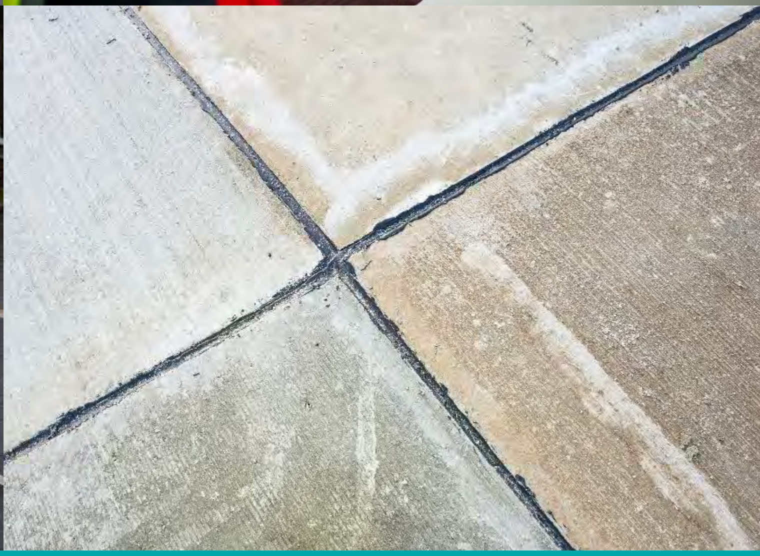
Concrete Pavement Maintenance Manual



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Foreword

Concrete pavements, or as our customers know them, road surfaces, can provide near maintenance free service for many years. However, as concrete pavements come to the end of their lives, they do not deteriorate in the more predictable way that flexible pavements do. When they do deteriorate, repairs are often much more complicated.

This new Concrete Pavement Maintenance Manual (CPMM) is the first of its kind for over 20 years and brings together the latest advice to provide comprehensive guidance on maintaining concrete pavements. It sets out the commonly used investigation techniques and how these can be used to investigate both the condition of, and diagnose defects in, concrete pavements.

This new manual includes descriptions and photographs of concrete pavement defects, examines the reasons why defects occur, and provides a step-by-step guide to repairs to facilitate right-first-time repair works. The manual also outlines the options available to improve the surface characteristics and structural capacity of concrete pavements.

By publishing this new manual as part of our concrete roads programme we will be able to maintain and improve these vital roads for years to come.

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Glossary and abbreviations

Glossary – General

Air entrainment: microscopic air bubbles intentionally incorporated in concrete during mixing, usually by use of a surfactant.

Aspect ratio: the ratio of the different dimensions of a shape.

Bay: section of jointed concrete pavement bound by longitudinal and transverse joints.

Bay replacement: full depth repair technique for jointed concrete pavements, involves the replacement of the full depth of concrete across single or multiple bays with joints reinstated at the same location as the original; where additional joints are introduced see 'full depth repair'.

Bond coat: proprietary polymer modified bituminous binder classified in accordance with BS EN 13808, BS EN 15322 or BS EN 14023.

Bound foundation: foundation incorporating one or multiple bound layers.

Bump cutting: mechanical removal of an area of pavement projecting above the level of the surrounding pavement to rectify a safety or serviceability issue i.e. poor ride quality or surface profile.

Chippings: single sized aggregate of nominal size between 3 mm and 20 mm. Applied to give surface texture.

Concrete: material formed by mixing cement, coarse and fine aggregate and water, with or without the incorporation of admixtures, additives and fibres, which develops its properties through a reaction between the cement and water.

Concrete pavement (or rigid pavement): pavement with a main structural layer of pavement concrete with either a concrete running surface or up to 50 mm asphalt surfacing.

Construction joint: a joint made in a concrete pavement at the end of a working day; similar joints may have to be introduced in an emergency when plant breaks down or when paving is stopped by bad weather.

Glossary – General

Continuously reinforced concrete pavement (CRCP): concrete pavement with continuous longitudinal steel reinforcement and no intermediate transverse expansion or contraction joints.

Continuously reinforced concrete base (CRCB): layer of continuously reinforced concrete, surfaced with a relatively thick (100 mm) asphalt overlay.

Contraction joint: a joint made in a concrete pavement to limit the effect of both contraction and bending on the pavement.

Crack and seat: process in which a concrete pavement is broken up into smaller 'bays' and rolled before overlaying in order to reduce relative movement at joints and cracks to inhibit reflective cracking of the subsequent overlay; aggregate interlock and load transfer is maintained between the small slabs as only hairline cracks are present between the 'bays'.

Crack stitching: treatment for longitudinal cracks (and joints), introducing additional tie bars along the crack (or joint) to maintain aggregate interlock and load transfer and prevent the crack (or joint) from widening further.

Crack inducer: an insert put into fresh concrete whilst it is still plastic to create a plane of weakness where it is intended for a crack to form that can subsequently be controlled and treated as a joint.

Deflection: the recoverable movement (elastic deformation) of the surface of a pavement under a transient load.

Design period: the number of years that a pavement is to carry a specific traffic volume and retain a serviceability level at or above a designated minimum value.

Design traffic: the commercial vehicle loading over the design period of a pavement; it is expressed as the number of equivalent 80 kN standard axles.

Dynamic plate test (DPT) device: equipment primarily used to assess the in situ performance and strength of pavements and foundations or granular materials using a test load dynamically applied through a circular plate and measuring the vertical deflections produced by the impact.

Early strength concrete: concrete designed to achieve adequate strength to permit trafficking within four hours after placement.

Glossary – General

Emergency repairs: rapid temporary repairs used to protect road users whilst minimising user disruption and delay.

Expansion joint: a joint made in a concrete pavement to limit the effect of both contraction and bending on the pavement, incorporating a compressible filler material that allows additional expansion of the concrete.

Falling weight deflectometer (FWD): a dynamic plate test (DPT) device for measuring the deflection response of road pavements. A load is applied to the pavement by dropping a mass onto a plate. Deflections are measured by sensors located at various positions relative to the load plate.

Formation: level upon which subbase is placed.

Foundation: all materials up to and including subbase.

Fractured slab techniques: techniques that require the concrete slab to be broken into smaller lengths prior to an overlay being applied. See ‘crack and seat’ and ‘saw-cut, crack and seat’.

