Improved design of overlay treatments to concrete pavements. Final report on the monitoring of trials and schemes

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Executive Summary

The current Highways Agency advice when overlaying jointed concrete pavements with asphalt is to apply a minimum of 180 mm in order to minimise the occurrence of reflection cracking, HD 30/99 (Design Manual for Roads and Bridges, DMRB 7.3.3). The objective of this research was to develop improved design guidance for asphalt overlays to concrete pavements by providing techniques that will reduce the thickness of overlay whilst still minimising the occurrence of reflective cracking.

The performance of an overlay to a jointed concrete pavement can be affected by the occurrence of reflection cracks above the joints. Reflection cracks are transverse and/or longitudinal cracks that occur in the overlay above the joints or cracks in the underlying concrete layer. Reflection cracks may also occur in pavements that have cement bound granular material (CBGM) layer under the asphalt overlay due to irregular spaced thermal shrinkage cracks that form during the curing of the CBGM. Extensive coring of trial sites has shown this reflection cracking in the overlay to occur from the top down.

This report summarises the performance to-date of a number of jointed concrete sites, with slab lengths varying from 5 m to 24 m, and various sites containing a lean concrete base. The full scale trial sites are located on the A1 Winthorpe, A14 Bury St Edmunds, A14 Quy, A259 Pevensey, A30 Launceston, M1 Barnet, M2 Kent, M5 Taunton. Data from a number of other schemes where various overlay treatments have been applied are also reported. The treatments applied included: variation in asphalt thickness; use of polymer modified binders; crack and seat techniques; sawcut and seal; interlayers; and joint treatments.

Structural assessment of the trial sites using the Falling Weight Deflectometer (FWD) has shown a soundness and consistency along the trials over the monitoring period. FWD load transfer measurements have shown that poor load transfer may occur if there is full depth cracking in the asphalt layer. Undertaking full depth concrete repairs prior to overlay can lead to the early development of transverse reflective cracking above the new joints because they are more likely to respond to thermal movements. However, the presence of voids under slabs needs to be remedied prior to overlay.

Some of the findings for the treatments applied are:

Crack and seat and Saw-cut, crack and seat

- The crack and seat technique has been used in the UK since 1992 and the saw-cut, crack and seat method since 1999.
- The crack and seat technique and the saw-cut, crack and seat technique are effective methods of reducing the occurrence of reflection cracking compared with control treatments of the same thickness and joint spacing.
- Overlays of 150 mm or above seem to be performing the best with minimal cracking present after 10 years service for crack and seat. The relatively recent use of saw-cut, crack and seat has shown no cracking present after 4 years service.

Geogrid/geotextile

- The performance of geogrids and geotextiles are very variable with some sites cracking before the control section with the same overlay thickness.
- The process of laying geogrids and geotextiles is labour intensive and requires good weather conditions and good control of the installation process is essential.
- The use of geogrids and geotextiles can give rise to problems when the surfacing is to be replaced because of the potential to cause problems when planing out the existing asphalt.

Modified asphalts

- Generally, the use of ethylene vinyl acetate (EVA) and styrene-butadiene-styrene block copolymer (SBS) modified binders did not prevent reflective cracking but did delay it by about six months. Compared to the equivalent control section, modified binder surface courses have been shown to exhibit good rut-resistant properties.
- The use of an EVA modified binder on the M2 Kent proved unsuccessful at preventing reflection cracking with cracking first being observed within four and a half years. The SBS modified binder used in the surface and binder courses on the eastbound carriageway of the A14 Bury St. Edmunds proved an effective treatment after ten years service. However, wide transverse cracking was observed on the westbound carriageway where the same technique was used for the same period.

Thin surfacing

- Thin surfacings were monitored on the A1 Eaton Socon where the treatment was applied directly to the concrete surface. The results showed that all sections exhibited signs of reflection cracking within 12 months. Therefore, thin surfacings are not considered a suitable overlay directly onto concrete due to cracking occurring within two years and the development of an on-going maintenance problem. For thicker overlays, there has not been a noticeable difference in performance to-date for different surface course materials for the same overall overlay thickness.

Saw-cut and seal

- The saw-cut and seal method is an effective treatment in reducing the occurrence of reflection cracking when compared with control sections with the same thickness of overlay and the same joint spacing.
- Good quality control during the installation of joint sealant is essential in the performance of the saw-cut and seal treatment.
- Performance of the saw-cut and seal treatment on various proprietary thin surfacings on the A1 Eaton Socon, constructed in 1994, have indicated that up to 30 mm

thickness of overlay is not sufficient for successful performance of the saw-cut and seal treatment. For practical reasons it is, therefore, recommended that the minimum overlay thickness for this treatment should be 70 mm comprising 40-50 mm binder course and 20-30 mm thin surface course.

- The survey results todate have shown that the saw-cut and seal treatment can be effective for slab lengths of up to 12 m.

The results from this work can be used to assist in preparing design guidance and specifications and also provide the highway engineer with information on the treatment options available. Guidance is given in this report for jointed unreinforced (URC) and jointed reinforced (JRC) concrete pavements and flexible composite pavements with a cement bound granular material (CBGM) base.

This guidance will enable the most cost-effective maintenance treatment to be selected, having regard to the resources available and the required life of the pavement. The recommended technique to be applied to an existing concrete carriageway will depend upon the existing construction and its current condition.

1 Introduction

The announcement of the Government's 10 year transport plan produced a commitment to using quiet road surfacing layers with the intention of ensuring that all concrete roads have a quiet surfacing by 2010 (Department for Transport, 2000). The main aims of this report were, by 2010, to:

- *complete a programme of local noise reduction measures to benefit those people living alongside noisy roads that are not subject to the latest noise mitigation standards; and*
- *have installed quieter surfaces on over 60 % (2,500 miles) of the network, including all concrete stretches, to benefit an estimated 3 million people living within around $^1/_3$ mile of trunk roads.*

To achieve this commitment, a variety of overlay techniques will be needed which can be applied directly onto structurally sound concrete as well as concrete pavements in need of rehabilitation. The purpose of this project was to investigate the available techniques currently being used and evaluate their performance; the results will enable engineers to select the most appropriate overlay options for future schemes, depending upon construction type and condition of the existing pavement, giving the best solution and benefit to the HA network.

2 Background

The performance of an overlay to a jointed concrete pavement is greatly affected by the appearance of reflection cracks at the joints. Reflection cracks are transverse and/or longitudinal cracks that occur in the overlay above the joints or cracks in the concrete layer due to the thermal expansion and contraction of the underlying concrete; see Figure 2.1. Reflection cracks may also occur in pavements that have a cement bound granular material (CBGM) base under an asphalt overlay due to the irregularly spaced thermal shrinkage cracks that form during the curing of the CBGM.

At the onset of reflection cracking, the crack widths are often barely visible to the naked eye and are not considered to significantly reduce serviceability. However, if the cracks are not promptly treated and left to widen and propagate to the full depth of the asphalt layer, the subsequent influx of water can weaken the foundation and fines can be pumped to the surface creating voids beneath the base. In the most severe cases, the structure of the pavement is compromised to such a degree that movement of the pavement structure occurs under normal traffic loading. In some cases, the surfacing can also ravel back from the crack with the reduced lateral support, impairing the ride quality. If allowed to progress to this state the maintenance implications are more serious.

Although Nunn (1989) identified three mechanisms that can result in the formation of reflection cracks, it is thought that only *surface initiated cracking* is typically found in asphalt overlays, at least in the UK. Surface

Figure 2.1 Reflection cracking present on minor road with 5 m concrete bays

initiated cracking is caused by the thermal expansion and contraction of the underlying concrete. The thick overlays provide a layer of thermal insulation, which may cause the contraction of the concrete slabs to be greater at the top of the slab than at the bottom which then results in a warping of the slab with the highest strains being found at the pavement surface. Age hardening also occurs more rapidly at the asphalt surface. Hence, cracking initiates at the surface and propagates down through the overlay until it reaches the joint or crack in the concrete, see Figure 2.2.

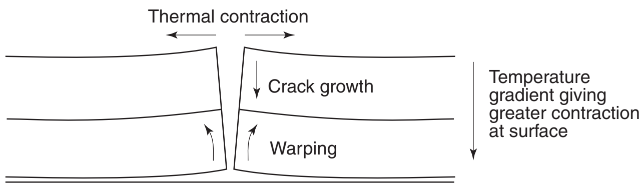


Figure 2.2 Mechanism of reflection cracking initiation

To minimise the cracking, the present Highways Agency advice for the overlay of jointed concrete motorways and trunk roads, HD 30/99 (Design Manual for Roads and Bridges, DMRB 7.3.3), is that approximately 180 mm of asphalt material is normally required following rehabilitation of any failed joints or slabs in the concrete.

The DMRB currently contains very little design advice for the selection of alternative overlay solutions. In order that engineers are able to select the most appropriate, cost-effective overlay solution for a concrete road, they need to be made aware of the range of techniques that are available, and any constraints that may reduce the applicability of each of them. This project has been designed to review the treatment options currently available and select techniques that show potential benefit for use on the HA network.

3 Review and descriptions of techniques

3.1 Categories

A literature review was conducted through the TRL Library into the various techniques used to reduce reflection cracking. These techniques can be divided into three categories:

- *Fractured slab techniques:* where the characteristics of the concrete construction are changed.
- *Interlayer techniques:* where materials are placed on the concrete surface or between layers of asphalt overlay.
- *Surfacing techniques:* where the surface course is treated or modified.

It should be noted that some of the techniques identified may not be suitable for all construction types considered in this report, these types being jointed reinforced concrete (JRC), unreinforced concrete (URC), and flexible composite pavement with a cement bound granular material (CBGM) base. Table 3.1 summaries the techniques researched.

Table 3.1 Summary of treatments

Treatment	*Technique type*	*Applicability*
Crack, seat and overlay (CSO)	Fractured slab	URC, CBGM
Saw-cut, crack, seat and overlay (SCCSO)	Fractured slab	JRC
Rubblisation	Fractured slab	URC, JRC, CBGM
Geogrids/geotextiles	Interlayer	URC, JRC, CBGM
Stress Absorbing Membrane Interlayer (SAMI)	Interlayer	URC, JRC, CBGM
Crack relief layer (CRL)	Interlayer	URC, JRC, CBGM
Modified asphalts	Asphalt overlay	URC, JRC, CBGM
Saw-cut and seal (SCS)	Asphalt overlay	URC, JRC
Slot-sealing	Asphalt overlay	URC, JRC
Programmed sealing	Asphalt overlay	URC, JRC

3.2 Fractured slab technique

3.2.1 General approach

These techniques require the concrete slabs to be broken into smaller lengths prior to an overlay being applied. By reducing the effective slab length, the strains resulting from thermal movements are reduced and distributed more regularly; therefore, reflection cracking is minimised. It is important to maintain good load transfer between fractured slab elements in order to maintain the load carrying capability of the pavement. This technique tackles the root cause of reflection cracking.

3.2.2 Crack, seat and overlay

The main principles behind the crack, seat and overlay technique is to reduce the effective length of the concrete prior to overlaying. If the effective lengths of the slabs are reduced, the horizontal strains resulting from the thermal movements should be evenly distributed throughout the pavement and, hence, the potential for reflection cracking to occur is reduced.

Two types of equipment have been used (Figure 3.1) on UK pavements:

- *Guillotine action:* uses a heavy, transversely mounted blade falling vertically under gravity to crack the pavement.
- *Whiphammer action*: uses a chisel type impact head powered by a spring loaded action.

There are a number of guillotine operators in the UK, and some contractors have also developed custom made guillotine devices that have been successfully trialled in the UK. An example of a custom-made guillotine is shown in Figure 3.2.

The cracking operation creates fine, close-spaced cracks generally between 0.5 m and 2.0 m spacings and changes the concrete pavement into the equivalent of a strong cement-bound base. It is extremely important that the cracks created within the concrete are fine so as to maintain good aggregate interlock needed for the load transfer. It is also important that the cracks produced are vertical (as shown in

Figure 3.1 Standard breaker equipment used for crack and seat

Figure 3.2 Custom made guillotine based on an adapted Ruston Bucyrus 38RB heavy-duty crane

Figure 3.3) because inclined cracks are likely inhibit thermal movements due to the aggregate interlock from the weight of the overlying pavement structure.

Figure 3.3 Core showing 'fine, full depth crack in concrete' after crack and seat

After cracking, the treated surface is then rolled with a minimum of six passes of a 20 tonne pneumatic tyred roller (PTR) prior to the application of the new overlay, as shown in Figure 3.4. The seating operation is carried out to minimise the occurrence of voids under the slabs prior to overlaying.

3.2.3 Saw-cut, crack, seat and overlay

Whilst crack, seat and overlay is effective for unreinforced concrete, it cannot adequately separate the concrete from the steel members present in reinforced concrete pavements. This lack of distinction results in the thermal contraction being concentrated at the existing transverse

Figure 3.4 20 tonne ballasted PTR used in the seating process

joints and, therefore, thermal movements are concentrated at discrete locations resulting in reflection cracking of the asphalt above the joints. It should also be noted that the length of the reinforced concrete bays are normally two to five times longer than standard jointed unreinforced concrete bays (that is, up to 25 m).

Saw-cut, crack, seat and overlay (SCCSO) was developed by TRL and the first UK trial took place in 1999 on the 25 m jointed reinforced concrete slabs on the A1 at Tuxford, Nottinghamshire.

The operation consists of sawing narrow cuts transversely across the slab to a depth of just below the reinforcement (determined by pre-scheme coring and ground penetrating radar (GPR) survey) in order to sever the longitudinal reinforcement steel (see Figure 3.5). The spacing used is intended to mirror that used in crack, seat and overlay.

Figure 3.5 Saw-cutting of the longitudinal steel reinforcement

Once cut, the remaining depth of the concrete is then cracked to retain satisfactory load transfer characteristics. It is important that the cracking operation takes place with minimum damage (spalling) to the saw-cut which could reduce load transfer between slabs. The use of a strike plate (see Figure 3.6) minimised spalling to the saw-cut compared to when struck directly. It was also found that, with the use of a strike plate, lower drop heights can be used which, in turn, reduces the vibration and possible damage to the lower layers.

Figure 3.6 Cracking of the saw-cut using a strike plate

After cracking, the treated surface is then subjected to a minimum of six passes of a 20 tonne PTR as performed in the CSO operation described in Section 3.2.2.

3.2.4 Rubblisation

Rubblisation is a technique, developed in the USA, which has been used since 1990. It is applied to both reinforced and unreinforced concrete nearing the end of its service life. It is also possible to rehabilitate an old flexible composite pavement by removing the existing asphalt overlay before pulverising the existing concrete pavement to effectively create a sub-base layer for the new pavement construction.

The pavement is broken into fragments, varying in size from about 25 mm at the surface to 380 mm at the bottom of the layer. The pavement is rubblised by the use of a Multi-Head Breaker (MHB), as shown in Figure 3.7.

The MHB consists of 12 hammers, each 300 mm wide and weighing between 544 kg and 680 kg. Typical output from the MHB is 3 m/min and up to 5,265 m^2 per shift for a 300 mm thick concrete layer. The rubble is then further pulverised by a grid roller (modified vibratory steel roller, as shown in Figure 3.8) and then seated using a pneumatic tyre roller, which finishes the surface. A finished surface is shown in Figure 3.9.

The overlay is then laid using a paver at a thickness ranging between 200 mm and 430 mm, depending on the design. The overlay thickness is controlled by:

- the traffic requirements;
- the quality of the material used in the original concrete pavement;
- the quality of the existing foundation;
- the thickness of the concrete pavement; and
- the size of the concrete fragments.

Figure 3.7 Rubblisation of the B1441 using the MHB

Figure 3.8 Grid roller (modified vibratory steel roller)

Figure 3.9 The finished rubblised surface, after compaction

Rubblising effectively eliminates the problem of reflection cracking. However, it also removes much of the strength of the old concrete and should only be considered as a viable option when there are major structural problems with the existing pavement. Caution is also needed when rubblising on weak foundations because the process may damage the subgrade and cause premature failure of the pavement. Figure 3.10 shows voids present in the underlying foundation.

Figure 3.10 Voids present under the rubblised concrete on the A110, Enfield

3.3 Interlayers

3.3.1 General approach

The introduction of a material layer between the concrete and the new overlay, called an interlayer, allows the overlay to move relative to the concrete. Examples of interlayer systems are geogrids, geotextiles and stress absorbing membrane interlayers (SAMIs) – a membrane layer that can deform longitudinally without breaking.

3.3.2 Geogrids/geotextiles

Geogrids and geotextiles have been used as interlayers between concrete and asphalt overlays to retard reflection cracking since the 1960s.

Geogrids are designed to enhance the tensile strength of the asphalt overlay by absorbing the horizontal tensile stresses above the joints in the concrete and distributing them over a wider area. They are said to result in reduced stress levels in the asphalt overlay at joint locations resulting from thermal effects.

3.3.3 Stress Absorbing Membrane Interlayer (SAMI)

The main aim of SAMIs is to provide a flexible layer that is able to deform horizontally without breaking. They generally consist of a thick layer of polymer modified binder that is sprayed onto the concrete surface and is sometimes used in combination with chippings.

3.3.4 Crack relief layer (CRL)

The use of crack relief layers have been pioneered within the USA for use on both jointed and continuously reinforced pavements, albeit mainly overlaid with a relatively thick layer of asphalt.

In the USA, 90 mm thickness of a coarse, open graded material is generally used for the CRL. Due to the large amount of interconnecting voids, the layer provides a medium to prevent differential movements of the underlying concrete by disconnecting the movement at joints from the overlaying surface course.

In the UK, TRL trials using porous friction course (PFC) as a CRL on military airfields have shown good performance in resisting reflection cracking, even with a CRL as thin as 20 mm (Langdale, 2006). PFC is the UK airfield equivalent to porous asphalt (PA). The current HA Specification for porous asphalt is found in Clause 938 of the Specification for Highway Works (MCHW 1). Due to the porous asphalt being used in a base rather than a surface course layer, aggregate with a high polished stone value (PSV) will not be needed, hence reducing material costs.

3.4 Surfacing treatments

3.4.1 General approach

Surfacing treatments are treatments that control reflection cracking within the surface course; they can be modified overlays with properties which retard the onset of reflection cracking. These techniques generally include polymer modified and fibre reinforced overlays, and the use of the saw-cut and seal technique to the finished surface.

3.4.2 Modified asphalts

To improve the performance of the bituminous binder, polymers are often added. These polymers often are added to help improve the viscosity of the binder in high and/or low temperatures. To date, several polymer systems are reported to improve resistance to reflective cracking when compared to conventional materials.

Polymers can be a natural compound (such as natural rubber), manufactured organic compounds (such as polystyrene), or even inorganic compounds (such as sulphur). The types of polymeric additives used can be divided into two categories: elastomers and plastomers. Elastomers result in a more resilient and flexible pavement, whereas plastomers give an increase in stability and stiffness modulus.

The polymeric additive is usually added to the binder using special plant prior to mixing. The asphalt material is then laid in the conventional manner using a paver.

Table 3.2 outlines some of the more commonly used polymer additives for asphalt.

Table 3.2 Polymer additives used

Generic name	*Acronym*	*Type*
Styrene-butadiene-rubber (latex)	SBR	Elastomer
Radial block styrene-butadiene-styrene	SBS	Elastomer
Vulcanised rubber	–	Elastomer
Ethylene vinyl acetate	EVA	Plastomer

3.4.3 Saw-cut and seal

The saw-cut and seal technique has been used in the UK since 1990, with the majority of the original trial sites being constructed between 1990 and 1994. The original trial sites were at A14 Bury St. Edmunds, M5 Taunton, A14 Quy (all unreinforced concrete) and M1 Barnet (jointed reinforced concrete).

The main principle behind saw-cut and seal is to accommodate the stress and strains associated with expansion and contraction of the underlying jointed concrete by introducing joints into the bituminous overlay directly above the joints in the concrete. These joints are then filled with an approved sealant, which creates a highly flexible reservoir and stops the formation of reflection cracks at the surface, whilst controlling their development from a crack initiation slot below the surface.

Currently, the Highways Agency guidance in the Manual of Contract Document for Highway Works, MCHW, (Volume 2 Notes for Guidance Clause 713) for jointed reinforced concrete pavements with bays exceeding 6 m in length may first be prepared by ‘saw-cut, crack and seat’. This guidance means that the effective length of some reinforced concrete needs to be reduced, which is often done by saw-cut, cracking and seating. The main principle behind this guidance is that the thermal movements are considered too great for the asphalt seal if the effective slabs were in excess of 6 m. However, the M1 Barnet trial and a trial on the A1 Winthorpe to Coddington, constructed in March 2004, have different effective slab lengths of up to 12 m which are performing satisfactorily.

The method requires that a slot is cut, cleaned, dried, bond breaker tape applied and an approved sealant installed in the asphalt overlay above each joint in the concrete, all carried out in one continuous operation. The technique also includes a fine saw-cut, below the sealant reservoir, which enables a crack to form between the bottom of the saw-cut and the joint in the concrete. It is essential that the location of the joints in the concrete are accurately identified and the centre of the slot in the asphalt layer is directly above the concrete joint. A schematic layout of the typical saw-cut and seal treatment is given in Figure 3.11.

Observations of the various cleaning and drying techniques would suggest that it is desirable to water jet the slot cutting in order to remove any detritus and then use a ‘hotdog’ type lance for drying the slots prior to the application of the bond breaker tape and sealant. The ‘hotdog’ type lance operates at a higher pressure than the larger lance and is more directional in applying heat to the cut slot with little or no heating of the surrounding asphalt surface.

In the UK, the sealant should comply to BS EN 14188-1 (BSI, 2004) and be fully compatible with asphalt material. Temperature control of the sealant during heating and application is important for the retention of the sealant properties. This control covers the safe heating temperatures and heating periods specified by the manufacturer and continued stirring of the sealant to ensure a uniform temperature throughout. The prefered method of application is through a re-circulating pump applied directly from a heating unit. This method avoids the need to transfer sealant to watering cans, which can be blocked as the sealant cools. The method for unblocking usually involves the application of a direct flame to the watering can spout, which is likely to overheat the sealant in this localised area and alter the properties of the sealant.

Good quality site control during the installation of the joint sealant is an important factor in the overall performance of the saw-cut and seal treatment. Figures 3.12 to 3.16 show the saw-cut and seal process used on site.

Performance of the saw-cut and seal treatment on various proprietary thin surfacings on the A1 Eaton Socon reported by Nicholls and Carswell (2004), constructed in 1994, have indicated that up to 30 mm thickness of overlay is not sufficient for successful performance of the saw-cut and seal treatment. Even with very accurate alignment of the saw cut in the asphalt with the joint in the concrete below, cracking can occur in the asphalt either side of the sealed slot as shown in Figure 3.17. For practical reasons it is, therefore, recommended that the minimum overlay thickness for this treatment should be 70 mm comprising 40-50 mm binder course and 20-30 mm thin surface course.

