

CURING AND INTERLAYER DEBONDING

Geoff P Ayton, Roads and Traffic Authority,
New South Wales, Australia

Edwin W Haber, Consultant
Queensland Department of Main Roads, Australia

SYNOPSIS

Current Australian practices in the areas of concrete curing and interlayer debonding reflect a blending of experience from leading overseas authorities together with that from substantial local experience since the 1970's. Australian practices were summarised (for the wider range of design and construction issues) in a paper presented at the Fifth Purdue Conference in 1993 [1] but significant developments have occurred since that time. In terms of curing practices, the major developments have been in the types of curing compounds being used (for both pavements and bases), and in the method of specifying and monitoring their quality in contracts. In the area of interlayer debonding, changes have been made to the specified materials and methods. With few exceptions, concrete pavements constructed within the past twenty years have been over a lean-mix concrete base. Ongoing experience confirms that the level of interlayer friction is critical to the early thermal movement and to joint induction. It is also critical to the avoidance of unplanned reflection cracking in the pavement and thereafter to its long-term performance. Particular attention has been focused on refining debonding treatments under continuously reinforced pavement following experience on several projects which exhibited undesirable cracking patterns which were thought to have been largely a consequence of too high a level of interlayer friction. This paper discusses the Australian specifications and practices in these areas. Comparison is also made to the international experience as reflected in the published literature, and a brief account is provided of current ongoing development work in this field.

A brief discussion is provided of factors (additional to interlayer friction) which appear to influence cracking patterns in continuously reinforced concrete pavements.

AUSTRALIAN PRACTICES

Since 1975, the Roads and Traffic Authority (RTA) has promoted the increased use of concrete pavements throughout the state of New South Wales (NSW). During the period up to mid-1997, approximately 1700 lane-km of concrete pavement have been constructed and a high level of planning continues for future works. The other Australian states have continued to construct almost exclusively in flexible pavements because of more favourable combinations of local geology and traffic loadings. However, the state of Queensland is currently constructing two projects in plain concrete pavement (PCP), and changing factors in that state appear to favour further substantial construction in concrete.

This paper focuses on NSW practices but, for the most part, the discussion also applies to the other states. Australian state road authorities maintain a high level of co-operative technology exchange, and this reflects in the consistency between states in terms of design and construction practices. American and Australian pavement terminologies differ, as shown in Table 1. American terminology has been adopted throughout this paper.

The Australian thickness design procedure is based on the Portland Cement Association (PCA) 1984 method. On the major arterial routes such as highways, freeways and motorways where concrete pavements are used, design traffic loadings are typically within the range of 5×10^7 to 2×10^8 ESAL's (7×10^7 to 3×10^8 commercial vehicle axle groups - CVAG).

PAVEMENT FORMATS

Four formats of rigid pavement have been constructed in Australia, viz plain concrete, continuously reinforced, dowelled reinforced, and steel fibre reinforced. Typical design aspects are provided below, with discussion generally limited to aspects which are relevant to the issues of interlayer friction and debonding.

Plain Concrete Pavement (PCP)

PCP (undowelled) constitutes approximately 80% of the total concrete pavement constructed in Australia since 1975.

Continuously Reinforced Concrete Pavement (CRCP)

Australia's CRCP inventory totals approximately 310 lane-km which constitutes almost 20% of the total rigid pavement constructed since 1975. The typical longitudinal steel content has recently been increased from 0.67% to 0.7%. CRCP is typically used in areas where maximum reliability is sought (ie minimum maintenance demand) and in areas of moderate differential settlement. It is also the preferred rigid pavement format where a thin porous asphalt surfacing is proposed.

Jointed Reinforced Concrete Pavement (JRCP)

The JRCP constitutes only a very small proportion of the "new" rigid pavement in Australia but its use continues to be promoted for applications such industrial areas, for piece-meal construction in urban areas (such as commercial/retail centres), for areas of potentially high differential settlement and for widening of existing JRCP.

Steel Fibre Reinforced Concrete Pavements (SFCP)

This format has found popular application throughout New South Wales in situations such as roundabouts [2] and intersections where high centrifugal loadings place extreme stresses on flexible pavements and where the unavoidable high incidence of odd-shaped slabs results in stresses which exceed the recommended limits for conventional concrete formats (plain and mesh-reinforced).

Local specifications currently require a minimum fibre content of 75 kg/m^3 but this limit, together with issues such as thickness design parameters, remain somewhat contentious amongst suppliers and designers [3]. Local confidence in this product has been boosted by its successful use on heavily trafficked routes in Europe, particularly in Belgium [4].

BASE FORMATS

With only a few exceptions, concrete pavements constructed since 1975 have included a lean concrete (econocrete) base (LCB).

The performance of LCB as reported by worldwide authorities has been somewhat variable. In Australia, with few exceptions, the performance of LCB has been exceptionally good and there have been only rare cases of the types of distress reported by researchers such as Smith et al [5]. Australian experience to date indicates that the following factors are important:

- (a) The provision of a sound subbase. Australian specifications typically require a controlled subbase within the top 150 to 300 mm.
- (b) Competent LCB qualities such as strength and shrinkage.
- (c) Controlled bond/friction levels at the pavement interface.
- (d) The provision of pavement edge drains.

LCB has proven economically competitive because of the shortage of suitable naturally occurring gravels along many of the heavily trafficked corridors and as a result of the ready availability of good flyash supplies from coal-fired electric power stations.

Rolled lean concrete (RLC) bases have been used in a small number of projects but this option has proven less attractive than conventional wet mixes for the following reasons:

1. The New South Wales paving industry favours the use of wet mixes to enable fuller utilisation of slipform pavers and mixing plants, and this is reflected in tender prices.
2. The difficulty of achieving uniform and adequate compaction throughout its depth.
3. Closer control of finished levels (and hence on thickness) is possible with slipformed wet mixes.

Given that the majority of concrete construction is of undowelled plain pavement, the properties and performance of the base are considered to be major factors in the overall design package. Rolled concrete is perceived to be inferior to LCB [6] because of its more variable insitu properties and because of concerns over its performance under undowelled joints (in erosion and shear). Consequently, under the Austroads design method, LCB is mandatory under undowelled pavements in highway applications. Thicknesses are typically 125 to 150 mm. (Austroads; Association of Australian State Road Authorities, similar in role to AASHTO.) LCB is also the preferred option under dowelled and continuously reinforced pavements but rolled lean concrete, cement treated base and asphaltic concrete (dense grade) are acceptable alternatives.

The materials design and construction specification for LCB have been developed with the aim of producing a base with sufficient strength to resist erosion and with very fine cracks at spacing typically of 4 to 7 m which will provide a satisfactory degree of shear transfer.

Testing by various authorities [6,7,8] on cement treated bases indicates that a minimum cement content of 5 to 7% is required to satisfy erodability requirements. Consistent with this, a minimum strength of 5 MPa was nominated for LCB in the formative years of the development of local specifications for the relatively mild climatic conditions which apply in Australia. A value of 5 MPa was set in the knowledge that later-age strengths would easily exceed 7 MPa. Under severe freeze-thaw conditions, German experience suggests that a minimum of 8 to 10 MPa is required, and this accords with Illinois experience [9].

To date, and with pavement ages up to 20 years, there have been very few cases in Australia of LCB surface erosion. (A rare case of surface erosion was recently discovered in an area where water was arrested on the uphill side of a pavement anchor on a steep grade.)

Investigations during slab replacement works generally indicate that the LCB provides an effective waterproofing (or at the very least, water-shedding) layer which protects the subgrade from significant moisture variation. Any water which enters the pavement appears to flow on top of the base and is readily collected in pavement edge drains. Consequently, there has been little incentive in Australia to trial permeable bases.

