a pavement and mixing the products of scarification with other aggregates or with a binder and placing and compacting the mixture thus obtained. Normally it is a cold process.</td>
</tr>
<tr>
<td>Re-texturing of surface</td>
<td>A mechanical impacting technique to improve the macro and/or micro texture.</td>
</tr>
<tr>
<td>Deep trenching of cracks</td>
<td>A process consists of full-depth trenching of severe transverse cracks, filling with compacted asphalt, planing the surfacing course from the whole road to obtain a level surface, and then replacing a new surface course.</td>
</tr>
<tr>
<td>Crack and seat</td>
<td>A process that consists of removing the asphalt layer, post-cracking the hydraulically bound base and then laying new asphalt layers.</td>
</tr>
<tr>
<td>Thin bonded concrete overlay (&lt; 50 mm)</td>
<td>An overlay consisting of a thin cement concrete layer bonded to an existing pavement.</td>
</tr>
</tbody>
</table>

Note: Certain treatments may not be applicable in certain countries
5.5 Conclusions

Efficient maintenance of road surfacing can have a significant role in ensuring that long-life pavements retain their long-life characteristics. The effectiveness of maintenance activities is improved by the reliable knowledge about every relevant condition parameter. Based on their measuring results, as well as applying well-established pavement performance models and intervention criteria, actual intervention needs can be determined.

The effective management of a proper combination of scheduled and reactive maintenance works is also a highly important task. The maintenance management can be ‘prevention of cracking’ or ‘living with cracks’. Maintenance techniques are to be selected taking into consideration the performance, longevity, efficiency and user delay issues (FEHRL, 2004).

5.6 Best practice

The following best practice advice for maintenance of long-life semi-rigid pavements can be given.

Every relevant condition parameter of pavement surfacing should be monitored using a frequency dependant on its rate of deterioration. Appropriate warning and intervention levels need to be defined to prevent surfacing conditions that may negatively influence the structural performance levels of underlying layers. The optimisation of maintenance considering also the minimum road closures should be preferred when maintenance techniques for implementation are selected.

The principal factor in determining the maintenance treatments for semi-rigid pavements is the type, extent and severity of cracking in the hydraulically bound base layer. For long-life semi-rigid pavements, only naturally forming cracks in the base are permitted; other cracks indicate some type of degradation in the structure. It is accepted that periodic transverse cracking can occur that is not necessarily structural deterioration. Transverse reflection cracking in isolation, at intervals of 5 m or more, generally indicates a pavement with a strong cement bound base which has cracked due to thermally induced strains, HD30 (DMRB 7.3.3).

Coring through the reflection cracks is recommended to determine whether the cracks are only in the surfacing (surface initiated) or through the full thickness of existing bituminous material. If cracking is confined to the surfacing, planing off the existing surfacing may be sufficient treatment prior to resurfacing; if badly cracked all of the existing bituminous materials may need to be removed.
6 Economics

6.1 Introduction

An economic assessment of the benefits of Long-Life Pavements (LLP) in comparison to Determinate Life Pavements (DLP), i.e. pavements that have a previously defined design life, may be achieved using an appropriate Cost Benefit Analysis (CBA) using a standardised methodology for each construction type.

In order to assess the economic benefits of long-life pavements, the following principal aspects should ideally be considered (this is a non-exhaustive list):

- initial construction costs
- costs of various maintenance treatments and the associated traffic management
- costs due to road user delays at road works
- costs due to accidents involving road users and workers at road work sites
- environmental economical impacts of road construction and maintenance, and
- residual value (loss of pavement capital value, or pavement deterioration).

The purpose of this Chapter is to provide economic justification for following a LLP construction option instead of a DLP, when the initial cost outlay for the former may be larger than for the latter. The evaluation is carried using a whole life cycle CBA model. (Obviously, the use of a CBA is only one possible route for assessing the benefits of an LLP, other possible methods could be based on a multi-criteria analysis.)

6.2 Cost Benefit Analysis (CBA) methodology

6.2.1 Selection of adapted CBA model

A review of existing CBA models which are suitable, or could be adapted, for determining an economic assessment of long-life pavements was undertaken in the Phase 1 of the ELLPAG project. The models identified were:

- The UK Highways Agency model,
- The OECD model (called PASI for Project Analysis System International) which is an adaptation of the earlier SAS model (UK),
- The FORMAT Project model.

However, it was required to select the model that could be most effectively adapted for the special case of long-life pavements. A general overview of major factors considered when making the selection is provided in Figure 6.1.
SAS model (UK) (Highways Agency, 2004)
This spreadsheet-based model was specifically developed for the economic assessment of treatment options for maintenance schemes on the UK’s core trunk road network and to identify the options that provide good value for money in whole-life cost terms. The model includes the capability to make allowances for user costs based on delay times, for residual value and for different treatments on each lane of the highway. Local background data can be entered or typical generic values from available data can be used. The user needs to enter a realistic maintenance regime for each of the pavement options being considered. All costs are discounted to a base year which allows a valid comparison of the Net Present Values (NPVs) for the options considered.

PASI model
The model is an adaptation of the SAS model, specially developed for whole life costing in a wide range of different environmental conditions extant in different countries. It was adopted by the OECD project investigating the potential for improving the life of long-life pavement surfacing; OECD Group, IM3 Group “Economic Evaluation of Long-Life Pavements” (OECD, 2005).

FORMAT model
The FORMAT model (FORMAT, 2004) is an integrated spreadsheet; so it means that each element can be examined and calculated separately. The maximum period of analysis offered by this model is presently limited to 30 years, which is a restriction. In addition, the FORMAT model is currently considered to be suitable for research purposes only. This fact does not fit in with the objective of the current described approach which is to provide a practical tool for the cost benefit analysis.

Selection of the analytical model
Each component (sub-model) of the PASI model is considered to be a “simple model”, i.e. it takes into account available data only and makes no assumption about missing data. In this case, where a comparative cost benefit analysis is required the results given by this model can be therefore considered acceptable. Therefore the PASI model seems to be the most suited for the CBA of
LLPs even though all the required elements, in particular environmental aspects, are not taken into account.

The FORMAT model can be used to provide data on environmental aspects, which are complex to quantify and they could vary a lot depending on the country. The results of the environmental component of the FORMAT model are more complex to justify because the standard inputs (elementary cost per ton of carbon concerning the fuel consumption impact, elementary cost per decibel concerning the noise impact, etc) could be very subjective. However, the environmental economic impacts relating to the construction and maintenance are expected to be very small in comparison to the construction costs.

In conclusion it was decided that a full CBA could be completed successfully in two stages by using both PASI and FORMAT. The suggested methodology for an analysis to determine if LLPs are a cost effective construction option, is shown schematically in Figure 6.2. Step 2 can only be completed if adequate environmental costs are available.

Figure 6.2. General methodology for undertaking a full CBA of LLPs
6.2.2 Applicability and assumptions

Future transport scenarios
Since the current analysis is being carried out on the basis of comparisons between different maintenance schedules, we have made the assumption that different future transport scenarios will not affect the comparisons significantly. In reality, some changes in transport policy could be expected that would alter the comparisons, e.g. traffic levels, but unless one has specific information about such changes, e.g. the introduction of new EC legislation, we cannot easily allow for this and have not attempted to do so within this study.

Applicability to different countries
A direct comparison of CBAs for the same activities/maintenance schedules from a range of European countries is not feasible, as the input data and therefore the results will be heavily dependent on the circumstances operating within each country. In certain situations predictions could be so different that LLPs may be economically justified in some countries but not in others, though this is thought to be unlikely within Europe. Primary sources of such discrepancies may be the availability of resources, a different pricing structure and different environmental conditions etcetera.

A basis for comparison
In order to demonstrate how a CBA analysis might be used and to illustrate the effect of factors such as different maintenance schedules for LLPs and DLPs, example schedules and costs etc. are provided from the UK’s experience of motorway management. (The term motorway as used in this report is in accordance with the definition provided within the Technical Directory of road terms (WRA, 2007) that is widely accepted throughout Europe.)

To enable readers of this report to more easily compare costs with those typical within their own countries, all costs in the following analyses are expressed in Euros and not GB pounds. Nevertheless readers are reminded that they should consider the analyses as an indicative exercise, which enumerates the aspects that should be considered when undertaking an economical comparison between a LLP and a DLP. These analyses are not a precise indication of the benefits that would be accrued by using LLPs in another country.

Assumptions
This cost benefit analysis focuses on Fully-Flexible (FF) and Semi-Rigid (SR) pavements from the end of construction onwards. In other words, the construction costs have not been explicitly included in this evaluation. However, when two potential solutions are being considered the likely difference between the initial construction costs should also be taken into account. It is assumed that all the maintenance treatments considered in this analysis occur exclusively in the bituminous layers for both types of pavements. Therefore, provided any degradation of the bituminous layers is not as a result of deterioration of the lower layers, a single CBA will be applicable to both FF and SR pavements.

6.3 Cost Benefit Analysis using PASI model

The main objective of this CBA is to assess the possible economic savings from the use of LLPs rather than DLPs. This is achieved by examining the effect on the whole life cost of the pavement due to variations in the value of certain key input parameters.
The first step to undertaking a CBA is to identify key factors which significantly affect the results of the analysis. The impact on the results by changes to these factors is dependent on changes to parameter values.

The CBA is undertaken by varying the values of these parameters above and below pre-selected base values that form a benchmark for comparative purposes, to evaluate the affect of these variations on the final cost. Different combinations of the parameter values are termed scenarios.

6.3.1 Development of PASI simulation scenarios

The primary source of expenditure for operating a highway over a given time period, is the maintenance cost that in turn is a function of the volume of traffic and the proportion of heavy goods vehicles (HGVs). Key parameters affecting traffic flows and maintenance costs are discussed in the following Sections; these are summarised schematically in Figure 6.3.

Other factors such as the discount rate and the time period covered review must be defined before any analyses may be completed.

6.3.1.1 Time period covered by the analysis

The analysis period is limited to 50 years in the PASI model. This duration seems to be a sufficient value to assess the different design options, and is used for all the analyses described within this Chapter. However, an option to increase this parameter, for further versions of the model, should be considered.

6.3.1.2 Discount rate

The discount rate is defined as the rate at which costs and benefits occurring in the future are converted to net present values. The discount rate may be expressed either in nominal terms where both the effects of inflation and real earning power of money are reflected or in real terms where the effects of inflation are excluded. Nominal discount rates are normally used in whole life cycle cost analyses.

The discount rate has a significant influence on the CBA results, especially for long analysis periods. Therefore, the rate must be carefully selected at a realistic level and should be considered as a variable of the analysis. The base or benchmark discount rate assumed by the CBA in this study was 5 per cent.

Discount rates of the 3, 5 and 7 per cent were considered as part of the analysis.
6.3.1.3 Design traffic

The design of a DLP pavement is based on a consideration of the number of standard axles which (theoretically) can be supported by the pavement before failure occurs and re-construction of the whole pavement structure is required. (The selection of an appropriate load equivalence relationship to accounting for damaging to the pavement is discussed in Section 2.) In the UK, the maximum asphalt layer thickness is designed to support 80 msa of 80 kN standard axles; however the base traffic value used in the CBA was 60 msa at 80 kN.

The relation between the design number of standard axles and the required asphalt layer thickness, reproduced from HD 26 (DMRB 7.2.3) is presented in Figure 6.4.
The base case for design traffic for the CBA is defined by:

- Design traffic: 60 msa of 80 kN standard axles,
- Annual growth rate: 1.0 %,
- Heavy goods vehicles proportion: 12.0 % of the total traffic flow,
- Traffic flow (AADT): 75,000 vehicles per day.

An understanding of the sensitivity of the CBA to traffic variations is achieved by varying one parameter at a time, either side of the base case values. The selected variations of the traffic parameters (including the base cases in **bold**) considered by the CBA are:

- Annual growth rate: 0 %, 1 % and 2 %,
- Heavy goods vehicles: 5 %, **12 %** and 20 %,
- Traffic flow (AADT): 50,000, **75,000** and 100,000.

In the United Kingdom AADTs in excess of 100,000 are normally accommodated on three lane dual carriageways. Therefore one traffic combination also considered is that of 100,000 vehicles/day carried by a dual three lane carriageway.

The different combinations of the variation in traffic parameters used in the analyses are summarised in Table 6.1.
Table 6.1. Sensitivity Cases for the Traffic Density parameter

<table>
<thead>
<tr>
<th>Growth rate (%)</th>
<th>Proportion of HGVs (%)</th>
<th>AADT#</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dual two lane motorway</td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>12</td>
<td>75,000</td>
</tr>
<tr>
<td>1</td>
<td>12</td>
<td>50,000</td>
</tr>
<tr>
<td>1</td>
<td>12</td>
<td>100,000</td>
</tr>
<tr>
<td>1</td>
<td>5</td>
<td>75,000</td>
</tr>
<tr>
<td>1</td>
<td>20</td>
<td>75,000</td>
</tr>
<tr>
<td>0</td>
<td>12</td>
<td>75,000</td>
</tr>
<tr>
<td>2</td>
<td>12</td>
<td>75,000</td>
</tr>
<tr>
<td>Dual three lane motorway</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>12</td>
<td>100,000</td>
</tr>
</tbody>
</table>

# Annual average daily traffic (AADT)

(The base case is highlighted in bold and variations from the base case are highlighted)

6.3.1.4 Maintenance schedules

Maintenance schedules for the service-life of a pavement represents a considerable expenditure and therefore must be considered as part of the CBA.

Initially a schedule was defined for both the long-life and determinate life pavements termed LLP1 and DLP1 respectively. Details of the maintenance schedules are shown in Table 6.2. For LLP1 it has been assumed that the only maintenance required is a regular replacement of the wearing course, i.e. an inlay of 40 mm every ten years, throughout the fifty year analysis period. For DLP1, it has been assumed that the pavement is initially designed for 20 year life with a strengthening overlay of 50 mm at that point and additional replacement of the wearing course at interim 10 year intervals. Eventually reconstruction is likely to become necessary within the 50 year analysis period but this can be delayed five years since the objective at this stage is not to preserve the asset.

Further maintenance schedules for determinate life pavements, DLP2 and DLP3, were defined to investigate the effect on the CBA of different maintenance tasks; the schedules are identified in Table 6.2. DLP2 continues to represent a basic 20 year design but the overlay has been delayed two years thereby necessitating an additional plane out and replacement of the top surfacing layer before overlaying. Such less than ideal treatment leads to a subsequent shorter surfacing life necessitating a further resurfacing in nine rather than ten years and a final partial reconstruction and overlay after a total of 45. DLP3 represents a basic 40 year design with alternate 40 mm and 100mm inlays before applying a strengthening overlay after 40 years and a final surface replacement at 50 years.

In addition two further maintenance schedules for long-life pavements, LLP2 and LLP3, were defined to investigate to examine the effect of different likely maintenance options; the schedules are identified in Table 6.2. However, their costs were only determined for the base condition.

Finally, because of the importance to the long-life design concept that the upper pavement layers maintain their integrity and protect the lower structural layers, two more maintenance schedules (LLP4 and LLP5) were defined for long-life pavements; these simulated a necessity to replace the
upper layers more frequently than for a determinate life pavement. The schedules are identified in Table 6.2, and were tested for all the scenarios considered by the CBA, except number five.

**Table 6.2. Maintenance schedules and work patterns for DLP and LLP for the first 50 years of service life**

<table>
<thead>
<tr>
<th>Maintenance schedule</th>
<th>Determinate life pavements</th>
<th>Long-life pavements</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>DLP1</strong></td>
<td>Year in service</td>
<td>Year in service</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>10</td>
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</tr>
<tr>
<td></td>
<td>20</td>
<td>20</td>
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<td>30</td>
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<tr>
<td></td>
<td>45</td>
<td>40</td>
</tr>
<tr>
<td>Maintenance task</td>
<td>NC</td>
<td>NC</td>
</tr>
<tr>
<td></td>
<td>IN40</td>
<td>IN40</td>
</tr>
<tr>
<td></td>
<td>OL</td>
<td>OL</td>
</tr>
<tr>
<td></td>
<td>IN40</td>
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</tr>
<tr>
<td></td>
<td>RC</td>
<td>OL</td>
</tr>
<tr>
<td></td>
<td></td>
<td>IN40</td>
</tr>
<tr>
<td><strong>DLP2</strong></td>
<td>Year in service</td>
<td>Year in service</td>
</tr>
<tr>
<td></td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td></td>
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<tr>
<td></td>
<td>IN40</td>
<td>IN40</td>
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<tr>
<td></td>
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</table>

**Key of abbreviations:**

**Abbreviation** | **Maintenance task and methodology**
---|---
NC | New construction
IN40 | Inlay 40 mm:- Resurface to same level
IN100 | Inlay 100 mm - Plane out 100 mm and replace
OL | Overlay:- P/O and overlay 50 mm
RC | Reconstruction, 300 mm
TS | Overlay with a thin surfacing, 25 mm
P/O 100 / 200 | Plane off and overlay 100 / 200 mm

**Dual 2 lane motorways (D2M)**

- NC: New construction
- IN40: Inlay 40 mm:- Resurface to same level
- IN100: Inlay 100 mm - Plane out 100 mm and replace
- OL: Overlay:- P/O and overlay 50 mm
- RC: Reconstruction, 300 mm
- TS: Overlay with a thin surfacing, 25 mm
- P/O 100 / 200: Plane off and overlay 100 / 200 mm

**Dual 3 lane motorways (D3M)**

- NC: New construction
- IN40: Inlay 40 mm:- Resurface to same level
- IN100: Inlay 100 mm - Plane out 100 mm and replace
- OL: Overlay:- P/O and overlay 50 mm
- RC: Reconstruction, 300 mm
- TS: Overlay with a thin surfacing, 25 mm
- P/O 100 / 200: Plane off and overlay 100 / 200 mm
6.3.2 Output from the CBA

The CBA determines the NPV (€k) of the cost of constructing and operating a dual two lane highway. The costs are broken down as follows.

Work Costs
- Maintenance

User Costs.
- Accidents
- Time delays
- Vehicle operation

6.3.3 Determination of scenarios for analysis

Simulation scenarios for investigation by the CBA were derived from different combinations of the parameters described in Section 6.3.1. Combinations that would not yield useful information were not tested.

The scenarios tested by the CBA are shown in Table 6.3. In the table, the base condition (Scenario 1) is shown in italics type.

Table 6.3. List of scenarios analysed in the PASI CBA

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Maintenance schedule</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>DLP1, 2, 3 and LLP1, 2, 3, 4, 5</td>
</tr>
<tr>
<td>2</td>
<td>DLP1 and LLP1</td>
</tr>
<tr>
<td>3</td>
<td>DLP1 and LLP1</td>
</tr>
<tr>
<td>4</td>
<td>DLP1 and LLP1,4,5</td>
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<tr>
<td>5</td>
<td>DLP1 and LLP1,4</td>
</tr>
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<td>6</td>
<td>DLP1 and LLP1,4,5</td>
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<td>7</td>
<td>DLP1 and LLP1,4,5</td>
</tr>
<tr>
<td>8</td>
<td>DLP1 and LLP1,4,5</td>
</tr>
<tr>
<td>9</td>
<td>DLP1,2,3 and LLP1,4,5</td>
</tr>
<tr>
<td>10</td>
<td>DLP1 and LLP1,4,5</td>
</tr>
</tbody>
</table>

6.3.4 Results of the CBA

The complete results from the CBA are summarised in Table 6.4. For each scenario the Table identifies the maintenance schedule and provides details of the Works Cost, User Costs and Total Cost for each scenario. All costs are expressed in Euros and are discounted to the Net Present Value (NPV).

The results are further presented in alternative tabular and graphical form to emphasise different aspects of the data.
These alternative tabulated results provide a ready appreciation of the relative cost benefit that may be achieved from using long-life pavements by presenting the ratio of total cost ($\frac{\text{DLP}}{\text{LLP}}$) for the different maintenance schedules.