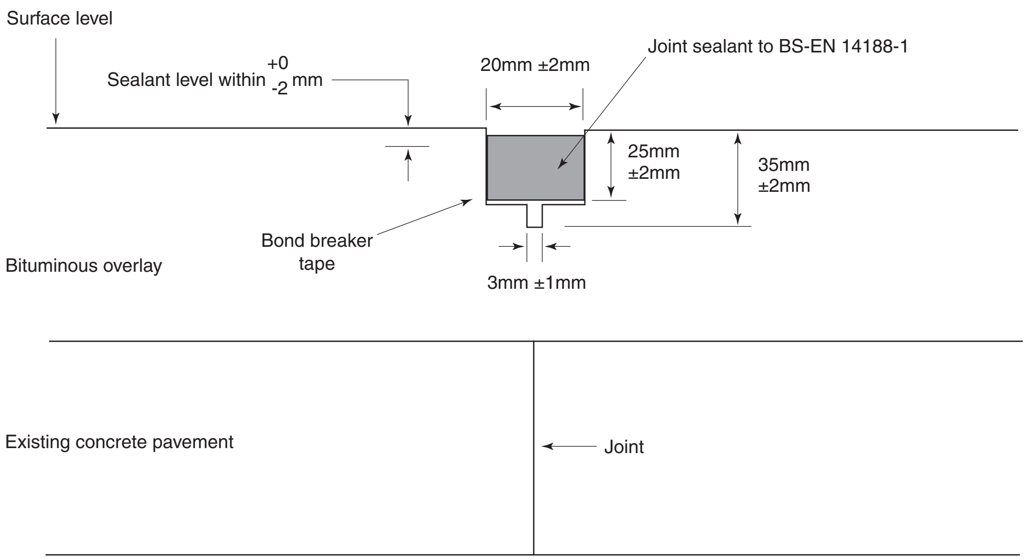


Figure 3.11 Schematic layout of saw-cut and seal technique (MCHW Volume 1: Figure 7/1)

Figure 3.12 Saw-cutting of the asphalt surface

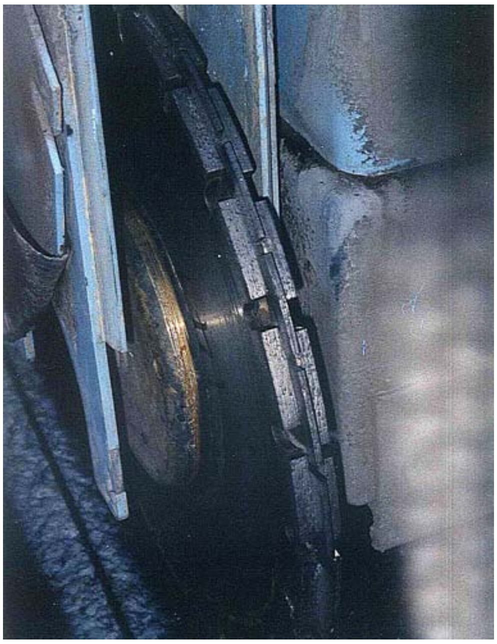


Figure 3.13 Close-up of the saw-cut blades creating the slot and groove

Figure 3.14 Slot being cleaned by water jet

Figure 3.15 'Hotdog' lance used for cleaning and drying of cut slots

Figure 3.16 Slot being sealed after the application of bond breaker tape using a re-circulating pump

Figure 3.17 Cracking on both sides of saw-cut and seal treatment on 30 mm thick thin surfacing applied directly to jointed concrete pavement

3.4.4 Slot-sealing

Slot-sealing is a technique used on thin surfacing, to overcome the problems described in Section 3.4.3 with overlays less than 50 mm. The technique involves bond breaker tape being applied across the concrete joint prior to overlay. After laying the surface course, a slot roughly 25 mm wide is sawn through the full depth of the asphalt and above the joint in the concrete. This slot is then filled with a sealant complying with BS EN 14188-1 (BSI, 2004). To date, this technique has not been trialled in the UK.

3.4.5 Programmed sealing

Programmed crack sealing is more of a maintenance regime than a treatment. The theory is that it is very hard to prevent reflection cracking occurring in thin overlays and, therefore, a certain proportion of unhindered reflection cracks may occur in the overlay before being sealed. This approach is an alternative to slot-sealing and the saw-cut and seal techniques where the position of the slots and saw-cuts need to be located very accurately over the joints in the concrete. However, this is an intervention treatment and several visits may be required for further or repeat treatments.

This maintenance regime was used on the M25 Junctions 16 to 17, where the 25 m long JRC slabs were overlaid with a Thin Surface Course System (TSCS) in 2000. Crack sealing was planned for 2002/03 but was not carried out until 2004/05. Due to the amount of deterioration (and hence more expensive treatments), only part of the scheme was completed with the remainder planned for 2007/08.

The cracks were treated by planing out 300 to 600 mm strips of the TSCS at the cracks locations and filling them with a Fibrescreed type material.

4 UK trial sites

4.1 Overview

In order to achieve the objectives of this project, it was necessary to collect a large amount of data from a number of sites. The following list of factors was borne in mind when determining which sites, other than TRL full scale trials, were monitored:

- *Construction records:* Are the original construction, overlay thickness and makeup of the overlay materials known?
- *Age.* Is the site likely to produce useful data? i.e. has it been in service for a sufficient time period?
- *Location.* Is the site accessible for daytime closures?
- *Multiple techniques.* Does the site contain multiple overlay techniques?
- *Previous visual surveys.* Are results from visual surveys prior to and post overlay available?
- *Control area.* Does the site contain a control area for a direct comparison?

From the list in Appendix A, 35 sites were selected and, where possible, surveys have been conducted under lane closures. However, for those sites where a lane closure was not available, a coarse visual survey was performed either from the edge of the carriageway or via a drive-over survey. Figure 4.1 shows the graphical location of the sites monitored while Table 4.1 summarises the sites monitored.

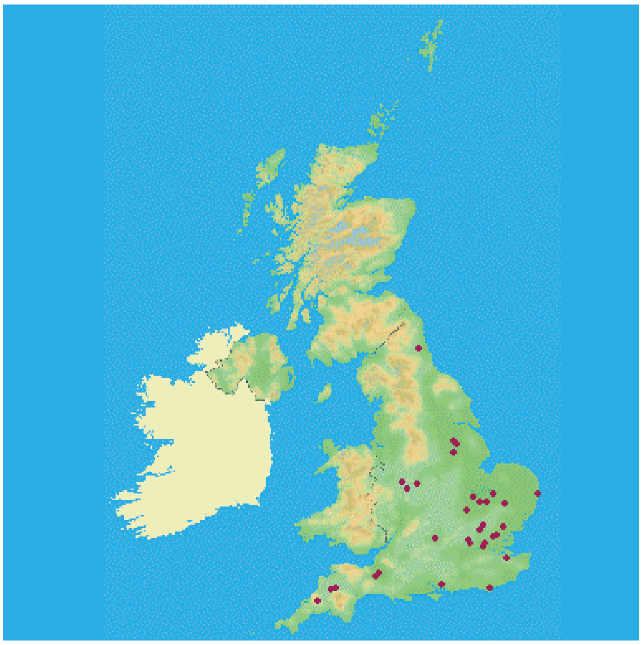


Figure 4.1 Graphical location of trial schemes monitored

4.2 Observations and performance

4.2.1 General

The performance of the trial sections after construction was monitored in order to establish the effectiveness of the techniques in inhibiting reflection cracking when compared to control sections. The monitoring was also to determine the effect of overlay thickness on the initiation and propagation of reflection cracking. The structural condition of the pavement was also monitored so that any progressive deterioration could be observed over time.

The methods used to assess the performance of the test sections after the asphalt maintenance treatments were applied include:

i Visual condition surveys (VCS).

ii Core extraction for inspection of crack propagation, to provide a measure of overlay thickness and for providing samples for material testing.

iii Load transfer efficiency across transverse joints using the FWD to assess the severity of reflection cracking.

iv Measurement of centre of slab deflection using the FWD to determine the overall stiffness of the pavement and the contribution from the bound and unbound components.

v Measurement of longitudinal profile using the TRL High-speed Survey Vehicle (HSV) and/or Highways Agency Road Research Information System (HARRIS) to identify structural deterioration.

Table 4.1 Summary of sites monitored

Scheme		*Constructed*	*Concrete type*	*Techniques monitored*
A1	Eaton Socon	Sep-91 & Jun-95	JRC, URC	Slot-sealing
A1	Markham Moor	Jan-00	JRC	SCCSO, SCS
A1	Tuxford	May-99	JRC	SCCSO, SCS, Geogrid
A1	Winthorpe-Coddington	Feb-04	JRC	SCS, SCCSO
A10	Ely	Sep-00	CBGM	CSO
A12	Boreham	Feb-00	URC	CSO
A12	Brentwood 1	Jan-01	JRC	SCCSO
A12	Brentwood 2	Nov-02	JRC, URC	CSO, SCCSO
A12	Hatfield Peverel	Feb-02	JRC	SCCSO
A12	Lowestoft	Mar-00	URC	SCS
A12	Mountnessing	Feb-99	URC	CSO
A12	Stanway	Apr-00	JRC, URC	CSO, SCCSO
A14	Milton-Fen Ditton	Aug-98	CBGM	CSO
A14	Quy	Jul-93	URC	CSO, SCS
A14	Spittals-Alconbury	Feb-99	URC	CSO
A14	Bury-St-Edmunds	Oct-90	URC	SCS
A167	Durham	1997	JRC	Geogrid
A259	Pevensey	Oct-03		Modified asphalts
A30	Bodmin	1989	CBGM	Geogrid, SAMI
A30	Launceston	1987	CBGM	Geogrid
A30	Plusha	1997/1998	ECOPAVE	CSO
A36	Ower	May-05	URC	CRL
A38	Swinfen-Weeford	Sep-00	CBGM	CSO
A46	Cossington-Six Hills	Jun-02	CBGM, JRC	CSO, SCCSO
A46	Kenilworth	Nov-98	URC	CSO
B1441	Weeley	Oct-04	JRC	Rubblisation
M1	Barnet J2-3	Aug-94	JRC	SCS
M1	J2-Deansbrook	Dec-99	JRC,URC	CSO, SCCSO, SCS
M2	Kent (Faversham)	1990	JRC	Geotextile
M27	J8-10	Jul-00	JRC, URC	CSO, SCCSO, SCS
M40	J6-7	Mar-97	URC	CSO
M5	Taunton 1992 Site	May-92	URC	CSO, SCS, Geotextile
M5	Willand	Sep-03	URC	SAMI, SCS

vi Measurement of rut depth using the TRL Transverse Profilometer (TP) and/or HARRIS to identify any differences in resistance to permanent deformation of the asphalt overlays.

Transverse Profilometer surveys were only undertaken where rut development had been noted from the visual condition and/or HSV surveys.

As part of the visual survey at each site, the location and length of every crack (where possible) was measured, together with a rating of the severity of the crack, defined as follows:

a Wide (greater than 2 mm), often with spalling or bifurcation.

b Easily visible (less than 2 mm wide), single crack.

c Fine (including cracking only seen when the road surface is drying).

To bring the results to a common denominator so that a direct comparison can be made of the performance of the different test sections, the amount of reflection cracking above the joints, or cracks, is expressed in terms of a Crack Index.

The Crack Index (CI) is a quantitative measure designed to take into account the visual severity rating in the assessment of reflection crack propagation. The CI is used to differentiate between sections with similar numbers of cracks and crack lengths, but with different severities. It uses the total length of reflection cracking weighted by a factor depending on the visual severity grading of the reflection crack. The CI value can be calculated using the following formula:

$$CI = \frac{\begin{pmatrix}\text{length of 'c' cracks} \times 1 + \text{length of 'b' cracks} \\ \times 1.5 + \text{length of 'a' cracks} \times 2\end{pmatrix}}{\text{total length of transverse joints within section} \times 2}$$

Hence, for a CI of unity, all cracks would be full width and category 'a', severe.

4.2.2 TRL trial sites on URC

4.2.2.1 A14 Quy

The A14 at Quy is a two-lane dual carriageway and the trial consisted of works to both lanes of the westbound carriageway. The existing pavement construction consisted of 250 mm thick unreinforced concrete slabs, 5 m in length, on top of 185 mm cement bound granular subbase on a chalk subgrade. The road was originally opened to traffic in 1977.

The trial consisted of three thicknesses of overlay (100 mm, 150 mm and 180 mm), and contained sections with SBS modified binder surface course, saw-cut and seal, and crack and seat each with the different overlay thickness. Control sections with conventional hot rolled asphalt (HRA) overlay were included in order to judge the performance of the trial sections. The trial was constructed in July-August 1993. Figure 4.2 shows the layout of the trial site, while Table 4.2 gives the details of each section and the proportion of each section affected by inlay since trial construction.

Concrete repairs were carried out as necessary, prior to overlay, and the existing joints in the concrete pavement

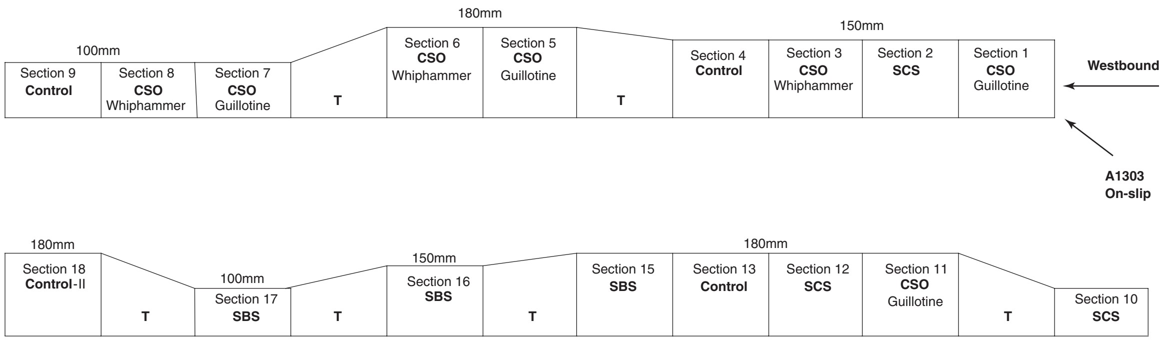


Figure 4.2 A14, Quy trial site

Table 4.2 A14 Quy trial site details

Section	*Treatment*	*Length (m)*	*No of joints/ natural cracks in concrete*	*% of section affected by inlay (lane 1)*
1	150 mm overlay with cracking by guillotine	240	48	14% 2003
2	150 mm overlay with saw-cut and seal	160	31	–
3	150 mm overlay with cracking by whiphammer	240	47	19% 1997 15% 2000 8% 2003
4	150 mm overlay only (control)	240	47	100% 1997 12% 2000 7% 2003
5	180 mm overlay with cracking by guillotine	250	49	4% 2000 5% 2003
6	180 mm overlay with cracking by whiphammer	250	49	72% 1997 4% 2000 2% 2003
7	100 mm overlay with cracking by guillotine	220	43	15% 2000 7% 2003
8	100 mm overlay with cracking by whiphammer	220	43	2% 2003
9	100 mm overlay only (control)	240	47	23% 2000 19% 2003
10	100 mm overlay with saw-cut and seal	160	30	–
11	180 mm overlay with cracking by guillotine	290	57	6% 2003
12	180 mm overlay with saw-cut and seal	160	32	10% 2003
13	180 mm overlay only (control)	405	80	20% 2000 13% 2003
15	180 mm overlay with SBS modified surface course	160	31	19% 2000 10% 2003
16	150 mm overlay with SBS modified surface course	160	32	27% 2000 21% 2003
17	100 mm overlay with SBS modified surface course	160	32	29% 2000 15% 2003
18	180 mm overlay with no joint treatment (control)	200	39	–

were routed out, with the exception of those in Section 18 where no repairs were made.

Due to excessive rutting, an inlay was applied to some areas of lane 1 in 1997. The inlay completely covered Section 4, much of Section 6 and a small area of Section 3. Further maintenance was carried out in 2000 and 2003, with a number of small areas being inlaid in both lanes of Sections 1, 3 to 9, 11 to 17 together with an extensive programme of overband sealing of cracks.

The latest detailed visual survey was carried out in November 2003 and Figures 4.3 to 4.6 shows the development of transverse cracking in lane 1 as a Crack Index value for the crack and seat by guillotine and whiphammer; the sawcut and seal; and the SBS modified binder overlay treatments respectively. Controls are included for reference on each treatment.

After ten years in service, it can be seen that, the crack and seat sections performed by whiphammer method are generally performing better than the control sections for each overlay thickness. However, the sections are not performing as well as those cracked using the guillotine method. These findings were the basis of the decision taken

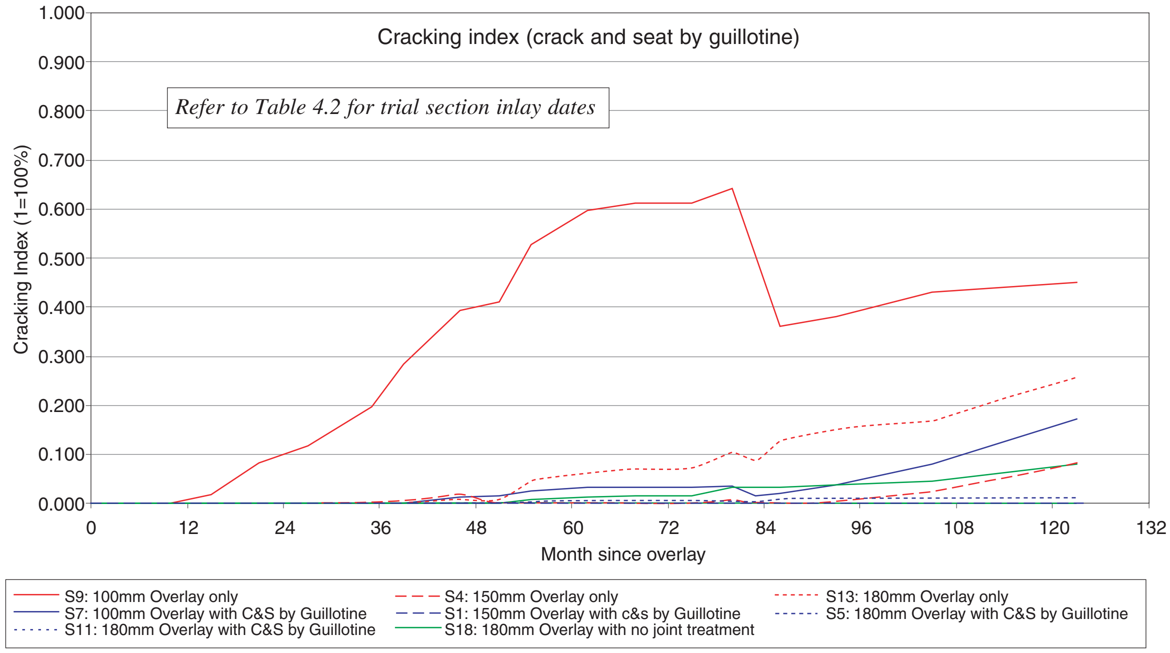


Figure 4.3 Development of reflection cracking, crack and seat by guillotine

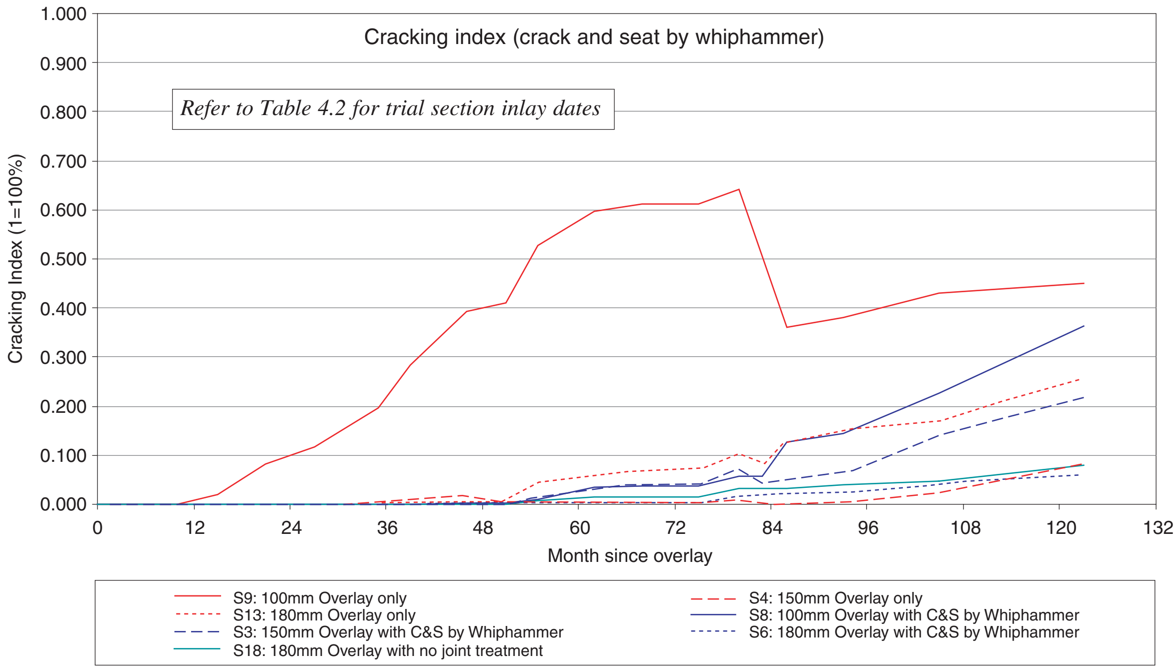


Figure 4.4 Development of reflection cracking, crack and seat by whiphammer

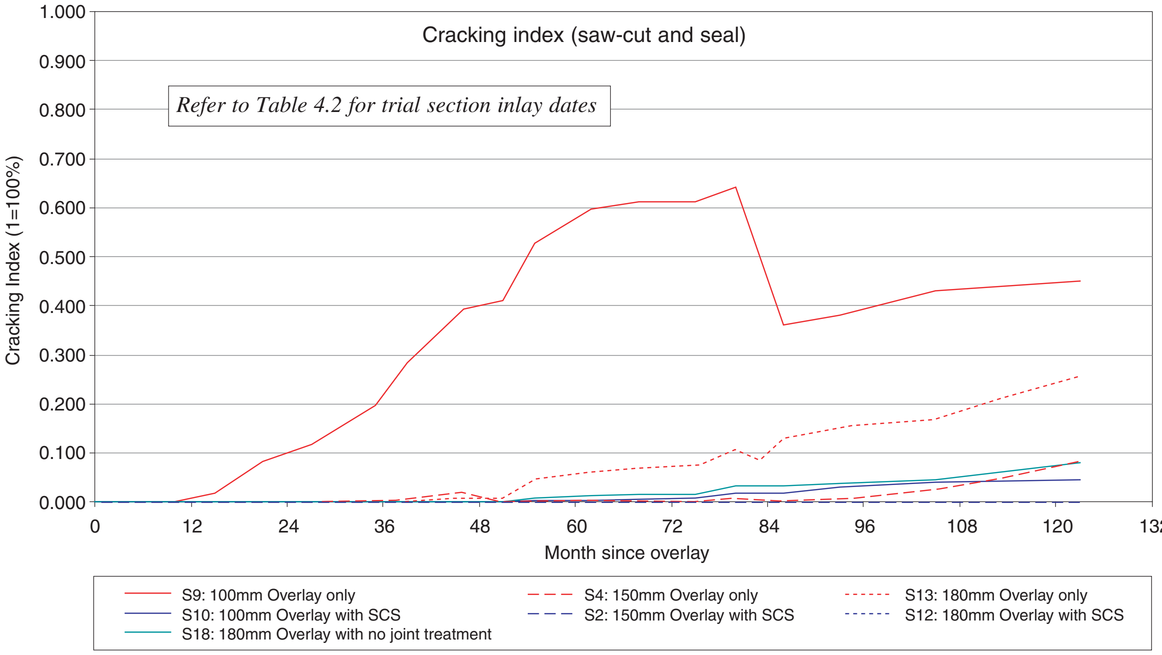


Figure 4.5 Development of reflection cracking, saw-cut and seal

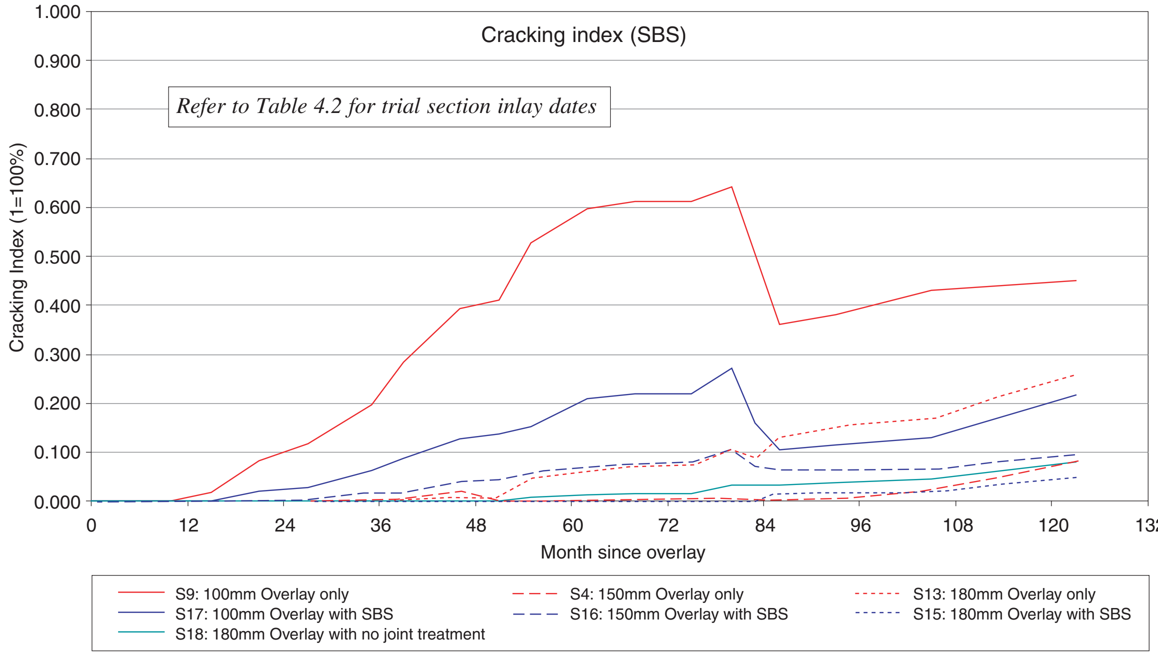


Figure 4.6 Development of reflection cracking, overlays with SBS

in early versions of the crack and seat specification to exclude the use of the whiphammer from the crack and seat specification. The possible reason behind the different performance of each method may result from the differing orientation of the cracks produced by the machines. The guillotine method produces transverse cracks whilst the whiphammer produces interlocking diamond shape patterns.