Full depth repair: full depth reinstatement of a discrete portion of concrete, and subbase as necessary, introducing joints in new locations; where full depth reinstatement is undertaken and joints are replaced in the location of existing joints, see ‘bay replacement’.

Full depth reconstruction: a maintenance treatment that involves replacement of all the bound pavement layers, including the subbase.

‘Green’ concrete: a concrete that has set but not appreciably hardened.

Highways England: government-owned company charged with operating, maintaining and improving England's motorways and major A roads.

Hot rolled asphalt (HRA): dense, gap graded bituminous mixture in which the mortar of the fine aggregate, filler and high viscosity binder are major contributors to the performance of the laid material. Used as a surface course or in the lower pavement layers.

Induced crack: a full depth crack intentionally induced in a concrete slab by providing a joint groove in the top surface and (sometimes) a crack-inducing insert in the underside.

Glossary – General

Inlaid crack repairs: holding repair undertaken over static or working cracks to seal the cracks and prevent water and detritus from entering the crack.

Isolation joint: a joint made between a concrete pavement and another structure, such as a drainage channel or ironwork, incorporating a compressible filler material to limit compressive forces that may result in cracking or compression failures.

Joint groove: groove provided at the top of a joint to receive the joint sealant.

Joint sealant: flexible material that adheres to the vertical faces of the joint groove, excluding water and detritus from entering the joint whilst accommodating opening and closing of the joint.

Jointed reinforced concrete pavement (JRC): concrete pavement with transverse joints at intervals generally from 7 m to 25 m. These joints may be either contraction or expansion joints. The pavement contains steel reinforcement within the bays in both the longitudinal and transverse directions.

Jointed unreinforced concrete pavement (URC): concrete pavement with transverse joints at intervals generally from 4 m to 6 m. The slabs do not contain steel reinforcement and the load transfer across transverse joints is provided either by round steel dowels (dowelled) or by aggregate interlock only (undowelled). Also referred to as unreinforced concrete pavement.

Laitance: a weak, milky layer of cement and aggregate fines on a concrete surface that is usually caused by an overly wet mixture, overworking the mixture, improper or excessive finishing or combination thereof.

Light weight deflectometer (LWD): a portable dynamic plate test (DPT) device for measuring the deflection response of exposed pavement foundations and subgrades. A load is applied to the layer by dropping a mass onto a plate and the deflection is measured by a sensor mounted on the equipment.

Load transfer: the distribution of load to an unloaded slab that occurs when the slab on the other side of a joint (or crack) is loaded.

Load transfer efficiency (LTE): the efficiency at which load is transferred from a loaded slab across a joint or crack to an unloaded slab. Typically measured with a falling weight deflectometer (FWD) as the ratio (expressed as a percentage) of vertical deflection of an unloaded slab (adjacent to a joint) to the deflection of an abutting loaded slab.

Glossary – General

Long life pavement: pavements designed to not require structural strengthening for 40 years.

Maintenance: treatment that helps slow the rate of deterioration by identifying and addressing specific pavement deficiencies that contribute to overall deterioration.

Next generation concrete surfacing: longitudinal noise-attenuating texture treatment; utilising longitudinal grinding and grooving techniques to provide a predominantly negatively textured surface.

Overlay: new material placed directly onto the surface of a pavement.

PAK test: a test, normally using a spray marker, to determine the presence of polycyclic aromatic hydrocarbons (PAH) within a bound layer; presence of PAH is an indication that materials are tar-bound.

Partial depth reconstruction: replacement of all the of the bound pavement layers, excluding the foundation.

Pavement concrete (or pavement quality concrete (PQC)): standard paving concrete in accordance with BS 8500-1 & -2.

Polymer modified bitumen (PMB): bitumen modified through the incorporation of one or more organic polymers.

Reconstruction (or recon): see ‘full depth reconstruction’ and ‘partial depth reconstruction’.

Renewal: process to improve the structural condition of the pavement and extend pavement life.

Retexturing: surface treatments that mechanically rework or reshape a sound road surface to restore skid resistance and / or texture depth; some of these techniques can also improve noise characteristics and profile.

Retrofitting dowel bars: technique to improve or restore load transfer efficiency across joints by incorporating new dowel bars.

Rigid pavement: see ‘concrete pavement’.

Glossary – General

Saw-cut and seal: treatment that introduces joints into an asphalt overlay at the location of the underlying joints to control reflective crack propagation.

Saw-cut, crack and seat: as 'crack and seat'; however, saw cutting is undertaken initially in JRC pavements to sever the existing reinforcement.

Shallow repair: treatment involving the removal of a partial depth (between 40 mm and one third of the slab depth) of deteriorated concrete and replacement with a suitable repair material.

Slab: the main structural element of a concrete pavement.

Slab lifting: treatment option to lift the level a discrete portion of concrete pavement to realign it with the rest of the carriageway.

Strategic road network (SRN): nationally significant roads used for the distribution of goods and services, and a network for the travelling public; managed by Highways England and, in legal terms, it can be defined as those roads which are the responsibility of the Secretary of State for Transport; any road on the SRN is known as a trunk road.

Stress absorbing membrane interlayer (SAMI): proprietary flexible material that is usually placed at or near the bottom of an asphalt overlay to reduce the tensile stress in the overlay in the vicinity of any joints or cracks in the underlying concrete pavement and hence minimise reflective crack propagation. Different types of SAMI include:

Asphalt SAMI: asphaltic layer with enhanced viscoelastic characteristics in comparison to conventional asphalt. Comprising fine aggregates and polymer modified bitumen (PMB), they are installed with a paving machine.

Spray applied SAMI: layer of bituminous material with enhanced viscoelastic characteristics compared with a conventional bond coat. May incorporate various fibres, applied with a spray tanker.

Geosynthetic SAMI: bitumen-saturated paving fabric (non-woven or purpose-built composite incorporating a grid) bonded in between two courses for the purposes of stress relief. The bitumen component may have enhanced viscoelastic characteristics compared with a conventional bond coat.

Subbase: a platform layer upon which the main structure of a pavement is constructed; the subbase is both part of the foundation and pavement.

Glossary – General

Subgrade: soil or fill underlying a pavement.

Substrate: prepared surface on which a material is placed.

Texture: visible and tangible characteristic of a surface. Categorised as:

Microtexture: the microscopic properties of the surface that enable it to develop friction. The dominant factor in providing wet skidding resistance at low speed.