The good performance of LCB is no doubt partly attributable to the high long-term strengths achieved by the high flyash mixes which are invariably used. Laboratory strengths are typically 5 to 7 MPa at 28 days, 10 to 15 MPa at 90 days and 15 to 20 MPa at one year. Cores taken from pavements at ages of 10 to 15 years have yielded strengths up to 30 MPa.

Acceptance is based on core strength (in lieu of cylinder strength) and an age of 42 days is set (in lieu of 28 days) for the achievement of the nominated 5 MPa core strength.

Binder composition is typically specified as:

- ☐ Portland cement: 90 kg/cu m minimum
- ☐ Flyash: 100 kg/cu m minimum
- ☐ Total binder: 250 kg/cu m minimum

These high flyash mixes have proven to be well suited to slipforming and their slow strength development in the early days after paving encourages the desired pattern of fine cracks at close spacings. This is further assisted by the stipulation of a shrinkage limit of 550 μ after one week moist curing and 3 weeks air drying. Further, under plain pavement a sprayed bituminous (chip) seal has typically been specified as an added insurance against reflection of cracks into the pavement. This seal is invariably placed within weeks of base paving in order to facilitate construction trafficking, but an added benefit is that the drying shrinkage is likely to be very low in the period up to placement of the pavement, after which the base remains in a stable moist condition [10]. Consequently, the insitu drying shrinkage is likely to be lower than that indicated by laboratory testing.

CURING AND DEBONDING

Curing and debonding of the LCB have proven to be important aspects of pavement construction, and local practices have been progressively refined with ongoing experience.

The two issues (curing and bonding) are necessarily inter-related in typical pavement applications. If, for example, positive debonding between layers is deemed necessary, it is logical and efficient to cure the lower layer with a material which contributes to the debonding.

Conversely, where it is proposed to place a bituminous asphalt surfacing to a concrete pavement, it is crucial that the curing compound used on the pavement does not compromise bonding, but rather enhances it. Interlayer bonding/friction impacts on several aspects of pavement design such as:

1. the design of longitudinal steel in CRCP,
2. cracking patterns within CRCP,
3. the design of transverse steel and joint tiebars in all pavement formats, in accordance with the subgrade drag model,
4. the risk of reflection cracking, and hence the warrant for “insurance” reinforcement in otherwise plain pavement,
5. the magnitude of stresses within the pavement as induced by contraction strains, and
6. as a consequence of (v), the safe level of total tied width.

Current Australian practices for curing and debonding are summarised in Table 2. Also included are the estimated interlayer friction coefficients yielded by each treatment. The values therein represent our best attempt at drawing the “line of best fit” through values compiled from several disparate sources, but we recognise the warrant for more work on this aspect. Attempts have been made to “test” these values against observed behaviour as predicted by the subgrade drag model, but the results have been inconclusive. A laboratory program is currently in progress in a further attempt to validate them or, at the very least, to rank them in magnitude order.

Care needs to be exercised in selecting friction values for design purposes because the insitu level of shear resistance is likely to be influenced by factors which are not commonly listed in friction tables but which are nevertheless studied in detail in the literature. Examples follow.

- ☐ In the case of RLC, CTB and BAM: variability in surface texture characteristics.
- ☐ In bound bases, the horizontal shear strength/resistance is likely to be variable and to increase with age. Hence, a relatively new BAM surfacing may yield a significantly lower effective friction than would an old, hardened BAM.
- ☐ The magnitude of interlayer displacement. In CRCP the magnitude of displacement (relative to the base) is significantly lower than in jointed pavements and this results in very different levels of effective friction. Timms [11] and others have reported on this.

- The effectiveness of debonding treatments. Substantial surface voids in LCB are not likely to be filled or masked by light materials such as curing compounds but can be effectively filled with bituminous materials.

A review by Ioannides and Salsilli-Murua [12] of the state-of-the-art in interlayer friction provides a coverage of the many studies undertaken since 1924. It shows that friction factors as high as 60 can be encountered over CTB.

DEBONDING

Given the variability (and somewhat unpredictability ~) of interlayer friction levels, it is established practice in Australia to apply treatments to the base which will provide moderate and relatively uniform friction conditions under the concrete pavement for risk minimisation. In the case of open-textured bases such as CTB, this has the added advantage of minimising the loss of mortar from the pavement concrete, an aim which is considered well justified in view of the sensitivity of pavement fatigue to "effective" thickness.

Under undowelled pavements, the influence of interlayer friction is potentially critical. Too low a friction results in uneven initiation of cracks at transverse sawcuts whilst too high a friction leads to reflection of both cracks and joints from the base into the pavement (and sometimes also from the pavement into the base, referred to locally as "*retro-reflection*").

On several Australian JPC projects, early crack initiation has occurred at spacings between 1 in 3 and 1 in 8 joints, and these openings are obviously of a magnitude where effective aggregate interlock is lost. This has also been regularly observed in steel fibre reinforced concrete, presumably because of the residual high tensile capacity after sawcutting. The designer would hope, and it would seem logical to expect, that joint widths would equalise after several full seasonal cycles. However, field observations suggest that this does not occur to the required extent.

The best interlayer treatments have proven to be either sprayed bituminous seals or a supplementary application of wax emulsion curing compound. The latter has been found to provide the optimum interlayer friction but has the disadvantage of not adequately preventing reflection cracking from more severe base cracks and joints, and is unlikely to mask any significant texture on the surface of the base. On one CRC project, such texture appears to have yielded too high a friction level and resulted in undesirable cracking patterns; this will be discussed more fully hereunder.

With this treatment (wax + wax), it is also occasionally necessary to "bandage" wider cracks and joints with a material such as bitumen coated geotextile prior to the placement of pavement concrete. However, crack widening will normally only be severe enough to warrant this treatment if there is a long delay between paving of the LCB and covering with the pavement.

The risk of reflection cracking appears to be higher from construction joints in the LCB than from self-induced cracks, an effect which appears to be related to their relative surface roughness. In order to minimise the risk of reflection cracking from joints, local specifications since 1984 have required that longitudinal joints in the LCB be located within 0.25 ± 0.15 m of a planned pavement joint. Under this arrangement, reflection cracking is not known to have occurred and hence bandaging was abandoned shortly thereafter. (Longitudinal joints are not mandatory; this requirement only applies where the contractor's construction method necessitates a joint.) Sprayed bitumen seals have proven very effective in preventing reflection cracking but at times they are too effective in debonding when applied to surfaces which have earlier been cured with wax emulsions. In accordance with expectations, slippage appears to occur at the wax-bitumen interface. The bitumen is typically applied at a rate of 1.0 l/sq m with a covering of 5 mm aggregate, but trials are currently being monitored to assess the influence of reducing the bitumen application rate, possibly through the use of bitumen emulsions. Technical aspects of curing and debonding are discussed in further detail below.

CURING

Curing is described as the maintenance of an environment favourable to the setting and hardening of concrete in order to achieve the expected service performance. On high output paving works, curing will necessarily involve considerable cost and effort. However, pavements are typically constructed in a relatively exposed and harsh environment and hence effective curing is essential in terms of strength, abrasion resistance, permeability, frost resistance and plastic shrinkage.

Curing practices warrant particular attention where the concrete is to form the running surface. Modern low-noise surfaces are created with reduced texture depths of only a few millimetres. The durability of those top few millimetres is critical to the long-term durability of friction and resistance to hydroplaning, and will be largely dictated by the quality of the surface mortar and the standard of its curing [2¹³14].

For this reason, techniques commonly accepted in the general construction industry (such as surface slurring and delayed curing) are unacceptable in road paving.

The same applies to a lesser extent to the surface of lean concrete bases because resistance to erodibility of that surface will be partly dependent on the quality of the surface.

Water-curing is indisputably the superior method of curing concrete surfaces but logistical considerations invariably render it impractical for road pavements. Plastic sheeting is also a proven effective curing method, but again, it presents practical difficulties in exposed windy conditions. (Notwithstanding, it may be required in addition to sprayed compounds as added protection against rainfall, low overnight temperatures or thermal shock from severe temperature falls).