- The ratios of the total cost of DLP1 to LLP1, and both DLP2 and 3 to LLP1, are presented in Table 6.5.
- The ratios of the total cost of DLP1, 2 and 3 to both LLP2 and LLP3 are presented in Table 6.6.
- The ratios of the total cost of DLP1, 2 and 3 to both LLP4 and LLP5 are presented in Table 6.7.

The results are presented in the form of a bar chart for each parameter varied within the CBA, to compare the total cost of the maintenance schedules tested. Note that the proportion of Works Costs and User Costs for each cost total are illustrated by a different pattern on the data bars.

- The total cost of each maintenance schedule, determined for the base conditions, is presented in Figure 6.5.
- The total costs of the maintenance schedules DLP1 and LLP1, for the different discount rates, are presented in Figure 6.6.
- The total costs of the maintenance schedules DLP1, LLP1, LLP4 and LLP5, for the different values of AADT, are shown in Figure 6.7. For comparison, an AADT = 100,000 for a dual 3 lane motorway (Scenario 11) is also shown in this Figure.
- The total costs of the maintenance schedules DLP1, LLP1, LLP4 and LLP5, for different percentages of HGVs, are presented in Figure 6.8.
- The total cost of maintenance schedules DLP1, LLP1, LLP4 and LLP5, for different the traffic growth rates, are presented in Figure 6.9.
### Table 6.4. Results of the PASI simulations for the different scenarios

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Maintenance schedule</th>
<th>Works cost [€k]</th>
<th>User costs [€k]</th>
<th>Total cost [€k]</th>
<th>Variation from parameter base value</th>
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<td>19,899</td>
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<td>(Base conditions)</td>
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<td>6,699</td>
<td>7,652</td>
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<tr>
<td></td>
<td>LLP3</td>
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<td>10,272</td>
<td>11,631</td>
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<tr>
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<td>14,696</td>
<td>16,523</td>
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<tr>
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<td>LLP5</td>
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The base conditions for the CBA were: Discount rate = 5%  
AADT = 75,000 vehicles per day  
HGVs = 12%  
Growth Rate = 1%

All costs are expressed in Euros and are discounted to the Net Present Value (NPV)
Table 6.5.  Ratio of total cost DLP1, 2 and 3 to LLP1

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<th>Parameter value</th>
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<th>Ratio of total cost DLP2/LLP1</th>
<th>Ratio of total cost DLP3/LLP1</th>
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<td>7%</td>
<td>2.2</td>
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<tr>
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<td>2.2</td>
<td>-</td>
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<td>100,000</td>
<td>2.4</td>
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<td></td>
<td>20%</td>
<td>2.4</td>
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<td>9</td>
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Table 6.6.  Ratio of total cost DLP1, 2 and 3 to LLP2 and LLP3

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<th>Parameter</th>
<th>Parameter value</th>
<th>Ratio of total cost DLP1/LLP2</th>
<th>Ratio of total cost DLP2/LLP2</th>
<th>Ratio of total cost DLP3/LLP2</th>
<th>Ratio of total cost DLP1/LLP3</th>
<th>Ratio of total cost DLP2/LLP3</th>
<th>Ratio of total cost DLP3/LLP3</th>
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</thead>
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<td>1.7</td>
<td>1.7</td>
<td>1.4</td>
</tr>
</tbody>
</table>

Table 6.7.  Ratio of total cost DLP1, 2 and 3 to LLP4 and LLP5

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Parameter</th>
<th>Parameter value</th>
<th>Ratio of total cost DLP1/LLP4</th>
<th>Ratio of total cost DLP2/LLP4</th>
<th>Ratio of total cost DLP3/LLP4</th>
<th>Ratio of total cost DLP1/LLP5</th>
<th>Ratio of total cost DLP2/LLP5</th>
<th>Ratio of total cost DLP3/LLP5</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>All</td>
<td>Base</td>
<td>1.2</td>
<td>1.2</td>
<td>1.0</td>
<td>1.9</td>
<td>1.9</td>
<td>1.6</td>
</tr>
<tr>
<td>2</td>
<td>Discount rate</td>
<td>3%</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td></td>
<td>7%</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>AADT</td>
<td>50,000</td>
<td>1.1</td>
<td>-</td>
<td>-</td>
<td>1.7</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>100,000</td>
<td>1.1</td>
<td>-</td>
<td>-</td>
<td>1.7</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>6</td>
<td>HGVs</td>
<td>5%</td>
<td>1.1</td>
<td>-</td>
<td>-</td>
<td>1.8</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>7</td>
<td></td>
<td>20%</td>
<td>1.2</td>
<td>-</td>
<td>-</td>
<td>1.9</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>8</td>
<td>Growth rate</td>
<td>0%</td>
<td>1.1</td>
<td>-</td>
<td>-</td>
<td>1.6</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>9</td>
<td></td>
<td>2%</td>
<td>1.3</td>
<td>1.3</td>
<td>1.1</td>
<td>1.9</td>
<td>1.9</td>
<td>1.6</td>
</tr>
<tr>
<td>10</td>
<td>3DM</td>
<td>100,000</td>
<td>1.6</td>
<td>-</td>
<td>-</td>
<td>1.8</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
Figure 6.5. Total cost for each maintenance schedule for the base conditions

Figure 6.6. Variation in total cost with discount rate, for the maintenance schedules shown
Figure 6.7. Variation in total cost with changes to the AADT, for the maintenance schedules shown. (Note: the costs for LLP5 were not determined for AADT = 100,000)

Figure 6.8. Variation in total cost with changes to the per cent of HGVs, for the maintenance schedules shown
6.3.5 Discussion of the results

User and Work Costs

(i) Works Costs
The construction of a LLP, particularly the lower layers, is critical to a successful long-term performance. Therefore, construction costs for a LLP may well be slightly higher than for DLP due to higher material costs and closer supervision of construction operations; however, the cost differential is likely to be less than 10 per cent more than the construction cost of a DLP.

The onerous maintenance schedules for long-life pavements LLP 4 and 5 cause their user costs to rise slightly above that for DLPs, but again the cost differentials are small.

The results of the CBA show that User and Works Costs typically comprise 88 and 12 percent of the total costs respectively. Therefore the variations in the Works Costs do not make any significant difference to the total cost of each Scenario.

(ii) User costs
On the presumption that accidents are equally likely to happen on DLPs as LLP, and that vehicle operating costs will also be the same for either pavement type, variations in user costs will be dominated by journey delays resulting from lane closures for maintenance works and increased traffic flows.
In all instances, the user cost for DLP1 is substantially greater than for LLP1, despite the fact that the LLP requires one more maintenance operation than the DLP in the 50 year service life considered. This wholly reflects the disruption to traffic flows and subsequent delays imposed by the 24 hour closures necessary to carry out overlaying and reconstructions operations, which become exacerbated by increased traffic flow. The increases are evident for increase in AADT and the annual growth rate for traffic, as shown in Figures 6.7 and 6.9 respectively. Whereas the increasing proportion of HGVs, has comparatively little effect on the total cost of the maintenance schedules, as shown in Figure 6.8.

**Total cost (NPV)**

The results of the CBA presented in Table 6.5 show unequivocal evidence of the substantial cost savings that can be accrued from the use of LLPs rather than DLPs; the data show that the cost of maintenance schedule DLP1 is between 2.2 and 2.7 times the cost of the LLP1. These cost differentials are pictorially represented in Figures 6.6 to 6.9.

The alternative maintenance schedules for determinate life pavements do not detract from the obvious benefits that LLPs can offer. The CBA results for maintenance schedule DLP2 are similar to DLP1 and the cost ratios shown in Table 6.5 are the same. The maintenance schedule DLP3 however, is almost 20 per cent cheaper than DLP1, and thus slightly reducing the apparent benefits of using LLPs; nevertheless the total cost of operating the maintenance schedule is about twice that for the LLP1.

Alternative maintenance schedules proposed for the long-life pavements (LLP2 and LLP3) were not fully tested by the CBA, only the costs for the base conditions were determined. The data presented in Table 6.6 show that LLP3 was a factor of about 1.5 times more expensive than LLP2; nevertheless both maintenance schedules were significantly cheaper than all the DLP maintenance schedules; the cost ratio ($\frac{DLP}{LLP}$) for the different schedules ranged between 1.4 and 2.6.

The performance of long-life pavements is dependent on the robustness and continued integrity of the foundation layers which should be protected by surface layers that are subject to strict maintenance schedules. The potential requirement for more onerous and expensive maintenance schedules was recognised and considered as part of the CBA by introducing LLP4 and LLP5 maintenance schedules. The cost of LLP4 for the base condition (see Figure 6.5) and for the other Scenarios (see Figures 6.7 to 6.9) is about twice that for LLP1; nevertheless, as shown in Table 6.7, that LLPs still offer an overall saving of between 10 and 60 per cent over the use of DLPs. The maintenance schedule LLP5 is approximately 75 per cent cheaper than LLP4 and the cost advantages ranges from 1.6 to 1.9.

**Dual three lane motorway**

For this motorway type, only one Scenario was considered, i.e. an AADT of 100,000 and a growth rate of 2%. As for many of the two lane results, long-life pavements with maintenance schedules LLP1, LLP4 and LLP5 were compared with the DLP1 scenario. LLP1 shows an enormous benefit compared with the determinate life scenario, DLP1, by a factor of over seven. This is because it is considered that the required maintenance treatment for LLP1 of single lane 40 mm inlays can be achieved overnight, without any significant interruption to the lower vehicle flows encountered at this time, thus generating relatively low user costs due to maintenance operations.

**Discount rate**

Changes in total cost resulting from changes in the discount rate were only examined for maintenance schedules DLP1 and LLP1, the results are presented in Table 6.5 and Figure 6.6. The CBA demonstrated a slight decrease in the cost ratio ($\frac{DLP}{LLP}$) with increasing discount rate; the ratio ranged from 2.7 to 2.2 for the discount values tested.

**Design traffic**

For the remaining factors, all traffic related, the data exhibited in Figures 6.7 to 6.9 show very similar trends in the relative costs. For each factor considered the results show that the costs values are in the order DLP1 > LLP4 > LLP5 > LLP1.
Typically the maintenance schedule DLP1 is:

- 2.4 times more expensive than LLP1
- at least 1.5 times LLP5, and
- 10 per cent more than LLP4

It is very clear from the results of the CBA that considerable savings may be generated by the use of LLPs and that the benefits are not compromised by fluctuations in traffic density. This suggests that a high level of confidence may be placed in the potential for a regular accrual of savings. The degree of saving is dependent on the nature of the maintenance schedule, but savings of up to 60 per cent are achievable.

**Limitations of the analytical model and the CBA**

The analytical model was selected as the most appropriate to the study in hand, but it was derived for DLPs and does not take full advantage of the reduced costs associated with LLPs.

DLPs need to be re-constructed at intervals and yet the model does not fully take an account of the residual worth of the DLP. Nor does it fully take account of the indefinite service life of the LLP. Therefore the CBA could be said to be unintentionally biased in favour of DLPs and against LLPs.

Experience has shown that DLP maintenance requirements are sometimes often worse than those assumed by the test Scenarios, e.g. the model assumes that reconstruction will only be required for Lane 1, whereas in reality sometimes Lane 2 will also require reconstruction.

The frequency of resurfacing (inlay) for LLPs has been assumed to be the same as for DLPs. However, LLP may not need re-surfacing as often as DLP due to a firmer foundation and a resulting reduced deformation. But it could be argued that LLPs require more frequent resurfacing to ensure there is no degradation of the foundation layers. Further research is required to clarify this requirement.

The CBA was conducted using typical costs for maintenance operation schedules that are typical for the UK. Therefore it may not precisely reflect conditions in other European countries. Nevertheless the trends identified in this study are likely to be applicable in most developed countries. Where precise information is required, CBAs should be undertaken using country specific data.

**Applicability of LLPs**

LLPs offer potential savings of up to 60 per cent over the use of DLPs. However, LLPs are not a panacea for all ills and there will be schemes where their use would be inappropriate. The successful performance of a LLP is crucially dependent on the quality of the foundation. On areas of very poor ground the additional expenditure required to improve the ground conditions to an acceptable level, may not be deemed appropriate and the designer could well use a DLP design irrespective of predicted future savings.

### 6.4 Conclusion

Different scenarios were examined to study the affect of variability of traffic density and discount rate (used to determine the NPV). Variations in the traffic density were achieved by independently varying the following factors:

- annual average daily traffic,
- proportion of heavy goods vehicles,
- traffic growth rate.
The results of the cost benefit analysis demonstrated that long-life pavements (LLPs) are substantially cheaper to operate and maintain than determinate life pavements (DLPs). For the 50 year period tested, the cost savings (calculated at the net present value, NPV) for the works and user costs of a DLP are likely to be as much as 2.4 times that for a LLP.

The precise level of saving will depend on local conditions and the maintenance schedule operated. The results of additional analyses, with different maintenance schedules specifically intended to (i) reduce the operating costs for DLPs, and (ii) increase the operating costs for LLPs, continued to demonstrate that LLPs were more cost efficient.

It is very clear from the results of the CBA that considerable savings may be generated by the use of LLPs and that the benefits are not compromised by fluctuations in traffic levels. This suggests that a high level of confidence may be placed in the potential for a regular accrual of savings.

The results and the conclusions from this study are considered to be broadly relevant to both fully flexible and semi-rigid constructions. Although the detailed maintenance techniques will in some cases be quite different, considerable cost savings are anticipated through the use of LLPs. The results are not considered applicable to rigid (concrete) pavements and will be considered in an alternative manner in the Phase 3 Report.

The use of LLPs is a very sustainable option and imposes less impact on the environment, by reducing the requirement for natural resources and reducing the carbon footprint of highway network maintenance operations.

At the current time the UK highway network operates many thousands of kilometres of pavements constructed in accordance with long-life principles, and is the only European country that has wide experience of acknowledged LLPs. Some other countries are actively considering their use, but at this time DLPs remain the most widely used.

The current limited use of LLPs across Europe is considered to result from a dearth of published information and experience on this subject. This may be compounded by the requirement to construct a robust high quality foundation to the pavement and the perceived additional costs that this would incur. It is hoped that pan European co-operative research programmes, such as the ELLPAG Project, will promote advances in pavement engineering.

The use of LLPs is a very sustainable option that imposes less impact on the environment by reducing the requirement for natural resources and reducing the carbon footprint of highway maintenance operations. However, despite their importance the economic analyses did not include a consideration of any environmental factors due to the paucity of data. The current lack of appropriate methods for quantifying the environmental and economic benefits of long-life pavements does not permit a true assessment to be made. In certain situations predictions could be so different that LLPs may be economically justified in some countries but not in others. Primary sources of such discrepancies might be the availability of resources, a different pricing structure and different environmental conditions. Once appropriate methods for quantifying the environmental and economic benefits of long-life semi-rigid pavements have been established a more wide-spread adoption of long-life pavements is possible.

### 6.5 Recommendations

The work reported in this Section of the report demonstrates clear benefits for the use of LLPs. Recommendations to promote the use of LLPs within Europe are outlined below.

1. Disseminate the knowledge of potential benefits of LLPs to engineers in all European countries, by undertaking a series of workshops and seminars by the members of ELLPAG.
Funding for this will be sought through the 7th Framework Programme for General Research, funded by the European Commission.

2. European countries (other than the UK) are recommended to consider the use of LLPs through undertaking similar cost benefit analyses (CBAs) with maintenance schedules and costs that are directly applicable to their own national practices.

3. Practical experience of LLPs is currently limited to the UK, where LLPs have been used extensively on major roads for over 10 years. It is recommended that full scale trials are undertaken by other European countries to investigate the concept and performance of LLPs. Such trials will generate greater confidence in the benefits of LLPs and promote their use.
7 Research Needs

A primary purpose of ELLPAG is to pool the available knowledge on long-life pavements. In many cases, this action will satisfy many of the gaps in knowledge that may be identified in a particular country; however there are cases where additional knowledge is required. The contributors to the ELLPAG project individually identified a large number of research needs in the area of long-life semi-rigid pavements. Five common themes were identified:

- Understanding the mechanism of reflection cracking,
- Improved techniques to control reflection cracking,
- Effects of traffic,
- Optimisation of maintenance,
- Economic analysis.

More than 40% of the research ideas concerned the improved understanding and prevention of reflection cracking. This is not surprising since, for semi-rigid pavement, reflection cracking is one of the main modes of deterioration and within the ELLPAG group, there has been lively debate regarding the actual mechanism for its formation.

Following the compilation of research ideas, recommendations for further research have been produced to reflect each of the five themes listed above. The recommendations are entitled:

- Investigation of the nature of reflection cracking in semi-rigid pavements,
- Assessment of techniques for the control of reflection cracking,
- Examination of the effect of traffic on semi-rigid pavements,
- Development of economic analysis tools,
- Optimisation of maintenance strategies for long-life semi-rigid pavements.

These recommendations for future research are examined in Section 8.
8 Recommendations

8.1 Reflection cracking

Reflection cracking is a major determinant of the performance of semi-rigid pavements. In as-laid pavements its development is controlled by temperature. Therefore, as part of any research study it would be advisable to carry out any investigations in climatic regions that cover temperate and the more extreme climatic conditions of eastern Europe.

8.1.1 Investigation of the nature of reflection cracking in semi-rigid pavements

Background

There has been debate regarding the nature of reflection cracking in semi-rigid pavements. Some countries take the traditional view that reflection cracking is caused by a fatigue mechanism which results in cracking from the bottom of the asphalt layer upwards; other countries claim to have observed top-down cracking in thick asphalt layers.

The nature of reflection cracking influences the choices for design and also for maintenance. For long-life pavements, the case of maintenance is more important since understanding the nature of reflection cracking will have a significant impact on how the asphalt layer is treated, particularly the amount of material that is replaced due to the presence of reflection cracking.

Objectives

The objectives of this research proposal are as follows.

- To determine whether cracks occur from the top-down or from the bottom-up;
- To show the existence of a thickness threshold above which only top-down cracking is seen;
- To appreciate the propagation speed of reflection cracking.

Methodology

This project will be split into two tasks:

Task A) Investigation of top-down cracking

This task involves the survey of semi-rigid pavements showing cracking on the surface. The investigation will be conducted Europe-wide on a range of semi-rigid pavements that could be considered long-life pavements. The cracks will be sampled by coring or a calibrated non-destructive technique to determine the nature of cracking and the crack depth; the age and properties of the surfacing will also be recorded.
Task B) Determination of the speed of propagation of cracking in semi-rigid pavements

This task will augment Task A using non-destructive crack-depth measuring techniques. Non-destructive testing (NDT) permits a greater number of cracks to be assessed on a regular basis. The NDT will be calibrated on the sites reviewed in Task A, and these sites will be visited on a bi-annual basis so that cracks at precise locations can be retested. This task will record the chronological development of crack depth and the time of year that these cracks are active. The results of this Europe-wide study will be pooled and the results will be analysed by age, material properties, climate, traffic and pavement construction.

Benefits

This work is fundamental for the understanding of cracking in long-life semi-rigid pavements. Establishing this understanding is an essential precursor in formulating other research recommendations for long-life semi-rigid pavements.

8.1.2 Assessment of techniques for the control of reflection cracking

Background

There are a number of techniques which are claimed to control the onset of reflection cracking including pre-cracking of the hydraulically bound base and stress-absorbing membrane interfaces (SAMIs). Historically, the demonstrations of the performance of reflection crack treatments have been variable and uncertainty still remains about which treatments provide positive benefits, or indeed economic benefits, if any at all.

This research should be carried out following the completion of research confirming the nature of reflection cracking.

Objective

The objective of this research is to identify the most appropriate treatments to control reflection cracking through the evaluation of the available techniques, and to understand the limitations of the various methods.

Methodology

This study should be conducted by a number of European partners in parallel. It is likely that historical studies of these crack treatments have been influenced by random effects and national differences in the materials and constructions used. Running the trials in parallel will reduce the impact of random effects. Carrying out the tests in a number of locations may help explain where local differences have affected the results of previous trials.