Macrotexture: the visible roughness of a surfacing material, enabling drainage of water and dissipation of noise. Factor in wet skidding resistance at high speed.

Megatexture: the degree of smoothness of the surface with wavelength between 50 mm and 500 mm.

Texture depth: measure of macrotexture of a surface.

Thin surface course system (TSCS): proprietary asphalt mixtures that are paver-laid onto a bond or tack coat to form, after compaction and cooling, a textured surface course less than 50 mm in thickness.

Thin bonded repair: treatment involving the removal of a partial depth (no more than 40 mm deep) of deteriorated concrete and replacement with a suitable repair material.

Treatment: the use of a material, procedure or method to improve pavement characteristics or repair or mitigate pavement deficiencies that contribute to overall deterioration.

Unbound foundation: foundation incorporating only unbound layers.

Under slab grouting: treatment where voids below pavement slabs are filled with a cementitious or resin grout to restore slab support and slow the deterioration of the pavement.

Vehicle restraint system: system installed on the road to provide a level of containment for an errant vehicle.

Warping joint: formed along the longitudinal joint to allow the pavement to 'hinge' but incorporate tie bars to prevent opening of the joint and provide load transfer across the joint.

Glossary - Pavement defects

Adhesion failure: failure of the joint seal through loss of adhesion between the sealant and the vertical faces of the concrete in the joint groove.

Cohesion failure: failure of the joint seal where cracks have occurred in the sealant at right-angles or parallel to the joint groove.

Compression failure ('blow-up'): localised upward movement or shattering of a slab at a transverse joint or crack; they generally occur when a crack or joint is not wide enough to permit thermal expansion of the concrete slabs.

Corner crack: a crack across a corner of a slab with crack length between 0.3 m - 2 m. If larger than 2 m, it is considered a diagonal crack (see 'diagonal cracks').

Cracks: a split or break in the pavement. May be shallow (see 'crazing' and 'plastic shrinkage cracking') or extend through the full pavement depth. Where extending through the pavement depth, cracks are categorised as:

Hairline cracks: cracks that are very narrow and detectable only with difficulty.

Narrow cracks: cracks less than 0.5 mm wide. Aggregate interlock across the crack likely maintained. Likely too narrow to permit ingress of water, de-icing salts and incompressible materials.

Medium cracks: cracks between 0.5 - 1.5 mm wide. Partial aggregate interlock, but may permit ingress of water, de-icing salts and incompressible materials.

Wide cracks: cracks exceeding 1.5 mm wide. Assumed to have no aggregate interlock and any reinforcement present likely to have yielded.

Cracks around ironwork: cracks that originate from ironwork such as surface water gullies and manholes penetrating through a concrete pavement slab. Controlled with an isolation joint.

Crazing (or 'map cracking'): network of shallow narrow cracks which extend only through the upper surface of the concrete.

Deep joint spall: a breakdown of the surface material at the slab edges / joints within 600 mm of the joint / corner, typically intersecting the joint at an angle and extending more than one third of the slab depth.

Defect: a shortcoming, imperfection, or deficiency. In pavements, a defect refers to evidence of an undesirable condition affecting the serviceability, structural condition or appearance.

Glossary - Pavement defects

Defective joint seal: all-encompassing term for joint seals which are not performing their function.

Depressions (or settlement): localised pavement areas with downward differential displacement compared with the surrounding pavement.

Diagonal crack: a crack that traverses between perpendicular joints in slabs and greater than 2 m in length; cracks traversing perpendicular joints less than 2 m are considered to be corner cracks (see 'corner cracks').

Feature: a distinctive attribute of a pavement that is expected under specific conditions.

Fatigue: damage to a pavement caused by repeated application of small stresses (traffic loading).

Heave: localised upward differential displacement of the pavement as a result of action from the pavement foundation.

Inherent or 'normal' (transverse) cracks in CRCP: regular closely spaced (at least 1 m apart) transverse cracks in CRCP ≤ 1 mm in width, with no spalling or bifurcations.

Longitudinal crack: a crack that is oriented roughly parallel to the pavement edge or longitudinal joint.

Plastic shrinkage cracking: surface cracks that form before the concrete hardens in a pattern of short cracks usually approximately parallel to each other, oriented diagonally to the bay sides and not extending to the edges of the slab.

Polygonal cracking: three or more intersecting cracks forming a closed plan resulting in a fragment of concrete isolated from the rest of the slab, likely to result in a punchout.

Ponding: shallow pools of water that accumulate during rain and, because of surface irregularity or inadequate gradient, disperse by draining slowly and / or evaporation.

Pop-out: isolated loss of surface material in a small area.

Glossary - Pavement defects

Pumping: water-borne fine-grained particles of subbase or subgrade material that have become saturated and are pumped out of joints or cracks where there are vertical deflections associated with heavy vehicular loading and insufficient support of the slab. Pumping can be seen as staining on a dried concrete pavement surface. As a consequence of these conditions, a void is formed in the subgrade and the volume of this void grows where these conditions continue.

Punchout: localised defects in which intersecting transverse and longitudinal cracks or joints create in loose fragments of concrete, normally at the pavement edge, which are 'punched out' by the action of traffic downwards into the underlying subbase layer.

Reflective crack: a crack in a concrete or asphalt overlay induced by movement at joints or cracks in an underlying layer.

Shallow joint spall: a breakdown of the surface material at the slab edges / joints within 600 mm of the joint / corner, intersecting the joint at an angle, extending up to one third of the slab depth.

Slab rocking: vertical movement at joints (or cracks) that occurs under the action of traffic loading.

Spall: breakdown of the surface material at joints. Can also occur at cracks.

Stepping: a difference in elevation across a joint or crack greater than 3 mm.