These problems have led to the development of liquid membrane-forming curing compounds. However the characteristics and quality of such products vary markedly and require careful regulation and specification. Australian summer conditions are relatively severe and mandate the adoption of very good curing practices. Australian specifications therefore justifiably require high quality curing compounds and a high standard of application to maximise their effectiveness.

Development of Specifications

The first large scale use in Australia of contemporary liquid membrane-forming compounds on a concrete road project was in 1975. The project comprised a CRC pavement over a lean concrete base. No Australian Standard Specification for curing compounds existed at the time, and so reference was made to ASTM C309: *Specification for liquid membrane-forming compounds for curing concrete*.

It was originally intended to use wax emulsion on both layers but this was altered to chlorinated rubber as a result of the observed behaviour of both compounds on an industrial site (the new Sydney Produce Markets at Flemington) over a large area of exposed concrete surface. Under exposed conditions of wind and rain, the chlorinated rubber provided a far more resilient film integrity, albeit at twice the cost. Most importantly, the chlorinated rubber better maintained its film integrity for the crucial initial period of three to seven days. This observed performance was at variance with their apparent equality under the then existing procedure of ASTM C156 (*Test Method for Water Retention by Concrete Curing Materials*) which formed part of ASTM C309.

Chlorinated rubber was thereafter adopted as the standard curing material for pavement course until the early 1990's when it was replaced by hydrocarbon resin because of its lower toxicity from health and environmental perspectives.

During the early 1980's, wax emulsion became the standard curing medium for the lean concrete base. It provides a satisfactory curing medium (at lower cost) and assists in debonding the overlying pavement. In parallel with the above developments in New South Wales, Woodstrom and Neal [14] simultaneously reported that new chlorinated rubber and hydrocarbon resin curing compounds were being developed which had properties superior to wax or wax resin base materials which had formerly been used.

An objection to wax or wax/resin-based formulations noted by Woodstrom and Neal was that on deeper broom textured surfaces the compounds tended to sag or run off the peaks and collect in the grooves. However, even on lightly textured surfaces, the use of wax emulsions on the pavement course is strongly discouraged in Australia for safety reasons and because of the difficulty of adhering bituminous surfacings, linemarking and raised pavement markers.

Hence, on the basis of both local and overseas experience, the dichotomy of using wax-based products for curing/debonding of the base surface, and chlorinated rubber or hydrocarbon resin for curing of the pavement became firmly established practice.

Australian Standard AS 3799 (Liquid Membrane-forming Curing Compounds for Concrete) was first issued in 1990 after further years of development work and experience gained on major road, airport and other infrastructure projects.

The following tests were derived directly from ASTM C309 and C156, without alteration except for Water Retention:

- (i) Water retention
- (ii) Reflectance (for white pigmented compound)
- (iii) Drying time
- (iv) Long-term settling (for pigmented compounds)
- (v) Non-volatile content (solids)

The following additional tests were added to AS 3799:

- (vi) Density (which was not included in the ASTM specification at that time).
- (vii) Viscosity (to control run-off on textured and vertical surfaces).
- (viii) Residue on 150µm sieve (as a protection against clogging of spray equipment).

The key aspect of AS 3799 is undoubtedly the water retention test. The curing compound is applied in its normal manner (most commonly by spraying) to the surface of a mortar which has been prepared in a standard reproducible method. For the material under test, the rate of application can be nominated by the supplier/manufacturer, or otherwise it is required to be applied at a standard rate of 0.2 l/sqm. The cured test specimens (3 for each material being tested) are placed into the test chamber for 72 hours, and the weight (m_{wt}) of water lost during this period is determined. Identical but uncoated control specimens are simultaneously placed in the chamber and the weight (m_{wb}) of water lost is similarly determined.

The efficiency index (e) for the curing compound is calculated as:

$$e = \frac{m_{wb} - m_{wt}}{m_{wb}} \times 100 \quad (1)$$

where

- e = efficiency index (%), to nearest whole number
- m_{wb} = average mass of water lost from mortar of the blank (control) specimen
- m_{wt} = average mass of water lost from mortar of the test specimen

A minimum value of 90% is required for compliance with the standard.

In the initial stages of the development of the water retention limits, consideration was given to a two-point test requirement, with an efficiency index of 80% after 7 days also being prescribed. This had been a long standing requirement of the Australian Government on airport pavements. However an examination of available data revealed sufficient equivalence between an efficiency index of 90% at 3 days and 80% at 7 days, and hence the latter was allowed to lapse, thereby saving much time and money.

ASTM C309 stipulates a maximum water loss of 0.55 kg/sq m which corresponds to a range on Efficiency Index of about 80% to 85%, depending on the absolute value of the control water loss. A higher standard was nominated in the Australian specification for the following reasons:

- (i) Existing products in the market which rated between 80% and 90% on test were considered as marginal or inconsistent in field performance.
- (ii) An appreciable number of existing products gave test results in excess of 90%, and these were rated by the RTA as consistently good in field performance.
- (iii) The remaining products sold as curing compounds were generally ineffective.

Several further refinements to the specification were required with ongoing field use, particularly of newly released products.

One example was that of a new product based on lanoline lambswool grease (and which smelled like a sheep dip) rather than on paraffin wax. It gave poor field performance which, for unknown reasons, had not been reflected in the standard testing regime and hence it became necessary to develop additional requirements applicable to wax emulsion products. In this case, the problem was addressed by specifying a softening point of the non-volatile material of not less than 45°C.

In another example, a material which performed well as a curing compound yielded an undesirably high bond with the overlying pavement. It was found that in winter conditions at temperatures in the range of -5°C to 0°C, some waxes undergo a solid-state phase change and thereafter functioned as an adhesive rather than as a debonder.

This was observed to apply particularly in instances where a saturated long-chain fatty acid such as stearate is used as the emulsifying agent rather than an unsaturated long chain fatty acid such as an oleate. To combat this, the additional requirement has been added that *“the emulsifying agent shall be an unsaturated long chain fatty acid capable of combining with lime to produce a calcium soap release agent to prevent bonding between the layers of concrete.”*

Classification of Curing Compounds

Curing compounds are categorised under AS 3799 as shown in Table 3. Table 4 lists the compound groups typically nominated by road authorities for curing of LCB and pavement layers. Table 5 lists road authorities' typical requirements for each compound group.

Resin-Based Curing Compounds

Until about 1992, the most commonly used and best performing resin-based curing compound was the chlorinated rubber material, a by-product of the paint industry. It usually comprised only 20% non-volatile material, with the balance being an aromatic solvent such as xylene or toluene. However, these solvents are now considered less acceptable both to site staff and from an environmental point of view and hence their use has now been discontinued on paving projects.

The replacement materials are hydrocarbon resins containing less offensive solvents such as white spirit or kerosene. Environmental objections are further lessened by the fact that they require around 50% solids in order to achieve the specified water retention efficiency index and hence contain a significantly reduced solvent content. More recently, some wholly water-based hydrocarbon resin products have appeared on the market, thereby overcoming any remaining environmental objections. Another variation of the hydrocarbon resin compound is a water-based product where the resin is pre-dissolved in solvent.

Water-based styrene butadiene resin (SBR) products have proven capable of meeting water retention requirements but, unlike the hydrocarbon resin, are quite unsuitable directly beneath any asphaltic concrete surfacing.

Acrylic resin based compounds have to date been unable to meet the 90% water retention requirement but have nevertheless gained limited acceptance in markets with less demanding specifications than those set by road authorities.

Monitoring of Quality

Ongoing experience shows that regular monitoring is required of the quality of curing compounds at construction sites. The detection of non-conforming product is surprisingly frequent given the long-established nature of the industry. Auditing and testing procedures need to target not only the product “*as delivered to site*” but also the product “*as sprayed onto the work*”.