The success of any treatment depends on the severity of the crack being treated. Therefore, the development of a method of characterising the severity of cracks should be considered. This will help to identify the range of conditions under which the various treatments are likely to be successful.

Ideally, the trials should be carried out using a full-scale trial on the road network. If this approach is to be adopted, the final results of the study may not be available for many years; however, interim reports on the performance of the trials should be regularly produced in order for the latest views to be immediately disseminated to European industry.

Where full-scale trials are not possible, pilot scale trials may be considered to establish the relative benefits of these techniques under traffic loading. However, it should be noted that these tests will be able to assess the impacts of traffic but will not be able to assess the long-term effect of environmental change.
Benefits

In order to optimise the benefits of long-life semi-rigid pavements, the frequency of maintenance of the surface can be reduced provided that occurrence of reflection cracking is slowed. Moreover, slowing the progression of reflection cracking provides further assurance that the structure of the long-life semi-rigid pavement is neither compromised through the ingress of water nor by the deterioration of the structural properties of the asphalt layer above the naturally forming cracks in the hydraulically bound layer.

8.2 Examination of the effect of traffic on semi-rigid pavements

Background

The wearing effect of traffic loading on pavements is usually accommodated in design by using a load-equivalence principle. Compared to flexible pavements, semi-rigid pavements are thought to be more sensitive to the traffic loading, this is demonstrated by some European countries using a much larger exponent for load-equivalence for semi-rigid pavements than for fully-flexible pavements. The wide range of load-equivalence rules in use in European countries suggests that there is some uncertainty about the actual effect of traffic on semi-rigid pavements. This is a fundamental requirement for the quantification of traffic loading for both the design of new pavements and for their maintenance.

Objective

To evaluate the effect of traffic on the structure of semi-rigid pavements so that the design methods in a number of European countries can be easily compared and the elements which form long-life pavements can be more readily communicated between countries.

Methodology

This study will collect information on the methods of assessing traffic on semi-rigid pavements and the design assumptions associated with this traffic assessment. Laboratory testing and pilot-scale testing will be used to examine the traffic assessment assumptions with due regard to the prevalent traffic conditions within national boundaries e.g. legal axle loads.

Benefits

Recommendations for traffic characterisation on semi-rigid pavements will lead to more efficient construction and maintenance. Errors in the quantification of traffic lead to inefficiency and improvements will enhance the management of the road asset.

8.3 Development of Economic Analysis Tools

Background

The lack of an appropriate model for the cost benefit analysis of long-life fully-flexible pavements was identified in the ELLPAG Phase 1 report (FEHRL, 2004); this review has also identified a need for semi-rigid pavement structures. The work in ELLPAG looks to the development of a suitable model in both the short and the long term. The model should adequately account for users and
environmental benefits. This research project will deal with the longer term goal of developing a dedicated cost benefit analysis model for long-life semi-rigid pavements.

Objective

To develop a model that is suitable for the analysis of costs and benefits for long-life semi-rigid pavements.

Methodology

A standardised model for the economic assessment of long-life pavements in Europe is urgently required, but practical experience of LLPs is currently limited to the UK. Financial information derived from full scale trials in other European countries will provide data necessary to establish generic guidelines and develop scenarios together with country specific data and identify the most appropriate analytical tool to undertake the cost benefit analysis. In addition any analytical tool should address influencing factors, such as noise and residual life of the structure, that are not considered in this report.

Benefits

A standardised and validated method for the analysis of the economic benefits of LLPs will generate greater confidence and promote their use. The use of LLPs, rather than DLPs, is more sustainable as they are significantly cheaper to maintain and offer reduced user delays.

8.4 Optimisation of maintenance strategies for long-life semi-rigid pavements

Background

A cornerstone of long-life pavements is that after construction, they should require only maintenance of the surfacing to remedy rutting and arrest reflection cracking. This maintenance should be carried out in a timely fashion such that the structural layers of the pavement are not compromised.

This research recommendation could potentially link the other four areas of research proposed for semi-rigid pavements; it should only be undertaken following the completion of the work to understand the nature of reflection cracking.

Objectives

To provide advice on the maintenance assessments that will lead to an optimised strategy for the maintenance of long-life semi-rigid pavements.

Methodology

The first task is to review the state-of-the-art in pavement assessment technology. This ELLPAG review provides some insight into the current methods in use, the scope of this review will also include new technologies for pavement assessment.

A library of available maintenance techniques for the assessment of deterioration will be catalogued, in order to direct maintenance operations. The state-of-the-art knowledge on the progression of deterioration (including the knowledge of crack propagation speed) will be used to predict the condition of the pavement so that a range of maintenance options will be available. The
most economic option of the available techniques will be chosen using the latest, most-appropriate life-cycle cost models.

Benefits

This project will ensure that the economic benefits of long-life semi-rigid pavements are realised by the selection of the most appropriate maintenance treatments to be carried out at the most appropriate time.
9 Summary

This review demonstrates that the technology available for long-life semi-rigid pavements is not as developed as that for fully-flexible pavement types. Whereas the concept of long-life fully-flexible pavements has been adopted in some European pavements and explicit design, assessment and methods are available, the equivalent long-life approach to semi-rigid pavements does not exist.

Two strategies for the management of long-life pavements covering construction, assessment and maintenance have been identified and these strategies are themes throughout the review. It is important to bear in mind the type of strategy employed when considering all aspects of long-life semi-rigid pavements.

- A ‘prevention of cracks’ strategy is widely employed in Europe. Within this strategy, pavements are designed to prevent the onset of reflection cracking. Thereby long-life pavements assessment and maintenance procedures concentrate on the functional aspects of pavement condition.

- A ‘living with cracks’ strategy accepts that the asphalt material can crack; however by the application of appropriate design, assessment and maintenance procedures the risk of structural degradation is controlled.

The ‘living with cracks’ strategy is becoming less favoured but may still be a viable option in some specific cases. Each strategy should be chosen based on local economic analysis.

Only the UK operates pavements that are explicitly labelled as long-life semi-rigid pavements. Pavements, that are considered to have a long, but indeterminate life, are in operation in the UK; in other European countries similar pavement constructions are available for heavy traffic.

It follows that since there are few explicit long-life semi-rigid pavement designs; there are few explicit methods for the assessment and upgrading of these types of pavement. In general, most countries use a measure of bearing capacity, such as the Deflectograph, to assess the structural condition of semi-rigid pavements but it is equally important to take into account the thickness of the layers and the visual condition of the pavement. The nature of the cracking that is seen on the surface is a strong indicator of the condition of the hydraulically bound base.

Three potential options were identified for upgrading a semi-rigid pavement to long-life. Where the hydraulically bound base was considered as suitable for upgrading a potentially long-life pavement can be produced either by placing an asphalt overlay to prevent cracking or by adopting a living with cracks policy. Where the base is assessed as not suitable for upgrading, a semi-rigid pavement may be upgraded to a long-life fully-flexible type of structure by adding a substantial overlay.

The approach to the maintenance of long-life semi-rigid pavements is identical to that for fully-flexible pavements with the important exception of how to treat reflection cracks. This exception depends upon the strategy employed; for example within a ‘prevention of cracks’ strategy, a long-life semi-rigid pavement should be free from reflection cracking.

From the information collected in the review and the discussions at ELLPAG meetings, five research themes were identified. Research recommendations have been constructed to cover these themes; these are:

- Investigation of the nature of reflection cracking in semi-rigid pavements,
- Assessment of techniques for the control of reflection cracking,
- Examination of the effect of traffic on semi-rigid pavements,
• Development of economic analysis tools,
• Optimisation of maintenance strategies for long-life semi-rigid pavements.

This review has identified best practice for the design, assessment, upgrading and maintenance of long-life semi-rigid pavements.

It is clear that there is a potential for the wider use of long-life semi-rigid pavements. One European country has adopted an equivalence between fully-flexible and semi-rigid design and will soon be considering some semi-rigid pavements to be equivalent to long-life pavements. However, the more wide-spread adoption of long-life semi-rigid pavements is only likely to occur once the appropriate methods for quantifying the economic and environmental benefits of these pavements are more widely available.
10 References


Design Manual for Roads and Bridges (DMRB). The Stationery Office, Norwich, United Kingdom.

- HD24: Traffic Assessment (DMRB 7.2.1)
- HD26: Pavement Design (DMRB 7.2.3).
- HD29: Structural Assessment Methods (DMRB 7.3.2)
- HD30: Maintenance Assessment Procedure (DMRB 7.3.3)


LCPC-SETRA, Paris.


A1. Austria

A1. Background

The Austrian high level Road Network (motorways and former national roads) has a total length of 14,000 km. The total length of the motorway network consists of 4,200 km of carriageways. 37% of the motorways are more than 20 years old (v. Figure A1.1). 55% of the network consists of flexible pavements, and 36% of concrete (rigid pavements). The remaining 9% are semi-rigid pavements (Figure A1.1).

![Figure. A1.1 Pavement types on the Austrian high level road network and their age.](image)

In Austria semi-rigid pavements are composed of an asphalt layer on a cement-stabilized base layer. Cement stabilized materials have been used for more than 50 years, mainly to guaranteed durability against repeated cycles of wetting, drying, freezing and thawing. In order to avoid reflective cracking strengths were kept as low as possible and the cement stabilized layer was vibration rolled to produce micro-cracks in the young cement base.

Recently reflective cracking has occurred especially on heavily trafficked roads. This phenomenon may be attributed to 2 major changes (Sommer, 2001): firstly modern cements harden more rapidly and hence, less cement is used (90 kg/m³ – the minimum allowed for mix-in-place construction – instead of 100-120 kg/m³); secondly, traffic volume has increased tremendously.

The reduction of the cement content had resulted from changes in the strength characteristics of the cement. Table A1.1 compares two typical cements widely used in Austria for stabilization in the 1960s and 1990s respectively.

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<th>Flexural Strength [N/mm²]</th>
<th>Compressive Strength [N/mm²]</th>
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<tbody>
<tr>
<td>7 days</td>
<td>28 days</td>
</tr>
<tr>
<td>7 days</td>
<td>28 days</td>
</tr>
<tr>
<td>1960-1970 3.8-5.4 (65%)</td>
<td>6.7-7.5 (100%)</td>
</tr>
<tr>
<td>17.2-25.0 (61%)</td>
<td>30.0-39.6 (100%)</td>
</tr>
<tr>
<td>1995/96 5.1-5.9 (77%)</td>
<td>6.8-7.5 (100%)</td>
</tr>
<tr>
<td>29.6-34.0 (72%)</td>
<td>42.7-45.8 (100%)</td>
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</tbody>
</table>

Within 25 years the 28-day strength had increased by 27%, the 7 day strength had increased by 50%, but the flexural strength at 28 days had remained the same. The higher cement strength at 7 days allowed a lower cement content to be used and this meant a strength loss at later ages: more important for flexural strength (which is more related to the performance of the road) than for compressive strength.

Today the pre-cracking method and slow hardening road binders (7 day standard strength at most 50% of the 28 day value) are used in Austria (see Chapter 2.2). Slow hardening road binders offer two advantages: Since their tensile strength is low at the beginning they will promote early opening of induced cracks (Figure A1.2).
Ultimately, tensile strength will be better than with normal Portland cement and this will improve the long-term performance of the road (Sommer, 2001).

![Graph showing splitting tensile strength over time](image)

**Figure A1.2. Splitting tensile strength of a granular material bound with a Hydraulic Road Binder (HRB) and a Portland Cement (PZ), respectively (Sommer et al., 2001).**

2. Design and Construction

2.1 Design

In the Austrian National Road Standards (RVS 03.08.63, 2008) there is no explicit design for long-life pavements.

The Austrian design standard, which is based on an analytical design procedure, uses a semi-rigid pavement type, consisting of an asphalt layer (bituminous wearing and base course) on a cement stabilized base layer over an unbound sub-base. The thicknesses of the layers refer to the different levels of traffic volume (100 kN ESALs), expressed by 7 load classes (S, I, II, ..., VI). The semi-rigid pavement types for the upper load classes S to III are shown in Table A1.2.

**Table A1.2. Austrian design standard for semi-rigid pavements, load classes S to III (RVS 03.08.63, 2008).**

| RVS 03.08.63 (design period 20 years, ESAL 100 kN) |
|-----------------|-----------------|-----------------|-----------------|
| load class      | I               | II              | III             |
| Msa             | > 10 to 25      | > 4 to 10       | > 1.3 to 4      |
|                 | > 0.4 to 1.3    |
| construction type 4 |
| mm              | 200             | 170             | 150             |

minimum subgrade-modulus $E_{V1,UP} \geq 35$ MN/m² gained from a load plate test

On motorways, typically load class S is used, thus having a cement-bound base course with a thickness of 300 mm and an asphalt layer with a total thickness of 170 mm. This type of pavement is considered to support up to 25 msa and to have a life of at least 20 years without any structural maintenance.

2.2 Construction

One possibility in order to avoid reflective cracking is to raise the thickness of the asphalt layer. To provide for durability Willberg (2001) gives a ratio 0.7 of the thickness of asphalt pavement ($h_1$) and the thickness of the cement stabilized base ($h_2$):

$$
\frac{h_1}{h_2} = 0.7
$$
For Austrian pavements, a thickness of 210 mm of the asphalt layer would be suitable for load classes S, and 170 mm for load classes II and III. However, in Austria two other methods are favoured:

(a) Micro-cracks

The cement-bound material is vibration-rolled, 12-24 hours after compaction. A slow hardening cement is used.

(b) Pre-cracking

Immediately after compaction notches are cut into the surface of the cement bound layer by means of a cutting disc mounted to the roller (Figure A1.3). Sub-bases often contain bigger stones, making the use of a CRAFT-machine a very destructive process. However, with this alternative process the cutting disc moves up when meeting a bigger stone and causes no damage. The notches are 1/3rd of the thickness of the base layer. A distance of 3.0 m to 3.5 m is recommended for these dummy joints (Sommer et al., 2001). A bond inhibitor is sprayed into the notch and a final pass of roller (with the disc raised) restores the evenness.

In order to ensure an early opening of the induced crack, a slow hardening hydraulic road binder is used. With a 7-day-compressive strength of 3 N/mm² on Proctor cylinders the tensile strength during first days will be only 1/3rd of the strength that would result if normal Portland cement were used and the notches will crack during the first nights. Ultimate tensile strength will be at least 50% higher than with Portland cement.

3. Assessment and Upgrading

In Austria there are no explicit methods for the assessment and upgrading of long-life semi-rigid pavements. Condition assessment of semi-rigid pavements usually includes a visual inspection, FWD-measurements (to gain individual layer thicknesses and stiffnesses) and laboratory tests (e.g. stiffness and fatigue test) on specimens that are cut from the pavement. Finally, the condition of the pavement is evaluated by means of engineering judgement. Provided that the cement stabilized layer is in good condition, upgrading becomes possible. An analytical design is used to estimate the residual life of the pavement and to design the required thickness of the new asphalt layers.

4. Maintenance

In Austria there is no explicit strategy for maintaining long-life semi-rigid pavements. Non-structural maintenance on semi-flexible pavements covers the full spectrum of maintenance options for fully-flexible pavements. The determination of a suitable maintenance treatment is often linked to the type and severity of cracking that occurs on the bituminous surface. Any surface crack that affects the whole asphalt layer, e.g. transversal
(thermal) cracks and bottom-up (fatigue or reflective) cracks, is considered as structural deterioration that will reduce service life. In the case of structural deterioration all asphalt layers need to be removed.

Top-down cracking may be judged as a non-structural deterioration and may therefore be accepted on a long-life pavement. In Austria top-down cracking is always thought to be linked to thermal and traffic overload, but not to reflective cracking.

5. Economics

In Austria there is no economical analysis for long-life pavements of any type.

6. Research Needs

Crack-mechanisms of semi-flexible pavements need to be further investigated. The theory of top-down reflective cracking is not yet convincing.

Mix-stabilization may be considered to be an interesting alternative to minimize reflective cracking, but experience with this method is rare. The higher investment costs of a mix stabilization may be compensated by a longer service life.

The need of economic tools to evaluate the cost effectiveness of long-life pavements is similarly essential for semi-rigid pavement types.

7. References


RVS 08.15.01, Technische Vertragsbedingungen, Oberbauarbeiten (ohne Deckenarbeiten), Tragschichten, Ungebundene Tragschichten. Richtlinien und Vorschriften für den Straßenbau, Österreichische Forschungsgemeinschaft Straße und Verkehr. Technical specifications (in contract) for road works, Pavements (excluding surfacings), Road bases and sub-bases, Unbound sub-bases, Vienna, Austria, 2004/05.

RVS 08.17.01, Technische Vertragsbedingungen, Oberbauarbeiten (ohne Deckenarbeiten), Tragschichten, Mit Bindemittel stabilisierte Tragschichten. Richtlinien und Vorschriften für den Straßenbau, Österreichische Forschungsgemeinschaft Straße und Verkehr. Technical specifications (in contract) for road works, Pavements (excluding surfacings), Road bases, Stabilized road base layers, Vienna, Austria, 2002.


A2. Belgium

A2.1 Background and History

Belgium is a federal state (final constitutional adoption in 1993) divided into three Regions: Flanders in the North, Brussels in the Centre and Walloon in the South.

Each region manages its own former state highways and roads belonging to its territory. The Region of Brussels capital is primarily concerned with urban types of roads.

<table>
<thead>
<tr>
<th>Table A2.1: Typical Belgian road categories</th>
</tr>
</thead>
<tbody>
<tr>
<td>TRAFFIC (ADT)</td>
</tr>
<tr>
<td>All vehicles categories Both ways</td>
</tr>
<tr>
<td>Road Category</td>
</tr>
<tr>
<td>Lanes / width</td>
</tr>
<tr>
<td>Base speed (km/h)</td>
</tr>
</tbody>
</table>

As of 2002, there is a total of 15678 km of state roads in Belgium of which 11% are motorways, 80% are regional roads and 9% are provincial roads.

A2.2 Design and Construction

Design standards have applied for over thirty years (ref. 1, 2). The target service life of road structures has been fixed to 20 years.

The designs which are dealt with in this report are those related to semi-rigid structures with a total design life bearing capacity of over 10^5 commercial vehicles (Nc >= 10^5).

25% of heavy vehicles (commercial vehicles) > 3.5 t are assumed in the normal traffic. The risks of axle overloads greater than the 13 t axle load acceptance threshold are also accounted for.
Referring to the adopted definition of layers:

The Belgian standard structures for semi-rigid pavements that are designed to take into account the risk of axle overloads are as follows.

**Motorways (since 1970):**
- Surface course: 2 x 5 cm Type I mix (bituminous wearing course).
- Binder course: 2 x 6 cm Type III (bituminous concrete).
- Base: 20 cm (steel mesh) reinforced lean concrete.
- Sub base: granular layer, crushed stones 0/40.
- Capping: drainage sand layer (minimum 20 cm).

**Other roads (Code of good practice, BRRC Recommendation 1983):**

The bituminous surfacing course thickness varies with traffic levels (Nc: number of commercial vehicles):

- $10^7 \leq 0.4 \times Nc \leq 10^8 \rightarrow 17 \text{ cm}$
- $10^6 \leq 0.4 \times Nc \leq 10^7 \rightarrow 15 \text{ cm}$
- $10^5 \leq 0.4 \times Nc \leq 10^6 \rightarrow 14 \text{ cm}$
- $10^4 \leq 0.4 \times Nc \leq 10^5 \rightarrow 12 \text{ cm}$

Remark: correction factor 0.4 expresses the reduction in traffic aggressiveness related to the transverse distribution of loads for lane widths > 3m.