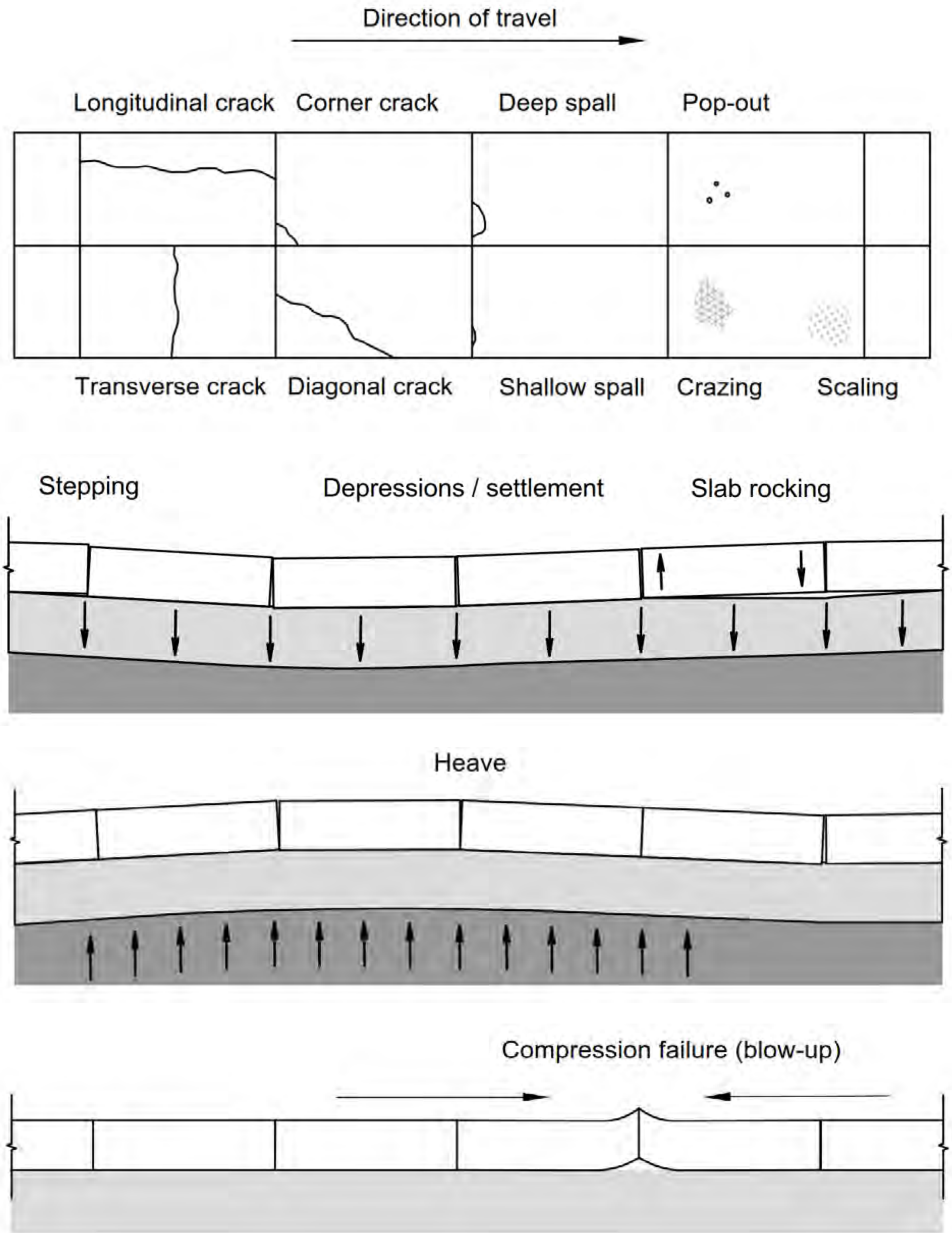
Surface irregularities: bumps or depressions in the pavement.

Surface scaling: delamination or disintegration of the slab surface to the depth of the defect, occurring over a portion of a slab.

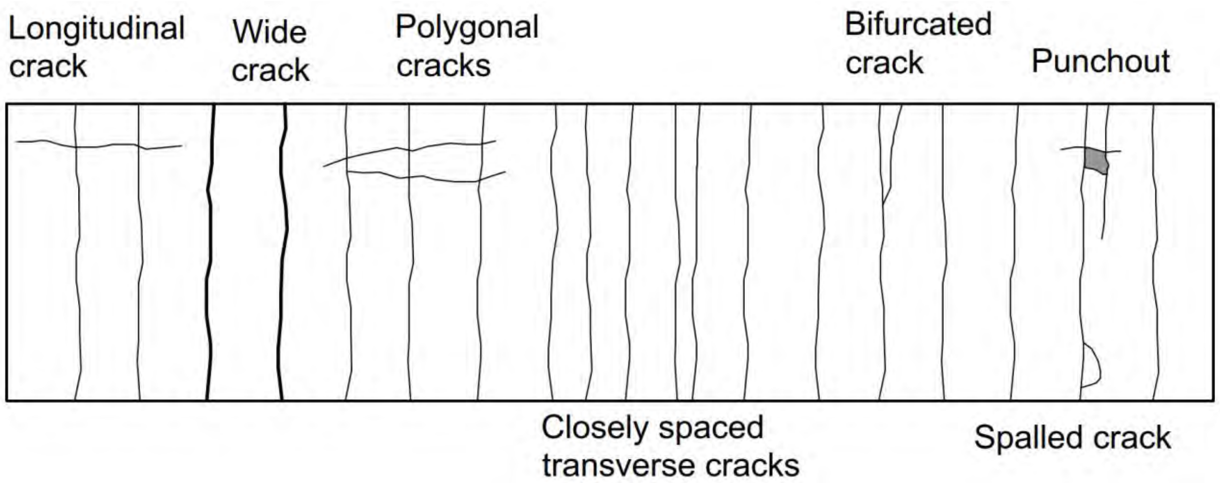
Sympathetic crack: a crack induced in a slab by movement at a joint or crack in an adjacent slab or a structure within or adjacent to the slab, such as ironwork.

Transverse crack: a crack that is roughly orientated perpendicular to the pavement centreline.