The majority of the cracking noted in the saw-cut and seal sections were due to separation of the sealant from the sawn slot. Only three joints in the 100 mm saw-cut and seal section showed actual cracking in the asphalt material adjacent to the seal. However, this cracking only appeared after the separation of the sealant had occurred. Compared to SCS techniques at other sites, the level of de-bonding at this site was higher than expected which reiterates the requirement for tight quality control during installation. Comparisons of the performance of the SCS technique with the controls show better performance even with the abnormally high de-bonding present. The use of the SBS modified binder has also performed well in comparison to the control sections.

Cores were extracted during April 2000 to ascertain the development of any cracking below the surface and to inspect the SCS techniques for alignment accuracy, seal integrity, and the development of any cracking beneath the seal. Locations were selected to give a variety of degrees of crack severity at the surface and a variety of load transfer values determined from FWD testing. The measured and nominal thickness of the asphalt layers in each section is presented in Table 4.3, with details of the extracted cores summarised in Table 4.4. Cores extracted over cracks with a severity rating of ‘c’ generally showed the cracking to be confined within the surface course layer. Cores extracted over cracks with a severity rating of ‘a’ or ‘b’ generally showed the cracking to have developed beyond the surface course and, in many cases, the cracking had developed to the full depth of the overlay. For cores extracted over SCS technique, a ‘c’ rating for separation of the seal at the surface often indicated cracking below the surface which, in many cases, were found to be the full depth of the asphalt overlay. Comparisons of FWD load transfer values with the development of cracking showed a load transfer below 0.75 (75%) after cracking had developed to the full depth of the overlay. The load transfer measured across cracks, which were confined within the surface course, were generally above 0.85 (85%).

Table 4.3 Asphalt layer thickness

Section		Number of cores	Nominal thickness (mm)	Average measured thickness (mm)
S2	150 mm overlay with saw-cut and seal	4	150	150
S4	150 mm overlay only (control)	2	150	145
S5	180 mm overlay with cracking by guillotine	1	180	175
S6	180 mm overlay with cracking by whiphammer	1	180	170
S7	100 mm overlay with cracking by guillotine	4	100	95
S8	100 mm overlay with cracking by whiphammer	4	100	93
S9	100 mm overlay only (control)	7	100	95
S10	100 mm overlay with saw-cut and seal	7	100	93
S11	180 mm overlay with cracking by guillotine	1	180	165
S12	180 mm overlay with saw-cut and seal	7	180	186
S13	180 mm overlay only (control)	6	180	179
S16	150 mm overlay with SBS modified surface course	3	150	145
S17	100 mm overlay with SBS modified surface course	4	100	99
S18	180 mm overlay with no joint treatment (control)	2	180	179

Of note, the results obtained for LTE from FWD will vary with the temperature of the materials being assessed. Most noticeably, in cold weather LTE is lower as the materials contracts and crack/joint widths increase thus reducing aggregate interlock across the crack. Conversly, in warmer weather LTE is generally higher as materials expand and crack/joint widths reduce, thus increasing the aggregate interlock and hence LTE.

It can be seen that the 180 mm control sections have an average thickness close to their nominal value. The associated crack and seat sections average marginally less than nominal, although it should be noted that only one core was taken from each section, and the saw-cut and seal section was marginally greater than nominal. This comparison of thicknesses would indicate that the comparisons of performance between the trial sections are reasonable.

The 150 mm control is, on average, marginally thinner, although still within the expected tolerances. It should, however, be noted that the actual thicknesses within this section vary greatly from 105 mm to 175 mm. The associated saw-cut and seal section averaged a thickness of 150 mm whilst no cores were extracted from the 150 mm crack and seat sections.

The cores from the 100 mm overlaid sections had average thicknesses marginally below the nominal thickness, with good consistency in the individual sections. The cores, therefore, justify the comparison between all the 100 mm sections.

4.2.2.2 A14 Bury St Edmunds

The A14 Bury St Edmunds is a two-lane dual carriageway with an annual average daily traffic flow (in 1990) of 38,000 vehicles, of which 15 % were heavy goods vehicles. The original construction consisted of a 200 mm thick cement bound granular sub-base overlaid by 250 mm thick unreinforced concrete slabs with joints at 5 m centres. The road was opened to traffic in 1974.

The concrete was generally in fair condition and very few repairs were carried out prior to overlaying. The bituminous overlay was constructed in October 1990 and the trial sections were located on the eastbound carriageway of this dual two-lane trunk road. The thickness of overlay was limited to 100 mm due to drainage restrictions. Details of the trial sections and layout are shown in Figure 4.7. Saw-cut and seal (SCS) was included in this trial although no crack initiation notch was cut in the bottom of the slot.

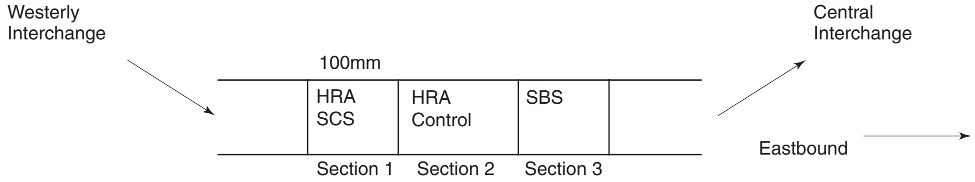


Figure 4.7 A14 Bury St Edmunds trial site

Table 4.4 Details of 100 mm diameter cores extracted from the A14 Quy in 2000

Location	*Section*	*Overall thickness (mm)*	*Detail*	*FWD load transfer*	*Crack severity or SCS debonding*
Jt 42	180 mm Control II	175	Cracking to 21/32 mm depth.	1.05	c
		183	Cracking to 11/17 mm depth.	0.89	c
Jt 161	100 mm SBS	99	Cracking to 18/13 mm depth.	0.94	b
Jt 172		Incomplete	Crack full depth, core in bits.	0.62	a/b
Jt 178B		99	Cracking to 52/37 mm depth.	0.71	a
Jt 184		98	Crack full depth.	0.39	a
Jt 202	150 mm SBS	140	Cracking to 32/34 mm depth.	0.91	c
Jt 211		Incomplete	Crack full depth.	1.00	a
Jt 218		150	No cracking seen.	n/a	c
Jt 517	180 mm Control I	180	Cracking to 47/71 mm depth.	0.92	a
		174	Cracking to 24/42 mm depth.	0.92	c
Jt 527		184	Cracking to 21/30 mm depth.	0.97	c
Jt 547		185	Cracking to 29/31 mm depth.	0.87	c
Jt 571		Incomplete	Cracking to 14/23 mm depth.	n/a	c
Jt 579		172	Cracking to 10/13 mm depth.	0.91	c
Jt 584	180 mm SCS	186	New seal ok. Cracking to 85/103 mm depth.	0.93	none, c*
Jt 585		198	New seal ok. Cracking to 66/98 mm depth.	0.89	none, c*
Jt 598		180	Seal looks ok, cracking from crack installation. slot to 75 mm depth on one side only.	0.92	none
Jt 599B		185	Crack full depth, seal failed, core split.	0.25	c
Jt 604		181	Cracking to 147/170 mm, seal possibly ok.	0.79	none
Jt 610		188	Crack full depth, seal failed, core separated at base layer.	0.75	c
Jt 613		183	Cracking to 122/144 mm depth, seal possibly still ok.	0.76	c
Jt 672	180 mm CSO by guillotine	165	Cracking to 18/45 mm depth, crack in surface of concrete (offset from joint by 1 m).	0.88	c
Jt 740	100 mm SCS	100	Seal failed, crack full depth.	0.73	c
Jt 747		100	Seal failed, crack full depth.	0.59	a/b
Jt 748		95	Seal partially failed, crack full depth.	0.82	c
Jt 751		Incomplete	Crack full dept, seal failed, core split.	0.55	b
Jt 755		85	Crack full depth, poor seal.	0.63	c/b
S 755		85	Cracking to 35/55 mm, crack forward of joint, occurred after sealant separation.	n/a	c
Jt 756		95	Seal partially failed, crack full depth, multiple cracks at mid-depth.	0.74	none, good
Jt 776	100 mm Control	94	Cracking to 15/54 mm depth.	0.82	b/c
Jt 791		97	Cracking to 40/27 mm depth.	0.88	c
Jt 792		96	Crack full depth, core split into two.	0.64	a
Jt 800		95	Crack full depth, core in bits.	0.59	a
Jt 802		98	Cracking to 28/30 mm depth.	0.90	c
Jt 809		91	Crack full depth.	0.39	a
Jt 810		95	Cracking to 22/30 mm depth, fibre screed on underside of core, plus cracking in full depth concrete (253 mm).	0.08	a
Jt 815	100 mm CSO by whiphammer	95	Cracking to 40 mm depth.	0.80	c
Jt 819		90	Crack full depth.	0.76	c
Jt 849		95	Crack full depth.	0.88	c
Jt 852		90	Cracking to 22 mm depth.	0.85	c

Continued ....

Table 4.4 ***(Continued)*** **Details of 100 mm diameter cores extracted from the A14 Quy in 2000**

Location	*Section*	*Overall thickness (mm)*	*Detail*	*FWD load transfer*	*Crack severity or SCS debonding*
Jt 867	100 mm CSO by guillotine	90	Crack full depth.	–	c
Jt 868-867		95	Crack full depth, plus cracking in full depth concrete.	–	c
Jt 886-885		95	Crack full depth, visible in concrete surface.	–	b
Jt 895-894		100	Cracking to 22 mm depth, plus cracking in full depth concrete.	–	c
Jt 960	180 mm CSO by whiphammer	170	Crack full depth, in thin surface patch.	0.63	b
Jt 1049	180 mm CSO by guillotine	175	Cracking to 35 mm depth.	–	c
Jt 1139	150 mm Control	145	Thin surfacing, no obvious cracking.	0.90	a
Jt 1148		Broken	Crack full depth, core in bits.	0.08	a
Jt 1211	150 mm SCS	148	Crack full depth, some horizontal cracking. one side of seal only.	0.75	c
Jt 1212		154	Crack initiation slot closed.	0.91	Good
Jt 1220		150	Cracking to 82/135 mm depth, seal failed.	0.85	Good
Jt 1223		146	Crack full depth, seal failed, base layer cracking.	0.77	c

** Assessment of SCS bonding made prior to resealing works.*
Crack severity: c= fine, b = medium, a = wide/multiple.

The HRA control and saw-cut and seal sections were inlaid with a thin surfacing material in lane 1 in spring 1997, following the occurrence of excessive rutting.

At a survey carried out in autumn 1995, prior to the inlay in 1997, no cracking was observed in 100 mm HRA control and the 100 mm SCS sections, although levels of rutting up to 25 mm were measured. The 100 mm SBS section, where minimal rutting has occurred, showed two joints with cracks during the 1995 surveys which remained unchanged to 2000, although the severity had increased from ‘c’ to ‘b’. This indicates that the 100 mm SBS overlay has performed well, increasing the resistance to rutting and reducing the effects of spalling where cracking has occurred. The SBS was included in the binder course and surface course at this site. A survey was carried out in February 2004 and it was observed that two cracks were present in the inlaid SCS section, three cracks present in the inlaid control section and two in the SBS overlay.

4.2.2.3 A36 circuit to Ower

The A36 at Ower is a dual two-lane carriageway with hardshoulder, the site is located between the M27 Junction 2 roundabout and the Ower roundabout. The nominal pavement construction consisted of 5 m jointed unreinforced concrete slabs, 290 mm in thickness, on 210 mm granular sub-base. The condition of the concrete was fair with minimal cracking and several spall repairs. Figure 4.8 shows the general condition of the A36 prior to overlay.

The proposed maintenance of the 800 m section consisted of a porous asphalt crack relief interlayer on the southbound carriageway and SCS and a 100 m control section of the northbound.

The works were carried out in March 2005 and the overlay to the southbound consisted of 50 mm porous asphalt binder course as shown in Figure 4.9 designed to Clause 938 of the *Specification for Highways Works* (MCHW:1). The surfacing consisted of 30 mm thickness of TSCS to Clause 942 as shown in Figure 4.10.

Figure 4.8 Typical condition of concrete joints on the A36, Ower

Figure 4.9 50 mm thick porous asphalt crack relief layer

Figure 4.10 30 mm thick TSCS

The overlay to the northbound carriageway consisted of 50 mm Heavy Duty Macadam (HDM) base course designed to Clause 931 of the MCHW:1. The surfacing consisted of 30 mm TSCS. Once overlaid, the saw-cut and seal technique was applied to roughly 700 linear meters of the surface in both lanes 1 and 2 (the hard shoulder remained un-sawn), the remaining 100 m being left as a control section.

A coarse visual survey undertaken in January 2006 showed no defects and further monitoring will be required to establish the potential of these treatments.

4.2.2.4 M5 Taunton: 1992 trial site

The M5 Taunton trial site was built on the northbound carriageway between Junction 26 and the Taunton Deane Service Area. The M5 here is a three-lane dual carriageway with a hard shoulder. The pavement construction consisted of 230 mm thick jointed unreinforced concrete with a slab length of 6 m. As part of a rehabilitation programme, sections of CSO and SCS were constructed with overlays of 100 mm and 150 mm, together with control sections. Sections including a geogrid system and a SAMI were also trialled with 150 mm overlay. This section of road had an annual average daily traffic flow (in 1992) of 52,000 vehicles, 20% of which were heavy goods vehicles.

In 1997, an inlay of HRA to Clause 943 of the MCHW 1 was applied to lane 1 of Sections G to J of the trial area to remove areas where the surfacing had exhibited excessive rutting. Subsequently in 1999, an inlay of thin surface course was applied to Sections A to G in lane 1, again to treat severe rutting. Figure 4.11 shows the layout of the M5 1992 trial site.

A detailed visual survey was conducted in October 2004, and several new cracks were observed and some of the existing cracks had grown in length and severity. Figure 4.12 show the Crack Index values for each section.

From the results prior to inlay, two sections did not contain any cracking – Sections F (C&S) and J1 (SCS). The survey undertaken in October 2004, showed the occurrence of reflection cracking in Sections B, C, D, H, J3 and I. The SCS treatment in lane 1 was effectively lost when the inlay was applied. However, the SCS treatment is still extant in lanes 2 and 3 with no visible signs of separation or deterioration.

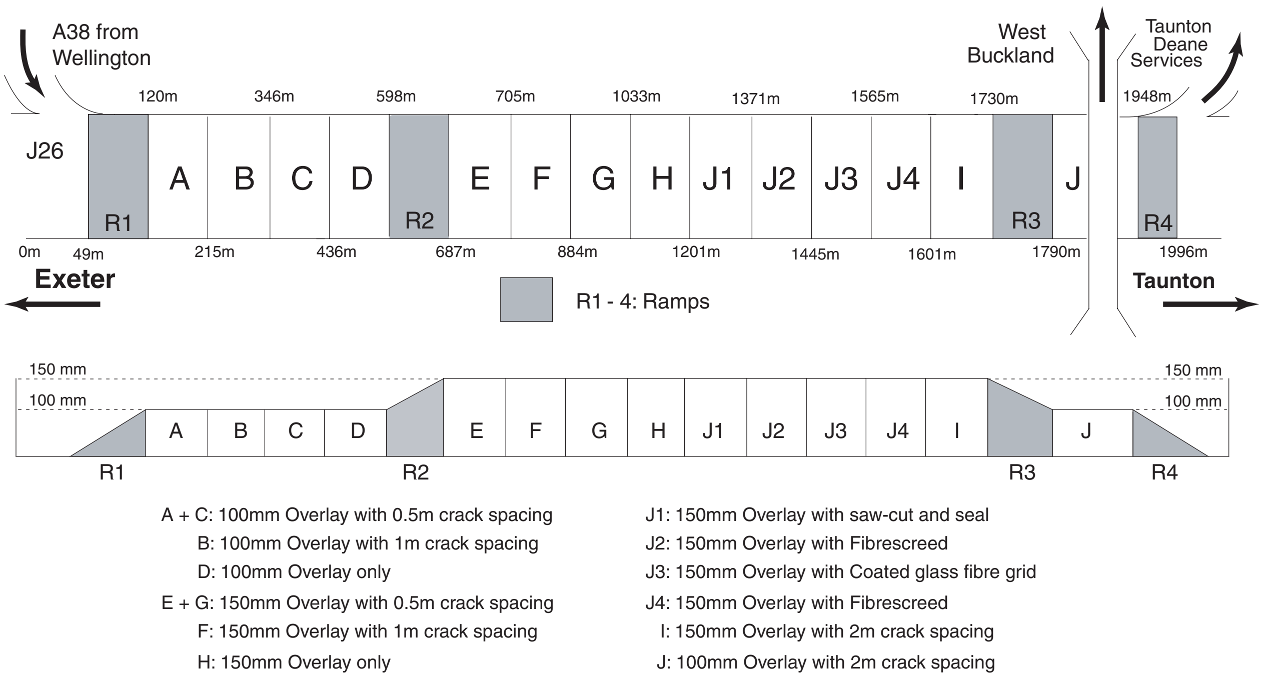


Figure 4.11 Schematic of M5 trial layout

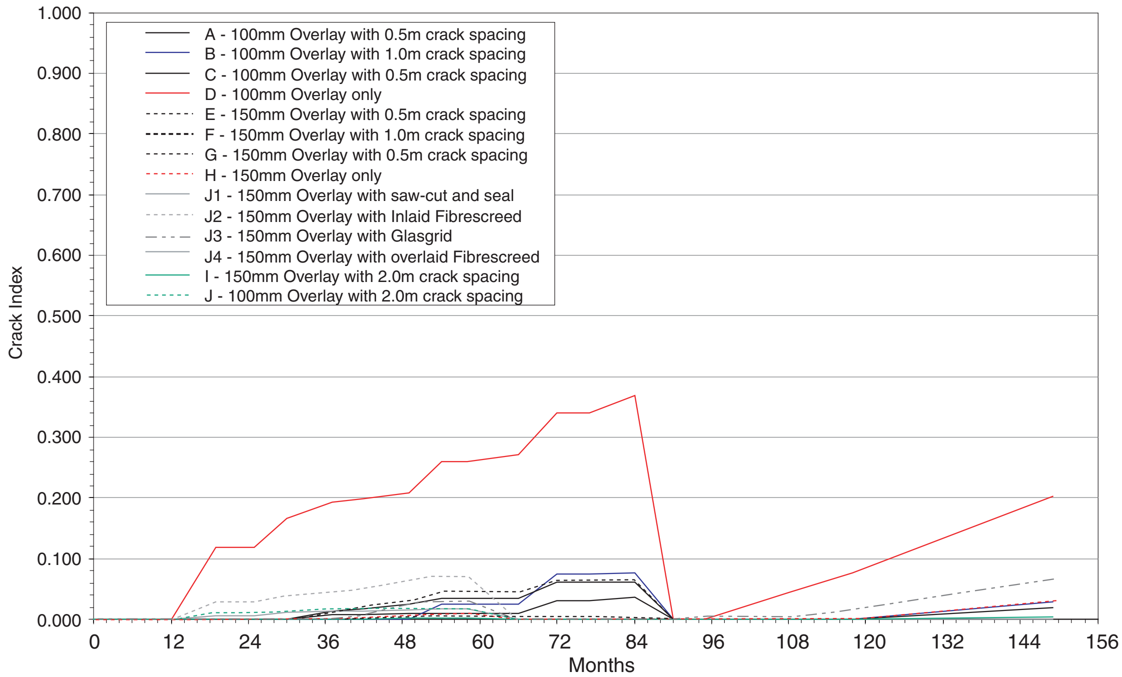


Figure 4.12 Comparison of trial sections, by Crack Index

4.2.3 TRL trial sites on JRC

4.2.3.1 A1 Eaton Socon

The A1 at Eaton Socon consists of a jointed unreinforced concrete pavement on the southbound carriageway and jointed reinforced on the northbound carriageway. The southbound carriageway was overlaid partly in 1991 with one and partly in 1995 with three different types of TSCS, while the northbound carriageway was overlaid in 1995 with the same three types of TSCS. The TSCS were applied directly to the jointed concrete and ranged in thicknesses from 15 mm to 30 mm. The saw-cut and seal treatment was then applied to some of these sections, and it was thought that the performance of these sections would provide guidance on the limit of thickness of overlay that can be applied for the treatment to be effective.

The monitoring of the trial site was split into three sections (all on the northbound carriageway). See Figure 4.13:

- *Section 1:* 20 mm thick TSCS (0/10 mm aggregate size) on 6 m URC and 24 m JRC.
- *Section 2:* 20 mm thick TSCS (0/14mm aggregate size) on 24 m JRC.
- *Section 3:* 30 mm thick TSCS (0/14 mm aggregate size) on 12 m and 24 m JRC.

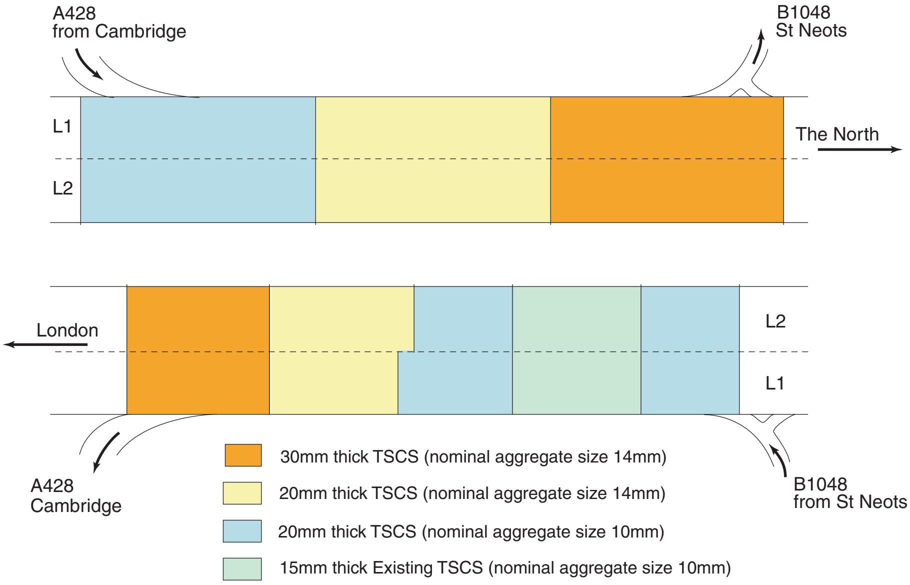


Figure 4.13 1995 A1 site, Eaton Socon

In September 2002 a series of repairs were carried out on the pavement. For calculation purposes, the joints where inlay and overband repairs had been applied were considered as wide cracking to the full width of the repair.

A detailed visual survey was carried out in September 2004 on 200 m sections from all three sites of the northbound carriageway. Table 4.5 shows the Crack Index values for each section, and indicates that severe cracking occurs relatively quickly with thin overlays directly over the jointed concrete.

Figures 4.14 to 4.16 show general views of each trial section on the A1 Eaton Socon.

Table 4.5 Crack Index

	Time after construction	
Section	*99 months*	*111 months*
1	0.66	0.82
2	1.00	1.00
3	0.65	0.81

4.2.3.2 A1 Markham Moor

The A1 Markham Moor is a dual-carriageway originally constructed in 1966-1967 as the Tuxford by-pass. The road is a two-lane dual carriageway with 7.9 m wide carriageways.

The pavement construction consisted of jointed reinforced concrete laid to a depth of 230 mm throughout on a composite base of 125 mm CBGM on 100 mm of crushed rock Type 1 sub-base. The subgrade consisted of silty clay.

The concrete was laid across the full width of the pavement with contraction joints formed at 24.5 m (80 ft) intervals and expansion joints at 73 m (240 ft) spacings. The longitudinal joint to form lanes 1 and 2 was inserted during the laying process.

Two SCS trial sections (with different thicknesses of overlay) were constructed at Markham Moor in 2000. Section 1, north of the overbridges, was overlaid with 100 mm of asphalt while Section 2, south of the overbridges, was overlaid with 120 mm of asphalt.

Figure 4.14 A1 Eaton Socon: Section 1

Figure 4.15 A1 Eaton Socon: Section 2

Figure 4.16 A1 Eaton Socon: Section 3

A coarse visual survey was carried out in February 2005 from the verge, no cracks were seen but a couple of seals had been pulled away from the saw-cut which, in turn, had caused the saw-cuts to close up. Figure 4.17 shows the general condition of the saw-cuts. Lengths of rutting were also noted along the entire length of the first trial section. Further investigation is needed under a lane closure to establish the cause of this permanent deformation.