The lean concrete road base has a constant thickness of 20 cm for all traffic levels. This design thickness is obtained taking into account the risk of encountering axle overloading leading to premature cracking. The elastic modulus of this material typically ranges between 12000 and 35000 MPa with a Poisson coefficient between 0.25 and 0.35.
The sub base thickness is dependant on both the traffic level and the soil (or capping) modulus $E_s$:

<table>
<thead>
<tr>
<th>TRAFFIC LEVEL</th>
<th>SOIL MODULUS (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$10^5 &lt; 0.4 \times N_c &lt; 10^6$</td>
<td>50 cm</td>
</tr>
<tr>
<td>$10^6 &lt; 0.4 \times N_c &lt; 10^7$</td>
<td>45 cm</td>
</tr>
<tr>
<td>$10^7 &lt; 0.4 \times N_c &lt; 10^8$</td>
<td>40 cm</td>
</tr>
<tr>
<td>$10^8 &lt; 0.4 \times N_c &lt; 10^9$</td>
<td>35 cm</td>
</tr>
</tbody>
</table>

The thickness design of pavements must be such as to deliver a total structure thickness of at least equal to the local freezing depth.

The approach to traffic characterisation used in Belgium which relates the number of commercial vehicles, $N_c$, and the number of 80-kN standard axles, $N_{st}$, is as follows:

$$N_{st} = k \times K \times N_c \quad (6)$$

where:

- $k = 1$ for channelled traffic; 0.4 for lane width > 3m.
- $K = n_{(av)} \times \alpha \times C$

- $n_{(av)} = 2.7 = \text{average number of axles per commercial vehicle}$;
- $\alpha = 0.143$ (distress mechanism considered: fatigue cracking, with rest periods between loads);
- $C = \text{traffic "aggressiveness" (or "pounding") factor (a function of the frequency spectrum of loads; 1991 update). Taking } f_i \text{ as the value of the proportion of loads } P_i \text{ in the aggregate set of commercial vehicle axles, } C \text{ is calculated from the relation } C = \Sigma_i f_i \times (P_i / P)^{\alpha}.$

For flexible structures: $P=8t$; $v=4$; $C = 0.82$
For semi-rigid structures: $P=13t$; $v=33$; $C = 3.1$

These are the figures retained for establishing solutions to the structural design where deterioration mechanisms taken into account are soil deformation, and fatigue cracking induced in the bituminous layers and in the lean concrete road base.

For the expected service life of 20 years, $N_c$ can be related to the average daily commercial traffic in one direction ($N_j$) by the equation:

$$N_c = N_j \times 300 \times [(1 + t)^{20} - 1] / t$$

Where $t$ is the annual increase of commercial traffic ($= 0.05$ for T1 to T4 levels)
A2.3 Assessment and Upgrading

Assessment

Conventional monitoring of the surface conditions on the network are carried out; these surveys measure: skid resistance, texture, evenness, rutting and surface deteriorations (cracking, ravelling). User oriented indicators are derived from the monitoring data and used to follow up on performance progression of the upper layer conditions. The results of these surveys will influence the maintenance of the bituminous surfacing layers.

Further investigations regarding structural conditions are implemented using deflection and thickness measurements: the deflectograph was regularly used; nowadays the curviameter or the falling weight deflectometer are more commonly used, as are radar devices. These observations, combined with results from visual inspection, lead to the diagnosis of possible structural disorders.

Structural assessment is derived from a combined surface condition evaluation with structure classes and traffic levels. Indicators are derived for structural intervention decisions. An alternative approach is also used through intensive radar monitoring; weak or suspect zones are then detected and further inquiry is conducted using falling weight techniques.

Upgrading

Although semi-rigid structures have been designed for a service life of 20 years (ref. 4), experience shows that some sections out-perform this life while others fail structurally within it.

Long term performance observations over 20 years are not readily available; nevertheless one can identify a series of factors, which have the potential to induce long-term service level performance: soil and drainage conditions, actual layer thickness, appropriate and timely bituminous surfacing maintenance, construction quality, and over estimated Nc figures.

For the failed sections, upgrading can be achieved through reinforcement (ref. 3), and in some cases the application of inlays. Inlays will be replaced by reconstruction of the pavement in the cases where the depth of the inlay exceeds the total thickness of the existing structure.

A2.4 Maintenance

The bituminous surfacing layers can undergo several types of maintenance interventions.

- Local repairs of deterioration such as sealing cracks (in particular transverse thermal cracks generated at base level in the lean concrete).

- Generalised repairs for the replacement of rutted or ravelled layers. These layer replacements will have to cope with the probable migration of thermal cracks from the road base by either providing sufficient thickness for the new surfacing layers or by making use of crack retarding interfaces such as bituminous membranes, non-woven interfaces, geo-grids and steel meshes.
A2.5 Economics

Recommendations regarding the economic aspects linked to design, construction and maintenance of semi-rigid structure have been formulated. Two basic types of strategy can be put forward (ref. 2).

- Construction of a high performance road with a significant initial investment, but with reduced maintenance costs (obviously a step towards long-life solution);
- Construction at the minimum initial cost and requiring a progressive increase of bearing capacity depending on traffic build up.

The selection of strategy is guided by the following:

- The type and nature of maintenance works that are linked to the main modes of structure deterioration.
- Enhancement with bituminous overlays can be constrained by imposed level heights.
- The presence of rights of way can hinder considerably some structural solutions.
- The upgrade of small section lengths can be uneconomical if the use of expensive construction equipment is required.

The design solutions can deliver alternative solutions depending on:

- The type and execution of delays.
- The use of special techniques e.g. soil replacement, water ingress protection.
- The types of measures undertaken to cope with freeze and thaw.
- The type of maintenance required.

Furthermore, the design should be accompanied by considerations on the needs for in-service monitoring and maintenance. This is especially true for new “private-public partnership” contracts that include both construction and maintenance by a private partner over a longer period (more than 20 years).

A2.5 Software

Recently, design and upgrading is also done using software packages designed to assist the designer in the evaluation of the optimal choice of materials and of layer thicknesses given the traffic expectations and the need for sufficient bearing capacity. This also resulted in the development of the DimMET© software package for design and back-calculation from gathered Curviameter and FWD data, and in the development of the multi-criteria based software tool EvalMET© for assistance in the choice of material for the upper layers when planning maintenance.

A2.6 Research Needs

- New procedures and methods for assessing traffic condition, particularly regarding the damaging effects of heavy vehicles (determination of load spectra).
- Models for forecasting traffic growth rates.
- Methods to protect the road base from moisture.
A2.7 References

1. V. VEVERKA, Renforcement des chaussées à revêtement hydrocarboné. Compte rendu de recherche CR12/80, Centre de Recherches routières, Bruxelles. (French)

2. Code de bonne pratique pour le dimensionnement des chaussées à revêtement hydrocarboné. Recommandations CRR - R49/83, Centre de Recherches routières, Bruxelles. (French)


A3. Czech Republic

A3.1 Background
Czech standards recognise only two pavement types: rigid (cement concrete) and flexible (asphalt, cobble-stone etc.). Asphalt pavements with a cement-bound base layer are classified as flexible pavements and the term “semi-rigid pavement” is not used routinely in engineering practice.

A3.2 Design and Construction
Flexible pavements, which could be characterized as semi-rigid pavements, are very common in the Czech Republic. The design methodology is given in the technical standard of the Czech Ministry of Transport – TP 170 – “Road Pavement Design”. This technical standard consists of general pavement requirements including the sub-grade, pavement design method and a catalogue of pavement constructions.

There are four types of rigid layers used in semi-rigid construction described in the Czech technical and quality standard of the Czech Ministry of Transport – TKP Chapter 5 – “Base and Sub-base Layers”:

- sub-base concrete,
- rolled concrete,
- gap-graded concrete (porous concrete),
- cement bound aggregate.

The most common used binder is Portland cement.

The technical standard TP 170 specifies three design levels of failure expected at the end of the 25 year design period of pavement - see following table:

<table>
<thead>
<tr>
<th>Design level of failure</th>
<th>Traffic importance of road</th>
<th>Percentage of pavement with failures in the end of design period (25 years of operation)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D0</td>
<td>motorways, expressways, incl. urban, 1st roads class</td>
<td>up to 1%</td>
</tr>
<tr>
<td>D1</td>
<td>2nd and 3rd road classes, urban roads, parking places</td>
<td>up to 5%</td>
</tr>
<tr>
<td>D2</td>
<td>urban roads, parking places, pavements in pedestrian zones, temporary pavements etc.</td>
<td>up to 25%</td>
</tr>
</tbody>
</table>

A3.3 Design levels of failure
In general, it is possible to say that roads with “better” design level of failure also have a higher quality sub-base layer. The material of these sub-base layers is usually mechanically bound aggregate (mineral concrete) or hydraulically bound layers as described above.

A3.4 Assessment, Upgrading and Maintenance
Assessment, upgrading and maintenance of semi-rigid pavements are carried-out with respect to technical standards effective for flexible pavements.

Several older roads with cement bound layers have reflection cracks and their reparation is a subject of considerable interest.

The following measures to prevent a propagation of reflection cracks to asphalt layers are carried out on new layers of cement bound material:

- binder adaptation; compaction of layer by roller during a setting process to release the contraction stress; creation of artificial reflection cracks in intervals up to 5 meters,
• realisation of compensation unbound layer (50 – 150 mm) on cement bound layer,
• usage of stress absorbing membrane interlayer (SAMI) with modified asphalt and capping layer (chipping, textile, microcarpet),
• usage of asphalt layer resistive to the creation of contraction cracks.

A3.5 Economics

There is no special economical analysis used for semi-rigid pavements in the Czech Republic. The semi-rigid pavements are analyzed within the Pavement management system and by using HDM-4 software on the same basis as for flexible pavements.

A3.6 Research Needs

There is no special requirement for research on semi-rigid pavements except for the development of maintenance and rehabilitation methods of reflection cracking.

References:
TP 170 – Road Pavement Design, Technical Standard of Ministry of Transport of the Czech Republic, 2005
TP 82 – Catalogue of flexible pavements failures, 1996
TP 87 – Maintenance and rehabilitation of flexible pavements, Technical Standard of Ministry of Transport of the Czech Republic, revision 2008
TP 115 – Cracks reparations of asphalt pavements, Technical Standard of Ministry of Transport of the Czech Republic, 1999
TP 147 – Usage of Stress Absorbing Membrane Interlayers and Reinforcing Elements in Road structure, 2001
TKP Chapter 5 – Base and Sub-base Layers, Technical and Quality Standard of Ministry of Transport of the Czech Republic, 1999
ČSN 73 6114 – Road Pavements – Basic Requirements for Design, Czech Standard, 2006
A4. Denmark

A4.1 Summary

Denmark does not have a National Report explicitly for semi-rigid pavements, but the Danish Road Institute did publish a Report Number 138 in 2004 entitled “Mechanistic Design of Semi-Rigid Pavements – An incremental approach.” This includes a review of the application of semi-rigid designs in Denmark, including the abandonment of such pavements in the early 1980’s and suggestions for its future revival and the reasons for this. The executive summary of the report is included below and recent developments since 2004 are described in A4.6.

A4.2 Executive Summary of Report 138

Throughout Europe, an increased interest in the pavement type commonly known as semi-rigid pavements has been observed in recent years.

This type of pavement is a composite design consisting of an unbound sub-base layer, a hydraulically bound base course layer and bitumen rich hot rolled asphalt concrete as binder and/or wearing course.

The reason for the growing interest in this type of pavement, which in Europe has been applied since the 1950’s, could well be related to the following:

• Increased traffic loads and widespread rutting in existing flexible pavements indicate the need for pavements with improved stiffness.

• Experience with existing semi-rigid pavements indicates economically feasible performance, provided relevant design and construction principles were applied.

• The design of Semi-rigid pavements lends itself to the use of marginal materials and, consequently, often a reduced environmental impact. In this way the design may be seen as a contribution to a sustainable infrastructure construction.

In Europe, the semi-rigid pavement concept is used to a varying extent. Some countries like Germany, France and Spain have a rather large proportion (30-50 %) of semi-rigid pavements on their main road network, whereas in other countries e.g. Denmark, less than 5 % of the total length is constructed with this pavement type. A typical pavement for heavy traffic applications consists of 15-25 cm cement treated base course (CTB) covered with 10-20 cm of asphalt concrete surfacing. The 7-day compressive strength requirement for the CTB layer is usually in the range 6-12 MPa.

A well-known disadvantage of pavements with cement-treated base courses is contraction cracking of the CTB layer causing reflective cracking through the asphalt concrete surfacing. This can be seen on the road surface as large transverse cracks with a typical spacing of 10-20 m. If these cracks are not maintained, e.g. by crack sealing, they will often act as the starting point for further deterioration.

The adverse effects of the contraction crack tendency exhibited by cement bound layers can be mitigated by different techniques for obtaining a more closely spaced crack system. One possibility is to use a binder combination with slower strength gain, thereby promoting a larger number of thermal contraction cracks formed while the early-age tensile strength is still relatively low. Another and by now widely agreed solution used by most countries is to pre-crack the CTB-layer. This is done by wet-forming transverse notches at 2-4 m distance in the layer immediately after the paver down to a depth corresponding to 1/3 – 1/2 layer thickness. After hardening, fine cracks will propagate through the rest of the layer. The resulting transverse cracks will develop relatively small cracks openings and good load transfer, thereby reducing the tendency for reflection through the asphalt layer.
In Denmark, around 100 km of motorway pavement with cement bound gravel base course was constructed in the 1970’s. The typical pavement structure was 20 cm CTB covered with 12 – 16 cm asphalt concrete surfacing. The CTB layer was required to have a minimum 7-day compressive strength of 5 MPa.

The use of this pavement type was abandoned in Denmark in the early 1980’s, mainly because of poor performance of a road section that suffered from critical transverse reflective cracking already at the time of opening to traffic.

Recent investigations by the Danish Road Directorate have revealed that a number of heavily trafficked motorways in Denmark have performed much better than expected at the time of design. The investigations show that in most cases the wearing course is bitumen rich asphalt concrete and in many cases the base course is cement treated.

Some of these high performance roads have served the traffic for more than 20 years without wearing course replacements and without any reflective cracking. It is assumed that the good performance of these stabilised base layers is related to use of moderate cement contents, which results in a crack pattern without discrete cracks.

These observations were the basis for the decision by the project partners to conduct the present data collection, research and development study with the following aims:

- Develop a realistic deterioration model for this type of pavement.
- Establish and verify a background for structural design of semi-rigid pavements.
- Draft Technical Specifications for construction of hydraulically stabilised base layers combined with relatively thin high performance wearing courses.

**A4.3 Project Work**

During 2003 the project was carried out as planned. Initially, a deterioration model was formulated, based on an incremental-recursive process, i.e. a “calculation rule” that determines the development of the layer’s E-modulus over a number of load repetitions as a function of the layer’s “critical reaction” to the actual load, its current E-modulus and the initial E-modulus. For the cement stabilised layer, the longitudinal strain at the bottom of the layer is chosen as the “critical reaction”.

The work with this deterioration model received valuable input from colleagues in CSIR Transportek, South Africa as well as from a DANIDA funded R&D project involving the Danish ‘Road Testing Machine’. The project partners express their thanks for these valuable inputs.

Upon formulation of the deterioration model the following procedure was followed:

- A full-scale testing series was executed that would allow determination of the deterioration parameters for different values of material strength and deformation
- The results from the full-scale testing was entered into the general model
- The model was finally verified against measurements from selected sections on Danish motorways with up to 30 years of service life

The full scale testing series was carried out on a test field at a motorway construction site near Fagerhult in southern Sweden. Three CTB-mixes with varying strength and composition were tested in a relatively thin pavement with 180 mm CTB covered by 30 mm asphalt surfacing. For each of the three materials two 15-m test sections were paved, one of them equipped with instruments for measurement of stress and strain in the pavement. Each of the six sections was
loaded with the HVS - Heavy Vehicle Simulator, which is a mobile Accelerated Load Testing Facility. During a 5-week period in the autumn of 2003 the test sections were loaded with up to 125,000 wheel passages with a load of 30 or 60 kN. The deterioration was monitored by measurements with both the Light Weight Deflectometer and Standard Falling Weight Deflectometer, along with recorded stresses and strains in the pavement.

The results from the full scale testing were used for development and calibration of the incremental-recursive model. The model constants were determined from data analysis for the six test sections.

The model was finally verified against Falling Weight Deflectometer data from Danish motorway sections. Along with this special verification a more general collection and analysis of data for construction, performance and traffic for a number of Danish CTB pavements was carried out with the aim of trying to establish why some of these pavements have shown superior performance and others not.

A few of the most interesting pavement sections were selected for supplementary studies. Pavement materials were sampled for determination of in situ strength and stiffness and Falling Weight Deflectometer (FWD) measurements on the pavements were performed. The main results are summarised in the following table:

<table>
<thead>
<tr>
<th>Pavement Section</th>
<th>Reflective Cracking</th>
<th>E-values from FWD [MPa]</th>
<th>Compressive strength [MPa]</th>
<th>Intact cores</th>
</tr>
</thead>
<tbody>
<tr>
<td>M3</td>
<td>Yes</td>
<td>12,000 – 20,000</td>
<td>17</td>
<td>14/14</td>
</tr>
<tr>
<td>M10-s</td>
<td>No</td>
<td>1,500 – 10,000</td>
<td>11</td>
<td>32/48</td>
</tr>
<tr>
<td>M10-n</td>
<td>No</td>
<td>2,000 – 12,000</td>
<td>21</td>
<td>28/43</td>
</tr>
</tbody>
</table>

The motorway section with extensive reflective cracking had a rather high and constant stiffness measured with the FWD, whereas the two sections with no reflective cracking showed lower values and greater variability. This was confirmed by the proportion of intact cores taken from the sections. For the M3 section all attempted cores were intact but for the other two sections only around 2/3 of the cores had an intact CTB-layer. The core strengths do not confirm that a lower strength should be favourable in relation to reflective cracking. It should be noted however, that the core strengths represent maximum values for the material in the intact cores, which for the last two sections does not give a true picture of the actual strengths. Furthermore, it is worth noticing the high in-situ strengths found after 20-30 years compared to the 5 MPa requirement at the time of construction.

The traffic analyses conclude, that some of the Danish motorway sections which were designed for 5-10 mil. 10-ton Standard Axles have now supported up to 27 mil. Standard Axles during a 20-30 year life time without need for strengthening.

The verification on the Danish motorway pavements confirmed the applicability of the incremental-recursive model developed from the full scale testing. With this model, traditional deterministic design criteria could be developed. These criteria relate the initial strain to allowable number of load repetitions for a given initial E-modulus of the stabilised layer and chosen deterioration level, i.e. terminal E-modulus.

Using the design criteria, a table with design examples for different traffic levels can be prepared. The failure condition is a terminal E-modulus of 3,000 MPa.
These designs are based on criteria equations developed from an incremental-recursive analysis that ensured that 75% of all measured E-moduli were above the prediction lines. It can therefore be assumed that 75% of all pavements that are designed according to these criteria will have E-moduli of the CTB layer that are above the design terminal E-modulus.

The CTB materials with E-moduli of approximately 16,000 MPa, prescribed for the heavily trafficked pavements must be pre-cracked at intervals of 1.5 – 3 m in order to avoid shrinkage and temperature cracks, while the 12,000 MPa material with lower tensile strength may be expected to develop a closely spaced (micro) crack pattern on it’s own. This type of material is therefore better suited to small-scale projects with more basic production methods.

During the project a number of uncertainties relating to properties of CTB materials were identified. Laboratory investigations were performed with the objective of contributing to a more rational approach in design of the materials and in determination of strength parameters, which should be specified in a Technical Specification to meet the requirements of the structural design.

An analysis of different methods for determination of compressive strength on laboratory specimens resulted in a recommendation of the method based on Standard Proctor compaction (ASTM D558, corresponding to prEN 13286-53).

In the design of CTB pavements, tensile strength, flexural strength and modulus of elasticity are all important parameters, but for reasons of simplicity usually only the compressive strength is determined. The quantitative relationships between all of these parameters were investigated for two different CTB-mixes.

Optimal design of compositions for hydraulically bound materials will be one of the requirements for the future. The use of local – sometimes marginal – materials along with a possible minimisation of the cement content contribute to the sustainability and cost effectiveness of the concept.

The relationship between dry density and compressive strength of a typical CTB material was investigated, and the results confirmed that an optimisation of the gravel and CTB composition is an important measure towards environmentally friendly and cost effective construction.