Diagrams - Concrete pavement defects



Diagrams - Concrete pavement defects – CRCP



Abbreviations

AD	Absolute deflection
ASR	Alkali-silica reaction
BS	British standard
CAD	Computer-aided design
CAUTS	Cold applied ultra-thin surfacing
CPMM	Concrete pavement maintenance manual
CRCP	Continuously reinforced concrete pavement
CRCB	Continuously reinforced concrete base
CSO	Crack, seat and overlay
DCP	Dynamic cone penetrometer
DPT	Dynamic plate test
DMRB	Design manual for roads and bridges
ECI	Early contractor involvement
EN	European norm (standard)
FWD	Falling weight deflectometer
GGBS	Ground granulated blast-furnace slag
GIS	Geographic information system
GPR	Ground-penetrating radar
HAPMS	Highways England's pavement management system
HBM	Hydraulically bound mixture
HBGM	Hydraulically bound granular mixture
JRC	Jointed reinforced concrete pavement
LCCA	Life cycle cost analysis
LiDAR	Light detection and ranging
LTE	Load transfer efficiency
LWD	Light weight deflectometer
MCHW	Manual of contract documents for highway works
MIT	Magnetic imaging tomography

MFV	Multi-function vehicle
msa	Million standard axles
NGCS	Next generation of concrete surface
NDT	Non-destructive testing
PMB	Polymer modified bitumen
PMS	Pavement management system
PQC	Pavement quality concrete
RCC	Roller compacted concrete
RCMI	Reflective crack mitigation interlayers
SAMI	Stress absorbing membrane interlayer
SRN	Strategic road network
TRACS	Traffic speed condition survey machine
TSCS	Thin surface course system
URC	Unreinforced concrete pavement
VCS	Visual condition survey (for concrete surfaced roads)
VI	Void intercept
VRS	Vehicle restraint system
WLC	Whole-life cost

1. Introduction

The Concrete Pavement Maintenance Manual (CPMM) provides comprehensive guidance on the maintenance of concrete pavements, including investigation to determine condition, identification of defects and causation, and maintenance options.

The CPMM has been developed to promote the efficient maintenance of concrete pavements within the Highways England legacy concrete pavement repair and renewal programme, and support good practice in meeting the requirements set in standards, in particular DMRB CD 227 [1], CS 228 [2], CS 229 [3], CS 230 [4], CM 231 [5], and MCHW Series 700 [6] and 1000 [7].

It follows on from the first edition, which was originally published in 2001 [8]. The content has been updated and restructured. Investigation and repair techniques, which are now standard practice, and findings from the monitoring of the strategic road network (SRN) over the past two decades and recent trials, have been incorporated.

In the CPMM, concrete pavements are those with an exposed concrete surface and those with up to 50 mm of asphalt surfacing; although the content may also be applicable to concrete pavements with a thicker asphalt surfacing.

Whilst written with specific focus on maintaining the concrete pavements on the Highways England SRN, the CPMM has wider applicability for the maintenance of other concrete pavements, such as those on the local authority road network, airfield pavements, and concrete hardstanding areas.

The CPMM contains photographs illustrating deterioration and defects that typically occur in concrete pavements. These are accompanied by advice on diagnosing the underlying cause of the defect using a wide range of established investigation and monitoring techniques.

Suitable repair techniques and materials are assigned to defects and guidance is given on applicability in consideration of the defect causation and factors such as asset management strategies, life cycle cost, efficiency, risk mitigation and appropriate uptake of innovation. Step-by-step guidance, figures and photographs of the repair techniques are included to facilitate effective, durable and right-first-time repairs.

The CPMM also provides information on the investigation and monitoring of the condition in terms of safety, serviceability and structural capacity of concrete pavements, which is complemented by figures and photographs of the techniques available.

Premature failure of repairs to concrete pavements have been observed on the SRN. The CPMM provides guidance which aims to minimise early failures of repairs through understanding defect causation and selection of the appropriate methods and materials. This should result in increased productivity, reduced disruption and smoother journeys for road users.

Guidance is consolidated within the CPMM on the maintenance of concrete pavements to address more widespread surface and structural defects. Techniques and materials outlined in the CPMM are contextualised in terms of their technology readiness level [9] to enable users to understand current and near future capability.

1.1. Background

Concrete pavements represent approximately 4 % of the SRN (see Figure 1.1), this mostly consists of jointed concrete pavements (both unreinforced and reinforced) with some sections of continuously reinforced concrete pavement. Jointed concrete pavements comprise a series of concrete bays separated by regular contraction joints and less frequent expansion joints, whereas continuously reinforced concrete pavement does not have intermediate joints.