4.2.3.3 A1 Tuxford

The A1 at Tuxford is a dual-carriageway originally constructed in 1968. The pavement construction consisted of jointed reinforced concrete laid to a depth of 230 mm throughout on a composite base of 125 mm of CBGM on 100 mm of crushed rock Type 1 sub-base. Transverse joints were formed in the pavement quality (PQ) concrete at 25 m intervals.

Two SCCSO trial sections (with different saw-cut spacings) and a control section were constructed at Tuxford in 1999. Cracking was performed by a guillotine with a steel strike-plate to minimise damage to the saw-cuts. Sections treated with a steel geogrid and saw-cut and seal (SCS) were also included in the trial. The test sections were overlaid with a total 100 mm thickness of high modulus base with a 35 pen binder (HMB35) and a TSCS. The trial also included sections with only a 50 mm thick TSCS containing the steel geogrid, SCS and a control. Figure 4.18 shows the layout of the A1 Tuxford trial.

Vacuum grouting was used to stabilise the transverse joints in each section apart from those treated by SCCSO. The method resulted in generally poorer load transfers and differential vertical movements at joints. Figure 4.19 shows the load transfer and differential vertical movements before and after grouting. The following was observed on site at the time of installation of the steel grid reinforcement:

- Holes were not drilled in the PQ concrete prior to nailing the steel grid, in order to ensure the grid was securely fixed.
- Tack coat was not applied to the concrete surfaces prior to the application of the slurry surfacing.
- The design of the spreader boxes resulted in the slurry surfacing being applied too thick.

Figure 4.17 Saw-cuts on the A1 Markham Moor trial

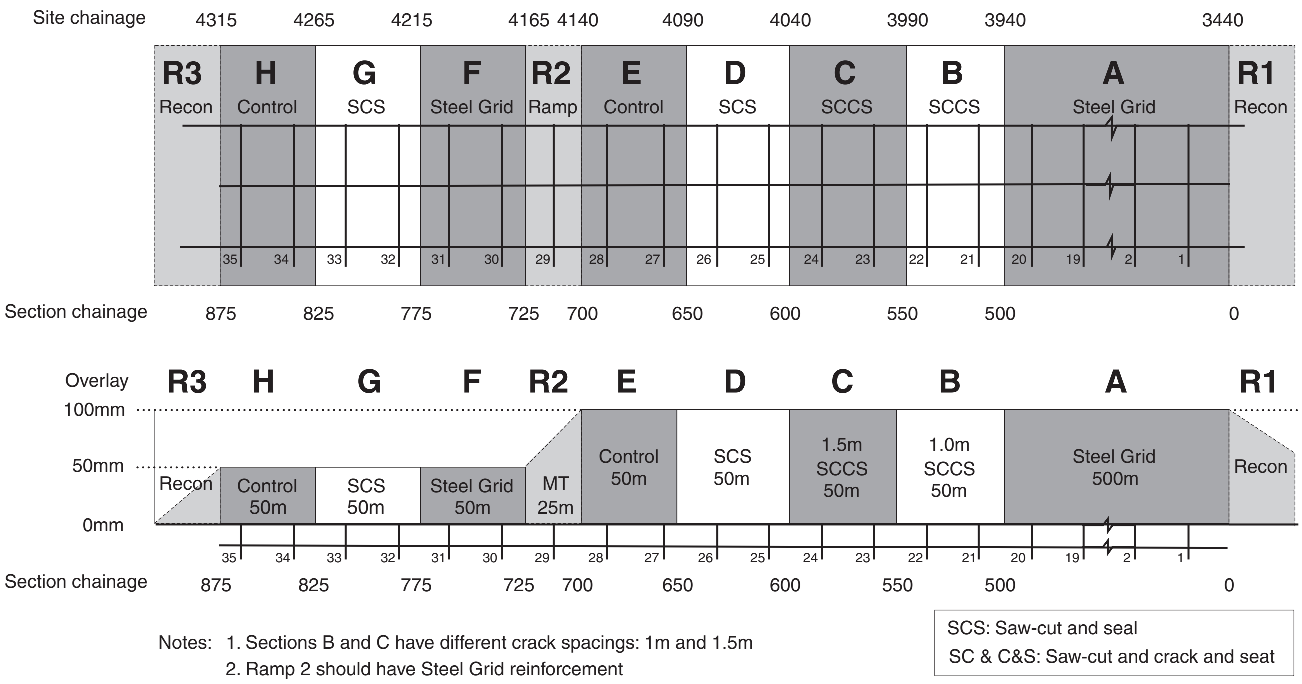


Figure 4.18 1999 A1 site, Tuxford

A detailed visual survey was carried out in February 2005 under a lane closure. Coring was carried out to investigate depth of cracking and whether the crack in the asphalt was above a joint or crack in the concrete. Crack lengths and severity of cracking were recorded and the results for Cracking Index are shown in Figure 4.20 for the 100 mm and Figure 4.21 for the 50 mm asphalt overlay sections.

From the Cracking Index values, it can be seen that, after 66 months service, the sawcut and seal sections were performing well with Crack Index values of 0.01 and 0.00 for the 100 mm and 50 mm overlay, respectively. All joints that had reflected through in the 100 mm control section contained wide cracks, and coring showed these cracks to extend the full depth of the asphalt layer and needed sealing to prevent ingress of water.

The 100 mm overlay with steel grid, Section A, also showed areas of failure in the form of depressions as shown in Figure 4.22. One possible cause for this defect may be debonding of the steel grid.

4.2.3.4 A1 Winthorpe

The A1 at Winthorpe is a dual 2-lane carriageway consisting of 25 m long reinforced concrete bays with a nominal thickness of 270 mm PQ concrete over 110 mm nominal thickness CBGM base.

Investigations carried out in May 2001 prior to overlay showed the pavement to be in poor condition with 50 % of the joints in lane 1 having load transfer of less than 50 %. Dynamic cone penetrometer (DCP) tests showed the top of the subgrade to be weak with California bearing ratio (CBR) value from 1 to 9 %, but that the CBR increases rapidly with depth.

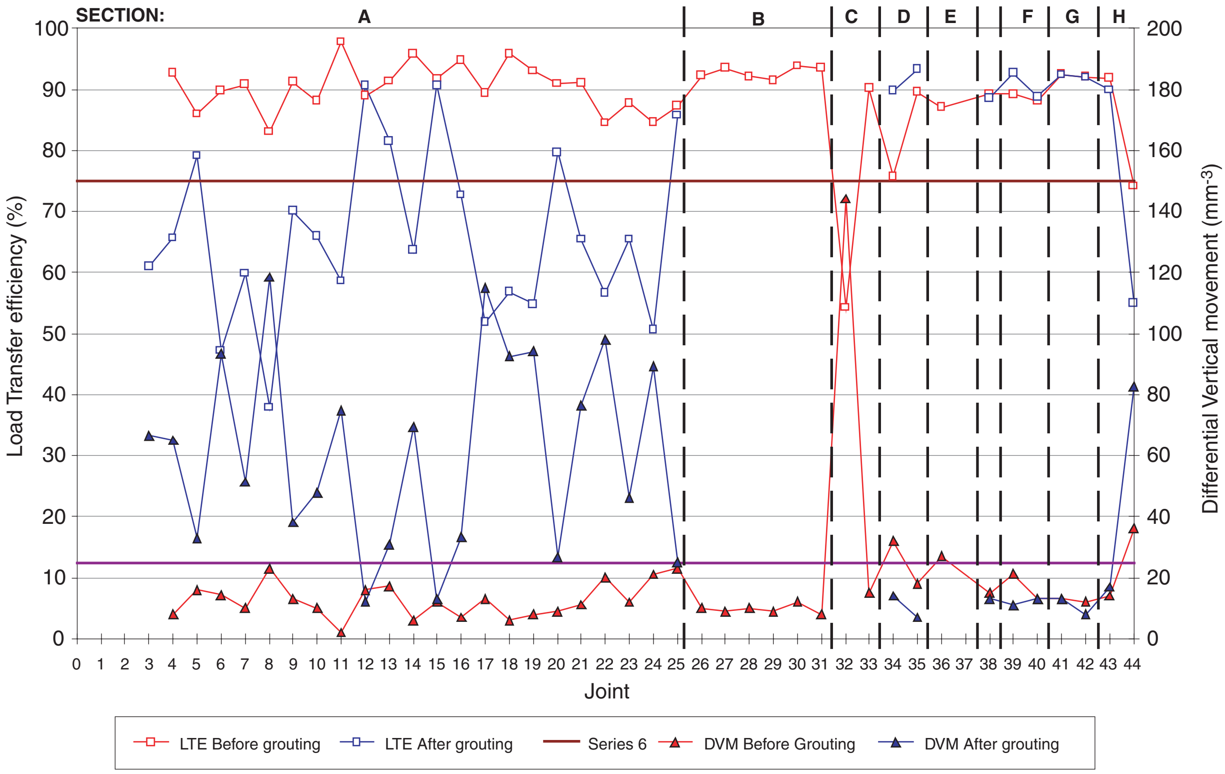


Figure 4.19 Load transfer efficiencies and differential vertical movements before and after vacuum grouting

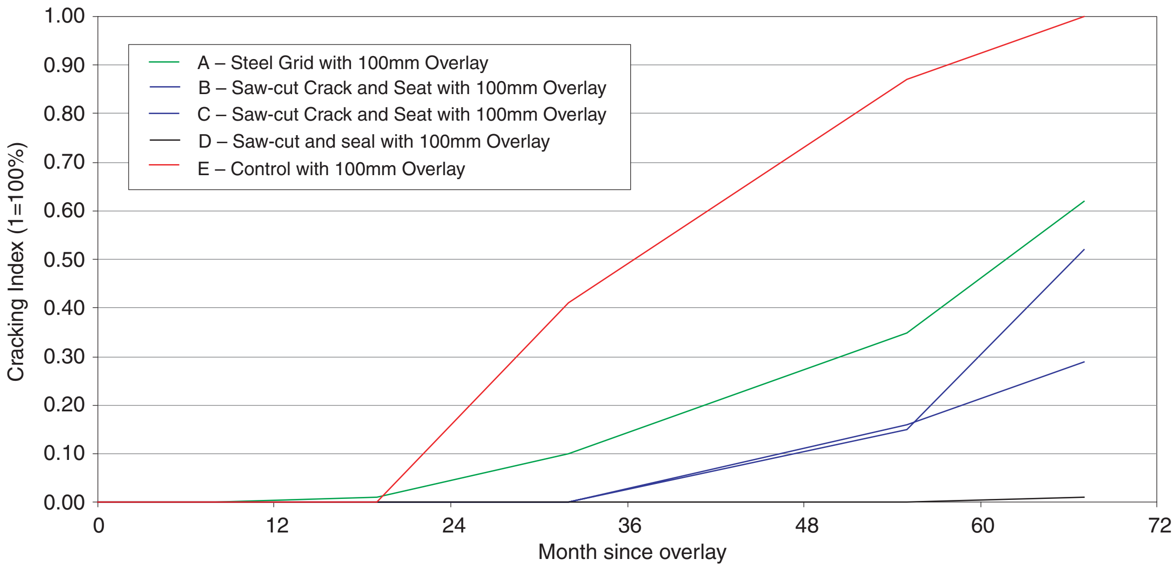


Figure 4.20 Development of reflection cracking, 100 mm overlay

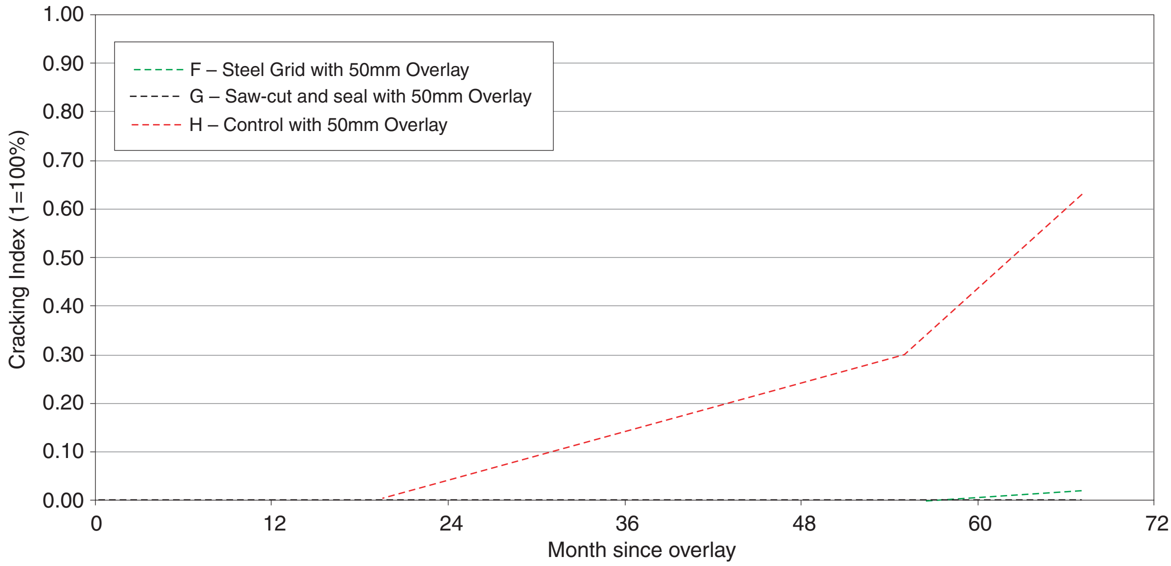


Figure 4.21 Development of reflection cracking, 50 mm overlay

Figure 4.22 Typical defects seen in Section A – steel grid with 100 mm overlay

The future anticipated traffic was calculated to be 137 million standard axles (msa) for a 20 year design.

Four SCS trial sections (with different spacing of saw-cuts and different thicknesses of overlay) were constructed at Winthorpe between October 2003 and March 2004. Trial site locations are shown in Figure 4.23.

A coarse visual survey was carried out from the edge of the carriageway on 19 February 2005. No defects were seen, although it appears that, in a number of saw-cuts, the sealant may have been delaminating in the nearside wheelpath of lane 1. Further investigation and monitoring was recommended.

4.2.3.5 M1 Barnet J3-4

The M1 Barnet site is a three-lane dual carriageway with hardshoulder, roughly 900 m in length. The site is located between London Gateway (formerly Scratchwood) services and Junction 4 northbound. The original pavement construction consisted of 280 mm thick reinforced concrete slabs with joints at 12 m spacing on 150 mm thick CBGM base on a 230 mm thick granular sub-base.

The trial, constructed between May and August 1994, consisted of three thicknesses of asphalt overlay (110 mm, 150 mm and 180 mm) with and without SCS applied to the finished surface. The overlay consisted of a HDM binder course and a HRA surface course, Figure 4.24 shows the layout of the trial.

A number of full depth concrete repairs were undertaken prior to overlay, replacing complete slabs and partial slabs at existing joint and mid-slab locations. These treatments effectively produced a site of varying slab lengths from 1.5 m to 12 m, with some 18 m in the 110 mm control where a series of existing 12 m slabs were replaced with longer 18 m slabs.

The latest detailed visual survey was carried out in July 2005 and Figure 4.25 shows the development of transverse cracking in lane 1 as a Crack Index value.

The 110 mm control section contains three nominally 18 m long bays replacing four original 12 m bays. Reflection cracks occurred over each joint within the first 12 months following overlay which could lead to an incorrect comparison between the performance of the control

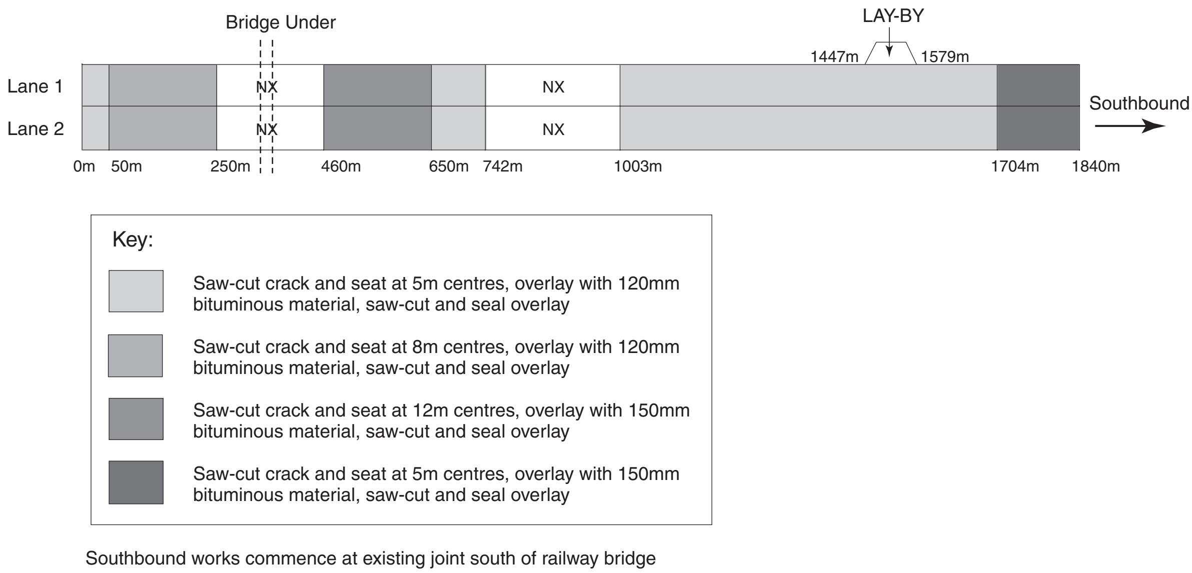


Figure 4.23 A1 Winthorpe to Coddington trial site

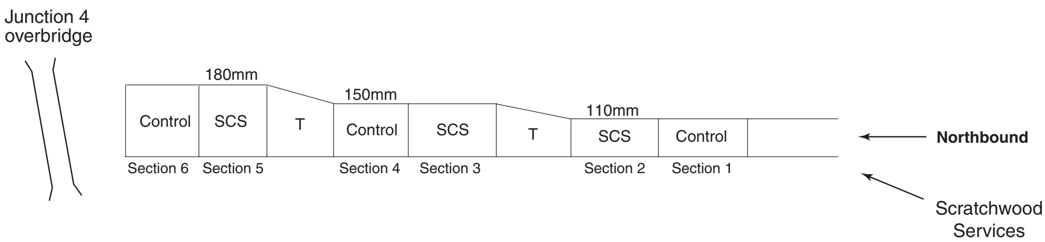


Figure 4.24 Layout of the M1 Barnet site

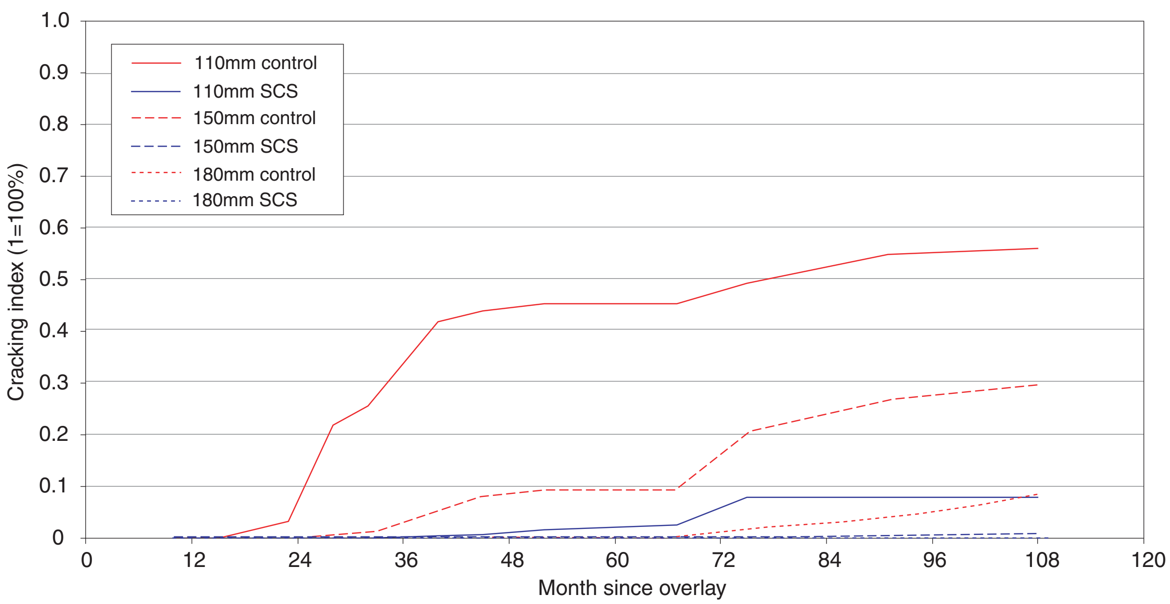


Figure 4.25 Development of reflection cracking, saw-cut and seal

section and the SCS treatment with 110 mm thick overlay. The SCS treatment sections were on bay lengths of 12 m. The performance comparisons between the two sections have, therefore, been made by omitting the crack development above the joints at the end of the 18 m bays in the control section. The results show that, even 180 mm thickness of asphalt may exhibit signs of reflection cracking within only four years of overlaying a jointed concrete pavement.

It can be observed that, after nine years service, minimal cracking and/or delamination of the sealant has occurred in the saw-cut and seal sections with all SCS sections having a CI value of less than 0.1. Figure 4.26 shows delaminating of the sealant observed on the M1 Barnet. However, the

Figure 4.26 Delaminating of the sealant observed

110 mm SCS section has performed better than both the 110 mm and 150 mm controls. The 150 mm sawcut and seal has performed better than all of the controls and the results demonstrate that this treatment can be effective on slab lengths up to 12 m.

4.2.3.6 M2 Kent

The M2 motorway is a two-lane dual carriageway with hardshoulder that links Dover with London. The original construction consisted of 250 mm thick jointed reinforced concrete pavement, with joints at 24 m spacings, on a 200 mm thick flint gravel sub-base. The road was originally opened to traffic in 1963.

In 1990, the condition of the concrete running surface was considered to be generally poor, particularly at the joints. Extensive spalling had occurred at some joints whilst, at some others, temporary repairs in the form of patching had been carried out. Consequently, about half the joints were considered to be in poor condition and were renewed prior to overlay. This action effectively divided the test sections into two categories:

- Those joints in fair condition prior to overlay that received 'minimal' treatment.
- Those joints in poor condition prior to overlay that received 'maximum' treatment.

The six trial sections were located in lanes 1 and 2 of the westbound carriageway of the M2 motorway. The asphalt overlay was constructed in May 1990.

The overlay consisted of a DBM binder course with a 40 mm HRA surface course. In some sections, the surface contained an EVA modified binder whilst a geotextile was used on Sections 4, 5 and 6 to act as a stress absorbing layer between the concrete road and the asphalt overlay. The geotextile was a needle punched fabric, and was bonded to the concrete carriageway using bitumen. Figure 4.27 shows the layout of the trial site.

Figure 4.28 shows the Cracking Index development from construction to 173 months service (September 2004). The

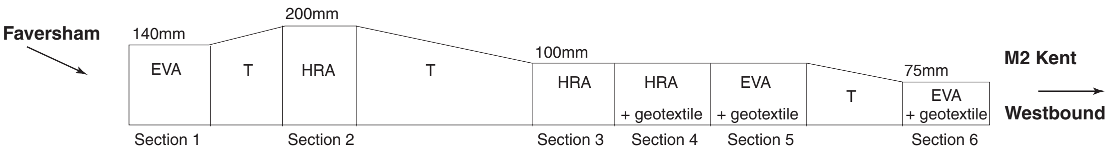


Figure 4.27 Layout of the M2 trial

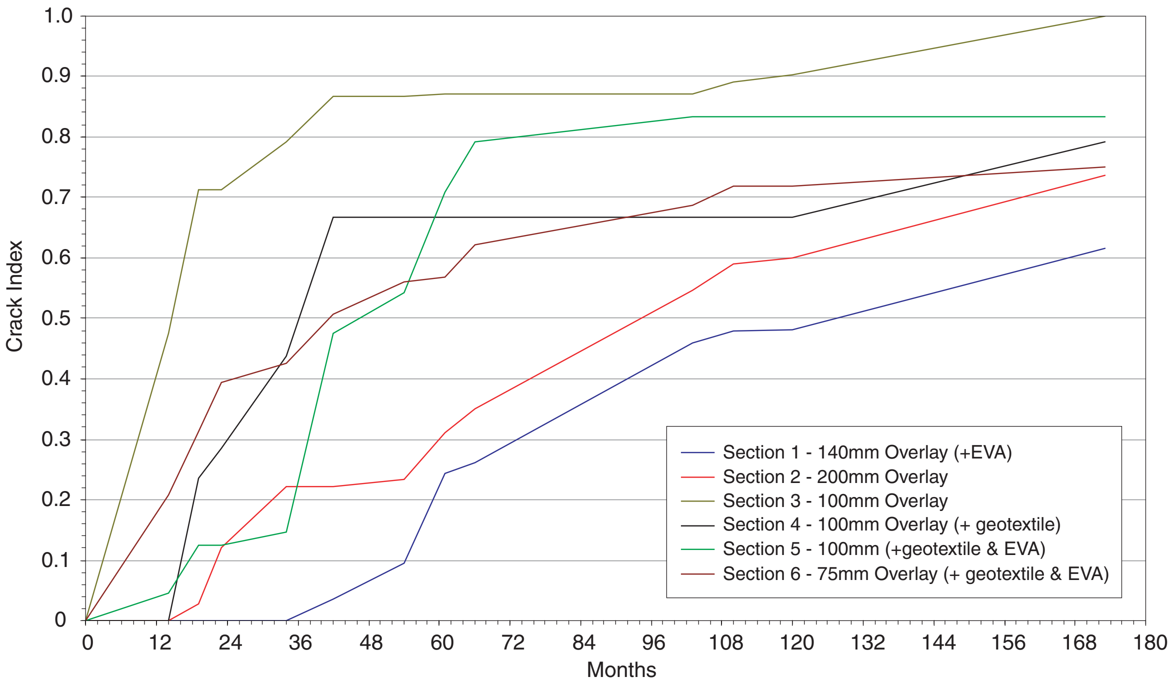


Figure 4.28 Crack Index graph

100 mm control showed a Cracking Index of 1.0, indicating full width wide reflections cracks above each of the joints below. Section 1, 140 mm with EVA, has given the best performance, although even this section has a CI > 0.6 after 173 months service. In the thinner overlay sections (75 mm to 100 mm), a number of trench repairs have been undertaken during its service life and some of these repairs have also failed, as shown in Figure 4.29.