It was established that for CTB aggregates the following parameters are beneficial: High density, low/moderate SE-value (non-plastic material) and high content of filler. Further it was demonstrated that by mixing two marginal sands which each had high cement requirements when used
separately, it was possible to obtain a good combined aggregate material with low cement requirement.

Finally, a special experiment suggested that an addition of 20 % 2-4 mm aggregate to a 0-2 mm CTB mix did not have any significant impact on the compressive strength. Consequently, such addition will result in a reduction in required cement content for a given strength requirement of 15-20 %.

A4.4 Conclusion

The deterioration model, which was developed as an incremental-recursive model has been transformed into a mechanistic pavement design guideline.

When this guideline was verified against measurements from the selected Danish motorway sections, it was found that the design guideline with good accuracy could predict the performance of these pavements without the need for any further corrections.

It may consequently, be concluded that a design methodology has been established which can predict the performance of Semi-rigid pavements in a realistic way.

A4.5 Recommendations

It is recommended that this new design approach, i.e. a mechanistic design based on incremental recursive approach, shall be demonstrated in full-scale on a Danish motorway section and that the work shall be performed based on Technical Specifications, which will be prepared on the basis of the work reported in the present report.

A4.6 Full-Scale Demonstration

After the conclusion of the project documented in report no. 138, a 500-m full scale demonstration was carried out in connection with the construction of a Danish motorway, using the design philosophy developed during the project.

Laboratory testing with locally available materials led to the formulation of an optimised mix with 60 kg cement and 60 kg fly ash per m$^3$. The cement bound base layer was paved in a thickness of 20 cm and supplied with wet-formed transverse crack inducers with a distance of 1.5 m. The base layer was covered with total 9 cm asphalt binder and wearing course. Based on results from core drilling and saw cuts in the pavement it has been established that the crack inducers were active and working.

The motorway section was opened for traffic in the autumn of 2006 and the first inspections indicate good performance. The demonstration will be followed in the coming years with visual inspections and bearing capacity measurements.
A5. France

A5.1 Background

In France, there are two types of pavement that use hydraulically bound material in the main structural layers in combination with bituminous upper layers. These are:

1. Semi-rigid pavements (chaussées assise traitée aux liants hydraulique) - These structures consist of an asphalt surfacing of up to 8 cm thick over a thick base of material treated with hydraulic binder.

2. Composite pavements (chaussées à structure mixte) - These consist of an asphalt surfacing and hydraulically bound base of comparable thicknesses.

Approximately 38.2 %, or 15,000 km, of the primary road network is of semi-rigid pavement construction, and about 100,000 km of the remaining network. While composite pavements represent about 5 % or 1,900 km of the primary network.

The majority of the primary network was constructed between 1970 and 1990 when the price of bitumen was high. Figure A5.1 gives the construction of semi-rigid pavements in the West of France [ref 1] during this period. A proportion of these semi-rigid pavements was constructed using sands treated with hydraulic binders.

![semi rigid pavements](image)

*Figure A5.1: length of semi-rigid pavements (Brittany)*

A5.2 Design

A5.2.1 General Outline

The general outline of the current French design method is as follows [2]:

- The cumulative number and types of heavy commercial vehicle over the pavement design period is determined. This information is converted into a cumulated number of passes of a 130 kN dual wheeled reference axle.
- Definition of environment (climatic data, wet and frost);
- Definition of the long term bearing capacity of the pavement foundation. In France, there are four stiffness classes for the long-term bearing capacity of the completed foundation (PF1 to PF4). For heavy traffic of greater than 14 rsn130, a PF3 foundation class is required. This corresponds to an equivalent surface stiffness of more than 120 MPa.
- Definition of pavement material defined by standards;
- Competence requirement of workmanship.
A5.2.2 Semi-rigid materials

Semi-rigid pavements principe
A semi-rigid pavement consists of two or three layers: Hydraulically bound sub-base and base and wearing course in asphalt concrete.

Hydraulic binder
The following binders (or mixture of binders) are the most widely used in France:
- Cement;
- Slag as a by-product from blast furnaces. It is a ground material that is activated with lime and alkaline sulphates;
- Silica-alumina pulverised fly ash fines derived from pulverised coal combustion;
- Vitrified, granulated or pelletized blast furnace slag. These are pozzolanic materials;
- Proprietary hydraulic binders that are mixtures of cement and slag and pulverised fly ash in proportions chosen by the contractor.

Mechanical performances
The mechanical performance properties used to classify materials treated with hydraulic binders are the values obtained at 360 days. These are the direct tensile strength $R_{360}$ and the secant modulus $E_{360}$. Reference values for the various materials are given in Table A5.1.

Table A5.1 Reference values of mechanical properties for design calculations

<table>
<thead>
<tr>
<th>Material</th>
<th>Nature of binder</th>
<th>$E_{360}$ (MPa)</th>
<th>$R_{360}$ (MPa)</th>
<th>Binder content</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bound graded aggregates</td>
<td>Cement</td>
<td>23 000</td>
<td>1.15</td>
<td>3 to 4 %</td>
</tr>
<tr>
<td></td>
<td>Proprietary binders</td>
<td>23 000</td>
<td>1.10</td>
<td>3 to 5 %</td>
</tr>
<tr>
<td></td>
<td>Pre-crushed slag and</td>
<td>15 000</td>
<td>0.9</td>
<td>8 to 15 %</td>
</tr>
<tr>
<td></td>
<td>setting agent</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Silico-aluminous fly</td>
<td>30 000</td>
<td>2.1</td>
<td>10 to 15</td>
</tr>
<tr>
<td></td>
<td>ashes and lime</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Pozzolans and lime</td>
<td>15 000</td>
<td>0.9</td>
<td>15 to 25 %</td>
</tr>
</tbody>
</table>

A5.2.3 Design of semi-rigid pavements
Generally, the semi-rigid structures have two base layers treated with hydraulic binders. The layer thicknesses are determined by calculating of failure due to fatigue at the base of each of the two treated layers. The criterion is that the stress, $\sigma_{f}$, predicted at the underside of the bound layer must be less than a permissible value in order to achieve the required design life.

The following examples give the pavement layer thicknesses for the reference design period of 30 years used for the primary road network, for a foundation class of PF3 (>120 MPa), and for a road designed to carry between $14 \times 10^6$ and $38 \times 10^6$ commercial vehicles.

- Example 1: Semi-rigid pavement with cement bound graded aggregate (CBGA):
  - Thin asphalt Concrete 2.5 cm
  - Asphalt concrete (binder course) 6 cm
  - CBGA 19 cm
  - CBGA 18 cm

- Example 2: Composite pavement with bituminous bound graded aggregate (BBGA) and cement bound graded aggregate (CBGA):
  - Thin asphalt Concrete 2.5 cm
  - Asphalt concrete (binder course) 6 cm
A5.2.4 Precracking techniques
Hydraulic bound material cracks naturally through thermal shrinkage, and this material will crack more readily if it has a high stiffness modulus and if the aggregates have high coefficient of thermal expansion. Within a few years, these cracks reflect through the upper layers of asphalt and appear on the surface. The time before cracks appear depends on the thermal movement in the stabilized material.

The onset of reflection cracking can be delayed by pre-cracking the hydraulically bound layers at the construction stage before the hydraulically bound material has set. A closer spacing of the transverse cracks reduces the thermal movements in the cracks. Pre-cracking at 3 m intervals can result in the appearance of reflection cracks at the surface being delayed for 10 years or more.

The following three pre-cracking processes available in France are illustrated in Figures A5.2, A5.3 and A5.4:

1. Craft (LCPC-EUROVIA patent) Grooving + Bitumen emulsion
2. Olivia (EUROVIA patent) Grooving + Plastic film at half-thickness
3. Joint-actif (SACER patent) Grooving + Corrugated profile + Vibration
Figure A4.5 - Stage 1: thin crack

Figure A4.6 – Stage 2: secondary cracking

Figure A5.2 - Craft Process

Figure A5.3 - Olivia Process

Figure A5.4 - Joint-actif Process
The Craft process (CRÉation Automatique de Fissures Transversales - automatic creation of transverse cracks) [ref 4] is applied before compacting. It consists of creating a discontinuity in the layer of hydraulically treated material by making a transverse groove in which a bitumen emulsion is injected.

The Olivia process consists of creating a discontinuity in the layer by placing a simple flexible plastic film vertically in the thickness. It is placed by means of a plough, which cuts a transverse groove to the mid-depth of the layer and through which the film is unreeled.

The joint-active process consists in placing a rigid insert of corrugated PVC within the layer of hydraulic material. Pre-cracking is carried out every 2 metres.

A5.3 Maintenance

To determine the maintenance schedule, the road manager relies on a knowledge of indicators of the state of the roadway under investigation. These indicators are calculated from measurements performed with high-yield survey equipments.

As a preliminary step, the road manager must have in his possession the following information:
- History of the various layers of the roadway, nature, thickness, age and type of construction,
- Traffic carried (all vehicles and heavy lorries) in each direction.

The most important problem for semi-rigid structure is the transverse reflection cracks. The evolution of cracks is illustrated in Figures A4.5, A4.6 and A4.7.

![Figure A5.7 – Stage 3: material breaking away to form potholes](image)

The maintenance must achieve the following objectives:
- Limit the development of cracks to fine cracks that not eroded.
- Preserve the water tightness of the road.
- Limit deterioration of the interface of surface layer/hydraulic layer.
The first line of defence is to seal cracks with a bituminous mastic. This will restrict water ingress and help to prevent deterioration of the interface between the asphalt concrete and hydraulic material.

If deterioration has proceeded to the second stage, one of the following techniques can be applied [ref 5]:

- Sand asphalt interlayer; the binder content is 10 to 12 %. Some mixtures contain fibres. The nominal thickness is 2 cm.
- SAMI (Stress Absorbing Membrane Interlayer); these comprise a layer containing between 2 and 2.5 kg/m² of bitumen binder rich in elastomer.
- Membranes covered by cold asphalt concrete mix generally with grain size 0/4 mm and then with an asphalt concrete wearing course.
- Geotextile interlayer; this is generally a non-woven fabric material. The geotextile used for this technique is intended to serve amongst other things as a reservoir for the binder applied at a dosage level of between 120 and 250 g/m². It is packaged in rolls of 100 to 150 m with a width that varies from 1.9 to 3.8 m. The glass grid technique associated that can be combined with geotextile is no longer employed to treat shrinkage cracks of hydraulic layers. It is more commonly reserved for the treatment of fatigue problems on weak roads.
- Fibre modified asphalt concrete; these asphalt concretes with grain size 0/6 mm or 0/10 mm are made using straight bitumen. The addition of fibres, generally organic (cellulose) fibres, allows a larger quantity of binder to be used while maintaining good rut resistance. This technique is similar to that of the sand asphalt interlayer with better rut resistance due to the aggregate skeleton. This technique, with 3 to 4 cm thickness of asphalt surfacing, also improves the structure of the road. The intended proportion of binder is of the order 6.8% to 7.0% by weight of the aggregate with a fines content greater than 10 %. The asphalt concrete is produced in traditional asphalt plants. Particular attention must be given to the proportion of fines, as well as the mixing temperature and the period of mixing to obtain homogeneity.

A5.4 Economics

To compare several types of structure during their life, taking account of the initial and maintenance cost, we will refer to a recent study [1], based on reliable, easily verifiable data, from known sources.

The whole-life cost of the pavement during its life is the sum of initial cost, maintenance cost, and user cost during maintenance.

A5.4.1 Criteria used for the capital-maintenance study

Period under analysis
In carrying out such studies, it is advisable consider a relatively long time-scale for the calculations. A period of analysis of 30 years is traditionally adopted for road projects in France.

Some sections under study had not achieved 30 years. Therefore, the expected time and form of the last maintenance treatment was predicted from an extrapolation based on data collected on similar projects. This ensured that, after 30 years, all the pavement sections had good structural behaviour and a surface layer that met the comfort and skid-resistance objectives of a new pavement.

Maintenance programmes
The routines and the work actually carried out is what has been analysed on each road section, except for the last maintenance treatment, as explained above. For the latter, an analysis of the pavement was made in 2000, which was used to determine the structural and surfacing treatment needed to restore the characteristics of a new pavement.

Economic criteria
The capital cost of the semi-rigid pavements was calculated using known data, taking the year 2000 as a point of reference. All the calculations of pavement costs were performed per linear metre for one side of a two-lane motorway. It accounts for the pavement itself, together with the hard shoulder and facilities such as roadside drainage for concrete pavements, the emergency lanes and half of the central reservation. The economic calculations were carried out in Euros adjusted to year 2000 values. This follows the normal practice of French economic studies.

The costs of maintenance are subjected to a discount rate. This coefficient, set by the Commissariat Général au Plan (an agency of the French Finance Ministry), was equal to 8% in 2000 and 4% in 2005.

The total updated cost of capital and maintenance is given by the following formula:

\[ C = I + E \]

Where, \( I \) is the cost of capital and \( E \) the accumulated cost of maintenance given by:

\[ E = \sum_{n=1}^{30} \frac{e_n}{(1+i)^n} \]

Where,  
\( i = \) discount rate;  
\( n = \) year of maintenance;  
\( e_n = \) maintenance expenditure in the year \( n \).

The discount rate expresses a collective national preference for spreading out over time the satisfaction provided by improvements. This is a political decision, and a private investor or a motorway management company, for example, may have a different policy. The discount rate is therefore different from the rate of interest and independent of inflation.

The results of this type of study are strongly influenced by the costs of the different techniques. In the same region and for the same work, prices may fluctuate with each tender, depending on the economic context in terms of cost of constituents, quantity of work to be carried out, etc.

The cost of investment was determined according to that of relevant French projects in 2000. The maintenance was determined from an analysis of all pavements of the French national network, with the following characteristics:

- New pavements longer than 1000 m;
- Heavy goods traffic of 600 HGV or more per day, in each direction;
- Pavements constructed between 1975 and 1995 (Sections constructed before 1975 do not comply with the 1977 pavement catalogue, and those built after 1995 are too recent to be eligible).

Each maintenance scheme was classified by thickness, as follows:

- Class A: Very thin overlay of less than 3 cm (very thin asphalt overlay, surface dressing, ultra-thin asphalt overlay);
- Class B: Thin overlay of between 3 and 5 cm (essentially a thin asphalt overlay);
- Class C: Asphalt overlay of between 5 and 9 cm (semi-coarse asphalt concrete or anti-cracking treatment plus a thin or very thin asphalt overlay);
- Class D: Overlay of between 9 and 14 cm (8 cm semi-coarse asphalt concrete + very thin asphalt overlay)
- Class E: Reinforcement with an overlay of 14 cm or more (10 cm road base asphalt + 6 cm semi-coarse asphalt concrete + thin or very thin asphalt overlay).

For each group of roads studied, the results can be plotted in terms of the cumulative frequency of maintenance per year, broken down by classes of maintenance, as illustrated in Figure A5.8.
Thus, once the average costs of maintenance of the pavement is known, their capital plus maintenance cost, based on the various thickness hypotheses, as given in the specification catalogue of the national network can be calculated [3].

Table A5.2. Costs over 30 years for semi-rigid pavements, per metre of pavement (ref. year 2000) in euros

<table>
<thead>
<tr>
<th>Initial cost</th>
<th>Maintenance cost</th>
<th>TOTAL COST in €</th>
</tr>
</thead>
<tbody>
<tr>
<td>278</td>
<td>56</td>
<td>334</td>
</tr>
</tbody>
</table>

A5.4.4 Economic conclusion
The study was based mainly on the behaviour of roads constructed according to the specifications contained in the 1977 Catalogue [ref 6]. However, the French government decided in 1975, after the first "oil crisis" to reduce the thickness of asphalt concrete wearing course to reduce bitumen usage. This maintenance practice for semi-rigid pavements was continued at an increasing rate thereafter (see Figure A5.8).

The impact of the capital investment remains important in calculating the overall cost of a highway during its lifetime. It represents about 83% of the total cost with maintenance accounting for the remaining 17%.

A5.5 Conclusion
To date, pre-cracking is specified for new semi-rigid pavements on the French network for pavements carrying over 750 lorries/day/lane, and it is recommended for all other roads.
thickness of wearing course in asphalt concrete must be at least 8 cm, with a minimum of 10 cm for heavy traffic.

French experience has shown that composite pavements (*mixte structure*) with 20 cm of asphalt concrete and a lower base and sub-base of hydraulically bound material perform well. For these pavements, there is less need for anti-reflection cracking treatments, like sand asphalt, fibre modified asphalt concrete or SAMI for the first maintenance treatment.

Innovative, purely hydraulically bound pavement solutions are currently under development, which hopefully will provide a long-life pavement. The High-Performance Concrete (HPC) Carpet [7, 8] which consists of laying a thin, unbounded, reinforced high-performance hydraulic flexible wearing course over a cracked hydraulic layer. Ingress of water into the pavement is prevented by laying a thick polyethylene sheet under the HPC, and owing to the high durability of HPC as a material, it is likely that such a concept will allow existing rigid or semi-rigid pavements to be upgraded to long-life pavements.
References

2 – SETRA-LCPC : Conception et dimensionnement des structures de chaussées (1994)
4 – M LEFORT Technique for limiting the consequences of shrinkage in hydraulic-binder-treated bases; Congress RILEM, Maastricht October (1996)
6 – SETRA-LCPC ; Catalogue 1977 des structures types de chaussées neuves (1977)
A6. Greece

Practice and experience on upgrading semi-rigid pavements into long-life pavements in Greece

Part of the Greek highway network consists of semi-rigid pavements and since the implementation of long-life pavements (LLP) has recently attracted great interest, particular attention has been given to the investigation of the possibility of upgrading the existing semi-rigid structures into LLP. Currently there is not a specific official guide for upgrading the semi-rigid structures, however there are a few pilot studies and projects towards this as well as several directions and recommendations (only in Greek) published by the related authorities of the Ministry of Public Works, Universities, etc.

Semi-rigid pavements in Greece were first considered during the 1970s, as a result of the Oil crisis which made asphalt based mixtures and materials expensive compared to cement based materials. This was also underscored by the fact that the Greek cement industry has always been at a high level of production and quality. The semi-rigid pavements used in Greek highways are based on the French practice (1984 French guide) with some necessary adaptations regarding local materials, climatic conditions, etc. The first constructions of pavements based on French practice were made around 1990; the related Greek design method was based not only in the aforementioned guidelines but also in analytical model calculations and all the available experience of that period. The design load was 13 tn on a single axle, and the designed structure consists of two layers of Cement Stabilised Base Material (CBM) with a total thickness of between 30-38 cm, and an asphalt concrete wearing course of between 8-10 cm. The design concept assumed that the load is taken by the CBM and not by the asphalt concrete and consequently the critical points of the structure were the bottom of the upper CBM layer and the bottom of the lower CBM layer (tensile strain).

A few years after construction, these aforementioned structures suffered from significant failures mainly due to reflective cracking and material pumping effect, but some fatigue cracking also appeared. It should also be noted that the pavement suffered from poor ride quality (due to roughness) but it was very difficult to identify whether this was caused by traffic or it due to poor construction practice.

Several projects were undertaken to deal with the problems of these structures and the National Technical University of Athens (NTUA) has participated in the majority of them. In recent years, a rational strengthening approach, which is comparable to upgrading to a long-life pavement, was undertaken. More specifically, in areas where the bound base material was significantly damaged the original structure was rehabilitated using in-depth cold recycling with foamed asphalt stabilisation. All of the asphalt concrete layer and part of the CBM was rehabilitated using the cold in place recycling (CIPR) technique and afterwards the pavement was additionally overlaid with asphalt concrete in order to assure a long pavement life. In the areas where the CBM was proved to be in a good condition, a special reinforcement with a modified asphalt concrete overlay (sometimes using geotextiles) was undertaken. A relatively thick asphalt concrete overlay was applied in order to ensure resistance to reflective cracking for a long-life. More information is available in (Loizos & Papavasiliou 2006, 2007).