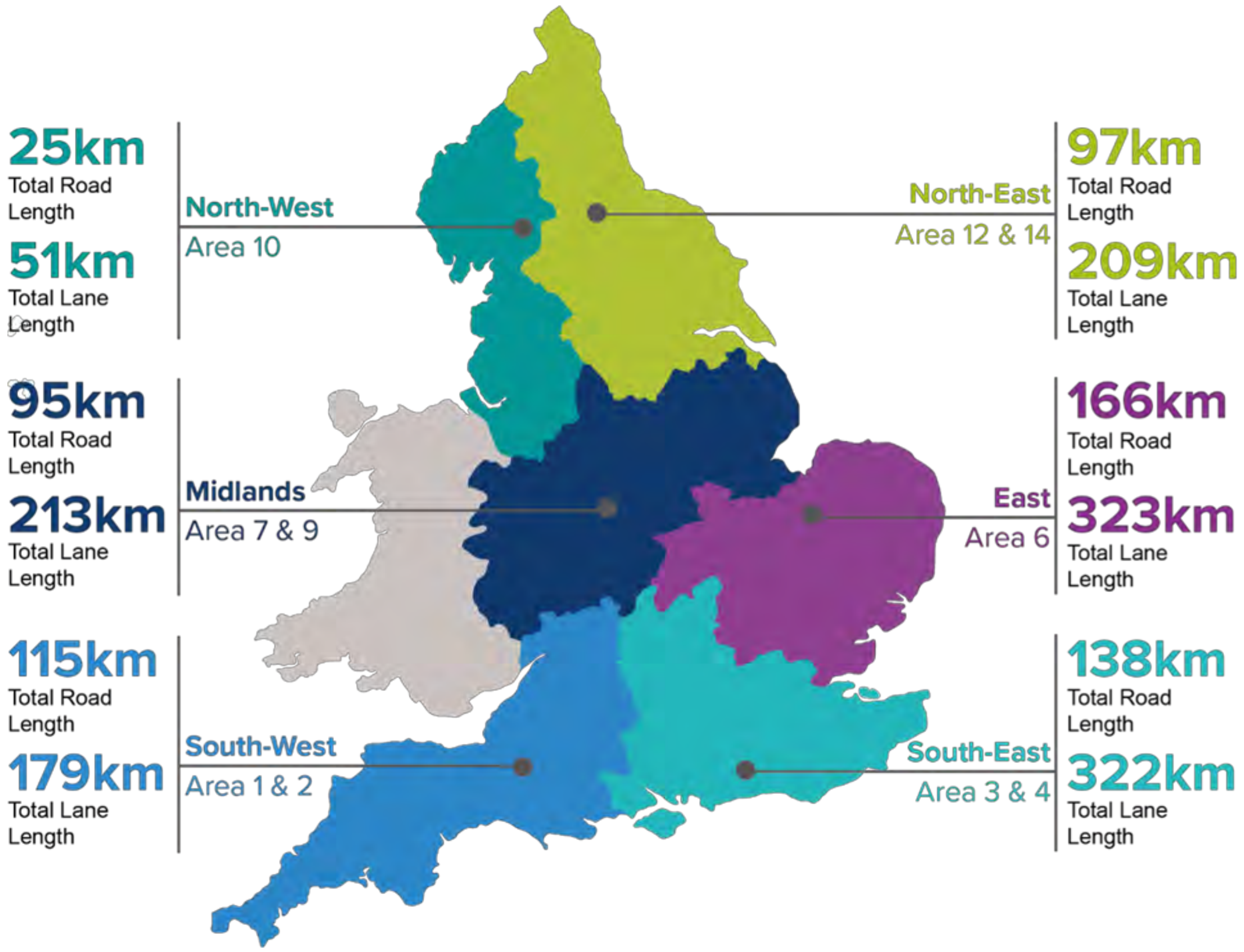


Figure 1.1 Highways England legacy concrete pavement extents by region [3]

Concrete pavements can provide near maintenance free service for many years; however, they do not deteriorate in the same, generally, predictable manner as flexible pavements. Jointed concrete pavements tend to deteriorate and fail at the joints between the bays. These failures can occur quickly and are difficult to predict, particularly when the pavement is at the end of life. Some failure mechanisms can be critical to road user's safety; therefore, it is important to maintain the asset in a timely manner to mitigate risk. Repairs to these failures can be expensive and complicated to undertake and often cause significant disruption.

The sections of jointed concrete pavement on the SRN were mostly constructed between 1950 and 1985 and have provided many years of satisfactory service with limited maintenance but are now beyond or nearing the end of their design life and design loading. Some of these pavements have exceeded their design life by as much as 20 years. Concrete pavements approaching or exceeding their design lives are known as ‘legacy concrete pavements’.

The maintenance of these legacy concrete pavements traditionally was limited to surface treatments and patching to ensure the safety of road users. However, the number of unplanned interventions on these legacy concrete pavements to maintain a safe surface increased. This offered poor value for money and impacted network availability. Therefore, during road period 2 (RP2) (2020 to 2025), Highways England began a programme to transform the legacy concrete pavement asset to long-life pavements.

1.2. Concrete pavements

Concrete is one of the most commonly used building materials in the world. It is used for a range of highway applications such as bridges, kerbs, retaining walls, buried foundations, pavements and drainage. Concrete is formed by mixing cement, coarse and fine aggregate, water, and sometimes admixtures and additives. It develops its properties through a reaction between the cement and water, which produces a hard mass that binds the aggregates together.

Concrete is inherently strong in compression, but weak in tension. It shrinks during curing and expands and contracts with changes in temperature. Stresses and deformations in concrete layers are generated from both traffic loading and environmental loading (e.g. temperature variations). Stresses generated by repeated loading applications over time can eventually lead to fatigue failures such as cracking.

Deterioration may be further exacerbated at cracks if they are full depth. A wheel load applied at the edge of a concrete slab generates a much higher stress in the concrete than if the same load was applied in the centre of a slab [10]. Cracks that extend through the full thickness of the concrete can effectively create edge loading conditions, so deterioration of the concrete will be further accelerated unless the load can be transferred across the crack.

Load transfer is the phenomenon whereby load is transferred across discontinuities which are connected through a natural (interlocking of aggregate) or designed (devices which span joint or cracks) mechanism. Indeed, where there is still aggregate interlock between a crack (when it is narrow (< 0.5 mm wide [10])), over 70 % of an applied load can be transferred across the crack, thereby reducing the stress generated in the slab [11].

Unsealed cracks greater than approximately 0.5 mm wide also make the concrete structure vulnerable to ingress of external substances including de-icing salts (corrosive to any steel in the concrete), water and ice, which can accelerate deterioration. Further deterioration can occur if incompressible materials enter cracks, filling the space into where the concrete normally expands.

The timing and type of cracking is influenced by factors including:

- early life initial shrinkage during curing of the installed pavement;
- repeated traffic loading generating material fatigue;
- expansion and contraction with temperature variations;
- cyclic warping through temperature gradients across slabs;
- insufficient strength or thickness relative to applied stresses;
- non-uniform support; and,
- being restrained from contraction and expansion.

Daily and seasonal expansion and contraction of the concrete that would otherwise cause potentially wide, bifurcated, random cracking can be accommodated by forming joints at appropriate frequencies where these daily and seasonal movements can occur. Alternatively, the pavement can be designed and constructed with less regular or no joints, but instead designed and constructed to allow narrow cracks (< 0.5 mm in width) to form at regular intervals (between formed joints) where this daily seasonal movement can occur. As the cracks are narrow, load transfer across the cracks is retained and foreign materials cannot enter into the cracks. This is achieved by:

- using the appropriate amount of steel reinforcement;
- limiting restraint from the underlying layers;
- material specification; and,
- good construction practices i.e. curing.

The design life of a concrete layer before significant cracking occurs is linked to the properties of the concrete (primarily tensile strength), its thickness, restraint and use of any steel reinforcement, and devices to transfer load across slabs (dowel bars and tie bars). Steel reinforcement is used to give additional tensile strength to the concrete layer, mitigating cracking by holding the concrete tightly together across the crack to maintain aggregate interlock.

The stress generated in a concrete slab partly depends on the stiffness ratio between the slab and its underlying support. To increase the pavement lifespan, a bound foundation is preferred as it erodes less readily than an unbound foundation, is less water susceptible and has a higher stiffness that provides additional structural contribution. There are, however, several existing sections of concrete pavement on the SRN with unbound foundations.