Figure 4.29 Failed repair on the M2, Kent

4.2.4 TRL trial sites on CBGM

4.2.4.1 A30 Bodmin

The A30 at Bodmin is a two-lane dual carriageway, originally constructed from 100 mm thick HRA over 200 mm CBGM base. The trial included three different types of geotextile (needle punched, heatbonded nonwoven and woven) together with a section containing cracks treated with proprietary SAMIs. The geotextiles were laid directly onto the surface of the cracked flexible composite construction after the application of a bitumen emulsion spray. An overlay of 40 mm thickness of HRA was then applied. In a second trial area, the geotextiles were placed on top of a new 40 mm thick HRA binder course then overlaid with a 40 mm thick HRA surface course. The final section contained different treatments to reflective cracks. The trial site was completed in April 1990 with details of the trial section and the layout being shown in Figure 4.30.

The latest detailed visual survey was carried out in May 2003 and Figures 4.31 and 4.32 show the development of transverse cracking in lane 1 as a Crack Index value for the 40 mm and 80 mm thick treatments, respectively. The performance of the 40 mm sections indicated comparable performance between control and geotextile sections. The 80 mm sections show the sections containing geotextiles to be performing well, although the CI for the 80 mm control section is < 0.1 after 13 years service.

4.2.4.2 A30 Launceston

The Launceston Bypass on the A30 is a dual two-lane carriageway which was built in 1975/76. The existing construction consisted of a 100 mm thick asphalt surface on a 200 mm thick CBGM base. Transverse cracks started to appear in 1981 and attempts were made to seal the worst areas by overbanding. The whole site was surfaced dressed in 1982, but cracking started to reappear through the surface after a further two years.

A deflectograph survey, carried out in April 1986, indicated that some areas required total reconstruction and other areas required a 50 mm overlay in order to carry the anticipated traffic for a further 20 years. A visual condition survey carried out in autumn 1986 identified a very large number of transverse cracks and

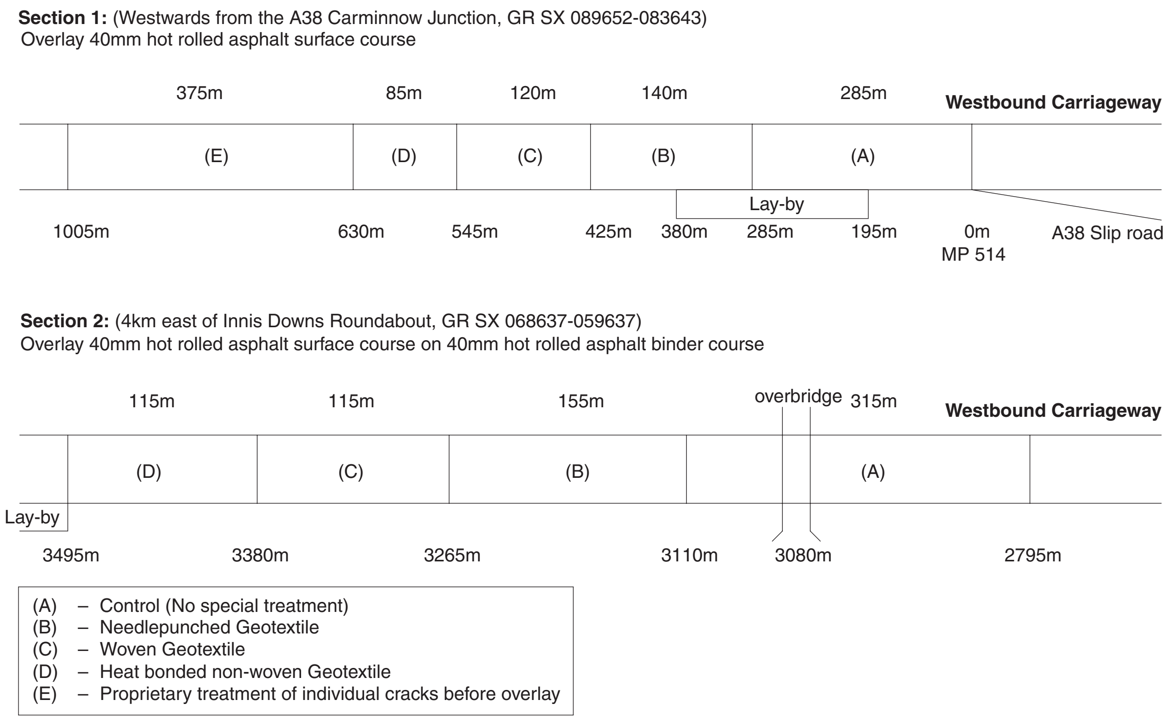


Figure 4.30 A30 Bodmin trial site

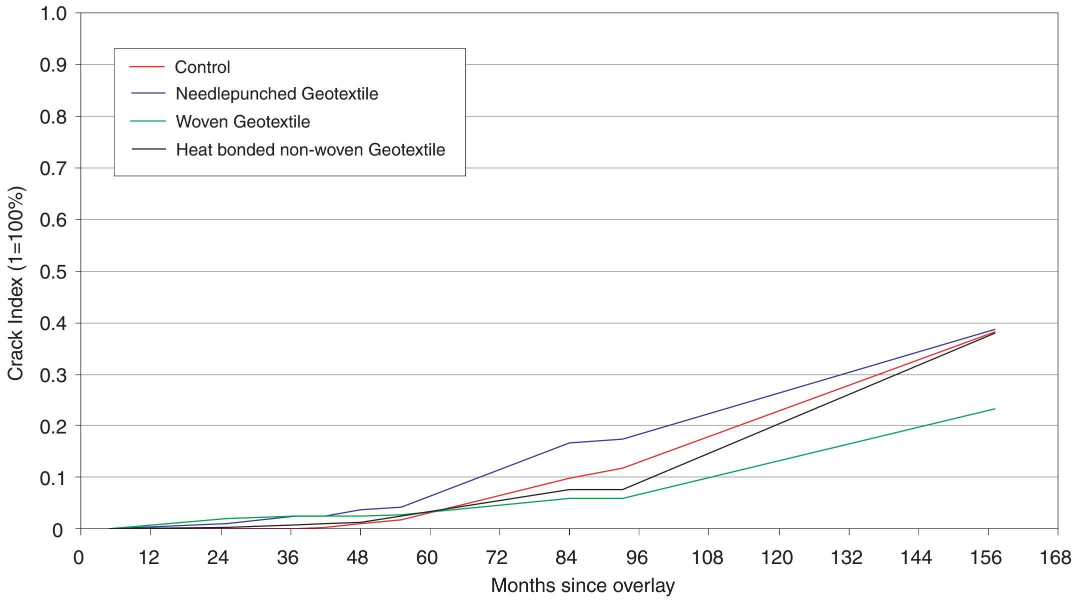


Figure 4.31 Development of reflection cracking, 40 mm overlay

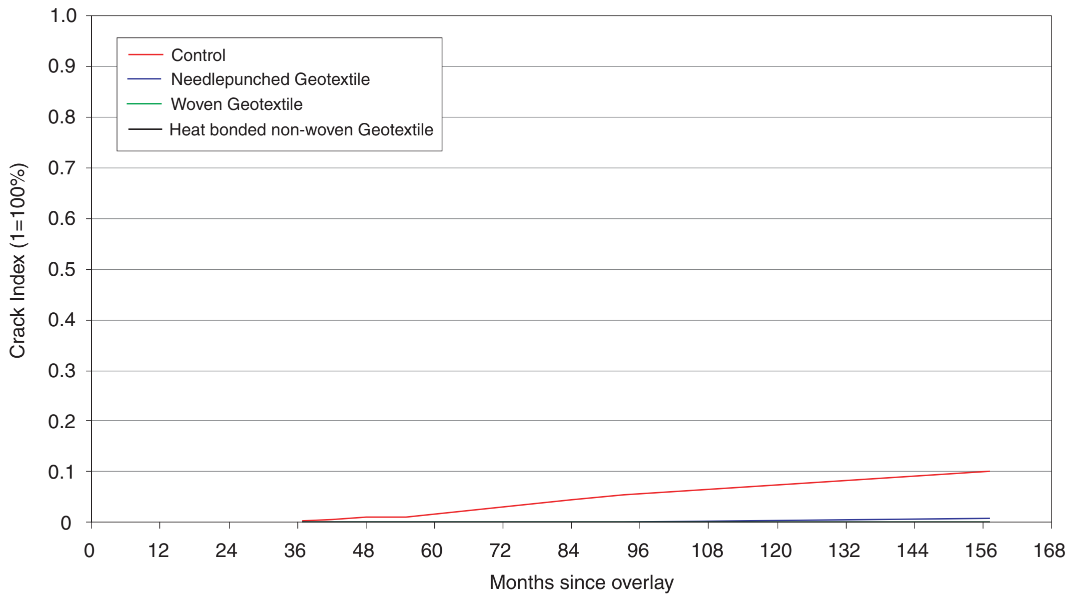


Figure 4.32 Development of reflection cracking, 40 mm overlay on new 40 mm base course

some longitudinal cracking. Cornwall County Council thought that a 50 mm overlay might not be sufficient to prevent further reflective cracking and they recommended that the thickness should be increased to 80 mm. The Eastbound carriageway was treated in 1987, with the incorporation of trials utilising a thinner 40 mm overlay in conjunction with trials of a bituminous coated polyester geogrid and a polypropylene geogrid.

Figure 4.33 shows the development of reflection cracking on the A30 at Launceston, with the performance of the geogrids being poor in relation to the control sections. At 113 months after construction, the polyester bituminous coated geogrid had a Crack Index of 0.81; the polypropylene geogrid had a Crack Index of 0.74 while the control had a Crack Index of 0.59. The trials incorporating a geogrid were shown to be ineffective in preventing reflection cracking compared with asphalt only controls.

The trial site was subsequently cracked and seated in May 2005.

4.2.5 Supplementary sites on URC

4.2.5.1 A12 Boreham

The A12 Boreham is a three-lane dual carriageway with a 20 year design life of 73 msa (in 1999). The nominal pavement construction consisted of 6 m long unreinforced concrete bays each approximately 265 mm thick on top of a 150 mm thick CBGM sub-base. Beneath this is 250 mm Type 1 sub-base on a 3 % CBR subgrade. Both carriageways between marker posts 130/6 and 134/4 were cracked and seated in January 2000.

The concrete was cracked at a spacing of 1.0 m, determined from an initial trial. The asphalt overlay consisted of a total thickness of 150 mm, comprising of 120 mm DBM50 and 30 mm TSCS.

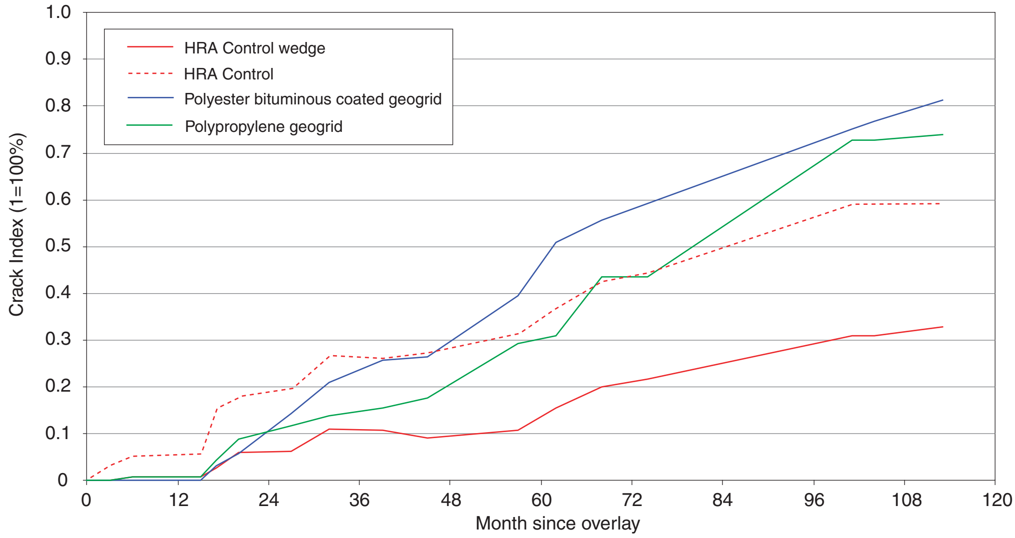


Figure 4.33 Development of reflection cracking

A coarse visual survey was carried out on both carriageways in September 2005 with only one possible crack being observed that was in lane 2 of the northbound carriageway at marker post 133/8 +40 m. Figure 4.34 shows the general condition of the A12 at Boreham while Figure 4.35 shows a close up of the surfacing.

Figure 4.34 General view of the A12 Boreham

Figure 4.35 Close-up of surfacing on the A12, Boreham

4.2.5.2 A12 Lowestoft

The A12 at Lowestoft scheme is located on the southbound section of the one-way system (Kirkley Cliff Road). The carriageway is 8.7 m wide and can accommodate parked cars on both sides and two lanes of traffic. The nominal pavement construction consists of jointed unreinforced concrete pavement, said to date from the 1940s, with an existing asphalt surface.

The works consisted of planing out the existing overlay and then relaying a total thickness of 50 mm overlay consisting of an HRA regulating layer and a TSCS. The saw-cut and seal technique was then applied to the new asphalt surface at 70 locations. It was observed that the spacing of the saw-cuts varied along the entire length of the site with a saw-cut possibly being missed at one location. The works were completed in March 2000.

Cracking was first observed in February 2004 (47 months after construction) and identified cracking either side of the saw-cuts, as shown in Figure 4.36.

A further detailed visual survey, undertaken in September 2005, showed the development of further new cracks and an increase in the severity rating of previously identified cracks. Table 4.6 summarises the cracking observed on the A12 Lowestoft. There was no information on the concrete or its condition prior to overlay made available to the authors, in particular on the condition of the ends of the concrete slabs below the saw cuts.

From the most recent survey, around 50 % of the joints showed signs of cracking and, at one location, rocking of the underlying slab under traffic could be felt. There was some doubt about the accuracy of the SCS locations given the irregular spacing along the site. In addition, given the age of the concrete it could be that some of the joints are in a poor condition leading to reflection cracking over a wider area, as shown in Figure 4.36. However, further investigation would be needed to confirm this hypothesis.

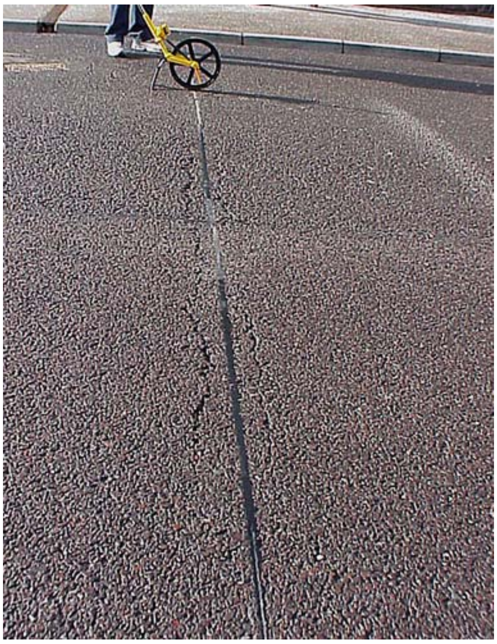


Figure 4.36 Typical cracking observed on the A12, Lowestoft

Table 4.6 Summary of cracking on the A12 Lowestoft

Date of survey	*Months after cons-truction*	*Number of full width cracks*	*Average crack length (m)*	*Number in severity category*			*Crack Index*
				A	*B*	*C*	
July 2000	4	0	0	0	0	0	0.000
February 2004	47	0	1.48	2	6	6	0.024
September 2005	66	1	1.98	10	21	4	0.094

4.2.5.3 A12 Mountnessing

The A12 Mountnessing is a three-lane dual carriageway with a nominal pavement construction of 5 m long unreinforced concrete slabs each approximately 165 mm thick on 140 mm thick sub-base. During the works, 25 m long reinforced concrete slabs were encountered either side of the culvert and subway, which were also cracked and seated. This treatment was performed prior to the development of SCCSO, in which the reinforcement in the slab is severed prior to cracking.

Both carriageways were treated for 2.5 km in length with CSO in February 1999. The overlay consisted of an HDM base with a TSCS laid to a total thickness of 150 mm.

A coarse visual survey was carried out on both carriageways September 2005 (79 months after construction). One crack was observed on the northbound carriageway at marker post 110/3 +50 m. On the southbound carriageway, several transverse and longitudinal patches were observed in lane 1 between marker posts 110/2 to 110/0. Figure 4.37 shows a general view of the A12 Mountnessing northbound carriageway and Figure 4.38 shows typical transverse patches observed on the southbound carriageway.

Figure 4.37 General view of the A12, Mountnessing, northbound

Figure 4.38 Transverse repairs observed on the A12, Mountnessing, southbound

4.2.5.4 A12 Stanway

The A12 Stanway scheme is part three-lane and part four-lane dual carriageway, roughly 2 km in length and including the Eight Ash Green and Lexden Interchanges on both carriageways. The nominal construction consisted of 6 m long unreinforced slabs with an average thickness of 400 mm and 350 mm (northbound and southbound, respectively) on 250 mm thick sub-base. The site contained a number of reinforced joint and bay replacements.

Both carriageways were treated with CSO and SCCSO in April 2000. The overlay consisted of a DBM50 base with a TSCS laid to a total depth of 150 mm.

A coarse visual survey was conducted on both carriageways on the September 2005 (66 months after construction). Figure 4.39 shows a general view of the A12 Stanway southbound carriageway. Four cracks were observed between marker posts 158/2 and 158/4, two short ‘b’ severity cracks in lane 1 and two full width lane 2 cracks (severity ‘b’) between saw-cut and seal treatments in lanes 1 and 3. Figure 4.40 shows the saw-cut and seal treatment to lanes 1 and 3 while lane 2 joint has reflected through.

Figure 4.39 General view of the A12, Stanway, southbound

Figure 4.40 Saw-cut and seal lane 1 and 3, transverse crack lane 2

4.2.5.5 A14 Alconbury to Spittals

The A14 Alconbury to Spittals Interchange is a two-lane dual carriageway and consisted of works to both lanes of both carriageways. The existing pavement construction consisted of 5 m long by 220 mm thick unreinforced concrete slabs on top of 225 mm Type 2 sub-base on a clay subgrade. The works consisted of CSO with a crack spacing of 1.25 m. The scheme had a 20 year design life of 135 msa and was overlaid with 150 mm HDM base and a 35 mm TSCS. The required stiffness threshold for the concrete after crack and seat was 9 GPa.

A detailed visual survey was conducted on the northbound section of the A14 Spittals Interchange between marker posts 115/1 and 116/3. No defects were seen during the survey carried out in December 2004 (after 58 months service).

4.2.5.6 A259 Pevensey

The A259 is a single carriageway trunk road that forms a south coast link between Eastbourne in East Sussex and the channel ports of Folkestone and Dover. The annual average daily traffic (AADT) based on a 2002 traffic count was 19,200 vehicles with 6 % HGVs. The existing pavement construction consisted of 130 mm asphalt surface on 200 mm thick PQ concrete slabs, 5 m in length, laid on 200 mm CBGM base. The works carried out in September to October 2003 consisted of the laying of three different sections of 15-18 mm thick TSCS using 6 mm aggregate, with a total length of 680 m.

A coarse visual survey was carried out in June 2004 (8 months after construction) from the roadside. From the survey it was observed that seven cracks had developed along the entire length of the site and material loss in the wheelpaths were observed in one section. A subsequent survey carried out in September 2004 (11 months after construction) identified seven new cracks had developed, bringing the total number of cracks up to 14. As cracking had developed within 12 months on the treatments applied, and the total asphalt thickness is greater than 130 mm, it is likely that some cracking was present in the existing surfacing prior to overlay which has subsequently led to the rapid onset of reflection in the new surfacings.

A further survey, carried out in September 2005 (23 months after construction), identified further cracking, with several cracks now full width and high severity. The majority of one trial Section had been inlaid, equivalent to 27% of the total length of the site. A general view of the site is shown in Figure 4.41, while Figure 4.42 shows an example of the reflection cracking in 2005.

4.2.5.7 A46 Kenilworth bypass

The A46 Kenilworth is a three-lane dual carriageway with a 1.0 m edge strip in Warwickshire that forms part of the Kenilworth bypass. This section of the A46 was constructed in 1973 with lanes 2 and 3 being laid in one pass whilst the longitudinal lane joint was formed afterwards and a wide lane 1 was laid in a separate pass. The construction consisted of 5 m unreinforced concrete bays, with an average thickness of 200 mm.

Figure 4.41 General view of site

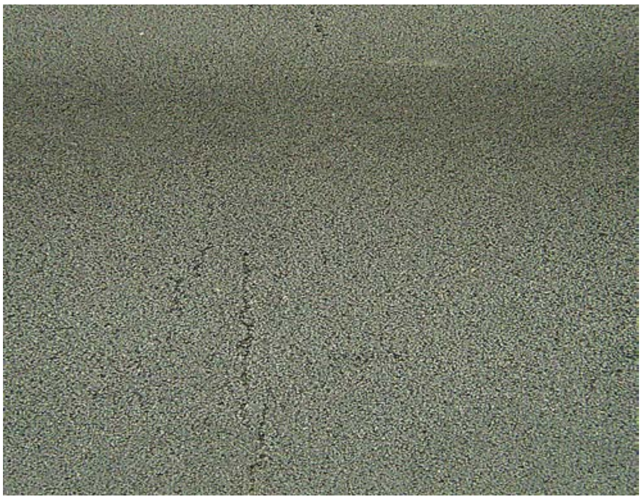


Figure 4.42 Reflection cracking in TAC 1

A visual survey was carried out on the A46 prior to crack and seat and showed the northbound carriageway to be in a very good condition, with only occasional spall repairs and longitudinal cracking in the near side wheel path (nswp) extending over two adjacent slabs. The southbound carriageway contained two full depth asphalt bay replacements in lanes 1 and 2. Both the northbound and southbound carriageways north of the University junction showed a wide separation of the L1/L2 joint with stepping. All transverse and longitudinal joints were cleaned and a new type N2 (BS2499-1:1993) sealant was applied prior to the works.

The works consisted of CSO of the concrete on all lanes of both carriageways over a length of 4 km. The overlay consisted of a total thickness of 170 mm, and comprised of HDM binder course with a thin surface course. The projected future traffic for a 20-year design life was 80 msa. Figure 4.43 shows a general view of the concrete on the A46 after crack and seat.