Since these rehabilitated structures are in sections of major highways with a significant, heavy traffic load, continuous monitoring is ongoing in order to certify whether the pavement will have the expected performance. It is also remarkable that some of these sections have been included in recent highway concession projects, in which the private sector has undertaken the maintenance and management of the road for a 40-year period.

Specifications for the procedure of upgrading semi-rigid pavements are currently under preparation; because of the high importance of the LLP concept for Greece, the NTUA has a leading national role in this process.

References:


A6.1 Research Needs

Due to the fact that the use of the LLP concept in semi-rigid structure is very challenging for Greece, especially in terms of private financed / concession projects, there are several aspects that have to be further investigated. The following are among the most important areas for developing understanding:

- The mechanism(s) of reflective cracking of thick and thin asphalt pavements.
- The interaction between CBM and asphalt concrete in terms of cracking initiation and propagation.
- Suitability of different reinforcing treatments (modifiers, geotextiles, etc) in upgrading existing semi-rigid pavements into LLP.
- Comparison of maintenance management alternatives for “Long – Life” concession projects.
A7. Hungary

A7.1 Background

Initially, mainly macadam-type pavements were built on the relatively highly-trafficked Hungarian roads. Later these pavements became less suitable for the increasing motor vehicle traffic; therefore, asphalt courses were laid on the macadam bases. Due to the low bearing capacity and ‘post-compacting’ nature of macadam-type base courses, the use of lean cement concrete bases gained popularity; these bases provided a strong, non-post-compacting base course type and they were considered and termed together with the uppermost asphalt layers as ‘semi-rigid pavement structures’.

Coincidentally, the construction of mixed-in-place cement stabilisation started in Hungary during the 1950’s in order to strengthen and to make the local soils impermeable (e.g. silt subgrade). Since the 1970’s almost all cement stabilisation layers were built using the mixed-in-plant method so that their homogeneity is ensured. Such structures with asphalt layers above are also considered semi-rigid pavements.

Semi-rigid pavement structures have been built on urban roads since the 1920’s. A lean concrete base course and one or two asphalt layer(s) is a typical structure which can provide a skid resistant and dust free pavement surface.

The Hungarian pavement terminology uses the terms of

• mixed pavement structure (20-30 cm one or two-layered hydraulically bound base course and 16-22 cm three-layered asphalt surfacing),
• semi-rigid pavement structure (15-20 cm lean concrete base and 4-12 cm one or two-layered asphalt surfacing).

Semi-rigid pavement structures are widely used in urban areas.

After World War II, almost all pavements were built using cement concrete (rigid construction) on the Hungarian main roads and later expressways. However, their construction discontinued in the mid-70’s, mainly due to the following problems:

• insufficiency of expansion joints,
• major surface damages due to de-icing agents applied (inappropriate cement concrete technique and poor cement quality).

The majority of existing concrete pavements were resurfaced with asphalt layers during the past decades. Therefore, another type of semi-rigid pavement evolved.

A7.2 Design and construction

The majority of the Hungarian pavement structures currently built is of semi-rigid type.

The current Hungarian Road Technical Specification for semi-rigid pavements specifies the following structure types in the highest category ‘K’ (10-22 million repetitions of 100 kN standard heavy axles [10-22 msa100] during the forecasted life cycle):

• 15 cm hydraulically bound stabilised base and 21 cm asphalt pavement,
• 20 cm hydraulically bound stabilised base and 19 cm asphalt pavements.
• 20 cm lean concrete base course and 17 cm asphalt pavements.

In 2003, several technological variants were developed and tested for a new traffic category ‘R’ (above the present highest category ‘K’) for traffic levels of 22-50 msa100. One of the variants was a semi-rigid alternative:
4 cm high modulus stone mastic asphalt (with modified binder, $D_{\text{max}}=12$ mm),
2x8 cm high modulus base courses (with modified binder, $D_{\text{max}}=12$ mm),
2 cm SAM (stress-absorbing membrane layer)
20 cm lean concrete base course, $D_{\text{max}}=12$ mm (crack initiation using KRAFT-method).

This variant together with a composite type of pavement and a rigid type of pavement was constructed in a 350 m long experimental section on a Hungarian main road in 2003; this section will undergo long-term monitoring.

A7.3 Assessment and upgrading

There is no special methodology in Hungary for the assessment of long-life semi-rigid pavements. Every five years, the bearing capacity of every pavement structure is determined by KUAB falling weight deflectometers.

The typical form of fatigue deterioration of semi-rigid pavements is the surface cracking immediately before the end of the service life. The main reason for this type of deterioration is that the mechanistic design methods used concentrate on the damaging effect of heavy vehicles (fourth-fifth power of axle loads!). However, recent experience and computer models prove that, in the case of semi-rigid pavement structures, even light axle loads (passenger car passes) can cause considerable stresses, and consequently cracks form in the uppermost asphalt layers due to the rigid base. New types of design methods are needed with different load power for asphalt and hydraulically bound layers.

Other observations show that surface pavement cracks can be initiated by the heavy vehicles with modern axle configurations by the so-called ‘anvil effect’ and by the additional loads coming from the insufficient bond between pavement structural layers.

The present distribution of semi-rigid pavement structures (859 km) in the Hungarian national highway network (total length of 30,000 km) can be found in the following tables.

<table>
<thead>
<tr>
<th>Road category</th>
<th>Length (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Motorways</td>
<td>38</td>
</tr>
<tr>
<td>Main roads</td>
<td>769</td>
</tr>
<tr>
<td>Secondary roads</td>
<td>52</td>
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<tr>
<td>Total:</td>
<td>859</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Traffic category (pcu/day)</th>
<th>Length (km)</th>
</tr>
</thead>
<tbody>
<tr>
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<tr>
<td>1001-5000</td>
<td>218</td>
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<tr>
<td>Min. 5001</td>
<td>625</td>
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<tr>
<td>Total:</td>
<td>859</td>
</tr>
</tbody>
</table>
### Asphal thickness (mm) and Length (km)

<table>
<thead>
<tr>
<th>Asphalt thickness (mm)</th>
<th>Length (km)</th>
</tr>
</thead>
<tbody>
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<td>41-59</td>
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<td>250-299</td>
<td>59</td>
</tr>
<tr>
<td>300-349</td>
<td>63</td>
</tr>
<tr>
<td>350-399</td>
<td>40</td>
</tr>
<tr>
<td>min. 400</td>
<td>21</td>
</tr>
<tr>
<td>Total</td>
<td>859</td>
</tr>
</tbody>
</table>

### Present age of surface layer (year) and Length (km)

<table>
<thead>
<tr>
<th>Present age of surface layer (year)</th>
<th>Motorways</th>
<th>Main roads</th>
<th>Secondary roads</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-5</td>
<td>0</td>
<td>60</td>
<td>2</td>
</tr>
<tr>
<td>5-10</td>
<td>17</td>
<td>268</td>
<td>16</td>
</tr>
<tr>
<td>10-15</td>
<td>5</td>
<td>192</td>
<td>4</td>
</tr>
<tr>
<td>15-20</td>
<td>0</td>
<td>86</td>
<td>5</td>
</tr>
<tr>
<td>20-25</td>
<td>0</td>
<td>89</td>
<td>11</td>
</tr>
<tr>
<td>25-30</td>
<td>15</td>
<td>38</td>
<td>14</td>
</tr>
<tr>
<td>30-35</td>
<td>0</td>
<td>36</td>
<td>0</td>
</tr>
<tr>
<td>Total</td>
<td>37</td>
<td>769</td>
<td>52</td>
</tr>
</tbody>
</table>

It can be seen that the majority of the Hungarian semi-rigid pavement structures can be found on main roads and the total thickness of asphalt layers is uniformly distributed in the range of 40-400 mm. Presently, the age of 36 km of Hungarian semi-rigid pavement structures (4 %) exceeds 30 years which can already be considered long-life pavements.

### A7.4 Maintenance

The typical non-structural form of deterioration in semi-rigid pavements is reflection cracking through asphalt layers above the thermally induced cracks in hydraulically bound courses. As it is known, the horizontal movements of cracked cement bound layers due to thermal effects and the vertical movement due to passing axle loads cause shear stresses in asphalt layers and can initiate cracking. Practical experience shows that the effect of traffic is more significant and more harmful since the high number of expansion joints and the use of stress-absorbing SAM and SAMI layers or geogrids cannot delay the creation of this type of crack.

Hungarian experience has identified four techniques that appear to be effective in avoiding reflection cracking:

- KFAFT-methodology (precracking),
- initiation of microcracks (by vibro rollers),
- inverse pavement structure (dense unbound crushed stone layer of 10 cm between hydraulically bound and asphalt courses),
- two-layered hydraulically bound base layer (this case, the two layers are cracked transversally not in the same vertical even, so no vertical movements are expected).
The maintenance requirements for non-structural deterioration in semi-rigid pavements are similar to fully-flexible pavements.

**A7.5 Research needs**

- A reliable solution for preventing (or at least managing) reflection cracking.
- Demonstration of the long-term economics of semi-rigid pavement structures versus fully-flexible or rigid variants.
- The economic comparison of semi-rigid and composite structures, e.g. thin asphalt layer on continuously reinforced concrete pavement.
A8. The Netherlands

A8.1 Introduction

In the Netherlands, semi-rigid pavements usually have a sand cement road base or a road base of asphalt granulate stabilised with cement or with cement and bitumen emulsion. Some older pavements also have a lean concrete road base. Road bases of furnace slags or phosphoric slags are usually considered as slightly bound road bases, leading to a flexible pavement structure. However these road bases are often found to have high stiffness values in practice.

Bound road bases traditionally enjoyed a limited popularity in the Netherlands due to the uneven settlements that occur in large parts of the country. However sand cement stabilisation received some increased attention during the seventies and eighties. In this period full depth asphalt concrete pavements were rather popular, but the road construction sands available in the Netherlands caused problems as these are generally of poor quality, with a rather uniform grain size and round grain shape. This has lead to a number of applications of cement stabilisation of the sand to increase the stability of the capping layers.

In recent years, sand cement has again become less popular. The main reasons for this are the experiences with reflective cracking and the resulting frequent maintenance. Recently, almost all pavement designs include only a lightly bound road base or (less frequently) an unbound road base, in almost all cases consisting of a re-used material.

Cement bound asphalt granulate has been very popular in recent years. In particular, the presence of tar in much of the reclaimed asphalt has prohibited hot recycling (i.e. in new asphalt) and has therefore lead to a wide use of asphalt granulate in road bases with cement stabilisation. However recent legislation implies that the stabilisation with cement is not sufficient as a measure to prevent tar components from leaching out and the use of tar contaminated asphalt in cement stabilised road bases is now forbidden. Only uncontaminated asphalt granulate can now be used in road bases but this is against the present policy of reusing materials in an as high value application as possible, in this case meaning hot recycling in new asphalt.

A8.2 Pavement design and construction

The design method for sand cement stabilisation dates back to 1983 - 1987 and was developed by a cooperation of contractors and the cement industry. The first edition (1983) [1] assumed that shrinkage cracks occur at distances of at least 3 m and was based upon multilayer calculations and analysed the risk of fatigue deterioration in the sand cement in uncracked parts of the pavement. These analyses were based upon a simple fatigue characteristic (see figure A8.1).

The full axle load spectrum is used. The reflection of existing shrinkage cracks was delayed by the application of sufficient asphalt thickness. This method was used for some time by the Ministry of Transport, observing an asphalt thickness of at least 14 cm for heavily trafficked roads to retard reflection cracking.

![Image of fatigue characteristic graph]
In 1988 a revised method was produced by the same parties [2]. It was now based both on fatigue and fracture mechanics and analysed the number of load repetitions until a crack had initiated in the sand cement and had propagated to the surface. It did not address the effect of thermal movements over the cracks. The traffic spectrum was now dealt with by using load equivalency factors which, however, were not provided. In some cases this led to the application of the fourth power law to arrive at an equivalent number of standard axles, which is very questionable considering the fatigue and cracking behaviour of sand cement. The Ministry of Transport, was losing interest in the application of cement stabilisation, and did not adopt this method. For the same reason, the policy regarding the minimum asphalt thickness has not been updated, although the maintenance frequencies for the reflection cracks experienced during the eighties would now be considered unacceptable.

Pavement design with cement bound asphalt granulate is based upon the simple equivalency law 1:2.5, implying that 25 cm of asphalt granulate road base allows a reduction of the asphalt thickness of 10 cm compared to the thickness required for a fully bituminous solution. Again a minimum thickness of the bituminous layers of 14 cm is observed for heavily trafficked roads. However the required asphalt thicknesses are usually considerably higher.

Recently the introduction of design and build contracts has led to a large number of designs with some form of cement bound road base. This is explainable as in many cases the main selection criterion is the lowest price and pavements with cement bound road bases are cheaper in the initial construction. Generally these contracts do not include a long warranty or maintenance period. Also the selection is usually not based on life cycle analysis. In a number of cases this has led to discussions about the long term durability and maintenance needs of the proposed designs, which however could not be solved due to lack of a product – independent, generally agreed design method for bound road bases.

**Mix design and quality control**

Both for sand cement stabilisation and for cement bound asphalt aggregate a mix design procedure [3] must be followed. This leads to a composition of the material including the required cement content.
Cement bound asphalt granulate must be designed to give a compressive strength of 1.5 MPa after 7 days and 2.0 MPa after 28 days. These requirements have been lowered relative to the former values (2.0 MPa and 3.0 MPa) to reduce the reflective cracking that was observed in practice. However occasionally reflective cracking is still observed. For this reason the addition of cement emulsion or foamed bitumen is under consideration.
Sand cement must be designed to give a compressive strength of 5.0 MPa after 28 days; there are plans to reduce this strength requirement to reduce reflective cracking, however this has not been implemented. A point of concern is to maintain the durability of the sand cement against the effects of frost and moisture.

After construction of the road base, drilling cores are sampled that should have an average compressive strength of at least 1.5 MPa.

**Construction**

Cement bound asphalt granulate road bases and sand cement road bases are usually constructed in one course. Cement bound asphalt granulate road bases can be produced by mix in plant, mix in place (materials brought in from elsewhere) or on site recycling (use of materials present on site). Sand cement road bases can be mixed in plant or mixed in place.

Usually the road base is first sprayed with a bitumen emulsion and sometimes covered with 2/6 chippings.

Occasionally the first asphalt layer is laid within 24 hrs after construction; this can help produce a finer pattern of narrow cracks instead of wide cracks at longer distances (usually about 5 m), it also protects the road base from dehydrating.

If the first asphalt layer cannot be applied within 24 hours, a waiting period should be observed until the road base can develop sufficient strength (at least 1.5 MPa average compressive strength) to carry the construction traffic. For cement bound asphalt granulate road bases this usually takes at least one week; the application of a surface dressing on the road base after construction is always necessary in this case.

The measures for prevention of reflective cracking consist of:

- aiming at low compressive strengths (see above)
- spraying the road base with emulsion immediately after construction
- preferably apply the first asphalt layers within 24 hours after laying the road base
- observing sufficient asphalt thickness
- sometimes using a SAMI
- adding bitumen emulsion or foamed bitumen to the asphalt granulate cement mix (however there is no mix prescription for this in the Dutch Standard)

**Economics**

In general it can be stated that pavements with a sand cement road base can be constructed at lower initial costs. Certainly for high traffic loading, significant savings can be achieved compared to flexible solutions which will require considerable asphalt thicknesses. However it is necessary that the problem of reflective cracking is controlled to maintain an economical benefit on a life cycle basis. Considering the facts that in the past many semi-rigid pavements have required considerable maintenance, and at present day the consequences of maintenance on traffic hindrance and environment are appreciated at high costs, in a life cycle analysis the initial costs will be much less important.
A8.3 Assessment

The structural assessment of semi-rigid pavements usually consists of FWD analysis and visual condition assessment. Falling weight analysis serves to determine the stiffness moduli of the bound road base. In many cases the asphalt stiffness has to be set at a fixed value for this purpose due to the usual back-calculation problems encountered with semi-rigid pavements. Usually the road base stiffness in a loaded (wheel path) and an unloaded situation are compared. High stiffness values for the road base in all locations are a sign that structural deterioration is not occurring. In this case strain levels in the road base are calculated. If this strain level does not exceed 50 µm/m, the pavement is considered to be free from any fatigue deterioration in the road base. Provided that reflection cracking is not significantly present or can be solved effectively, the pavement can meet the long-life definition. However the effectiveness of measures to repair cracks is often not satisfactory.

In some cases the stiffness values in the wheel tracks is found to be considerably lower than those between the wheel tracks. This is obviously a sign of traffic-related deterioration in the road base. If this does not lead to apparent cracking, the road base is sometimes considered to disintegrate into an unbound road base and an overlay thickness is calculated, assuming an end value for the stiffness of the road base.

In some cases the stiffness value in unloaded sections is found to be low. This is a sign of non-structural deterioration, which can be an effect of frost and/or moisture. In this case again, if no serious reflection cracking is present, the usual approach is to consider the road base as an unbound road base and to determine the necessary overlay thickness.

A8.4 Maintenance

The maintenance of semi-rigid pavements is similar to that of the maintenance of flexible pavements but there is an obvious emphasis on crack sealing. Transverse reflection cracking is usually treated by overbanding. In many cases it has proven necessary to treat the cracks more or less annually (i.e. especially after the winter period). It is believed that crack sealing is essential to maintain the bearing capacity of semi-rigid pavements.

Several experiments have been done with inlays and/or overlays with reinforcing grids (steel, synthetics, glass) textiles and SAMI’s, with varying success.

A8.5 Research needs

To use fully the potential of semi-rigid pavements as an economic long-life solution for heavy duty pavements, it is necessary to control the reflection cracking problem. For this reason, the effectiveness of the possible measures for reflection crack prevention (precracking, microcracking with rollers, SAMIs/reinforcement, asphalt laying within 24 hours, strength reduction, addition of emulsion etc) must be determined. The main questions are: what are the experiences in other countries and more important, what are the reasons for different experiences.

A second important question is how the risk of crack formation and propagation can be determined on the basis of performance properties of pavement materials. This is necessary because of the introduction of design and build contracts. In this situation, semi-rigid solutions are often used for which it is difficult to judge the long-term behaviour. Effective models in combination with crack growth properties derived from performance tests could serve as important decision-support systems in these new contractual relations.
A8.6 References

[1] Dimensioneren met zandcement. (Pavement design with sand cement). Association of Stabilisation Contractors (SAG) and Dutch cement Industry Association (VNC), November 1983

[2] Funderen met zandcement. (Road bases with sand cement). Association of Stabilisation Contractors (SAG) and Dutch cement Industry Association (VNC), November 1988

A9. Poland

A9.1 Background

About 15% of all national roads in Poland are of semi-rigid pavement construction (a total length of about 18,000 km). These pavements typically have a base of hydraulically bound material or lean concrete base and asphalt surfacing.

A9.2 Design and Construction

Semi-rigid pavements in Poland typically have a base of cement bound material that varies from 16 to 22 cm and an asphalt surfacing that varies from 10 to 29 cm; 29 cm is required for designs for cumulative traffic of over 14,6 msa (100 kN ESALs). The design period for semi-rigid pavements is typically 20 years.

Standard designs for new semi-rigid pavements are given in the “Flexible and Semi - Rigid Pavement Design Catalogue”. The thickness of the base layer is dependent on the design traffic, base material type and strength, and modulus of elasticity of the sub-base layer. The thickness of the surfacing asphalt layer is dependent on the design traffic only, the asphalt layer is principally designed to minimise the risk of reflection cracking.

The requirements for the cement bound bases are as follows:

• cement stabilised soil or cement bound macadam:
  - Cube strength: 2,5 to 5,0 MPa,  
  - Modulus of elasticity: greater than 4,5 GPa,  
  - Poisson's ratio: 0,25

• lean concrete:
  - Cube strength: 6,0 to 9,0 MPa,  
  - Modulus of elasticity: greater than 12,9 GPa,  
  - Poisson's ratio: 0,20

There are recommendations for the use of a stress-absorbing membrane layer at the interface between the rigid base with flexible layers and with pre-cracked cement base.