The procedures currently mandated under NSW specifications are as follows.

(a) *Prior to commencement of work*

The Contractor is required to provide to the Client a report, supported by test certificates, to certify that the proposed curing compound has been tested and that it complies with the Specification. The sample so tested is termed the “reference sample”. Testing is required of :-

- ☐ efficiency index,
- ☐ non-volatile content,
- ☐ density, and
- ☐ drying time.

Additionally, an infrared spectrum must be provided to enable future comparison with results from compound actually used in the works.

This testing program is comprehensive but need not be repeated for each project; results for a specific product are taken to be valid whilst ever the formulation remains unchanged.

Note that the report to the Client must be provided and signed by the Contractor rather than the material manufacturer or supplier, because the Client has no contract with either of those companies.

(b) *From the initial delivery to site*

The Contractor is required to randomly sample and test for the properties listed under (a) above, including infrared spectrum but excluding the efficiency index.

On the basis of these results, the Contractor is required to provide written certification (accompanied by the test results) that the delivered batch has the same formulation as that of the reference sample.

(c) *From subsequent deliveries*

The Contractor is required to provide written certification that each delivered batch has the same formulation as that of the initial delivery. The certification may be made on the basis of the manufacturer’s Certificate of Analysis, for uniformity of the following properties :-

- ☐ non-volatile content
- ☐ density, and
- ☐ viscosity.

In addition to the above mandatory testing, the Client’s site representative will intermittently undertake audit sampling and testing of product both “*as delivered to site*” and “*as sprayed onto the work*”. All testing is required to be undertaken by a laboratory holding appropriate Quality certification.

Application of Curing Compounds

Australian specifications require that the compound be applied in two separate applications. This is intended to counter any variability resulting from influences such as surface moisture condition, variable output from a nozzle or intermittent wind gusts. In combination with the specified viscosity limit, it is also thought to reduce the extent of runoff into the troughs of surface texture.

Each application is required to be at the rate stated on the Certificate of Compliance or at 0.20 l/m², whichever rate is the higher. In effect, this constitutes a total application of twice the rate which is applied on a flat surface in the water retention test but this is considered to be a reasonable safety margin to compensate for the effect of variable site conditions combined with the effect of surface textures. Ideally, the two applications would be in perpendicular directions but this will not be feasible with large-scale mechanised equipment.

The first application is required to be within 15 minutes of the “low sheen” surface condition being reached, a condition which indicates that the bulk of the free bleed water has evaporated. The second application is required to be 10 to 30 minutes later, or otherwise in accordance with the manufacturer's recommendation. On fixed-formed surfaces, the first application is required to be within half an hour of removal of the formwork and the second to be 15 to 45 minutes later.

Early curing requires that texturing be carried out as close as possible behind the paver and the hessian (burlap) drag. This is achieved in the fresh concrete by optimising the length and orientation of tines so that they stroke the surface rather than rake it.

However, timing of the spray should also recognise the effect of surface moisture on different compound types. Testing of chlorinated rubbers and wax emulsions [15] has shown that their effectiveness can be significantly influenced by this moisture condition. Results are shown in Figure 1.

Water-miscible products (such as wax emulsion, but not chlorinated rubber) are diluted by free water, resulting in reduced viscosity and increased variability of film thickness. Figure 1 indicates the deleterious effect of applying water-miscible compounds whilst bleed water is present.

With chlorinated rubber compounds (which are organic solvent based and not miscible with free water) application must be prior to significant surface drying. Laboratory testing showed that “... *after approximately 145 minutes the free surface water was no longer present and the concrete surface porosity subsequently increased. As a result, (chlorinated rubber) curing compound applied after this time was absorbed into the concrete surface, thereby preventing the formation of a continuous film.*”

Testing of this type is not known to have been carried out on hydrocarbon resins.

In practice, evaporation of bleed water under field conditions is usually much more rapid than is indicated by the above laboratory testing. A delay of 145 minutes at a paving speed of 1.3 m/min would leave about 180 m of concrete exposed. Fortunately, the ideal “low sheen” condition will typically be reached within about 20 to 45 minutes, depending on temperature, humidity, wind and the mix's bleeding characteristics.

Specifications require that application be in a fine spray and by the following means:

- (i) For paving widths < 2.5m: by hand lanced, with optional single or multiple nozzles.
- (ii) For paving widths ≥ 2.5m: by spray bar fitted with multiple nozzles spaced to give a uniform cover for the full paving width in a single pass. Additionally,
- (iii) For paving widths > 4.5 m which are paved by mechanical means (ie slipformer), application is required to be by a mechanical sprayer fitted with a spray bar with multiple nozzles spaced to give a uniform cover for the full paving width in a single pass.

Category (ii) frequently applies on tapered widths in congested areas where a hand-held spray bar will be the only practical means of application. Multiple nozzles are specified in order to minimise the degree of variability in application rate.

For areas sprayed by single nozzle lance, the application rate is required to be increased by 25 per cent. Included in this category are the faces of formed joints and sections of slipformed edges which may have been supported by temporary forms at the time of initial spraying.

The specified 25% surcharge is consistent with the results of testing by Woodstrom & Neal [14] using hand-held lances and which showed that even where coverage looks uniform to the eye, large variations are likely. Coefficients of variation of up to 20% were measured, with isolated applications as low as 65% of the mean rate. If the mechanical sprayer under category (iii) becomes inoperable, concrete paving by mechanical means must be terminated and must not recommence until the mechanical sprayer becomes fully operable again.

Sprayers must incorporate a device for continuous agitation and mixing of the compound in its container during spraying. Spray bars are required to be fitted with protective hoods to minimise losses from the target application rate in windy conditions and to minimise the drift of compounds to workers, to passers-by and to roadside areas.

The application rate is required to be checked as follows:

- (a) by calculating the average application rate from the total measured quantity of compound applied over each paving run, and
- (b) by testing the local amount of curing compound as measured on test mats placed on the pavement.

The application rate within a test section is deemed to conform if both the average rate and the mean local rate equal or exceed the specified rate, and if no single local rate is more than 5% below the specified rate. Any section on which the application does not conform must be resprayed within 6 hours of testing, at an application rate not less than twice the deficiency in the original application.

This testing regime is by no means foolproof, because application under all categories (i) to (iii) is subject to operator influence. This is seen as further justification for the somewhat liberal margins added to the application rates and justification for regular auditing of paving operations.

The curing membrane must be maintained intact in a continuous and unbroken film for 7 days after placing the concrete. Any damage to the curing membrane must be made good by handspraying of the affected area. Additionally, at the commencement of each paving run (and notwithstanding that film damage may not be readily apparent), any adjoining concrete of age less than 7 days must be resprayed with a single application for a minimum distance of 7 m in the area trafficked by staff during placement at the construction joint; in other words, the terminal section of afternoon paving which by necessity is trafficked the following morning during start-up operations.

CRCP PERFORMANCE AND THE INFLUENCE OF DESIGN PARAMETERS

The following is a brief discussion of Australian experience in the design and performance of CRC pavements, with emphasis given to the apparent influence of interlayer friction and bonding on cracking behaviour. The authors have endeavoured to limit the discussion in this paper to the topics "*curing and debonding*". However, with reference to CRC pavements, a discussion of debonding and friction in isolation would appear to be of limited value because of their apparent interaction with other design parameters such as concrete strength and steel content.

The discussion relates predominantly to transverse CRC cracking because procedures in CRCP for the initiation of longitudinal joints (and tiebar design therein) are identical to those for jointed pavements.