As different methods are available for controlling cracking, various types of concrete pavement are used. The three common types of concrete pavement on the SRN, and covered in the CPMM are:

- jointed unreinforced concrete;
- jointed reinforced concrete; and,
- continuously reinforced concrete.

Roller compacted concrete (RCC) has a history of use internationally and was introduced to the UK in 2002, with the material being used mainly to construct hardstanding areas for the waste industry, bulk materials handling and heavy goods vehicle parking where it is used as a surfacing. Since 2020, RCC has been a standard option for new pavement designs on the SRN with an asphalt surfacing [12]. However, the maintenance of RCC is not covered in the CPMM as its performance behaviour differs from conventional concrete pavements.

1.2.1. Jointed unreinforced concrete (URC)

Jointed unreinforced concrete (URC) pavements (see Figure 1.2) have transverse joints induced or constructed at regular intervals to prevent excessive stresses occurring in the concrete that would result in the occurrence of random and relatively wide crack patterns.

The joints are sealed to prevent foreign materials entering the void space and water from reaching the lower pavement layers. These joint seals have finite lifespans and require replacement at regular intervals to avoid deterioration of the pavement.

The following three types of movement joints are used, they all permit warping movement (i.e. rotation of the joint faces):

- transverse contraction;
- transverse expansion; and,
- longitudinal warping joints.

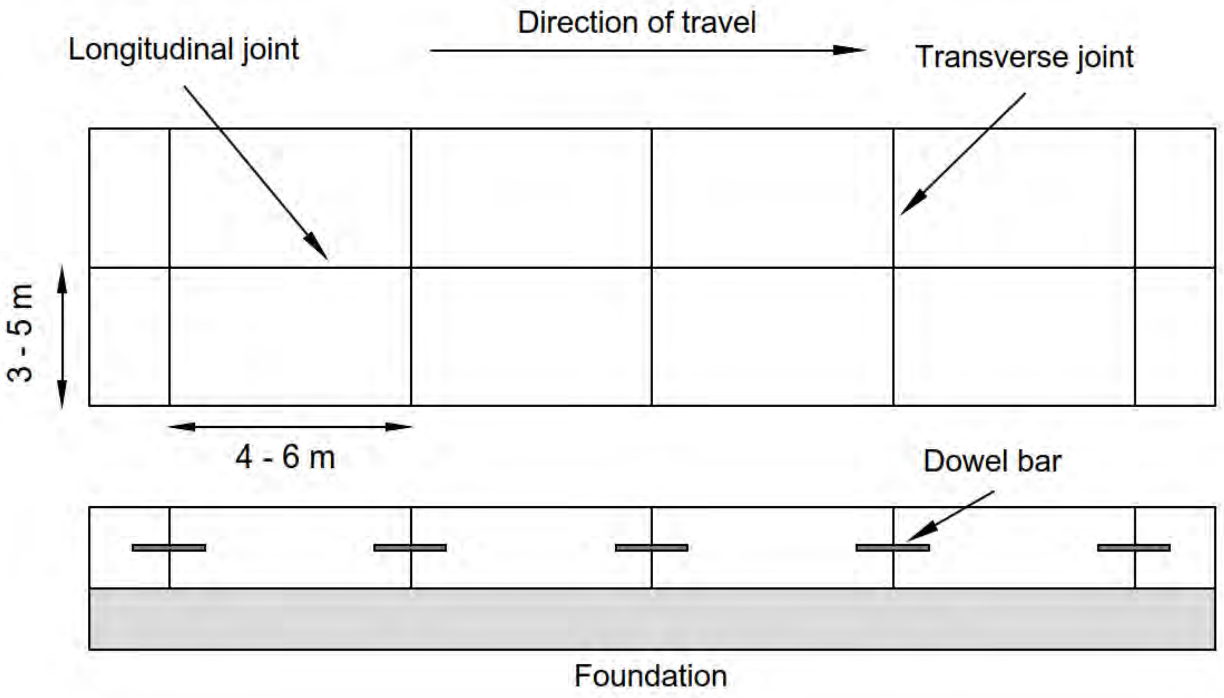


Figure 1.2 Typical URC construction

Transverse contraction joints (see Figure 1.3) enable the slab to contract during the initial curing period and thereafter when its temperature decreases; and to expand when its temperature increases.

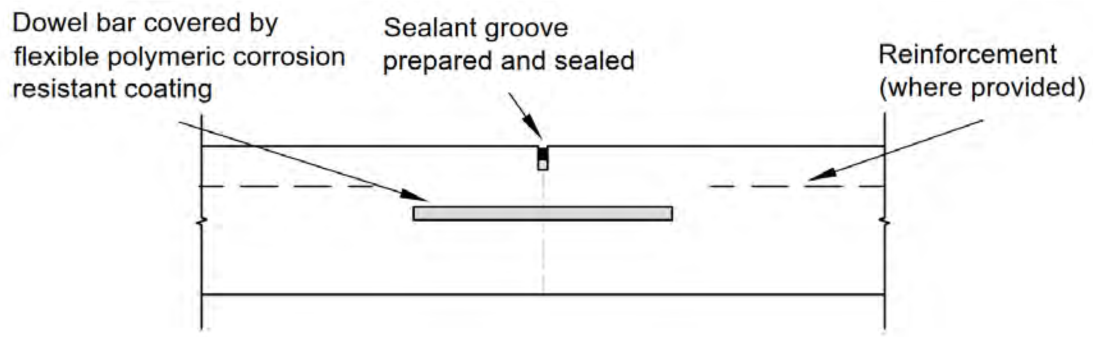


Figure 1.3 Transverse contraction joint

Transverse expansion joints (see Figure 1.4) cater for the expansion that would naturally occur at temperatures higher than that of the concrete when the slab was laid. They also allow the slab to contract. Major highways in the UK normally omit expansion joints unless there are special circumstances, such as at fixed structures, around ironwork (also known as isolation joints) and within transitions to fully flexible pavement construction.