The current Polish technical specification in “Flexible and Semi - Rigid Pavement Design Catalogue” specifies the following structure types for the highest traffic category KR6 (over 14,6 msa100 or 8,3 msa115 during the forecasted life cycle):

• With bound stabilised base:
  22 cm hydraulically bound stabilised base,  
  29 cm asphalt layers pavement,

• With lean concrete base course:
  22 cm lean concrete base course  
  25 cm asphalt layers pavements.

A9.3 Assessment and Upgrading

In Poland there are no explicit methods for the assessment of long-life semi-rigid pavements. The general method for the assessment of semi-rigid pavements is described in the “Flexible and Semi - Rigid Pavement Maintenance and Rehabilitation Catalogue” and “Assessment of Condition Pavement System” (“SOSN” System).
To evaluate the road pavement condition the following parameters are taken into account: cracking, depths of ruts, bearing capacity (FWD), condition of the surface, evenness and skidding resistance. The assessment of pavement condition is provided every 3 years for all national main roads.

The results obtained from the visual assessments and from automated measurements enable the definition of each particular parameter of the pavement condition.

According to the established parameters, pavement condition is classified into one of four categories: A, B, C or D. Category A is the highest category for which the pavement is free from structural deterioration. Category B permits some type of deterioration. The roads ranked to Category C (unsatisfactory condition) require rehabilitation within the next 2-3 years, road ranked to category D (bad condition) require immediate rehabilitation. Rutting and low bearing capacity are the major deterioration modes in Polish roads.

A pavement that satisfies a Class A condition will have the following characteristics:

- Visual condition: widely spaced cracks that are of a fine width
- No evidence of foundation degradation.

In Poland, two upgrading methods are used for semi-rigid pavements. One method is based on deflection, which determines the required additional reinforcement of the flexible upper layer. The thickness of the upper layer is dependent on the current deflection measurement. This method is used for the low traffic category KR1 and KR2 (to 0,51 msa100). The other method is a mechanistic method that is based on stress and strain analysis in the structure pavement. This method is used for the high traffic category from KR3 to KR6 (over 0,51 msa100).

The “Flexible and Semi - Rigid Pavement Maintenance and Rehabilitation Catalogue” contains general advice for the assessment of maintenance requirements.

A9.4 Maintenance

The “Flexible and Semi - Rigid Pavement Maintenance and Rehabilitation Catalogue” covers the maintenance requirements for both semi-rigid and fully-flexible pavements. Generally, the maintenance requirements for semi-rigid pavements are similar to the requirements for fully-flexible pavements.

As it is well known, reflection cracking is a particular type of degradation in the structure of semi-rigid pavements. The “Flexible and Semi - Rigid Pavement Maintenance and Rehabilitation Catalogue” contains 23 techniques of maintenance treatment; among these, four techniques are treatments for reflection cracking in semi-rigid pavement. For the repair of reflection cracking geogrid and geotextiles are used.

A9.5 Economics

There are no specific issues for the economic evaluation of long-life semi-rigid pavements.

A9.6 Research Needs

1. Problems associated with reflection cracking:
   - preventing of reflection cracking,
   - optimise the maintenance strategy for reflection cracks,
   - mechanism of reflection cracking in different pavement structures.

2. An optimised strategy for managing surface distress.
3. New methods for the design and construction of semi-rigid pavements.
   • of particular importance in long-life pavements.

   • developing an adapted cost benefit model for long-life semi-rigid pavements.

5. Problems connected with the influence of waves arising in the structure of multi-layered pavements, especially in long-life pavements.
   One of the crucial impacts, which act on a multi-layered pavement, is wave character of loads for moving vehicles. Neglecting the influence of waves on the pavement results in not fully appreciating its influence on one hand, especially on its durability; and the difficulty of computational character requiring application of complicated mathematical mechanism on the other hand. However, through not considering wave matching of layers and propagation of surface and volume waves coming from moving vehicles, there is a risk of premature fatigue damage and at the same time to an uneconomical construction of pavements. Wave reflections cause pulsation of stresses and strains in a layer, the magnitude of which depends on wave velocity, thickness and impedance of wave layer.

A9.7 References

1. “Flexible and Semi - Rigid Pavement Design Catalogue” - General Directorate for Public Roads and Road and Bridge Research Institute, Warsaw 1997
2. “Flexible and Semi - Rigid Pavement Maintenance and Rehabilitation Catalogue” - General Directorate for Public Roads and Road and Bridge Research Institute, Warsaw 2001
A10. Spain

A10.1 Background

The complete Spanish motorway network has a total length of about 165,000 km. The State Road Network consists of about 25,000 km of the most heavily trafficked sections of this network with approximately 25% of these being semi-rigid pavements (See Figure A10.1).

![Figure A10.1. Distribution of pavement type in the Spanish national road network.](image)

A10.2 Design and construction

There are three types of semi-rigid pavement that are classified according to the nature of the cement-bound material used as structural base layer (see section on material characterisation for details of these materials):

The first consists of a compacted concrete base with a typical thickness of between 20 to 25 cm, and an asphalt surfacing of between 8 and 10 cm thick. Only about 1% of the semi-rigid pavements have this construction. In 1989, the revised Spanish pavement design catalogue included this pavement type in which it was compulsory to pre-crack every 7 m, to mitigate reflection cracking. More than 120 km of highway were constructed using this pavement type. However, serious problems with reflection cracking resulted in a ban on these pavements and they were not included in the recently updated version (2003) of the Spanish pavement design standard (Ministerio de Fomento, 2003a). [1]. Lately, with the development of better pre-cracking techniques to mitigate reflection cracking, this kind of pavement has made a come-back, albeit with some reservations.

The second pavement type consists of a cement bound granular base with a typical thickness of 20 to 25 cm, and an asphalt surfacing of between 10 to 20 cm thick. The first pavements of this type were built in the early 60's and since then it has been used profusely for high traffic sections of the network. Now, this pavement type accounts for more than 5,000 km or about 57% of the semi-rigid pavement network. Again, the reflective cracking problem has resulted in fewer pavements of this type being constructed in recent years. However, experience has demonstrated that if these pavements are well constructed and maintained, they can have a very long-life. For example, the N-I highway in northern Spain has carried more than 22 million trucks in 12 years without any structural rehabilitation.

The third type of semi-rigid pavement consists of a soil-cement base, with a thickness typically in the range 20 to 30 cm, and an asphalt surfacing thickness of between 5 and 25 cm, that depends on the traffic volume and the subgrade bearing capacity. This type of pavement has been used
since the sixties for all levels of traffic, with almost 40% of the total national road network being built with this pavement type. Nowadays, it is the preferred option since it is more economic and there are environmental benefits that result from utilising treated soil.

The Basque region in northern Spain have adopted a variation of these semi-rigid pavements types. A hydraulic binder derived from Slag is used instead of cement to treat the granular material. The gravel-slag base is typically between 20 to 25 cm thick if a soil-cement treated sub-base is used, or it can be up to 35 cm without. The asphalt surfacing thickness is typically 12 to 18 cm, depending on the traffic volume and the subgrade bearing capacity. This type of pavement has performed very well in recent years with no serious reflection cracking problems because of the low initial strength of slag bound materials.

Table A10.1 gives the design of semi-rigid pavements from the Spanish pavement design catalogue for the national road network.

### Table A10.1 Spanish semi-rigid pavement design catalogue for the national road network.

<table>
<thead>
<tr>
<th>Traffic *</th>
<th>Soil - cement base</th>
<th>Granular cement treated base</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><strong>Ev2&gt;60</strong> (Mpa)</td>
<td><strong>Ev2&gt;120</strong> (Mpa)</td>
</tr>
<tr>
<td><strong>Ev2&gt;60</strong> (Mpa)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.000 &gt; AADTp &gt; 2.000</td>
<td>25 AC</td>
<td>20 SC (1)</td>
</tr>
<tr>
<td>2.000 &gt; AADTp &gt; 800</td>
<td>20 AC</td>
<td>20 SC (1)</td>
</tr>
<tr>
<td>800 &gt; AADTp &gt; 200</td>
<td>15 AC</td>
<td>15 AC</td>
</tr>
<tr>
<td>200 &gt; AADTp &gt; 100</td>
<td>12 AC</td>
<td>12 AC</td>
</tr>
<tr>
<td>100 &gt; AADTp &gt; 50</td>
<td>10 AC</td>
<td>10 AC</td>
</tr>
<tr>
<td>50 &gt; AADTp &gt; 25</td>
<td>8 AC</td>
<td>8 AC</td>
</tr>
<tr>
<td>AADTp &lt; 25</td>
<td>5 AC</td>
<td>5 AC</td>
</tr>
</tbody>
</table>

* AADT of heavy lorries in the design lane in the first year.

(1) Pre-cracking is mandatory with distances from 3 to 4m.

AC: Asphalt concrete; SC: soil-cement $CS_{7\text{days}}$ > 2.5MPa; GC: gravel-cement $CS_{7\text{days}}$ > 4.5MPa.

Thicknesses in cm.

In the past, problems associated with reflection cracks resulted in very low usage of semi-rigid pavements with granular cement bound bases with high initial strength, especially for heavily trafficked roads in severe continental climates. In recent years, the introduction of pre-cracking techniques has revived interest in these pavements.

### A10.2 Materials

Two cement-treated materials (CTM) are used in Spain: soil-cement and gravel-cement. Table A10.2 shows the main strength specification of these materials. The design of CTM is based on strength and workability requirements. Soil-cement is usually made from soils with a Plasticity
Index (PI) limited to 15 and with less than 35 % fines, or granular materials. Typically, for soil-cements the amount of cement used varies from 3 – 7 % by mass, depending on the characteristics of the basic material. The gravel-cement is made from crushed granular materials with a continuous gradation. The amount of cement typically varies from 3.5 – 5 % by mass of the aggregates.

Table A10.2 Strength specifications for cement-treated materials.

<table>
<thead>
<tr>
<th>Material</th>
<th>Spanish Standard</th>
<th>Compressive Strength (MPa)</th>
<th>Minimum</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil-cement</td>
<td>Art. 513 PG-3</td>
<td>(R_{CT7d} \geq 2.5)</td>
<td>(R_{CT7d} \leq 4.5)</td>
<td></td>
</tr>
<tr>
<td>Gravel-cement</td>
<td>Art. 513 PG-3</td>
<td>(R_{CT7d} \geq 4.5)</td>
<td>(R_{CT7d} \leq 7.0)</td>
<td></td>
</tr>
<tr>
<td>RCC</td>
<td>6.1 y 2 –IC (1989)</td>
<td></td>
<td>(R_{RT28d} \geq 3.3)</td>
<td></td>
</tr>
</tbody>
</table>

**Notas:**
- Minimum of 98% the Modified Proctor density.
- \(R_{CT7d}\) – Compressive strength at 7 days.
- \(R_{RT28d}\) – Indirect tension strength at 28 days.

A10.3 Techniques to reduce reflection cracking

Several measures have been taken to reduce the risk of reflection cracking in the asphalt layer. One of the main methods is pre-cracking the hydraulically bound layer during construction. Pre-cracking consists of creating transverse cracks, typically every 2 to 4 m, in the fresh layer just before compaction. The closer transverse crack spacing reduces the thermal crack movements in the completed pavement and delays or prevents the onset of reflection cracking in the overlying asphalt layer.

Initially, techniques developed in France were solely used for pre-cracking (mainly CRAFT, see figure A10.1), but now other systems are being developed by Spanish contractors to make the treatment more competitive.
Recommendations on the best use of pre-cracking techniques have been included in a recently published manual on semi-rigid pavements edited by CEDEX and the Spanish cement association IECA (CEDEX-IECA, 2003). Table A10.3 gives these recommendations for different traffic levels, material strength, and climate.

Table A10.3  Recommendations for pre-cracking cement treated material layers.

<table>
<thead>
<tr>
<th>TRAFFIC</th>
<th>CLIMATE</th>
<th>CS7 &lt; 4 MPa</th>
<th>CS7 ≥ 4 MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>AADT₁₀₀ ≤ 200</td>
<td>CONTINENTAL</td>
<td>Necessary[^1]</td>
<td>Necessary</td>
</tr>
<tr>
<td></td>
<td>COASTAL</td>
<td>Advisable[^2]</td>
<td></td>
</tr>
<tr>
<td>AADT₁₀₀ &lt; 200</td>
<td>CONTINENTAL</td>
<td>Not necessary</td>
<td>Advisable[^2]</td>
</tr>
<tr>
<td></td>
<td>COASTAL</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

CS7: Compression strength at 7 days.
[^1]: Not necessary if asphalt concrete thickness ≥ 18 cm.
[^2]: Not necessary if the Annual Average Daily Traffic of heavy lorries on the design lane (AADT₁₀₀) is less than 25 in the first year of service.

In addition to pre-cracking, other techniques are being employed in association with pre-cracking to reduce the reflection cracking. For example, the use of low hydration heat cements, specifying a minimum thickness of asphalt concrete to reduce the temperature variation in the cement treated material or by limiting the strength of the CTM.

A10.3 Construction

The construction quality of semi-rigid pavements is extremely important. Therefore, if a few rules of good practice are followed (apart from pre-cracking), an important improvement in pavement performance and enhanced life of the infrastructure can be attained. The following is important:

- Good compaction of the CTM layer is essential; especially in the region close to the bottom of the CTM layer. After an exhaustive study of many kilometres of in-service roads in Spain, it has been observed that many of the cores had poor compaction in this region where the largest tensile stresses are induced by traffic. This will result in a strength reduction which will have a
serious impact on pavement life. The poor compaction was largely the result of poor workability of the CTM during the construction stage.

- Good adhesion between the asphalt layer and the CTM layer is important. A loss of bond will result in a large decrease in pavement life.

### A10.4 Assessment and Upgrading

In Spain, the methods of assessment, upgrading and maintenance of semi-rigid pavements are described in the standard for pavement rehabilitation (Ministerio de Fomento, 2003b). Strengthening the pavement structure needs to be considered when there is structural failure of the pavement, a strong increase in traffic loading is anticipated or when the costs of the ordinary maintenance works become excessive.

Condition assessment of semi-rigid pavements usually includes visual inspection, deflection measurements (typically FWD), coring and laboratory tests (strength and stiffness). The standard (Ministerio de Fomento, 2003b) includes a best practice guide and recommendations for interpreting deflection measurements. This guide includes correlations between several devices for measuring deflection and deflection corrections for temperature and subgrade moisture condition.

The strengthening is based on overlays. The thickness of overlay can be determined from the deflection level, which can also be used to assess the necessity for pavement reconstruction as a result of subgrade failure. Subgrade failure is frequently related to drainage failures and any drainage deficiencies need to be remedied prior to reconstruction.

### A10.5 Maintenance

The maintenance of semi-rigid pavements is similar to that of the maintenance of flexible pavements but with greater emphasis on crack sealing. Transverse reflection cracking is usually treated by overbanding to prevent the ingress of water weakening the foundation and compromising the bearing capacity of the pavement.

### A10.6 Economics

There is no specific economical analysis for long-life semi-rigid pavements.

### A10.7 Research Needs

The following research needs are identified:

- Crack-mechanisms of semi-rigid pavements.
- Economic tools to evaluate the cost effectiveness of long-life pavements.
- Long-term assessment of pre-cracking techniques to avoid reflection cracking.

### A10.8 References


A11. Switzerland

A11.1 Background
The Swiss National road network total length is about 1700 km. The part of semi-rigid pavements does not exceed 88 km (5% of the National network length, see figure 1). It is mainly due to the problem of reflecting cracking (for this reason the standards suggest a “sandwich” structure type: the unbound layer is situated between the asphalt layer and the cement stabilized base layer). The high quality of the subgrade and the level of frost are reasons that this type of pavement is not much used in Switzerland.

Figure 1: Part of semi-rigid pavement on the National road network

The Swiss semi-rigid pavements consist in an asphalt overlay on a concrete “base” layer.

A11.2 Design and Construction

Design
The Swiss design standard is based on the results of the AASHTO (American Association of State Highway and Transportation Officials) Guide for Design of Pavement structures (Washington D.C. 1986) and the design period is a minimum of 20 years for each type of pavement. The Swiss design standard describes a semi-empirical procedure which takes into account the traffic load (traffic classes T1 to T6) and the quality of the subgrade (bearing classes S1 to S4). The resistance of the selected design to frost must be checked.
Table A11.1: Swiss design standard for semi-rigid pavements, traffic classes T4 to T6 and subgrade class S3 (high bearing capacity)

<table>
<thead>
<tr>
<th>Traffic class</th>
<th>T4 (heavy)</th>
<th>T5 (very heavy)</th>
<th>T6 (extremely heavy)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ESAL 8.16t / day</td>
<td>300 to 1000</td>
<td>1000 to 3000</td>
<td>3000 to 1000</td>
</tr>
<tr>
<td>Structure type 4</td>
<td>.</td>
<td>.</td>
<td>.</td>
</tr>
<tr>
<td>Structure type 5</td>
<td>12 cm</td>
<td>15 cm</td>
<td>18 cm</td>
</tr>
<tr>
<td></td>
<td>17 cm</td>
<td>16 cm</td>
<td>18 cm</td>
</tr>
<tr>
<td></td>
<td>29 cm</td>
<td>15 cm</td>
<td>16 cm</td>
</tr>
<tr>
<td></td>
<td>.</td>
<td>45 cm</td>
<td>49 cm</td>
</tr>
<tr>
<td></td>
<td>.</td>
<td>.</td>
<td></td>
</tr>
</tbody>
</table>

The total thickness of the asphalt layers does not depend on the quality of the subgrade, only the thicknesses of the cement stabilised base and the unbound sub-base layers are varied for the subgrade.

**Case for long-life pavement**

There is no specific design for long-life pavements in the Swiss design standard. However, there may be a way to design a pavement structure for a long design period using a Structural Number (SN) approach. The basis of the Swiss approach (respectively the AASHTO approach) is the bearing capacity of the structure; so, if the only type of distress which is expected to occur during its life cycle is a loss of bearing capacity it is theoretically possible to design a long-life pavement.

The principle of the design is to obtain the SN which corresponds to the measured traffic and subgrade classes. Each centimetre of each material contributes to the expected bearing capacity (respectively the SN). Each material corresponds to a specific weight or bearing capacity value which is multiplied by its layer thickness to obtain the contribution of each layer.

\[ SN = \sum_{i=1}^{n} a_i \cdot D_i \]

Where:
- \( a_i \) = bearing capacity value for a specific material \( i \)
- \( D_i \) = thickness of the layer which corresponds to the material \( i \) [cm]
Even if theoretically the only design parameter is bearing capacity and all the materials and the construction of the structure are correctly made, two restrictions occur:

1. For a design life of 40 years and traffic class $\leq$ T6: The limit is the lack of knowledge of long-term performance of the bearing capacity.
2. For a design life of 40 years and traffic class $>$ T6: The limit is the lack of knowledge in term of relationship between a traffic load more than 10000 ESAL 8.16t per day and the need of bearing capacity (respectively SN).

**Construction**

• Cement stabilised base

The minimum proportion of cement is 60 kg/m$^3$ and the minimum compressive strength must be between 2 and 4 N/mm$^2$ depending on the depth of the layer. The minimum thickness of one stabilised layer is 15 cm or 3 times the nominal diameter of the largest aggregate size.

• Joints

The minimum offset distance between two longitudinal joints of two superimposed stabilised layers must be 0.5 m and the longitudinal joint of the stabilised layer which is just below the asphalt layer must not be under the wheel tracks. The transversal joints show an incline of 60°.

To minimise and retard reflective cracking, pre-cracking of the stabilised layer every 5-15 m immediately after the construction is carried out or a SAMI is installed between the stabilised and the asphalt layer.

**A11.3 Assessment and Upgrading**

**Assessment**

The assessment of the bearing capacity is carried out using the Benkelman Beam or the Lacroix Deflectograph. The FWD method is often used, but it does not currently form part of the Swiss standard for assessment.