CRCP Cracking Behaviour

The ideal cracking pattern in CRCP is one of parallel and discrete transverse cracks at spacings between about 0.8 m and 2.5 m. The upper spacing target of 2.5 m is imposed because, for a given shrinkage strain, wider spaced cracks are likely to open to widths exceeding 0.5 mm. The lower target of 0.8 m reflects concerns over the increased risk of block cracking from the higher stresses associated with elongated slabs under traffic loading. However, these values are guidelines only, and international surveys show that many CRC pavements of high quality have provided excellent long-term performance notwithstanding that they exhibit (parallel and discrete) cracking at spacings as low as 0.3 m, or even 0.2 m.

Given favourable conditions, the propensity for discrete, parallel, transverse cracking in CRCP without skewing or branching is remarkably strong. However, experience shows that the ideal pattern can be adversely affected by influences such as interlayer bonding, concrete mix characteristics and pavement curing procedures. Indeed, the sensitivity of CRC to early-age cracking is demonstrated by the fact that the taking of a core (of, say, 75 mm diameter) within the first month after paving will invariably induce a transverse crack.

Cracking Patterns in CRCP

Several CRC pavements constructed within NSW have exhibited cracking patterns which do not conform to the model pattern, and overseas literature reflects similar experience. As an example, the AASHTO pavement design guide [16] Fig 3.15 shows several types of undesirable cracking. A Transportation Research Board survey of experience in 33 US states classed "*erratic crack patterns*" as the most common "*distress manifestation*". [17] Such cracking typically becomes apparent within weeks of construction, in which case it is clearly not associated with parameters such as trafficking or embankment support conditions. (Where large temperature differentials occur in summer paving, a well developed crack pattern can form within the first few days whilst concrete strengths are still low.)

Australian experience on several major CRCP projects has indicated a high correlation between undesirable cracking and too high a level of interlayer friction, particularly in summer paving. However, other factors (such as mix design) also have a significant influence on its incidence and severity.

In an attempt to eliminate this cracking, modifications have been progressively made to the interlayer systems. In one summer project, the type of curing compound on the LCB was changed and, on another project (again summer), extra effort was devoted to the finishing techniques on the LCB to ensure a finish which was smooth and free of significant surface (air) voids. In both cases, these actions were successful in almost totally eliminating the undesirable cracking.

The following is a brief discussion of undesirable types of CRC cracking. For the purpose of this discussion, a distinction is made between "early-age" and "later-age" cracking, as summarised in Table 6. At the risk of oversimplifying the many inter-related factors involved, the intention is to distinguish between:-

- (i) those cracking types which occur independent of traffic loading, and
- (ii) those which occur at later age, and only under the influence of traffic loading.

In many winter paved projects the full "early-age" cracking pattern does not stabilise until the passing of one or two winters, but this cracking is thought to be largely independent of traffic loading and hence remains within the definition of "early-age".

In seeking to remain within the intended scope of this paper, the following discussion is confined as much as possible to early-age, "self-initiated" cracking because of its apparent close correlation with interlayer bonding and friction. By comparison, later-age cracking appears to be relatively independent of friction.

(a) *Close-spaced (parallel) cracking*

The international literature contains several reports, most notably from Belgium [18], which verify that parallel close-spaced cracking is unlikely to compromise the performance of CRC pavement as long as other factors are favourable. Most significant of those factors are insitu concrete strength, support strength and crack widths. Substantial lengths of Belgian CRC pavements have crack spacings as low as 0.2 (and even 0.1 m) which show no distress at ages up to 20 years. Belgium's strength criterion is typically quoted as "60+ MPa" [19].

Similarly, several local NSW projects have shown no distress in areas of close-spaced or cluster cracking unless other adverse factors were also present, such as low strength concrete or poor support. Consequently, close-spaced cracking would seem to be undesirable not so much for its own influence but because of the risk of premature distress wherever other unfavourable factors are also present (such as marginal concrete strength or poor subgrade support).

(b) *Cluster cracking*

“Cluster” typically refers to groups of cracks (commonly in threes) with close spacing of the order of 0.2 to 0.4 m. For the purpose of clarity, the discussion will be limited to cluster cracks which are both parallel and discrete. (In reality, the cracks may intersect or skew, in which case they will obviously fall within other categories discussed hereunder.)

Conventional crack initiation theory suggests that new cracks should form approximately mid-way between preceding cracks at a cross-section where contractional tensile forces are maximised. Under such conditions, cluster cracking would theoretically not occur.

However, their occurrence in otherwise (apparently) uniform CRC pavements suggests that movement near the initial crack was restrained by a cause such as base bond/friction to the extent that the stress close to that crack exceeded the stress level within the zone mid-way between cracks.

This appears consistent with the work by Timms [20] and others on the relationship between friction coefficient and displacement magnitude. The magnitude of displacement each side of an early CRC crack would be relatively high whereas there is theoretically no displacement at the point mid-way between existing cracks.

Local experience [21] indicates that, in the case of LCB, cluster cracking is usually centred about a prominent base crack, and that those CRC cracks initiated by base cracks are amongst the earliest to form. Whatever the causal influences, the first cracks are obviously induced in concrete of (early) low strength. Within areas of higher friction, movement away from the first crack appears to be restrained, seemingly generating enough tensile stress to induce companion cracks on each side at offsets which are lower than can otherwise be explained by “conventional theory”.

(c) *Skew cracking*

Australian experience suggests that any significant degree of skewing is indicative of non-uniform conditions such as an adversely high level of friction with the base. Skewing will not necessarily be linear from edge to edge, but might occur as a deflection mid-way across the paved width (of either single or double lane width).

(d) *Edge cracking*

In CRC, edge cracks typically extend up to 1 m from the edge and for the full depth of the pavement. (By comparison, in jointed pavements they are often only partial depth cracks and hence could reasonably be called “arris” cracks.)

The causes of edge cracking have not been fully explained but they appear to be associated with severe or differential drying and/or shrinkage effects at the exposed upper edge of the slab. Consistent with plastic shrinkage cracking, susceptibility to edge cracking appears to vary with different concrete mixes. Mixes with poor aggregate shape and/or grading appear more susceptible. The risks also appear to increase with any delay in the application of curing compound at times of high evaporation potential. This influence is avoidable because techniques are available to facilitate early texturing and curing.

On some CRC projects, there has been a high incidence of edge cracking where tiebars have been inserted into a slipformed edge. However, investigations have indicated that the tiebars have not been the initiator of the cracking but have merely influenced the location of what would otherwise have been randomly spaced cracks initiated by other effects.

(e) *Branch cracking*

It appears that branching can occur by either divergence or convergence of individual cracks. Converging cracks are an advanced and consequential form of skewed cracking. The causes of divergent cracking are yet to be explained but the major factor, again, appears to be restraint against free and uniform transverse cracking.

The first serious local example of branch cracking was observed on a project where (contrary to previous local practice) plain concrete shoulders were constructed ahead of the CRC traffic lanes. In addition to the natural CRC cracking, additional cracking in the CRC was initiated by movement within the shoulder contraction joint and, in many cases, cracks of each type converged. Similar experience has been reported by overseas authorities. For this reason, local specifications now require that the through-carriageway CRC lanes be paved ahead of any abutting shoulders (or other pavement such as ramps) regardless of whether the abutting pavement is jointed or CRC.

The greatest risk with branch cracking appears to be the potential for spalling and/or corner cracking at junctions, but punch-outs can also occur in severe cases.

(f) *Map cracking*

This is the extreme case of combined skew and branch cracking and it typically forms blocks with dimensions of 1 to 3 metres. The common cause of map cracking appears to be bonding with the underlying base. However, one case has been observed where late sawcutting of longitudinal joints appeared to have an influence in an environment where cracking was simultaneously occurring in both the longitudinal and transverse directions.

The few sections of local CRCP affected by early map cracking (totalling about 2 carriageway kilometres) have performed for five years with only one known failure which can be attributed to that cracking. In areas displaying map cracking, the CRCP has effectively created discrete “working” transverse cracks by the selection of paths through the cracking network which are most nearly square to the pavement. Thereafter, all other cracks seem to remain relatively dormant.