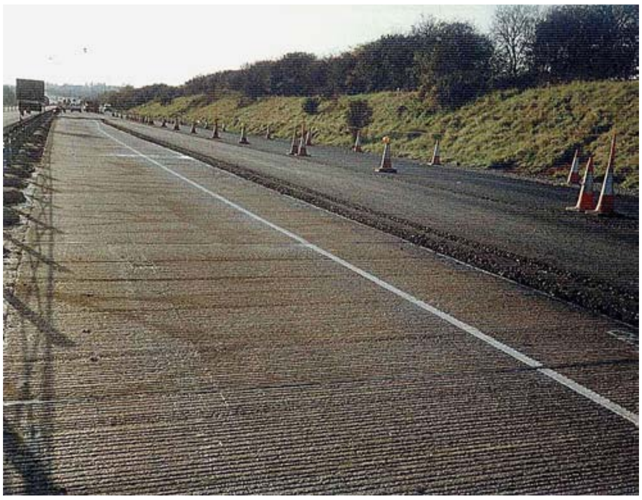


Figure 4.43 General view of the concrete after crack and seat

A coarse visual survey was carried out in May 2004 along the complete length of both carriageways. It was observed that occasional longitudinal cracking was seen on the southbound carriageway 0.3 m away from the lane joint between marker posts 91/3 and 90/5, as shown in Figure 4.44. From the original pre-crack and seat visual survey, it was noted that this location contained wide separation and stepping of the lane 1/lane 2 joint. Visual, coring and ground penetrating radar (GPR) surveys under a lane closure were undertaken in February 2005 to investigate this cracking further. The GPR in use is shown in Figure 4.45.

Figure 4.44 Longitudinal cracking on the A46 marker post 91/2

Figure 4.45 Survey of the longitudinal crack, A46

The core logs showed cracking to be the full depth of the asphalt layers and directly above the stepped longitudinal lane joint. Figure 4.46 shows the separation and stepping of the original concrete lane joint. This cracking would indicate that the crack and seat method may not be suitable for sites containing wide separation and stepping of the joints without additional treatment to the slabs.

The GPR results showed the majority of the cracks to be the full depth of the asphalt layer and tied in with the core results.

Figure 4.46 Coring of the longitudinal crack, showing stepping of the concrete below

4.2.5.8 M1 Junction 2 to Deansbrook

The M1 Junction 2 to Deansbrook viaduct is a three-lane dual carriageway with hardshoulder. The original construction consisted of unreinforced concrete slabs and reinforced concrete slabs (discussed in Section 4.2.6.6), constructed in 1975. The nominal thickness of the unreinforced concrete was 295 mm thick and 5 m in length, on top of a gravel sub-base.

The trial comprised of CSO to the unreinforced sections, with crack spacings at 1.0 m and a total overlay of 150 mm. Figure 4.47 shows the scheme layout.

The scheme was designed for a 40 year life with cumulative traffic of 230 msa for the northbound carriageway and 270 msa for the southbound. The overlay consisted of a DBM50 binder course with 30 mm TSCS. The scheme was completed and opened to traffic in January 2000.

A coarse visual survey was carried out in July 2005 (66 months after construction) and no defects were observed within the CSO trial section.

4.2.5.9 M27 Junctions 8 to 10

The M27 Junctions 8 to 10 is a three-lane dual carriageway with hardshoulder, roughly 8 km in length. The original construction consisted of 250 mm thick unreinforced concrete bays, 5 m in length, on 200 mm thick Type 1 sub-base.

The majority of the pavement construction was rigid, although small lengths of hard shoulder near the on and off slips were in flexible composite construction. A large number of joints and slabs had been replaced in reinforced concrete as part of a previous maintenance contract.

The works consisted of CSO to the unreinforced concrete with a crack spacing of 1.0 m. SCS was initially applied to the overlay at the reinforced repairs but, after the first phase, it was decided to use SCCSO instead due to the number of reinforced repairs. The saw cutting for the crack initiation slot and the sealant reservoir was performed in two operations, with the wider 20 mm wide sealant reservoir slot cut first. The finished slot was then water jetted, and blown out and dried with a ‘hotdog’ air lance prior to the application of bond breaker tape and pouring of the sealant. The hot-applied joint sealant was indirectly heated in a Breining vessel fitted with an automatic horizontal stirring mechanism and thermostatic control. The sealant was poured directly from the heating vessel into the prepared slot via a re-circulating pump. No problems were reported with the installation of the sealant at this site. The SCS section is located around marker post 31/4 on the eastbound carriageway. A total overlay thickness of 150 mm was used, consisting of 120 mm HDM binder course with a 30 mm thick TSCS. The process of slot cutting and cleaning and drying with the ‘hotdog’ lance are shown in Figures 4.48 and 4.49, respectively.

A coarse visual survey conducted from the hard shoulder was carried out in October 2004, no defects were observed and all the SCS were in good condition as shown in Figure 4.50.

4.2.5.10 M40 J6-7

The M40 Junctions 6 to 7 is a three-lane dual carriageway with hardshoulder. The works were carried out as part of the initial maintenance work for the design,

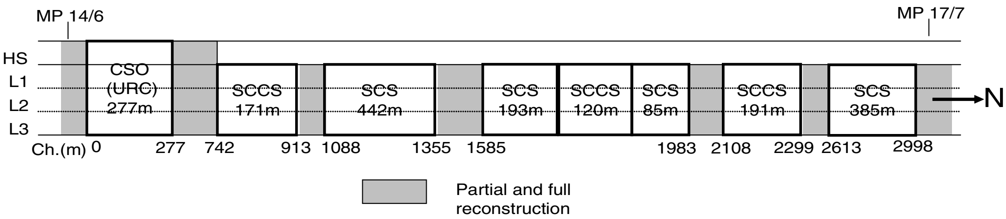


Figure 4.47 Layout of the M1 Deansbrook site, northbound

Figure 4.48 Cutting of crack initiation slot on M27, Fareham

Figure 4.49 'Hotdog' lance used for cleaning and drying of cut slots on M27, Fareham

build, finance and operate (DBFO) M40 contract. The original pavement construction consisted of 250-280 mm thick unreinforced concrete slabs, 5.0 m in length, on a 150 mm Type 1 sub-base.

The trial was carried out in early 1997, using three types of crack and seat equipment:

- Arrow Hammer.
- Badger Breaker.
- Whip Hammer.

The trials were carried out in the hardshoulder of the carriageway prior to the main works. The contractor reported that the Arrow Hammer (whiphammer) caused a certain amount of surface spalling. The Badger Breaker (guillotine) was considered to provide the highest output whilst producing a satisfactory crack pattern and was, therefore, selected for the main works.

The total overlay was 140 mm, consisting of 110 mm thick stone mastic asphalt (SMA) binder course with a 30 mm thick TSCS. The SMA binder course consisted of 20 mm aggregate and was laid in two lifts.

A coarse visual survey was carried out in May 2006 (over nine years after construction) and no defects were observed.

4.2.5.11 M5 Willand

The M5 Willand is a three-lane motorway with hardshoulder, located between Junctions 27 and 28. This section was originally constructed in 1976 and carries around 45,000 vehicles per day. Due to headroom restrictions from three overbridges, it was considered that crack and seat would not be practical so, with the approval of the Highways Agency, a trial was conducted with the use of a continental SAMI overlay system. The overlay system is a two-layer system consisting of a sprayed polymer modified membrane (2.5 to 3 mm thick) which is overlaid with a nominally 10 mm thick microsurfacing which is then overlaid with hot-mix binder and surface course layers as required. Figure 4.51 shows the application of the spray applied membrane and Figure 4.52 shows the microsurfacing layer being applied.

On the M5, the overlay consisted of 3 mm membrane, 10 mm microsurfacing, 30 mm SMA binder course and 25 mm TSCS. The trial was completed in November 2003. Table 4.7 shows trial section construction and lengths.

Figure 4.50 Condition of the saw-cut and seal treatment on the M27, eastbound

Figure 4.51 Laying of the spray applied membrane

Figure 4.52 Laying of 10 mm thick microsurfacing layer

Table 4.7 Layout of the M5 Willand trial section

ID	*Extent*	*Length (m)*	*Treatment*	*Thickness (mm)*
1	Marker posts 232/840-232/740	100	Surface course/binder course/SAMI	80
2	Marker posts 232/740-232/640	100	Surface course/binder course	75
3	Marker posts 232/640-232/600	40	Surface course/binder course ramp	75-35
4	Marker posts 232/600-232/500	100	Surface course	35
5	Marker posts 232/500-232/460	40	Surface course/binder course ramp	35-80

A detailed visual survey was undertaken in June 2005 (19 months after construction) and showed a number of new cracks in Sections 1, 3, 4 and 5. No cracking was observed in Section 2. Cracking was first observed on the M5 section in October 2004 (11 months after construction). Tables 4.8 and 4.9 shows the Crack Index values and proportion of joint reflected.

A coarse visual survey was carried out on a 3 km stretch of the southbound M5 Willand. Several cracks were observed with an average of 2 cracks present every 100 m. The majority of the cracks present were twinned and were similar in appearance to the cracking present with overlays less than 50 mm. Figure 4.53 shows typical twinned cracking observed on the southbound carriageway.

The amount and severity of cracking is still reasonably minimal and not yet at a stage where intervention would be considered necessary. Where cracking has occurred in the surfacing above the SAMI sections, it would be worth coring to establish whether the SAMI layer is still intact and providing a seal to the pavement layers below.

4.2.6 Supplementary sites on JRC

4.2.6.1 A12 Brentwood I and II

The A12 Brentwood bypass is a busy two-lane dual carriageway, carrying 74,000 vehicles per day with 12 % HGVs. The road was built in 1966 using 25 m long reinforced concrete bays. The nominal pavement

Table 4.8 Crack Index values

	Months since construction		
Section	*0*	*11*	*19*
1	0.00	0.00	0.01
2	0.00	0.00	0.00
4	0.00	0.06	0.07

Table 4.9 Proportion (%) of joints reflected

	Months since construction		
Section	*0*	*11*	*19*
1	0.0	0.0	5.6
2	0.0	0.0	0.0
4	0.0	12.5	17.7

Figure 4.53 Twinned cracking observed on the M5, southbound

construction consisted of PQ reinforced concrete laid to a thickness of 260 mm on a 150 mm CBGM base. In cuttings, a variable thickness of granular sub-base was also included below the CBGM. The subgrade consisted of silty clay.

Both carriageways, 3 km in length, were treated with SCCSO in early 2001 with 1.0 m crack spacings. The overlay consisted of a DBM50 base with a TSCS laid to a total depth of 150 mm. Figure 4.54 shows the A12 Brentwood works in progress (SCCSO) in 2001.

A coarse visual survey was conducted on the northbound carriageway between marker posts 100/3 and 106/8 in September 2005 (56 months after construction of Brentwood I and 34 months after construction of Brentwood II) when no cracking was observed. Figure 4.55 shows a general view of the A12 Brentwood, northbound carriageway.

Figure 4.54 Saw-cut crack and seat of the A12, Brentwood

Figure 4.55 General view of the A12, Brentwood, northbound

4.2.6.2 A12 Hatfield Peveral

The A12 Hatfield Peveral bypass is a two-lane dual carriageway, carrying 57,000 vehicles per day with 15 % HGVs. The road was built in 1966 using 25 m long reinforced concrete bays, 7.3 m wide. The nominal construction consisted of PQ reinforced concrete laid to a thickness of 260 mm on a 120 mm CBGM base. The CBR values of the subgrade, as measured by DCP, were around 2 %.

Both carriageways, 1.5 km in length, were treated with SCCSO in January 2002 with 1.0 m crack spacings. The overlay consisted of an HDM base with a TSCS laid to a total depth of 160 mm.

A coarse visual survey was conducted on both carriageways between marker posts 134/5 and 136/0 in September 2005 (44 months after construction). No cracking was observed, only a patch repair visible in lane 1 towards the end of the site on the northbound carriageway. Figure 4.56 shows a general view and condition of the A12 Hatfield Peveral.

Figure 4.56 General view of the A12 Hatfield Peveral, northbound

4.2.6.3 A167 Durham

The A167, formerly the A1 trunk road, is a two-lane dual carriageway consisting of 300 mm thick jointed reinforced concrete, overlaid by a number of courses of HRA with a total thickness of approximately 140 mm. The pavement was suffering from rutting and reflection cracking. Maintenance work on the northern carriageway was started in 1996 and consisted of removing the old cracked asphalt, installation of a steel grid, and then overlaying with HRA to a nominal thickness of 140 mm. A control section without a steel grid was also included.

A detailed visual survey was carried out on the A167 Durham in September 2003 and no cracking or defects were observed in either treatment. A further survey in February 2006 showed cracking in both the control and steel grid sections of severity ‘a’ and ‘b’, with no discernable difference noted in their relative performance. Examples of the cracking observed in the control and steel grid sections in February 2006 are given in Figure 4.57.

4.2.6.4 A46 Cossington to Six Hills

The A46 Cossington to Six Hills is a two-lane dual carriageway with a 20 year design life of 37 msa (in 2002). The original pavement construction consisted of 150 mm thick asphalt surfacing on JRC and 180 mm thick asphalt surfacing on a CBGM base (described in Section 4.2.7.4).

The JRC was overlaid with 150 mm asphalt with the SCS technique applied above the joints in the concrete. The technique was applied to the southbound carriageway only, and carried out in July 2002.

A coarse visual survey was conducted in February 2005 along the complete length of the scheme and no defects were seen and all SCS treatments were in good condition. Figure 4.58 shows typical condition of the saw-cut observed.

4.2.6.5 B1441 Weeley

The B1441 at Weeley is a single two-lane carriageway constructed in the 1930s. The original pavement construction consisted of 100 mm asphalt surfacing on 300 mm thick jointed reinforced concrete slabs on a clay foundation. Over recent years, the pavement has suffered due to extremes of moisture within the clay causing the clay to heave and shrink.

Figure 4.57 Cracking in control (left) and steel grid (right) sections on the A167 Durham

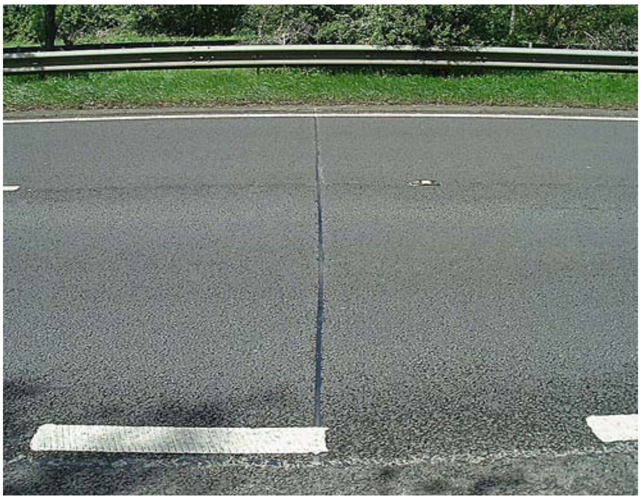


Figure 4.58 Typical saw-cut observed on the A46, Cossington

Figure 4.59 General view of the B1441, Weeley

The proposed maintenance consisted of rubblisation of the concrete base turning it into a strong foundation and then overlaying with 160 mm DBM base and 40 mm surface course system. The rubblisation technique was carried out in October 2004.

Falling weight Deflectometer (FWD) measurements were made prior to and after rubblisation, Table 4.10 shows the average and 15th percentile measurements (conservative value) for both northbound and southbound.

Table 4.10 Average and 15th percentile surface modulus (MPa) prior to and after rubblisation

	15th Percentile (MPa)		*Average (MPa)*	
	Prior to rubblisation	*After rubblisation*	*Prior to rubblisation*	*After rubblisation*
Northbound	328	97	684	165
Southbound	404	148	915	282

The northbound carriageway is in worse condition than the southbound both prior to and after rubblisation. The target surface modulus measurement value for an Unbound Class 2 foundation is 80 MPa (design 100 MPa), Interim Advice Note (IAN) 73/06, when compared with the minimum 15th percentile measured value of 97 MPa shows the rubblised concrete to be a good Class 2 foundation.

A detailed visual survey was carried out along the entire length of the site in September 2005 (11 months after construction) and no cracking or defects were observed. Figure 4.59 shows a general view of the site.

4.2.6.6 M1 Junction 2 to Deansbrook

The M1 Junction 2 to Deansbrook viaduct is a three-lane dual carriageway with hardshoulder. The original construction consisted of reinforced concrete. The reinforced concrete slabs (constructed in 1963) were nominally 285 mm thick, 12.2 m in length over a 140 mm thick CBGM base.

The trial comprised of SCCSO to the reinforced section, with saw-cuts at 1.5 m spacings and a total overlay of 200 mm. SCS was applied to the asphalt overlay in the reinforced sections where joints were in good condition. The scheme was designed for a 40 year life with traffic being 230 msa for the northbound carriageway and 270 msa for the southbound. The overlay consisted of a DBM50 binder course with 30 mm thickness of surface course. The scheme was completed and opened to traffic in January 2000. Figure 4.60 shows the scheme layout.

A coarse visual survey was carried out in July 2005 (66 months after construction) and no defects were observed within the SCCSO and SCS trial sections.

4.2.7 Supplementary sites on CBGM

4.2.7.1 A10 Ely

The A10 Ely is a single carriageway trunk road in Cambridgeshire forming part of the Ely ring road. This section of the A10 was built roughly 30 years ago using flexible composite construction and consisted of 100 mm of asphalt over 200 mm of CBGM.

Both lanes of a 700 m length of road were treated with CSO in September 2000. The concrete was cracked at a spacing of 1.5 m, determined from an initial trial. The overlay consisted of a total thickness of 120 mm of asphalt comprising of a DBM50 base with a TSCS. The asphalt thickness was limited by existing finished levels.

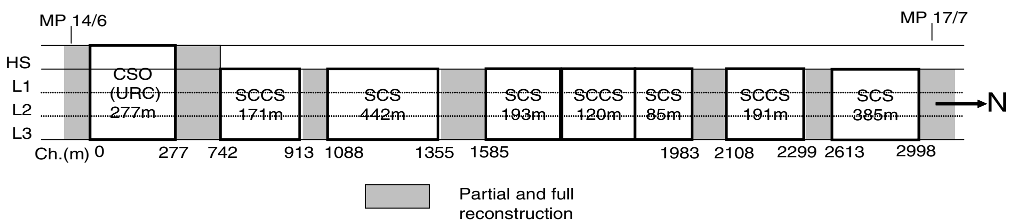


Figure 4.60 Layout of the M1 Deansbrook site, northbound

A coarse visual survey was carried out in September 2005 (60 months after construction) and identified many wide full width cracks. Comparison with the existing survey on the CBGM base identified the cracking observed in the asphalt overlay was at the same locations where wide existing transverse cracks were observed in the CBGM. Figure 4.61 shows typical cracking observed on the A10 at these locations. In this instance, it would appear that the limit on the overlay thickness, imposed by the need to keep existing finished levels, was inadequate in preventing reflection cracking.

Figure 4.61 Cracking observed on the A10, Ely

4.2.7.2 A14 Milton to Fen Ditton

The A14 Milton to Fen Ditton is two-lane dual carriageway and the scheme consisted of works to both lanes of the eastbound carriageway of the Cambridge northern by-pass. The existing pavement was of flexible composite construction and the works, undertaken in August 1998, consisted of planing off the existing overlay and CSO of the existing lean concrete using 2 m crack spacing. The overlay consisted of 30 mm thickness of TSCS on 190 mm thickness of HMB15 base.

Prior to crack and seat, regular transverse cracking and lengths of longitudinal cracking were observed in the existing overlay. Figure 4.62 shows longitudinal cracking present in the existing asphalt surface.

Following planing, the existing lean concrete base was found to contain substantial longitudinal cracking across the width of the carriageway. This longitudinal cracking was believed to be representative of shrinkage cracks in the embankment and that the cracking present suggested that the pavement was well drained and that no moisture had been allowed to enter the embankment, thus very dry gault clay was present which had shrunk with time. TRL recommended that a ‘quick setting’ fluid grout be poured into the longitudinal cracks and an increased load used for

Figure 4.62 Longitudinal cracking present in the existing surface prior to crack and seat

the seating process of the lean concrete base. Figure 4.63 and 4.64 shows the severity of the existing longitudinal cracking in the lean concrete base.

A detailed visual survey carried out in September 2000 that revealed no signs of distress over the 830 m length of the scheme between the A10 on-slip at Milton and the B1047 off-slip at Fen Ditton. However, a third party observed transverse cracking in the TSCS in January 2001, 30 months after construction. As a result of this observation, a detailed visual survey was carried out in March 2002. The survey revealed six transverse cracks, most being relatively short, and some longitudinal cracking of over 1.0 m in length.

A coarse visual survey was carried out in November 2003 that revealed 15 new transverse cracks, with the majority being medium ‘b’ severity, and long lengths of longitudinal cracking, with the majority being high severity and in excess of 10 m in length. A further detail visual survey was carried out in September 2004 and revealed the existing transverse cracking had increased in both length and severity, with the majority being observed as high, ‘a’ severity and several new longitudinal cracks

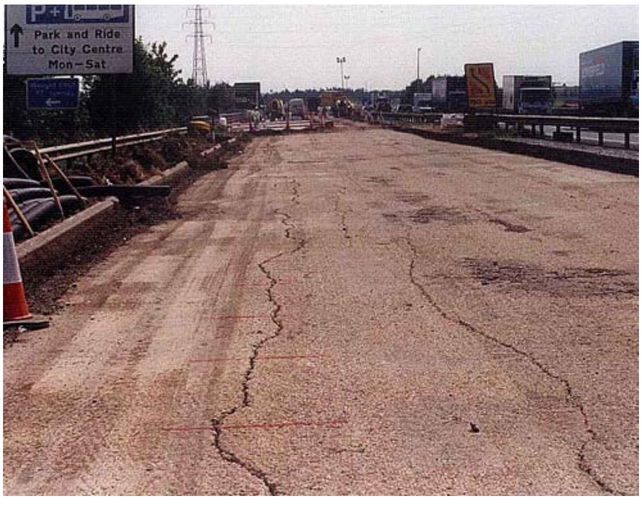


Figure 4.63 Longitudinal cracking present in the existing CBGM base

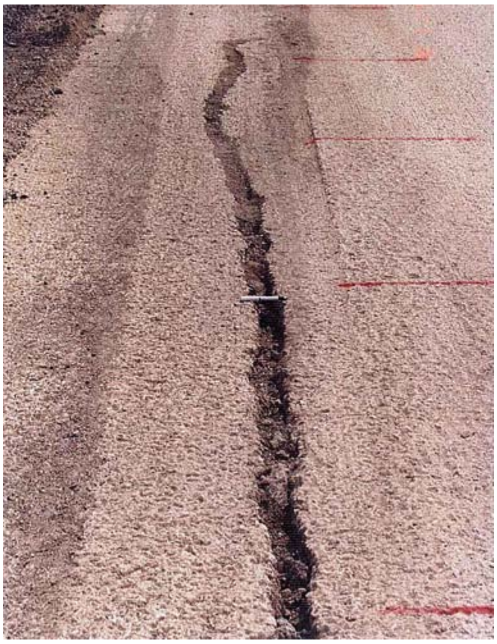


Figure 4.64 Longitudinal cracking present in the existing CBGM base

also being observed. Patch repairs had been undertaken on several existing cracks.

Problems occurring with HMB 15 have been well documented with early failures occurring as a result of water ingress and subsequent stripping of the binder from the aggregate (Sanders and Nunn, 2005). These problems would explain the early and rapid onset of reflection cracking at this site.

A detailed visual survey of the lean concrete base was not carried out prior to overlay and, therefore, it was not possible to ascertain whether the cracks observed in the 220 mm thick asphalt overlay are present over natural cracks or induced cracks in the underlying lean concrete.

Reflection cracking has not been observed at any other sites treated by CSO after two years in service; even those with a total overlay thickness of 150 mm or less.

4.2.7.3 A38 Swinfen to Weeford

The A38 Swinfen in Staffordshire is a two-lane dual carriageway with a 20 year design life of 44 msa. The existing pavement construction consisted of between 175 and 180 mm asphalt with a 175 mm CBGM base on a 10 % subgrade CBR.

Both carriageways were cracked and seated in June 2000, and overlaid with 180 mm combined HDM base and TSCS.

A detailed visual survey was carried out in October 2004 (52 months after construction) and no cracking was observed. However, several locations of fatting up of the asphalt were observed and slight chip loss in the lane 1 wheelpaths. Figure 4.65 show a general view of the A38, northbound.

4.2.7.4 A46 Cossington to Six Hills

The A46 Cossington to Six Hills is a two-lane dual carriageway with a 20 year design life of 37 msa. The original pavement construction consisted of 180 mm thick asphalt surfacing on 180 mm thick CBGM base.

Figure 4.65 General view of the A38, northbound

The techniques were applied to the southbound carriageway only, and carried out in July 2002. All the exiting asphalt was removed, the CBGM layer was cracked and seated and overlaid with 180 mm of new asphalt.

A coarse visual survey was conducted in February 2005 along the complete length of the scheme and no defects were seen.