**Upgrading**

Measurement of the deflections permit the evaluation of the need for upgrading (in terms of centimetres of unbound sub-base with rounded aggregates). Two methods exist:

• Upgrading by overlaying
• Upgrading by partial reconstruction

Both cases use the approach of the Structural Number SN which was introduced in chapter A11.2. Table A11.2 shows some values for the parameter $a_i$.

**Table A11.2: Upgrading - comparison of the value of the parameter $a_i$ in function of the material.**

<table>
<thead>
<tr>
<th>Layer</th>
<th>$a_i$ for a new layer</th>
<th>$a_i$ for an old layer in function of its level of distress</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Soft distresses</td>
</tr>
<tr>
<td>Asphalt</td>
<td>4.0</td>
<td>3.4</td>
</tr>
<tr>
<td>Cement stabilised</td>
<td>2.4</td>
<td>2.0</td>
</tr>
<tr>
<td>Unbound sub-base</td>
<td>1.0</td>
<td>1.0</td>
</tr>
</tbody>
</table>
A11.4 Maintenance

In Switzerland no specific maintenance method exists for semi-rigid pavements. The proportion of semi-rigid pavements in the national roads network is very small (almost 0%) due to the problems of reflecting cracking. Two other reasons (the high quality of the subgrade and the level of frost) contribute to the low level of interest in the use of such pavement types in Switzerland.

A11.5 Economics

There is no specific economical analysis carried out for long-life pavements in Switzerland.

A11.6 Research Needs

- Comprehension of crack-mechanisms specifically linked to semi-rigid pavements need to be further investigated in order to better prevent their occurrence.
- Specific performance indicators for semi-rigid pavements could be developed (reflective cracking).
A12. United Kingdom

A12.1 Design and construction prior to the 2006 revision

Semi-rigid pavements in the UK are generally termed ‘Flexible-Composite’ pavements. Prior to 2006 designs for semi-rigid pavement were provided for traffic levels of up to 200 msa80; above 20 msa80 the design thicknesses remain constant, i.e. the pavement can be considered an indeterminate or long-life pavement. Pavements typically were typically constructed with a base of cement bound material (CBM) that varied from 150 to 250 mm depending on the strength of material used and the properties of the foundation. The asphalt surfacing typically varied from 100 to 200 mm (200 mm was required for designs for cumulative traffic of over 20 msa80).

Below 20 msa80 the design curves were determined empirically and the pavement was expected to deteriorate gradually under traffic loading. Above 20 msa80, the pavement was designed to have a long but indeterminate life. It was designed to resist longitudinal cracking in the wheel-path under traffic loading. This was achieved by ensuring that the combined traffic induced stress and the restrained thermal warping stress was less than the flexural strength of the cement bound base (a safety factor was applied to this ratio).

Standard designs for semi-rigid pavements prior to 2006 were given in the then existing version of HD 26 (DMRB 7.2.3), the design curves are reproduced in Figure A12.1. Six types of CBM base were included in the standard designs. The base types were defined according to three levels of compressive strength (measured at 7 days) and two aggregate classes that had different thermal properties. Materials labelled ‘R’ contained crushed rock and had a low thermal expansion coefficient, while ‘G’ denoted other aggregates such as gravel.

The thickness of the base layer was dependent on the design traffic, base material type and strength of the sub-base layer. Only cement bound material containing more coarsely graded aggregates (CBM2 and CBM2A) could be used in the sub-base layer for the heaviest traffic conditions (greater than 80 msa80).

The thickness of the asphalt surfacing layer was dependent on the design traffic only, and was principally designed to minimise the risk of reflection cracking. For traffic levels greater than 20 msa80, a 10 mm reduction in the thickness of asphalt, i.e. from 200 mm to 190 mm, was permitted because pre-cracking of the base at 3 m intervals was employed to reduce the risk of reflection cracking; pre-cracking was required for all cement bound bases and sub-bases with a strength greater than 10 MPa at 7 days.

Revised UK semi-rigid designs, after the 2006 revision

The UK design method for the semi-rigid pavements was revised in 2006. These pavements consist of a lower base of hydraulically bound mixtures (HBM) designed to withstand traffic induced stresses and an asphalt upper base and surfacing which both insulates the HBM and contributes to load spreading. (The design method of semi-rigid pavements prior to 2006 only considered a base constructed from CBM, whereas the revised method is applicable to pavements with bases formed from other hydraulically bound materials provided that there is empirical justification for the calibration of the design criterion.)
Figure A12.1. UK design curves for semi-rigid pavements, reproduced from HD 26 (DMRB 7.2.3 prior to the revision in 2006)
The new method is analytically based for all traffic levels, and for simplicity, it considers only the traffic induced stress or strain at the underside of the base layer. It uses a multi-layer elastic response model to calculate the critical stresses and 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. This is multiplied by two factors $K_{Hyd}$ and $K_{safety}$ to obtain the critical stress or strain which is then compared with a permissible value to achieve the required design life. The two multiplying factors are described below.

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

$K_{safety}$ The factor can be used to accommodate any inherent risk in pavement design. The value may be adjusted for very heavily trafficked roads, roads constructed in sensitive areas, to give added conservatism to design using new materials or new construction practices. The default value is 1.0; a lower value produces a more conservative design.

The current pavement design for semi-rigid pavements is provided for traffic levels of up to 400 ms$^{-1}$a$^{80}$. The transition to the long but indeterminate life zone is at 80 ms$^{-1}$a$^{80}$ to fall in line with the transition to long-life design for fully flexible pavements. Material properties for design are taken as the 360 day value rather than the 28 days value previously used. The adoption of 360 day design values brings the characteristics of the HBM more in line with the asphaltic material, and also to allow both slow curing of the HBM and the asphalt to be accommodated more easily. The following curing periods have been assumed to achieve the relative compressive strength of CBM shown.

<table>
<thead>
<tr>
<th>Curing Period</th>
<th>Relative compressive strength of CBM*</th>
</tr>
</thead>
<tbody>
<tr>
<td>7 day</td>
<td>0.67</td>
</tr>
<tr>
<td>28 day</td>
<td>0.80</td>
</tr>
<tr>
<td>360 day</td>
<td>1.00</td>
</tr>
</tbody>
</table>

* Ratio of the strength at the day shown to the 360 day strength

The current design uses the concept of foundation classes and the retention of a subgrade strain criterion to predict long lives for pavements incorporating stronger HBM sub-bases. This will allow the thickness of the HBM base layer to be reduced if a superior type of foundation is used. A minimum allowable HBM base thickness is 150 mm for flexible construction with a HBM base.

The foundation stiffness classes are defined in terms of the equivalent half-space stiffness of the composite foundation. The foundation classes are:

- Class 1 $\geq$ 50 MPa
- Class 2 $\geq$ 100 MPa
- Class 3 $\geq$ 200 MPa
- Class 4 $\geq$ 400 MPa

For designs in excess of 80 ms$^{-1}$a$^{80}$ a total of 180 mm thickness of asphalt overlay to the HBM base is required to ensure adequate delay to the onset of reflection cracking, provided transverse cracks have been induced in the HBM base at 3 m intervals. Below 80 ms$^{-1}$a$^{80}$ a variable thickness of asphalt is specified. Without pre-cracking, an additional 20 mm of asphalt is required over the full design range.
Standard designs for new semi-rigid pavements are provided in HD 26 (DMRB 7.2.3) and are reproduced in Figure A12.2. The left hand portion of the nomograph gives the thickness of the HBM for a given strength, and the thickness of the overlaying asphalt is shown in the central portion. Improved performance is expected for those mixtures made with a crushed rock coarse aggregate that has a coefficient of thermal expansion less than 10x10^{-6} per °C (typically limestone).

Figure A12.2. Design curves for semi-rigid pavements, reproduced from HD 26 (DMRB 7.2.3)
A12.2 Assessment and Upgrading

Assessment

The UK does not have an explicit method for the assessment of long-life semi-rigid pavements. The method for the assessment of semi-rigid pavements (called flexible-composite pavements in the UK) is described in the HD 30 (DMRB 7.3.3). The assessment of pavements with a HBM base requires special consideration because it is highly dependent on the findings of the visual survey and coring. The method involves the classification of structural condition using a suite of tests, and based on the results from the tests flexible pavements with HBM bases are divided into four condition classes (Class A, B, C and D). The principle determining factor are the type, extent and severity of cracks in the HBM base, but the determination of the condition classes is based on all the criteria given in Table 6.2 of HD 30 (DMRB 7.3.3). The Falling Weight Deflectometer (FWD) stiffness values apply to the combined asphalt and HBM base. Other defects such as rutting, fretting and surface cracking should be assessed in the same way as for flexible pavement with an asphalt base. Class A defines a pavement free from structural deterioration; other classes permit some type of structural deterioration. Details of visual and structural conditions for the assessment of treatment of flexible pavements with a HBM base are given in HD 30 (DMRB 7.3.3). A pavement that satisfies a Class A condition will have the following characteristics:

- **Visual condition:** widely spaced cracks that are of a fine width, no evidence of foundation degradation
- **HBM strength:** cube strength of hydraulically bound base is greater than 10 N/mm².
- **Pavement structure:** combined stiffness greater than 10 GPa.

The primary transverse shrinkage cracks, which are formed in a layer of medium to high strength HBM at the time of construction, often cause deflection cracks in the road surface. The development of such cracks in the surface is influenced by the age and thickness of the overlaying asphalt layers and partly by other factors such as strength of mix, subgrade strength, and weather conditions during and immediately after construction and traffic loading. Forecasts of long residual lives, which are derived for HBM bases in combination with low deflections and moderate traffic loading, need to be treated with caution as they depend on the pavement remaining substantially uncracked.

Using the FWD, an assessment of individual layer stiffness values can be made in accordance with HD 29 (DMRB 7.3.2). Good integrity is sought in each layer, therefore the asphalt layer stiffness is expected to be greater than 7 GPa, the HBM base layer greater than 15 GPa and the foundation layer greater than 100 MPa (HD 30 DMRB 7.3.3)).

Upgrading

The suitability of upgrading an existing structure for consideration as a long-life semi-rigid pavement can be applicable provided that the HBM base is assessed to be in good condition with no significant deterioration. Tests will be required to determine the current properties of the base, unless structural records exist to prove compliance with certain classes of material in HD 29 (DMRB 7.2.3).

For a mature pavement, it is unlikely that the structural layers of the pavement will have undergone formal pre-cracking treatment at the time of construction. Therefore, the thickness of asphalt surfacing for a long-life semi-rigid structure must be at least 200 mm. No attempt should be made at post-cracking the structure, e.g. removing the asphalt layer and inducing cracks prior to treatment. The existing naturally forming cracks may be difficult to locate and are unlikely to be uniformly spaced; localised weak areas could be produced if the further cracks are introduced.

The upgraded pavement structure should be free from deterioration. The UK upgrading methodology does not take into account any existing deterioration within the asphalt layer.
The new post 2006 revised design method for both fully flexible and for semi-rigid pavements adopts superior foundation classes within the UK design standards. This allows for the additional support that could be provided by the adopted sub-base layer to result in savings of asphalt thickness in the upgraded pavement structure. If the pavement is structurally deteriorated, the pavement is only considered as a part of the foundation and a replacement pavement thickness is constructed over it. If such a pavement is required to be upgraded, the adopted sub-base is treated as equivalent to unbound granular layer.

The design document HD 30 (DMRB 7.3.3) contains general advice for the assessment of maintenance requirements. Flexible pavements with HBM bases in Classes B and C generally require an overlay but local reconstruction may be considered if the HBM base is severely cracked. Flexible pavements with HBM bases in Class D will need to be reconstructed, or a thick overlay may be an alternative, because the HBM will have deteriorated to small slabs with poor load transfers and moreover deterioration is likely to continue until the HBM is reduced to little more than a granular sub-base.

A12.3 Maintenance

The design document HD 30 (DMRB 7.3.3) covers the maintenance requirements for both flexible-composite and fully-flexible pavements in one section entitled 'flexible pavements'. This indicates that in general, the maintenance requirements for semi-rigid pavements are similar to that of fully-flexible pavements.

The principal factor in determining the maintenance treatments for semi-rigid pavements is the type, extent and severity of cracking in the HBM base layer. For long-life semi-rigid pavements, only naturally forming cracks in the base are permitted; other cracks indicate some form of deterioration in the structure. It is accepted that periodic transverse cracking can occur and is not necessarily a result of structural deterioration. Transverse reflection cracking in isolation, at intervals of 4 m or more, generally indicates a pavement with a strong HBM base which has cracked due to thermally induced strains.

The document HD 29 (DMRB 7.3.2) advises that once reflection cracks have formed the road may deteriorate more quickly under the action of traffic. Water may enter the pavement structure and weaken the road foundation around the crack; the loss of support will depend on moisture susceptibility of sub-base and subgrade. The action of traffic may cause vertical movements at the crack as the wheel load is transferred from one side of the crack to the other. Interlock in the HBM base may be eroded and eventually secondary cracks can form around the transverse crack as the pavement deteriorates.

Evidence from coring has demonstrated that reflection cracks can initiate at the surface of the asphalt and propagate downwards to the crack in the concrete base and this mechanism is gaining acceptance in the UK.

Coring through the reflection cracks is recommended to determine whether the cracks are only in the surfacing (surface initiated) or through the full thickness of existing bituminous material. If cracking is confined to the surface, planing off the existing surface and then re-surfacing may be a sufficient treatment. If badly cracked all of the existing bituminous material layers may need to be removed.

A12.4 Economics

There are no specific issues for the economic evaluation of semi-rigid pavements.
A12.5 Research Needs

What is the influence of the HBM base and the bituminous layer on the nature of reflection cracking?

The strength and thickness of the HBM layer has a significant influence on the progression of deterioration. The current design method for semi-rigid pavements acknowledges that provided a suitable HBM base layer is constructed, 200mm of bituminous material is sufficient to prevent reflection cracking that originates from the base of the asphalt layer. Other forms of roadbase where the distance between cracks is shorter can permit thinner asphalt surfacing without increasing the risk of reflection cracking. The interaction between crack frequency, crack opening, the thermal and mechanical properties of the asphalt surfacing and the concrete base, age hardening characteristics of the surface course and the risk of reflection cracking is yet to be determined; although it has been empirically determined from certain pavement types.

What is the mechanism of reflection cracking in thick asphalt layers?

The UK generally accepts that reflection cracking in thick asphalt layers laid on HBM bases originates from the surface and propagates downwards. The mechanisms of its formation are as yet unclear, as is knowledge on the rate of propagation of these cracks.

Are adjustments necessary to optimise the maintenance strategy for reflection cracks for different thicknesses of asphalt layer?

It is thought that thick asphalt layers tend to experience surface cracking while thinner layers can crack right through the layer. Therefore, when reflection cracks appear at the surface of thick asphalt layer they may not need such immediate treatment as semi-rigid pavements with thin asphalt layers. Alternatively, inlaying the pavement with a new surface course, at or about the time when cracks are just initiating, may be cost effective in delaying the onset of serious reflection cracking for a considerable time.

How to detect and accommodate superior sub-bases?

The influence of the foundation on the performance of the semi-rigid structure is not known. Stronger sub-bases are requested in the UK to reduce the risk of structural deterioration on heavily trafficked pavements. The new revised semi-rigid pavement design includes use of foundation classes. The upgrading and assessment techniques do not take into account the nature of the foundation. With these changes to the UK semi-rigid design procedure, a suitable method of accommodating the positive effect of superior foundations is becoming more significant.

A12.6 References

Design Manual for Roads and Bridges (DMRB). The Stationery Office, Norwich, UK.
   HD 26 – Pavement Design (DMRB 7.2.3).
   HD 29 – Structural Assessment Methods (DMRB 7.3.2)
   HD 30 – Maintenance Assessment (DMRB 7.3.3).


ROAD PAVEMENT

**Surface Course**
- the interface between tyre and surface course.

**Surface course**
- the layer at the top of the pavement which is in contact with the tyre

**Binder course**
- a layer between the surface course and the base

**Base or Roadbase**
- the main structural layer of the pavement.

**Sub-base**
- a layer designed to protect the formation from traffic loading.

**Subgrade**
- the natural soil beneath the pavement layers.

**Formation**
- the level at the bottom of the pavement layers.

**Foundation**
- comprises all layers below the base layer (Sub-base, capping and subgrade)

**Pavement**
- comprises all layers above the formation including the sub-base layer.

**Fully-flexible pavement**
- A pavement with a bituminous surfacing and with a roadbase with or without a hydrocarbon binder.

**Semi-rigid pavement**
- A pavement with a bituminous surfacing and one or more courses treated with cementitious binders and which make a significant contribution (or courses treated with hydrocarbon binders and which by their stiffness or thickness cannot be considered as structurally flexible).

**Rigid pavement**
- A pavement substantially constructed of cement concrete.

**Primary road network**
- Generally, the primary road network consists of the major long distance through routes linking principal ports and airports, cities and large towns which serve major geographical and industrial regions.

REFERENCE AXLE

The reference standard axle used in the computed design traffic is denoted by a numerical subscript as follows:

- $msa_{80}$ = million standard 80kN axles
- $msa_{100}$ = million standard 100kN axles
- $msa_{130}$ = million standard 130kN axles
### APPENDIX C: ELLPAG Membership

<table>
<thead>
<tr>
<th>Core Member Organisations</th>
<th>Country</th>
</tr>
</thead>
<tbody>
<tr>
<td>BRRC</td>
<td>Belgium</td>
</tr>
<tr>
<td>CRR - Centre de Recherches Routières</td>
<td>Belgium</td>
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<tr>
<td>OCW - Opzoekingscentrum voor de Wegenbouw</td>
<td>Belgium</td>
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<tr>
<td>CDV</td>
<td>Czech Republic</td>
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<td>CDV - Centrum Dopravního Vyzkumu</td>
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<td>DRI</td>
<td>Denmark</td>
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<td>DRI – Danish Road Institute (of the Danish Road Directorate)</td>
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<td>DVS</td>
<td>The Netherlands</td>
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<td>DVS - Dienst Verkeer en Scheepvaart (formerly DWW - Dienst Weg-en Waterbouwkunde)</td>
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<td>IBDiM</td>
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<td>IBDiM – Instytut Badawczy Dróg i Mostów</td>
<td>Poland</td>
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<td>CEDEX – Centro de Estudios y Experimentación de Obras Públicas</td>
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<tr>
<td>TRL</td>
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</tr>
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<td>TRL – Transport Research Laboratory</td>
<td>United Kingdom</td>
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</table>

Contact with the above organizations can be made through their respective FEHRL research coordinators whose current names and addresses are available via the FEHRL website (www.fehrl.org).
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<thead>
<tr>
<th>Associate Member Organisations</th>
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<tbody>
<tr>
<td>BASI – Bundesanstalt für Straßenwesen</td>
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<tr>
<td>EAPA – European Asphalt Paving Association</td>
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<td>FEHRL – Forum of European National Highway Research Laboratories</td>
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<td>IGH – Civil Engineering Institute of Croatia</td>
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<tr>
<td>Asfalttiilto</td>
<td>Finland</td>
</tr>
<tr>
<td>Kolo Veidekke</td>
<td>Norway</td>
</tr>
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</table>
**Affiliate Member Organisations**

<table>
<thead>
<tr>
<th>Organisation</th>
<th>Country</th>
</tr>
</thead>
<tbody>
<tr>
<td>AAPA – Australian Asphalt Paving Association</td>
<td>Australia</td>
</tr>
<tr>
<td>ARRB – Australian Road Research Board</td>
<td>Australia</td>
</tr>
<tr>
<td>JTRI – Jiangsu Transportation Research Institute</td>
<td>China</td>
</tr>
<tr>
<td>NAPA - National Asphalt Paving Association</td>
<td>United States of America</td>
</tr>
<tr>
<td>NCAT - National Centre for Asphalt Technology</td>
<td>United States of America</td>
</tr>
<tr>
<td>TUD – Technical University – Delft</td>
<td>The Netherlands</td>
</tr>
</tbody>
</table>