(g) *Longitudinal cracking*

(This discussion is confined to early-age cracking and hence does not consider fatigue longitudinal cracking.)

This is often indicative of late sawing of longitudinal joints or inadequate depth of induction. However, even when joint induction procedures are satisfactory (or where joints are formed rather than induced), it can occur by reflection from a longitudinal joint in the underlying LCB.

Local experience shows that the latter type of reflection cracking is much less likely (or even fully eliminated) if all LCB joints are located close to joints in the pavement. In the mid 1980's, local specifications were revised to require that all LCB joints be located within 0.25 ± 0.15 m of a pavement joint and, to date, this appears to have been very effective.

Another major lesson has been the value of transverse reinforcement in the event of unplanned longitudinal cracking. On the first CRCP constructed in NSW, a longitudinal crack was initiated over the full 5.5 km by an underlying LCB joint at an offset of 1.2 m from the central joint. This pavement is now 22 years old but shows no deterioration whatsoever and the crack goes unnoticed by most motorists. The good performance of this cracked CRC appears to be attributable to the uniform high quality of concrete throughout the project (albeit only 28 MPa, by design) in concert with a competent supporting LCB, together with adequate transverse reinforcement across the crack.

Further confirmation of this is available from the observation of longitudinal cracking which often occurs over terminal anchors, presumably caused by restraint to curling. Local experience is that this cracking has not compromised the pavement integrity as long as the concrete is of sound quality.

Parameters of Early-age Cracking Patterns

A brief discussion follows of factors which influence early-age cracking patterns, including those which have been observed to mitigate against model cracking.

(1) Base friction

The international literature appears inconclusive on the role played by interlayer friction. Issues such as concrete strength, shrinkage, steel ratio and temperature regimes appear to be given greater emphasis.

Formulae for the design of longitudinal reinforcement derive from work by Vetter [22] on stresses in vertical walls. His original formulae did not consider friction, presumably because of its relatively minor influence on walls, and a friction factor still does not appear in the formulae used by some major road authorities.

Under the formulae currently used in Australia [23,24], friction is input to the calculation of the critical steel ratio p_{crit} (the content required to avoid yield of reinforcement) but is not input to the calculation of crack spacing except indirectly through its influence on p_{crit} .

Computer models developed by Chypin Chiang, McCullough and Hudson [25], by Ma and McCullough [26] and by Hunt [27] take account of friction, but analysis showed that the effect of friction on predicted average crack spacing was small and it was excluded from regression equations because its inclusion did not improve correlation [28]. However, in contrast to this seemingly minimal influence on average spacing, work by McCullough [29] showed that friction does have an influence in evening out the crack spacings.

There is also suggestion in parts of the literature of a possible link between high friction levels and cluster cracking [28] but there appears to be little discussion of the possible influence of friction on other undesirable types such as branch cracking.

NSW experience suggests that friction/bond does have a significant influence on undesirable cracking patterns and hence substantial effort has been devoted to analysing its effects. However, the findings are certainly not conclusive and the feedback gathered over more than twenty years has at times appeared somewhat contradictory. Indeed, on the first CRCP constructed within NSW [21], positive action was taken to generate interlayer friction by transverse texturing of the LCB. The only forms of undesirable transverse cracking sustained were of the cluster and close-spaced types, and the full 11 lane-km length remains totally free of maintenance (and of block cracking) after 22 years in service.

Notwithstanding, several projects since that time have provided evidence of a clear link between friction and cracking. The direct consequences of friction/bond are sometimes masked by other issues but, increasingly with time, the evidence indicates that it is just one of several influencing factors and that if all other factors (such as strength and temperature regime) are favourable then the CRCP may be forgiving of a high friction level. Conversely, in combination with unfavourable influences from other factors, a moderate to high friction level will invariably induce undesirable cracking patterns.

In summary, there is mounting confirmation that a CRC pavement needs to be largely free of external constraints and influences for it to reliably form a natural and favourable cracking pattern. One such constraint is base friction.

(2) Concrete strength and compaction

Computer modelling by Ma and McCullough [26] shows that "... even a modest variation in the tensile strength of the concrete in the analysis would greatly alter the crack pattern, leading to close-spaced cracks in the weaker areas with long lengths of uncracked concrete between".

Jackson [28] reports that "Few computer studies have considered concrete variability. However the author has analysed some recent tension tests by Williams [30] with and without considering the effect.

Including it greatly improved the correlation with the test results. It appears that including concrete variability in the analysis of CRCP would greatly alter the predicted crack patterns whilst having little effect on the predicted average crack spacing."

It must be remembered that concrete strength is itself influenced by several factors. The major influences on uniformity of "as supplied" concrete are uniformity of mixing and slump control. (CRCP is therefore potentially very sensitive to retempering of the mix.)

However, not only is the "*as delivered*" strength important (as reflected in cylinders) but also the "*end product*" strength (as reflected by cores). The latter is influenced by handling of the concrete up to the end of the curing period, with major influences being compaction, curing and early weather/temperature protection.

Performance surveys by Nussbaum & Lokken [31] indicate a high correlation between cluster cracking and poor consolidation of the concrete, particularly under the reinforcement.

Inadequate compaction has a profound impact on the strength of concrete and has been an ongoing cause of premature distress in concrete pavements worldwide. CRC pavements seem to be particularly sensitive to marginal compaction (and thereby strength) because of the inherently high stress levels generated by the continuous reinforcement. Additionally, it is possible that compaction levels in CRC pavements are typically lower (albeit only marginally) than in plain pavements because of the "vibration shadow" effect of the steel mat.

Particular care is required in compaction at transverse construction joints. Under Australian specifications, thorough supplementary manual vibration is mandatory in the area which cannot be vibrated by the slipform paver. Premature failures in this zone have occurred with unacceptably high repetition in local projects.

(3) *Steel reinforcement*

Variations in steel content will influence crack spacings (and hence also crack widths) but are not known to influence the formation of undesirable cracking types (with the obvious exception of the close-spaced type). A recent local review has led to a marginal increase in steel content from 0.67% to 0.7%.

As part of this review of CRCP design and construction practices, and in an attempt to moderate crack widths, lap details in longitudinal reinforcement have also been revised in accordance with concerns raised by Jackson [28].

(4) *Thermal, curing and drying effects*

The early-age shrinkage of concrete is largely dictated by thermal and drying effects.

In order to minimise drying effects, it is important that curing be initiated at the earliest possible time after texturing and after the bleed-water sheen has left the surface. Texturing can be brought forward behind the paver by optimising the length and orientation of tines so that they stroke the surface rather than rake it. (Timely and effective curing of the surface is critical on several accounts, not the least of which is the need to ensure very high quality surface mortar to maximise long-term skid resistance.)

The critical thermal effects are the rate and magnitude of ambient change from the maximum daytime temperature to the reduced overnight temperature. Typically, the spacing of transverse cracking will reduce with increasing severity of temperature drop. Large temperature changes at early ages, especially during the first 24 hours, impose stresses on the low strength concrete and appear to exacerbate the influence of other factors such as high or uneven base friction.

However, in the absence of other unfavourable factors, the thermal influence on transverse cracking is thought to be limited to crack spacing only, and a severe diurnal change by itself is not thought to cause adverse crack types such as branch or skew cracking.