4.2.8 Supplementary sites on ECOPAVE

4.2.8.1 A30 Plusha

The A30 at Plusha is a dual two-lane carriageway. The site was originally an ECOPAVE trial site. ECOPAVE is a system of laying concrete through a modified bituminous paver, inducing transverse cracks into the concrete and overlaying with a thin bituminous surfacing. The system was developed through the BRITE (Basic Research in Industrial Technologies in Europe) project with partners in the UK and Denmark after extensive laboratory testing, feasibility trials and a full-scale trial on the trunk road network. Originally, the ECOPAVE was induced cracked, using a variety of techniques to prevent reflection cracking and different spacings.

The A30 trial pavement consisted of four different concrete mixtures, induced crack spacings of 1, 2, 3 and 5 m and various methods of inducing the cracks, all using the Whiphammer. In addition, over 500 m was allowed to crack naturally in order to determine if sufficient microcracking could be developed to eliminate the discrete transverse cracks which normally form in this type of construction.

The control sections, and a section where induced cracks at 3 m spacing or greater resulted in reflection cracks, have been maintained using CSO with 1 m crack spacing.

The A30 was cracked and seated in two phases in 1997 and 1998. The first phase consisted of a 70 mm overlay and was applied to a 200 m length of ECOPAVE with natural cracking. The second phase consisted of a nominal 150 mm overlay and was applied to 430 m section of ECOPAVE with induced cracks and a 310 m section of ECOPAVE with natural cracking. All sections were surfaced with an HRA surface course.

A coarse visual survey was carried out on the A30 Plusha in May 2005 and no cracking was observed. However, lengths of chip loss were observed and a pothole (25 m past marker post 767/15) was noted in the offside wheelpath in lane 2.

4.3 FWD measurements

4.3.1 Surveys

The FWD was used to measure the load transfer across the joints and structural integrity of the slabs in accordance with HD 29/94 (DMRB 7.3.2). These measurements have been undertaken on an annual basis on the M5 Taunton 1992 trial site, A14 Quy and M1 Barnet through to 2002 and on the M2 Kent up to 1995. Single surveys were also carried out on the A30 Plusha, A1 Tuxford, A38 Swinfen and the A14 Spittals to Alconbury.

For Quy and Taunton, the loading plate for the FWD was positioned close to the centre of the underlying slabs so that the geophones were located within the boundary of the underlying slab being tested. For other sites, measurements were made at 5 m centres throughout the sections. No temperature corrections have been applied to the deflections at any site due to the main structural element of the pavement being concrete, for which there are currently no corrections that can be applied.

4.3.2 Deflection measurements

The structural integrity at the mid-slab locations was assessed by:

i comparison of profiles of the central deflection (d_1);

ii difference between the central deflection and deflection at 900 mm from the loading plate (d_1-d_4); and

iii deflection at 1800 mm from the loading plate (either d_6 or d_7), depending upon the FWD geophone spacing. The measurements were made so that any differences could be determined on an annual basis.

All measurements were made at a target loading plate pressure of 700 kPa, with all deflections being normalised to that value.

The d_1 profile indicates the stiffness of the pavement as a whole, while increases in the d_1-d_4 deflection profiles provide an early indication of structural deterioration within the pavement layers. An increase in the deflection profiles at 1800 mm (d_6 or d_7 depending upon geophone configuration) from the plate is an indication of a problem developing within the foundation.

A substantial amount of data has been collected from the M2 Kent, M5 Taunton, A14 Quy and M1 Barnet trial sites. The variation in deflection profile with time initially suggested some variation in the bound layers. However, further analysis showed that higher deflections were measured during warmer weather with variations of up to 20 ^oC between surveys, indicating that there had been little variation in structural condition over the monitoring period.

The results from the M5 Taunton showed consistent deflection profiles for the first 10 years of monitoring, and then an increase in the d_1-d_4 deflection for one section (Section E, 0.5 m C&S with 150 mm overlay) where longitudinal cracking had developed, indicating a weakening of the bound layer.

The surveys at the A14 Quy have confirmed the findings from the M5 Taunton, showing that whilst the C&S treatments reduced the overall stiffness of the bound layers when compared with the control sections, this reduction did not lead to any deficiencies in the performance, with the exception of one section where longitudinal cracking had developed after 10 years service. Therefore, in the medium term, C&S of jointed concrete pavements does not lead to progressive structural weakening.

Some sections on the A14 Quy showed higher initial central deflections and deflections at 1800 mm from the loading plate compared with other sections along the site, implying that the initial condition of the subgrade was worse than in other sections. However, with time the deflection levels have reduced substantially to the levels found in other test sections, which indicated that the drainage works carried out at the time of overlay improved the condition of the subgrade over time.

These results would indicate that FWD testing at mid-slab locations of overlaid jointed concrete trial sites should be undertaken soon after overlaying in order to obtain a baseline measurement, and then only repeated if a significant deterioration in overlay condition occurs.

From the significant amount of FWD deflection data that has been collected from trial sites and other schemes, a comparison was made between the bound layer stiffness and d_1-d_4 deflection, as shown in Figure 4.66. This figure clearly shows the expected trend of lower deflections with increasing bound layer thickness, with the C&S sections generally showing slightly higher deflections.

4.3.3 Load transfer efficiency (LTE)

For surveys up to Autumn 1995, the LTE was measured in accordance with the Strategic Highways Research Program (SHRP) Long Term Pavement Performance (LTPP) procedure (SHRP, 1999) which utilises measurements made 300 mm either side of the plate (d_7/d_2) for the assessment of joint efficiency. Thereafter, the surveys were carried out in accordance with the Highways Agency specification given in HD 29/94 (DMRB 7.3.2). The HD 29/94 technique uses d_7/d_6 which generally gives lower efficiencies than using the d_7/d_2 approach. The HD 29/94 method is also more likely to detect poor edge support on the loaded slab. HD 29/94 states that satisfactory LTE ranges from 75 % to 100 % (0.75 – 1.0) for inservice concrete pavements.

The measurements were only made across saw-cut and seal treatments and at crack locations where the position of the joint could be identified so that there could be reasonable certainty that the FWD geophones were located either side of the joint. For joints with cracks, FWD measurements made on successive surveys were not necessarily located at the same position along the crack, so that variations in the measured load transfers could occur. If cracking only occurred in the off-side wheelpath of lane 1, then no measurements were made for safety reasons.

The temperature of the pavement can have a significant effect on the measured load transfer efficiency. Generally, higher efficiencies are obtained at high pavement temperatures as the concrete expands and locks cracks and joints together. HD29/94 recommends that joints or cracks be tested at a similar temperature, ideally below 15 °C, when the width of the cracks are greater and the degree of movement is more severe. However, it has not always been possible to carry out the surveys on the trial sections at low temperatures due to traffic management constraints.

In order to grade the LTE of each crack or joint, the LTE has been assigned a severity rating according to the following criteria:

- Severity A: LTE < 0.5
- Severity B: 0.5 ≤ LTE < 0.75
- Severity C: 0.75 ≤ LTE < 0.9
- Severity N: 0.9 ≤ LTE

Cracks or joints with LTE ratings of either C or N are considered to be performing well.

The FWD LTE measurements made on concrete pavements prior to overlay showed a significant number of LTE values below 0.5 on the M5 Taunton and above 0.75 on the A14 Quy. Following overlay, with these joints being untreated prior to overlay, there was little difference in the measured LTE at either of these sites, indicating that poor load transfer before overlay may not be an indicator of poor performance after overlay. Furthermore, the preoverlay FWD LTE survey on the M5 Taunton was carried out in November when the temperature was < 15 °C whilst the preoverlay survey at the A14 Quy was undertaken in July when the temperature was well above 20 °C. The comparable performance post overlay would suggest that the differential vertical movement (DVM) across the slabs was low prior to overlay, as high DVM would indicate poor slab support and possible the presence of voids beneath the slab which would be expected to lead to the rapid onset of reflection cracking.

The LTE measurements were made regularly on cracks and saw-cuts between 1994 and 2001. In order to analyse the data, the minimum value of the measured LTE for each crack and saw-cut showing delaminating of the sealant has been selected from the surveys in 1999, 2000 and 2001. In addition, the 2001 testing was not carried out on the cracks that had been treated by wide overbanding due to the difficulty in ensuring the geophones were either side of the crack. Temperatures were around 10 °C for each of the surveys and comparisons should be unaffected by temperature variation. From the data, 40 cracks have shown an LTE less than 0.75. A summary of the condition of 40 cracks characterised as having an LTE less than 0.75 are given in Table 4.11.

It is apparent from Table 4.11 that the visual condition of the cracks appears to correlate well with the lower LTE of the cracks, with only six (20 %) visual severity level ‘c’ cracks appearing in the table. From the LTE data 51 SCS have shown an LTE less than 0.75. A summary of the condition of four SCS characterised as having an LTE less than 0.75 are given in Table 4.12.

In general, as a crack develops there is a measured fall in LTE as the interlock between the asphalt matrix deteriorates. For SCS sections, a fall in LTE is an indicator that the crack has developed from the crack initiation slot beneath the surface seal. As long as the surface seal remains intact, then there is no cause for concern. 20 joints show no seperation of the sealant at the surface and only four joints are considered to have separated at category ‘b’ or ‘a’ severity.

4.4 Longitudinal profile and development of rutting

4.4.1 Longitudinal profile measurements

Measurements of the longitudinal profile variance have been made on an annual basis at five trial sites, and at less frequent occasions on the other monitored sites. Measurements since construction were made using the TRL High-speed Survey Vehicle (HSV) up to 2000. Since 2000, the surveys have been carried out using the Highway Agency Routine Road Investigation System (HARRIS) vehicle. HARRIS is a prototype vehicle used for the production of the Traffic Speed Condition Survey (TRACS) machine currently used for the monitoring of the Highways Agency trunk road network.

The 3 m and 10 m averaging lengths reflect the fundamental response of vehicle suspension systems to road surface unevenness and, therefore, are a measure of ride quality. Increases in these values at these shorter averaging lengths are associated with deterioration of the pavement layers, while changes at the 30 m averaging length yield evidence of subsidence or settlement.

Threshold values of in-service longitudinal profile are given in HD 29/94 (DMRB 7.3.2) for various categories of roads with the levels applicable being shown in Table 4.13. However, Highways Agency Interim Advice Note (IAN) 42/05 has since amended the threshold values for measurements produced by TRACS and HARRIS machines. The revised values for motorways and rural dual carriageways are shown in brackets in Table 4.13 where they differ from those applicable.

Descriptions of the categories are quoted in HD29/94 as follows:

0 *Sound:* no visible deterioration.

1 *Some deterioration:* lower level of concern. The deterioration is not serious and no action is needed unless extending over long lengths or several parameters are at high levels at isolated positions.

2 *Moderate deterioration:* warning level of concern. The deterioration is becoming serious and needs to be investigated. Priorities depends on extent and values of parameters.

3 *Severe deterioration:* intervention level of concern. Immediate action is required. This condition should not occur very frequently on the motorway and trunk road network because earlier maintenance should have prevented this state from being reached.

Most surveys were category ‘0’ for the duration of the monitoring period.

Table 4.11 Condition of reflection cracks with LTE <0.75 at Quy

			Lowest LTE measurement of crack/saw-cut (d6/d7)		*Condition of crack from 2003 visual survey*	
Joint ref.	*Section*	*Treatment*	*LTE*	*Severity rating*	*Length (cm)*	*Visual severity*
J1191	S3	150 mm CSO by Whiphammer	0.71	B	180	c
J1189	S3	150 mm CSO by Whiphammer	0.71	B	160	c
J1188	S3	150 mm CSO by Whiphammer	0.66	B	190	c
J1186	S3	150 mm CSO by Whiphammer	0.60	B	370	b
J1179	S3	150 mm CSO by Whiphammer	0.73	B	370	b
J1177	S3	150 mm CSO by Whiphammer	0.62	B	370	b
J960	S6	180 mm CSO by Whiphammer	0.47	A	370	a
J886	S7	100 mm CSO by Guillotine	0.07	A	370	a*
J809	S9	100 mm Control	0.39	A	370	a
J804	S9	100 mm Control	0.52	B	370	a
J803	S9	100 mm Control	0.69	B	370	a
J802	S9	100 mm Control	0.55	B	140	c
J801	S9	100 mm Control	0.68	B	230	b
J800	S9	100 mm Control	0.59	B	370	a
J797	S9	100 mm Control	0.74	B	370	b
J796	S9	100 mm Control	0.57	B	370	a
J792	S9	100 mm Control	0.57	B	370	a
J791	S9	100 mm Control	0.69	B	370	c
J787	S9	100 mm Control	0.65	B	370	a
J786	S9	100 mm Control	0.63	B	370	a
J784	S9	100 mm Control	0.59	B	370	a
J781	S9	100 mm Control	0.55	B	370	a
J779	S9	100 mm Control	0.69	B	370	a
J777	S9	100 mm Control	0.57	B	370	a
J775	S9	100 mm Control	0.66	B	370	b
J774	S9	100 mm Control	0.71	B	370	a
J772	S9	100 mm Control	0.73	B	370	a
J769	S9	100 mm Control	0.67	B	370	a
J767	S9	100 mm Control	0.73	B	370	a
J765	S9	100 mm Control	0.63	B	370	a
J763	S9	100 mm Control	0.69	B	370	b
J178a	S17	100 mm SBS	0.53	B	370	b
J178b	S17	100 mm SBS	0.71	B	370	a
J175	S17	100 mm SBS	0.71	B	370	a
J172	S17	100 mm SBS	0.50	B	370	a
J169	S17	100 mm SBS	0.32	A	370	a
J164a	S17	100 mm SBS	0.70	B	370	b
J164b	S17	100 mm SBS	0.52	B	370	b
J157	S17	100 mm SBS	0.71	B	370	b
J58	S18	180 mm SBS	0.70	B	60	c

Table 4.12 Condition of SCS sections with LTE <0.75 at Quy

			Lowest LTE measurement of crack/saw-cut (d6/d7)		*Cracking or delamination of the sealant from 2003 visual survey*	
Joint ref.	*Section*	*Treatment*	*LTE*	*Severity rating*	*Length (cm)*	*Visual severity*
J1224	S2	150 mm SCS	0.52	B	30	c
J1223	S2	150 mm SCS	0.73	B	190	c
J1222	S2	150 mm SCS	0.67	B	140	c
J1221	S2	150 mm SCS	0.71	B		
J1220	S2	150 mm SCS	0.72	B		
J1219	S2	150 mm SCS	0.69	B		
J1217	S2	150 mm SCS	0.70	B	180	c
J1215	S2	150 mm SCS	0.74	B	160	c
J1214	S2	150 mm SCS	0.67	B		
J1212	S2	150 mm SCS	0.63	B		
J1211	S2	150 mm SCS	0.71	B	220	c
J1209	S2	150 mm SCS	0.51	B	120	c
J1208	S2	150 mm SCS	0.59	B		
J1205	S2	150 mm SCS	0.70	B	130	c
J1204	S2	150 mm SCS	0.64	B		
J1203	S2	150 mm SCS	0.68	B		
J1201	S2	150 mm SCS	0.58	B		
J1200	S2	150 mm SCS	0.71	B	100	c
J1199	S2	150 mm SCS	0.60	B	10	c
J1198	S2	150 mm SCS	0.61	B		
J1197	S2	150 mm SCS	0.63	B		
J761	S10	100 mm SCS	0.65	B	20	c
J760	S10	100 mm SCS	0.46	A	20	c
J759	S10	100 mm SCS	0.53	B		
J758	S10	100 mm SCS	0.55	B		
J757	S10	100 mm SCS	0.63	B	70	c
J756	S10	100 mm SCS	0.67	B		
J755	S10	100 mm SCS	0.63	B	150	c
J753	S10	100 mm SCS	0.70	B		
J752	S10	100 mm SCS	0.67	B	10	c
J751	S10	100 mm SCS	0.52	B	220	b
J750	S10	100 mm SCS	0.63	B		
J749	S10	100 mm SCS	0.59	B	80	c
J748	S10	100 mm SCS	T0.69	B	30	c
J747	S10	100 mm SCS	0.52	B	180	a
J746	S10	100 mm SCS	0.67	B	20	c
J743	S10	100 mm SCS	0.50	B	370	b
J742	S10	100 mm SCS	0.60	B	20	c
J741	S10	100 mm SCS	0.73	B		
J740	S10	100 mm SCS	0.63	B	120	c
J739	S10	100 mm SCS	0.73	B		
J735	S10	100 mm SCS	0.58	B	370	a
J613	S12	180 mm SCS	0.60	B	40	c
J610	S12	180 mm SCS	0.71	B	50	c
J607	S12	180 mm SCS	0.71	B	40	c
J602	S12	180 mm SCS	0.74	B		
J599a	S12	180 mm SCS	0.25	A	50	
J599b	S12	180 mm SCS	0.49	A	30	c
J598	S12	180 mm SCS	0.27	A		
J596	S12	180 mm SCS	0.72	B	20	c
J595	S12	180 mm SCS	0.73	B		

Table 4.13 Longitudinal profile threshold levels

		Profile variance (mm^2)		
	Category	*3 m averaging length*	*10 m averaging length*	*30 m averaging length*
1	Sound	<1.25 (0.7)	<4 (1.6)	<55 (22.0)
2	Lower level	<3.75 (2.2)	<16 (6.5)	<165 (66.0)
3	Warning level	<7.50 (4.4)	<36 (14.7)	<275 (110.0)
4	Intervention level	>7.50 (4.4)	>36 (14.7)	>275 (110.0)

4.4.2 Rut depth measurements

Detailed measurements of permanent deformation (i.e. rutting) in the wheelpaths have been made at the trial sites when the visual condition surveys or the indicative measurements produced by the HSV or HARRIS have shown the site may be experiencing measurable levels of rutting. The measurements were taken to assess whether the rutting was consistent over all sections at a given site. Differences in rut depth for sections with similar levels of loading may be the result of variability in the properties of the asphalt overlay. Surfacings containing higher binder content, a softer binder, or a mortar matrix may be more susceptible to rutting. However, such differences in the properties of the surfacing may also have a marked effect on the materials resistance to reflection cracking. Areas showing high levels of early-life rutting often show greater than normal resistance to reflection cracking.

Most of the rutting problems that have occurred at trial sites have been confined to the surface course layer and have been remedied by planing out and replacing with a more rut-resistant surface course layer.

5 Treatment options

5.1 Cost benefit assessment

Based on a knowledge of the prodecures involved, a rough assessment was made of a range of maintenance techniques in terms of initial cost, duration of construction activity, expected maintenance intervention, and overall life as shown in Table 5.1. Each technique was analysed against each criteria and was awarded a mark between 1 and 5 (5 being the lowest/worst case). From Table 5.1 it can be seen the best three techniques adopted on UK roads are CSO, SCCSO and SCS. However the table does not include techniques such as rubblisation and CRL as no longer term performance data is available. The treatment option to be applied will also depend upon a number of other factors as follows:

Table 5.1 Cost benefit analysis of maintenance techniques

Maintenance technique	*Initial cost*	*Project duration*	*Maintenance intervention*	*Design life*	*Total*
Crack and seat	1	1	1	2	5
Saw-cut, crack and seat	1	2	1	2	6
Saw-cut and seal	1	2	1	2	6
Modified asphalt	2	1	2	2	8
Thin surfacing only	1	1	4	3	9
Thick overlay (180 mm)	3	2	2	2	9
Geogrid – thick overlay	3	3	2	2	10
Geogrid – thin overlay	2	3	3	3	11
Full reconstruction	4	5	1	1	11

Assessment made by simple ranking of each technique against each criterion.

- Concrete pavement construction type.
- Condition of the existing concrete.
- Relative estimates of initial cost and level/cost of expected maintenance interventions.
- Existing headroom and drainage and barrier levels.

5.2 Treatment guidance

In order to make the best maintenance decision for the existing pavement, surveys of the existing concrete pavement will provide the data necessary to follow the decision options given in Figures 5.1 to 5.3 for unreinforced jointed concrete pavements, reinforced jointed concrete pavements and CBGM pavements, respectively.

Each of these options charts gives the type of defects that may be encountered, and classifies these into ‘major’ and ‘minor’ categories. A series of questions are then answered in order to determine the most appropriate treatment option(s).

Examples of potential treatments for each construction type could be as follows:

i For an unreinforced jointed concrete pavement with general transverse cracking but no pumping, the treatment option would be to C&S the pavement and overlay with 150 mm of asphalt (Figure 5.1).

ii For a reinforced jointed concrete pavement without general longitudinal or transverse cracking but with pumping, the treatment options are either to SCCSO with 150 mm of asphalt or to grout and adjust slab lengths to 12 m or less, apply a minimum overlay of 70 mm and SCS over the joints (Figure 5.2).

iii For a CBGM concrete pavement with general longitudinal cracking in the asphalt above longitudinal cracking in the in the concrete, the options are to either recycling using TRL611 (Merrill *et al.*, 2004) or to reconstruct the full depth of the pavement (Figure 5.3).

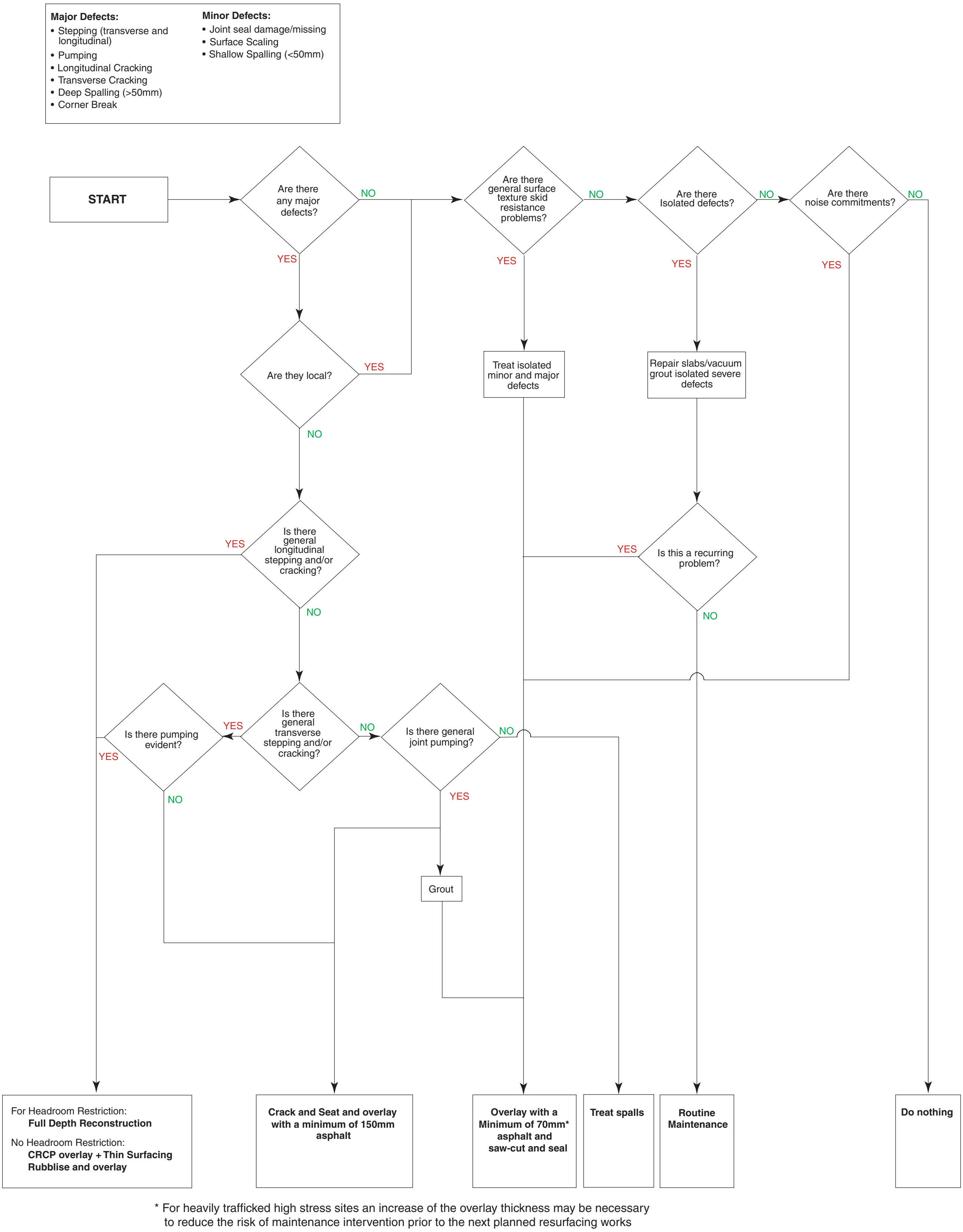
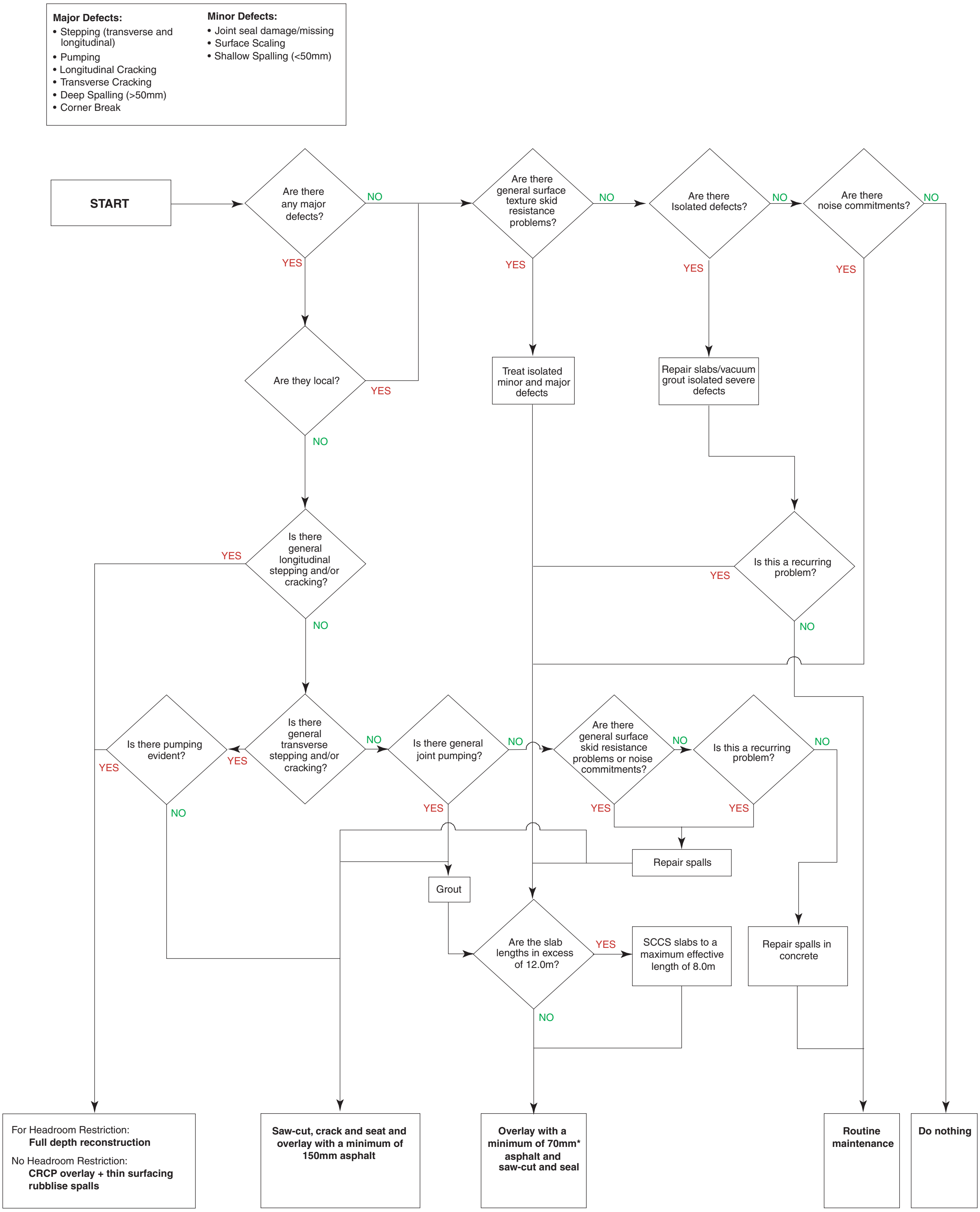