Summary of CRCP Design and Performance Parameters

The major issues from the foregoing discussion are seen to be as follows:

1. The cracking behaviour and subsequent performance of CRC pavements is influenced by many factors, and much scope remains for refinement of current practices.
2. In terms of cracking behaviour, experience indicates that interlayer friction can be a significant influence. If other factors such as strength and temperature regime are favourable then the CRCP may be forgiving of a high friction level. Conversely, in combination with unfavourable influence from other factors, a moderate to high (and/or non-uniform) friction level will invariably induce undesirable cracking patterns.
3. Of the cracking types deemed to be undesirable, some present a higher risk than others. All other factors being equal, close-spaced parallel cracking is less likely to cause premature distress than is branch cracking.
4. There is ample evidence in the international literature that close-spaced parallel cracking is unlikely to compromise the performance of CRC pavement as long as other factors are favourable. Most significant of those factors are insitu concrete strength, support strength and crack widths.
5. Notwithstanding this reassurance, the existence of undesirable cracking patterns of any type serves to increase the risk of premature distress where they occur in concert with other unfavourable factors. Hence, there is a strong incentive for refinement of the technology to minimise the occurrence of such patterns.

LIST OF TABLES AND FIGURES

Table 1: Pavement Terminology

Table 2: Curing and Debonding Treatments for Base (5)

Table 3: Categories of Curing Compounds

Table 4: Choice of Compound Type

Table 5: Specification of Compound Groups

Table 6: Cracking Types in CRCP

Figure 1: Water loss related to delay in time of application

Table 1. Placing conditions of the various test cases.

	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6	Case 7
Air Temp.	21-35	21-35	21-35	21-35	21-35	21-35	21-35
Batch Temp. at Placement	32.2	23.9	15.6	32.2	15.6	26.7	15.6
Soil Temp.	26.7	26.7	26.7	26.7	26.7	21.1	21.1
Time of Placement	8 AM	8 AM	8 AM	4 PM	4 PM	8 PM	8 PM

$$1^{\circ}\text{F}=(\text{f}-32)/1.8 \quad ^{\circ}\text{C}$$

TABLE 2 Curing and Debonding Treatments for Base

Base format	Pavement format			
	CRC	JPC	JRC	SFC
LCB	Wax + Wax [1.5] ⁽⁴⁾ or HCR + Wax [1.7] ⁽⁵⁾ WHCR + Wax [1.7] ⁽⁵⁾ (4)	Wax + Wax [1.5] ⁽⁴⁾ or Wax + BSS [1.2] or Wax + BE [1.5] or HCR + Wax [1.7] ⁽⁵⁾ or WHCR + Wax [1.7] ⁽⁵⁾	Wax + Wax [1.5] ⁽⁴⁾ or Wax + BSS [1.2] or Wax + BE [1.5] or HCR + Wax [1.7] or WHCR + Wax [1.7] or SBR + Wax [1.7]	Wax + Wax [1.5] or HCR + Wax [1.7] ⁽⁵⁾ or WHCR + Wax [1.7] ⁽⁵⁾ (4)
RLC ⁽⁹⁾ or CTB ⁽⁹⁾	BE + BE [2.5] or BE + BSS [2.5]	N/A ⁽²⁾	BE + BE [2.5] or BE + BSS [2.5]	N/A ⁽²⁾
BAM ⁽⁹⁾	Nil + BE [3.0] or Nil + BSS [3.0]	N/A ⁽²⁾	Nil + BE [3.0] or Nil + BSS [3.0]	N/A ⁽²⁾

Notes

1. Treatments are listed as "C + D [F]", where C= curing type, D = debonding treatment, F = estimated friction coefficient.
2. LCB is the only base permitted under undowelled jointed pavements for highway applications.
3. This table applies only to curing of the base and debonding under the pavement. Curing treatments for the pavement surface are covered in Table 5.
4. Wax emulsion can be slippery (particularly when wet) and requires extreme care for both pedestrian and vehicular traffic.
5. A BSS surfacing yields an added benefit in enabling relatively safe trafficking by construction personnel and vehicles.
6. Under CRCP, the combination of {Wax + BSS} or {Wax + BE} would probably also be satisfactory, but given that the presence of reinforcement will limit the trafficking period, the added cost may not be warranted.
7. The intention is to substantially fill all surface voids in the base in order to moderate the coefficient of friction and to minimise loss of pavement mortar. Spray application rates should be designed accordingly.

Key

- **Wax** = wax emulsion
hydrocarbon resin.
- **BSS** = bituminous sprayed seal with cover aggregate (typically 5 to 7 mm)
- **SBR** = styrene butadiene resin (water-borne)
- **BE** = bitumen emulsion, typically with a sand surfacing.
- **LCB** = lean concrete base
- **RLC** = rolled lean concrete
- **CTB** = cement treated base
- **BAM** = bituminous asphalt

TABLE 3 Categories of Curing Compounds

Classes	Types
A: wax-based ⁽¹⁾	1: clear or translucent without dye
B: resin-based	1-D: clear or translucent with fugitive dye
C: synthetic rubber-based	2: white pigmented or containing aluminium reflective pigments
Z: other	3: black or colored, other than white

Notes

- (1) Waxes can be emulsified with water or dissolved in solvents.

TABLE 4 Choice of Compound Type

Pavement type	Pavement curing	LCB curing (beneath the listed pavement type) ⁽²⁾
JCP	HCR or	Wax or HCR ⁽¹⁾
JRC	WHCR or	Wax or HCR
CRCP or SFCP	SBR	Wax

Notes

1. HCR must be covered by a bitumen debonding (chip) seal in this situation.
2. Refer to Table 2 for further details of allowable combinations of base curing and interlayer debonding

Key

Wax = wax emulsion; **HCR** = hydrocarbon resin; **WHCR** = water-borne hydrocarbon resin; **BSS** = bituminous sprayed seal; **BE** = bitumen emulsion; **SBR** = styrene butadiene resin (water-borne).

TABLE 5 Specification of Compound Groups

Compound group	Requirements ⁽¹⁾	
	On base	On pavement
Wax emulsion	Class A, Type 2 ⁽²⁾	Not permitted
Hydrocarbon resin	Class B, Type 2	Class B, Type 1-D
Water-borne hydrocarbon resin	Class Z, Type 1-D or 2	Class Z, Type 1-D
Styrene butadiene resin (SBR) compounds	Class Z, Type 1-D or 2	Class Z, Type 1-D
Bitumen emulsion	Grade CRS/170 ⁽³⁾	Not permitted

Notes

1. ~ in accordance with AS 3799.
2. Additionally, when tested for stability in accordance with RTA T862, the rate of separation in seven days shall not exceed 4%.
3. ~ complying with specification RTA 3254.

TABLE 6 Cracking Types in CRCP

Early-age	Later-age
(a) Close-spaced (b) Cluster (c) Skew (d) Edge (e) Branch (f) Map (g) Longitudinal	(h) Close-spaced (i) Block (j) Longitudinal fatigue
Influences include: (i) steel content (ii) thermal conditions (iii) base friction/bond (iv) concrete strength (early) (v) shrinkage & creep (vi) concrete rheology	Influences include: (i) traffic loading (ii) support conditions (iii) concrete strength (mature) (iv) crack widths

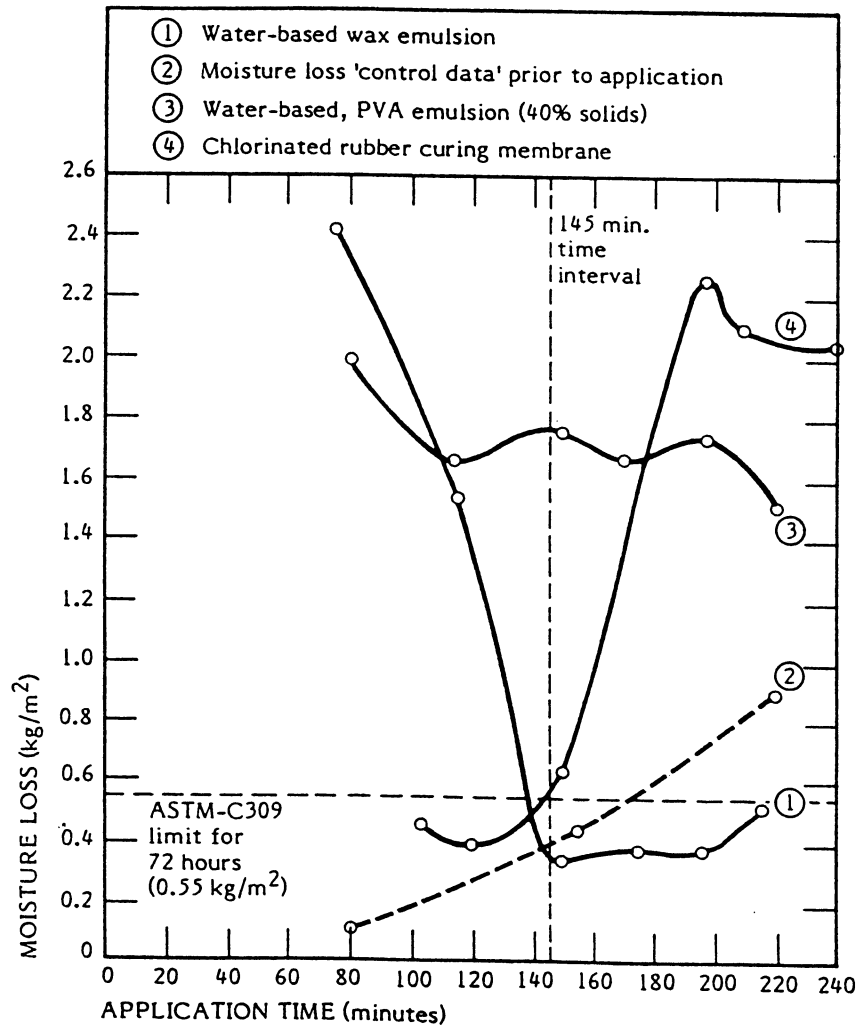


Figure 1: Water Loss Related to Delay in Time of Application

REFERENCES

- 1 Ayton G P. *Concrete Highway Pavements in Australia*. 5th International Conference on Concrete Pavement Design & Rehabilitation. Purdue University, Indiana USA. April 1993.
- 2 *Concrete Roundabout Pavements, a Guide to their Design & Construction*. Roads & Traffic Authority, New South Wales (Australia). April 1996.
- 3 Bernard E S and G Ayton. *Design and Performance of Steel Fibre Reinforced Concrete in Pavements*. Concrete Institute of Australia 18th Biennial Conference, Adelaide Australia, May 1997.
- 4 Verhoeven K. *Thin Overlays of Steel Fiber Reinforced and Continuously Reinforced Concrete: State of the Art in Belgium*. 4th International Conference on Concrete Pavement Design & Rehabilitation, Purdue University USA, April 1989.
- 5 Smith K D, D G Peshkin, M I Darter & A L Mueller. *Effect of Design Features on Concrete Pavement Performance*. 4th International Conference on Concrete Pavement Design & Rehabilitation. Purdue University USA, April 1989.
- 6 van Wijk A J & C W Lovell. *Erosion of Subbase Materials under Rigid Pavements*. 3rd International Conference on Concrete Pavement Design and Rehabilitation, Purdue University USA, April 1985.
- 7 Phu N C & M Ray. *The Erodability of Concrete Pavement Sub-base and Improved Subgrade Materials*. Bulletin de Liaison des Laboratoires des Ponts et Chaussees, Special Issue 8. July 1979.
- 8 Howard M D. *Erosion of Cement Treated Pavements*. Proceedings 14th Australian Road Research Board (ARRB) Conference, Canberra Australia, August 1988.
- 9 Zollinger D G & E J Barenberg. *A Mechanistic Based Design Procedure for Jointed Concrete Pavements*. 4th International Conference on Concrete Pavement Design & Rehabilitation. Purdue University USA, April 1989.
- 10 Janssen D J. Moisture in PCC. *66th Annual Transportation Research Board Meeting*, Washington DC, January 1987
- 11 Timms A G. Evaluating Subgrade Friction-Reducing Mediums for Rigid Pavements. *Transportation Research Record* No 60, TRB, National Research Council, Washington DC, 1964, pp 28-38.
- 12 Ioannides A M & R A Salsilli-Murua. *Interlayer and Subgrade Friction: A Brief Review of the State-of-the-Art*. Study for FHWA, Washington DC, August 1988.
- 13 Cather B. How to Get Better Curing. *Journal Concrete* Sept/Oct 1992. The Concrete Society's Materials Group, UK.
- 14 Woodstrom J H & B F Neal. *Curing Compounds for Portland Cement Concrete*. California Department of Transportation Laboratory. Final Report, December 1976.
- 15 Clarke S R. Test Method for Liquid Membrane-Forming Curing Compounds. *CIA News* (Concrete Institute of Australia) Vol 10 No 2, July 1984.
- 16 AASHTO *Guide for Design of Pavement Structures*. American Association of State Highway and Transportation Officials, Washington DC, 1986.
- 17 Transportation Research Board. *Distress Manifestations in Continuously Reinforced Concrete Pavement: Report of the Subcommittee*. TRB Information Series: Group 2, No 13, 1979, 8pp.
- 18 Verhoeven K. *Cracking and Corrosion in CRC Pavements*. 5th International Conference on Concrete Pavement Design & Rehabilitation, Purdue University USA, 1993.

- 19 PIARC. *Continuously Reinforced Concrete Pavements*. Technical Committee on Concrete Roads, Permanent International Association of Road Congresses, Paris France, 1994.
- 20 Timms A G. Evaluating Subgrade Friction-Reducing Mediums for Rigid Pavements. *Transportation Research Record* No 60, 1964, Pp 28-38.
- 21 Leask B E, H E Penn, E W Haber & B A Scala. Continuously Reinforced Concrete Across Clybucca Flat. *Australian Road Research Board (ARRB) Proceedings* Vol 9, 1978.
- 22 Vetter C P. Stresses in Reinforced Concrete due to Volume Changes. *Transactions, American Society of Civil Engineers*, 1933, No 8, Pp 1039-1080.
- 23 Haber E & J Cruickshank. *Design Procedure for CRCP Based on Theoretical Consideration and Service Behaviour*. 2nd International Conference on Concrete Pavement Design, Purdue University USA, 1981.
- 24 *Pavement Design – A Guide to the Structural Design of Road Pavements*. Austroads, Sydney Australia, 1992.
- 25 Chypin Chiang, B F McCullough, & W R Hudson. *A Sensitivity Analysis of Continuously Reinforced Concrete Pavement Model CRCP-1 for Highways*. Research Report No 177-2, Centre for Highway Research, University of Texas at Austin, August 1974, 106pp.
- 26 Ma J & B F McCullough. CRCP-2, An improved computer program for the analysis of continuously reinforced concrete pavements. *Research Report No 177-9*, Centre for Highway Research, The University of Texas at Austin, Aug 1977. 152pp.
- 27 Hunt J G. *Temperature Changes and Thermal Cracking in Concrete Pavements at Early Ages*. Cement and Concrete Association Technical Report 460, Wexham Springs, April 1972, 24pp.
- 28 Jackson P A. *Continuously reinforced concrete pavement: A literature review*. Transport and Road Research Laboratory (UK), Contractor Report 127 (1988).
- 29 McCullough B F. Criteria for the Design, Construction and Maintenance of Continuously Reinforced Concrete Pavement. *Australian Road Research*, Vol 13 No 2, June 1983, Pp 79-99.
- 30 Williams A. *Tests on large reinforced concrete elements subjected to direct tension*. Cement and Concrete Association, Technical Report 562, Wexham Springs, 1986. 56pp.
- 31 Nussbaum P J & E C Lokken. *PCC Pavements; Performance Related to Design, Construction & Maintenance*. Portland Cement Association (USA) for the FHWA, Final Report, August 1977.