Figure 5.1 Treatment options for jointed unreinforced concrete pavements



* For heavily trafficked high stress sites an increase of the overlay thickness may be necessary to reduce the risk of maintenance intervention prior to the next planned resurfacing works

Figure 5.2 Treatment options for jointed reinforced concrete pavements

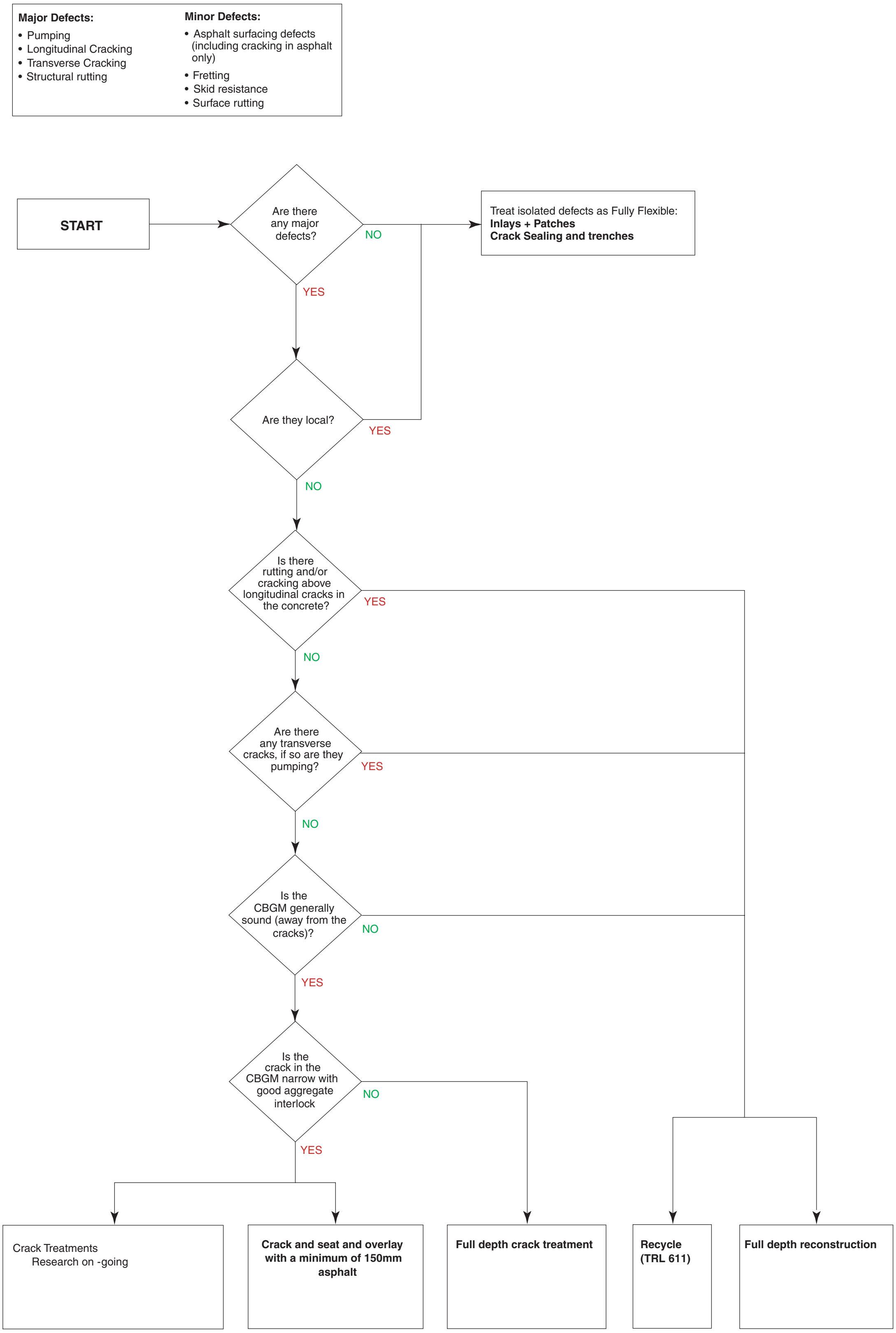


Figure 5.3 Treatment options for a flexible composite pavement with a CBGM base

6 Conclusions and recommendations

In conclusion, it has been identified that reflection cracking is a frequent problem when overlaying jointed concrete and CBGM pavements and can be minimised by the use of an appropriate maintenance technique. Much progress has been made in understanding the causes and consequences of reflection cracking in concrete pavements in recent years. In particular, the following summary sets out the current status of the options available:

Crack and seat and saw-cut, crack and seat:

- The crack and seat technique and saw-cut, crack and seat technique results to date show these are an effective way of reducing the occurrence of reflection cracking when compared with control sections with the same thickness of overlay, and are appropriate to concrete roads in a fair condition.
- Longitudinal cracking was present at the A46 Kenilworth and from original surveys prior to the application of the crack and seat technique, wide separation and stepping of the longitudinal lane joint was visible. An investigation into cracking of the overlay (75 months after construction) showed the cracking was located above the joint in the concrete. Therefore the technique may not be appropriate for sites with severe stepping of joints, where further treatment may be required prior to overlay.
- Overlays of 150 mm or more seem to be performing the best with minimal cracking present after 10 years service for crack and seat and no cracking after 4 years service for saw-cut, crack and seat.
- The depth of steel in reinforced concrete pavement needs to be accurately determined using GPR and coring when applying SCCSO.
- Crack and seat techniques should be applied where the existing pavement is in a fair or moderate condition.

Saw-cut and seal:

- The saw-cut and seal technique has proved to be very effective in reducing the occurrence of reflection cracking, and is an appropriate treatment for jointed concrete pavements in a generally good condition.
- Performance with overlay thicknesses of less than 50 mm has not been satisfactory and it is recommended that this is the minimum thickness for this technique to be applied. It is preferable to have a thickness of at least 70 mm.
- Good quality control during the application of the saw-cut and seal technique is an important factor in the performance.

Geogrid/geotextiles

- The performance of geogrids and geotextiles are very variable with some sites showing cracking before the control section with the same overlay.
- The process is very labour intensive and requires good weather conditions for satisfactory results and good control of the installation process.
- The use of geogrids/geotextiles can give rise to problems when the surfacing is to be replaced if used between the surface and binder course layers.

Modified asphalts

- The use of an EVA modified binder used on the M2 Kent proved unsuccessful at preventing reflection cracking with cracking first being observed within four and a half years.
- The SBS modified binder used in the surface course on the A14 Quy shows better performance in preventing reflection cracking with a 180 mm overlay. However, the use of 100 mm and 150 mm overlay is performing similar to the control sections.
- The SBS modified binder used in the surface and binder course on the eastbound carriageway of the A14 Bury St. Edmunds proved an effective treatment after ten years service. However, wide transverse cracking was present on the westbound carriageway where the same technique was used during the same period.
- There is scope for further use of modified binders but a clearer understanding of how to achieve good performance is required.

Thin surfacing

- Not considered a suitable overlay directly onto concrete due to cracking normally occurring within two years and the development of an on-going maintenance problem.

Crack relief layers

- Application of the technique is relatively easy and, because the porous asphalt does not need to be designed for a surface course, no high polished stone value (PSV) aggregate is needed.
- Further data is needed on the durability of this treatment.

Rubblisation

- This technique is only considered viable for pavements nearing the end of their serviceable life and is considered a more sustainable alternative to full depth reconstruction for pavements in a generally poor condition.
- Care needs to be exercised when rubblising the concrete due to possible soft spots in foundation and also to minimise damage to the foundation.
- Overlays for rubblisation are generally 200 to 430 mm thick; depending on the performance of the granular layer produced, the traffic requirements and the asphalt material.
- Further data is needed on the durability of this treatment.

The results from this work will be used to assist in preparing design guidance and specifications and also provide the highway engineer with information on the treatment options available. This advice will enable the most cost-effective maintenance treatment to be selected, having regard to the resources available and the required life.

7 Further work

Observations to date have shown trends in the performance of various maintenance techniques and methods for inhibiting reflection cracking in asphalt overlays. Continued monitoring of the existing trials and recent trials (rubblisation and crack relief layers) is required to enable better relationships in the results to be established. This monitoring will form the final basis for recommended overlay treatments to concrete pavements.

Further data is needed to provide more robust guidance on treatments based on FWD LTE measurements and measured DVM across slabs.

Other developments also need to be tracked, such as incorporating a UK trial to improve the performance of the saw-cut and seal technique by reducing sealant pull-out. This reduction is achieved in the USA by the addition of a closed-cell 'backer rod' to the saw-cut prior to the application of the sealant. The 'backer rod' is a flexible, compressible and chemically inert polyethylene rope with a circular cross section. It eliminates the need for bond breaker tape, due sealants not being able to adhere to it. The theory behind the use of the 'backer rod' is that the circular cross-section of the material increases the ability of the overlying bituminous sealant to expand laterally and, therefore, better accommodate thermal movements at the joint. This hypothesis, in theory, reduces the possible problems with long term adhesion to the side of the saw-cuts.

8 Acknowledgements

The work described in this report was carried out in the Infrastructure and Environment Division of TRL Limited (Head: Bob Collis). The authors are grateful to Dr Cliff Nicholls who carried out the quality review and auditing of this report. The authors are also grateful to Kent, Suffolk, Somerset, and Cambridgeshire County Councils and to the London Borough of Barnet for their assistance during the construction of full scale trail sites.

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Appendix A: UK trial sites and schemes

Site	Location	Technique	Road type	Length	Built	Concrete	Overlay (mm)	Surface type
A1 Markham Moor†	Newark, Notts	SCCSO, SCS	D2L	3.4	Jan-00	JRC	100, 120	TSC
A1 Tuxford	Newark, Notts	Geogrid	D2L	1.2	1999	JRC	50, 100	SMA
A1 Tuxford*	Newark, Notts	SCCSO	D2L	0.5	May-99	JRC	50, 100	TSC
A1 Tuxford*	Newark, Notts	SCS	D2L	0.2	May-99	JRC	50, 100	TSC
A1 Winthorpe-Coddington	Newark, Notts	SCS	D2L	8	Mar-04	JRC	150	TSC
A10 Braughing	Herts	CSO	D2L	2	Sep-00	CBM	160	TSC
A10 Ely	Cambs	CSO	S2L	1	Sep-00	CBM	120	TSC
A12 Boreham	Chelmsford, Essex	CSO	D3L	22	Feb-00	URC	180	TSC
A12 Brentwood 2	Essex	SCCSO	D2L	17.2	Nov-02	JRC	150	TSC
A12 Brentwood 2*	Essex	CSO	D2L	4	Nov-02	URC	150	TSC
A12 Brentwood I	Essex	SCCSO	D2L	14	Jan-01	JRC	150	TSC
A12 Hatfield Peverel	Chelmsford, Essex	SCCSO	D2L	4	Feb-02	JRC	150	TSC
A12 Hopton	Gt Yarmouth, Norfolk	CSO	D2L	4	Jan-00	CBM	150	TSC
A12 Lowestoft	Suffolk	SCS	S2L	0.7	Mar-00	URC	70	TSC
A12 Mountnessing	Essex	CSO	D3L	15	Feb-99	URC	170	TSC
A12 Stanway*	Colchester, Essex	CSO	D2L	10	Apr-00	URC	150	TSC
A12 Stanway*	Colchester, Essex	SCCSO	D2L	10	Apr-00	URC	100	TSC
A12 Stanway*	Colchester, Essex	SCS	D2L	0.4	Apr-00	URC	150	TSC
A14 Bury St Edmunds*	Suffolk	SCS	D2L	0.3	Oct-90	URC	100	HRA
A14 Milton-Fen Ditton	Cambridge	CSO	D2L	3	Aug-98	CBM	200	TSC
A14 Quy*	Cambridge	CSO	D2L	3.4	Jul-93	URC	100 150 180	HRA
A14 Quy*	Cambridge	SCS	D2L	1	Jul-93	URC	100 150 180	HRA
A14 Spittals-Alconbury	Huntingdon	CSO	D2L	14	Feb-99	URC	170	TSC
A153 Anwick	Sleaford, Lincs	CSO	S2L	6.4	Apr-93	JRC	200?	HRA
A167 Durham	Durham	Geogrid	D2L	0.4	1997	JRC	140	HRA
A17 Swineshead	Boston, Lincs	CSO	S2L	5.4	Apr-02	CBM	130	TSC
A180 Grimsby	North Lincolnshire	CSO	D2L	4	Mar-05	URC	150-180	TSC
A30 Bodmin*	Cornwall	Geogrid	D2L	0.6	1989	CBM	40, 80	HRA
A30 Cummery	Exeter	CSO	D2L	1.6	Mar-87	CBM	175	HRA
A30 Exeter	Exeter	CSO	D2L	1	Feb-91	CBM	75 155	HRA
A30 Launceston	Cornwall	Geogrid	D2L	0.2	1987	CBM	40	HRA
A30 Launceston	Cornwall	CSO	D2L	2.8	May-05	CBM	180	TSC
A30 Pennygillam-Tavistock Rd	Launceston, Cornwall	CSO	D2L	4	May-03	CBM	175	TSC
A30 Plusha 1	Launceston, Cornwall	CSO	D2L	0.3	Oct-97	Ecopave	70	HRA
A30 Plusha 2	Launceston, Cornwall	CSO	D2L	1	Oct-98	Ecopave	150	HRA
A36 Ower	Hampshire	PA Interlayer, SCS	D2L	0.8	Apr-05	URC	80	TSC
A38 Swinfen-Weeford	Lichfield, Staffs	CSO	D2L	5.3	Sep-00	CBM	180	TSC
A40 Whitchurch	Herefordshire	CSO	D2L	1.2	Mar-92	CBM	175 355	HRA
A449 Culdra-Usk	Cardiff	CSO	D2L	10	Oct-98	URC	220	EAC
A46 Cossington-Six Hills*	Leicester	CSO	D2L	6	Jun-02	CBM	180	TSC
A46 Cossington-Six Hills*	Leicester	SCS	D2L	8.4	Jun-02	JRC	140	TSC
A46 Kenilworth	Warwickshire	CSO	D3L	22	Nov-98	URC	170	TSC
A5 Wibtoft	Hinckley, Leics	CSO	D2L	6	Mar-03	CBM	180	TSC
B1441	Weeley	Rubblisation	S2L	2	Oct-04	JRC	200	TSC
M1 Barnet J2-3	Barnet, N London	SCS	D3L	1.1	Aug-94	JRC	110 150 180	HRA
M1 J2-Deansbrook*	Barnet, N London	CSO	D3L	8.6	Dec-99	URC	150+	TSC
M1 J2-Deansbrook*	Barnet, N London	SCCSO	D3L	5.3	Dec-99	JRC	150+	TSC
M1 J2-Deansbrook†*	Barnet, N London	SCCSO, SCS	D3L	3.3	Dec-99	JRC	150+	TSC
M11 J6-7 NB*	Harlow, Essex	CSO	D3L	23.1	Mar-02	URC	180	TSC
M11 J7-6 SB*	Harlow, Essex	CSO	D3L	5.4	Mar-03	URC	180	TSC
M11 J7-8 Phase III*	Stansted, Essex	CSO	D3L	21.6	Feb-01	URC	150	TSC
M11 J7-8 Phase III*	Stansted, Essex	SCCSO	D3L	21.6	Feb-01	URC	150	TSC
M11 J7-8 Phases I-II*	Harlow, Essex	CSO	D3L	42	Jan-00	URC	150	TSC

Site	Location	Technique	Road type	Length	Built	Concrete	Overlay (mm)	Surface type
M11 J7-8 Phases I-II*†	Harlow, Essex	SCCSO	D3L	42	Jan-00	URC	150	TSC
M11 J8 Stansted*	Stansted, Essex	CSO	D3L	23	Oct-02	URC	150	TSC
M11 J8 Stansted*	Stansted, Essex	SCCSO	D3L	23	Oct-02	URC	150	TSC
M11 J8-9 Phase 1*	Saffron Walden, Cambs	CSO	D2L	58.9	Apr-03	URC	150	TSC
M180	North Lincs		D2L	1	2003	URC	150	TSC
M2 Kent*	Faversham	Geogrid	D2L	1.8	1990	JRC	75, 100, 140	HRA
M20 J3-5	Maidstone. Kent	CSO	D3L	48	Aug-01	URC	150	TSC
M20 J9-10	Ashford, Kent	CSO	D3L	40	Jul-00	URC	150	TSC
M27 J2-4	Southampton	CSO	D3L	67	Oct-01	URC	150	TSC
M27 J8-10*	Fareham, Hants	CSO	D3L	48	Jul-00	URC	150	TSC
M27 J8-10*	Fareham, Hants	SCCSO	D3L	48	Jul-00	URC	150	TSC
M27 J8-10*	Fareham, Hants	SCS	D3L	3.1	Jul-00	URC	150	TSC
M40 J6-7	Thame, Bucks	CSO	D3L	60	Mar-97	URC	150?	TSC
M42 J9-10 NB	Tamworth, Staffs	CSO	D2L	19.2	Feb-01	URC	150	TSC
M5 J24-J26	Taunton, Somerset	CSO	D3L	8	Jan-01	URC	150	TSC
M5 J26-27 scheme 104A/B	Tiverton, Devon	CSO	D3L	20.4	Feb-02	URC	160-170	TSC
M5 J26-27 scheme 134	Wellington, Somerset	CSO	D3L	10.8	Feb-03	URC	175	TSC
M5 Taunton 55a (NB)	Taunton, Somerset	CSO	D3L	3.2	Nov-97	URC	150	HRA
M5 Taunton 55c (NB)	Taunton, Somerset	CSO	D3L	8	Mar-98	URC	150	HRA
M5 Taunton 57 (SB)	Taunton, Somerset	CSO	D3L	6.4	Apr-98	URC	150	HRA
M5 Taunton 58 (SB)	Taunton, Somerset	CSO	D3L	5.6	Oct-98	URC	150	HRA
M5 Taunton 92*	Taunton, Somerset	CSO	D3L	2.2	May-92	URC	100 150	HRA
M5 Taunton 92*	Somerset	SCS	D3L	0.5	May-92	URC	100 150	HRA
M5 Taunton C56 (NB)	Taunton, Somerset	CSO	D3L	6.5	Nov-99	URC	150	TSC
M5 Taunton C59 (SB)	Taunton, Somerset	CSO	D3L	4	Dec-99	URC	150	TSC
M5 Taunton*	Somerset	Geogrid	D3L	0.4	1992	URC	150	HRA
M5 Taunton (J26-27)	Somerset	CSO	D3L	3	Feb-05	URC	150	TSC
M5 Willand*	Devon	SAMI, SCS	D3L	0.3	Sep-03	URC	70	TSC
M54 J6-7	Telford, Salop	CSO	D2L	12	Feb-00	URC	150	TSC
M69 Phase 1*	Hinckley, Leics	CSO	D3L	30	Feb-00	URC	150	TSC
M69 Phase 1*	Hinckley, Leics	SCS	D3L	0.2	Feb-00	URC	150	TSC
M69	Leics	CSO	D3L		Feb-05	URC	150	TSC

SCCSO – Saw-cut crack and seat and overlay
SCS – Saw-cut and seal
CSO – Crack and seat and overlay
SAMI – Stress absorbing membrane interlayers
D2L – Dual carriageway 2 lanes
D3L – Dual carriageway 3 lanes
S2L – Single carriageway 2 lanes
** Denotes scheme also contains areas with other treatments.*
† Denotes scheme where SCCS used to prepare pavement for SCS.

URC – Unreinforced concrete
JRC – Jointed reinforced concrete
CBM – Cement bound material
TSC – Thin surface course
SMA – Stone mastic asphalt
HRA – Hot rolled asphalt
EAC – Exposed aggregate concrete

Abstract

The performance of an overlay to a jointed concrete pavement can be affected by the occurrence of reflection cracks above the joints. Reflection cracks are transverse and/or longitudinal cracks that occur in the overlay above the joints or cracks in the underlying concrete. If the cracks are not promptly treated and left to widen and propagate to the full depth of the asphalt layer, the subsequent influx of water can weaken the foundation and fines can be pumped to the surface creating voids beneath the base. In the most severe cases, the structure of the pavement is compromised to such a degree that movement of the pavement structure occurs under normal traffic loading. In some cases, the surfacing can also ravel back from the crack with the reduced lateral support, impairing the ride quality. If allowed to progress to this state the maintenance implications are more serious. This report summaries the performance to-date of a number of overlaid jointed concrete sites, with slab lengths varying from 5 m to 24 m, and various sites containing a lean concrete base. The treatments applied included: variations in asphalt thickness; use of polymer modified binders; crack and seat techniques; saw-cut and seal; inter-layers; and concrete joint treatments. Guidance is given in this report for jointed unreinforced (URC) and jointed reinforced (JRC) concrete pavements and flexible composite pavements with a cement bound granular material (CBGM) base. This guidance will assist the highway engineer in preparing maintenance design options, enabling the most cost-effective maintenance treatment to be selected, having regard to the existing construction and its current condition, the resources available and the required life of the pavement.

Related publications

TRL636 *The application of Enrobé à Module Élevé in flexible pavements* by P J Sanders and M Nunn. 2005 (price £50, code HX)

TRL611 *A guide to the use and specification of cold recycled materials for the maintenance of road pavements* by D Merrill, M E Nunn and I Carswell. 2004 (price £50, code HX)

TRL606 *Durability of thin asphalt surfacing systems: Part 2, Findings after 3 years* by J C Nicholls and I Carswell. 2004 (price £40, code EX)

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