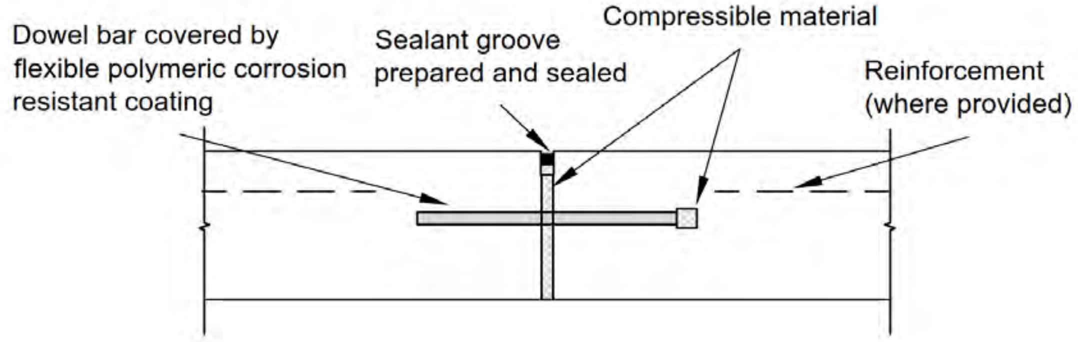


Figure 1.4 Transverse expansion joint

On the SRN, transverse joints are bridged by dowel bars to ensure that the contraction and expansion movement at joints is perpendicular to the joint length, that there is reasonably good load transfer between successive slabs and that steps do not form in the surface profile. This may not be the case for this type of pavement in other applications, for example most UK airfield pavements do not use dowel bars.

The required spacing of transverse joints for URC depends on slab thickness and aggregate type. For example, concrete incorporating limestone aggregate has a lower coefficient of thermal expansion than for most other aggregate types. As less expansion and contraction occurs, the spacing between joints can be greater.

Longitudinal warping joints (see Figure 1.5) use tie bars to ‘tie’ the slabs together and can be thought of as acting as ‘hinges’. They permit relative rotation between adjoining slabs but not opening or closing of the joint.

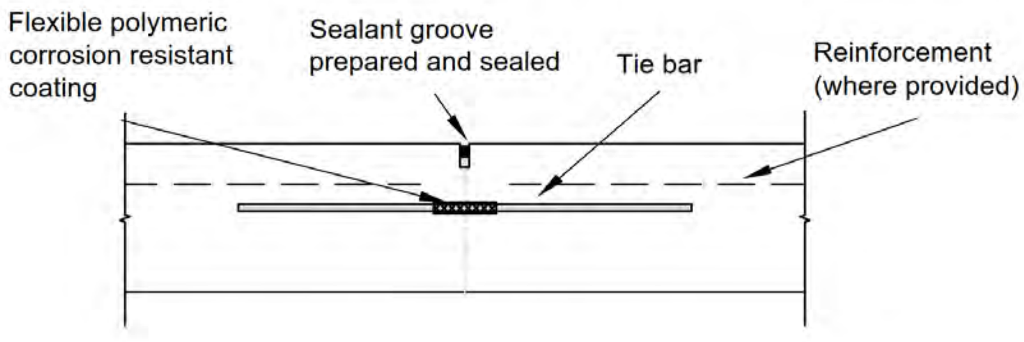


Figure 1.5 Longitudinal warping joint

Other variations of concrete joints may be used on heavy duty pavements such as airports and ports. Examples include movement joints without load transfer devices and keyed joints to provide interlock between bays. ‘Longitudinal’ joints may also be dowelled to provide additional load transfer where regular trafficking is anticipated along or across the line of the joint, such as lane gain / widening or lane drop / narrowing locations, junctions and laybys.

1.2.2. Jointed reinforced concrete (JRC)

Jointed reinforced concrete (JRC) pavements (see Figure 1.6) are constructed in a similar manner to unreinforced concrete pavements but differ in that steel reinforcement is added.

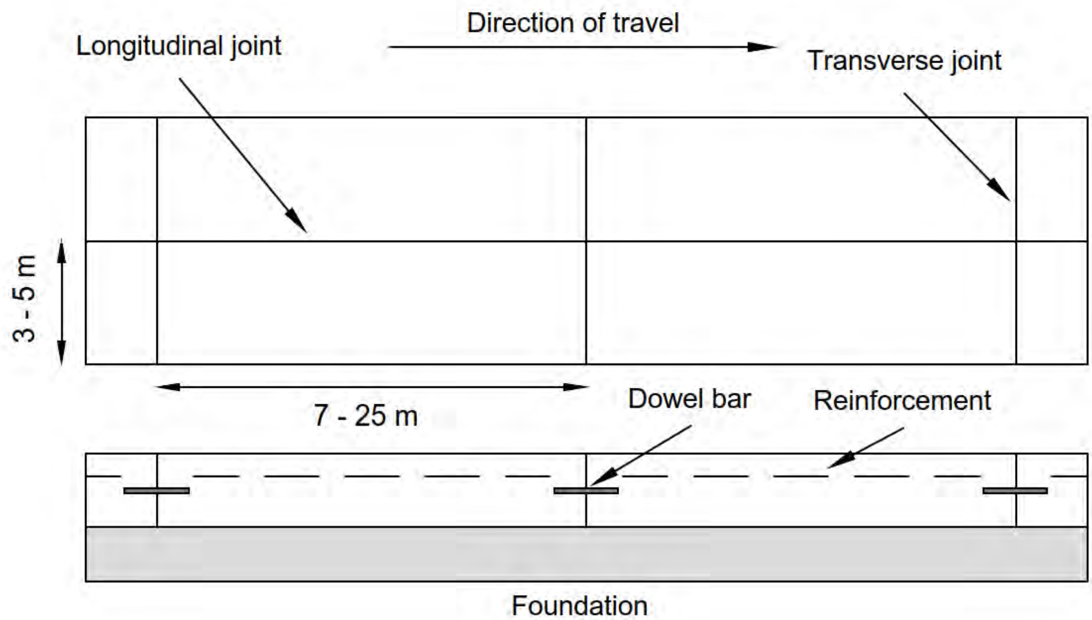


Figure 1.6 Typical JRC construction

The reinforcement is used for two functions:

- ensuring cracking remains narrow; and,
- reducing the thickness of the slab by offsetting thermal stresses during the pavement life.

Fundamentally, reinforcing the concrete allows the length of slabs to be significantly increased compared with URC, with transverse joints permitted at spacings up to 25 m depending on the quantity and type of reinforcement, its location within the slab, as well as slab thickness and aggregate type.

In JRC, transverse cracking is expected at regular intervals (approximately 4 m) shortly after construction in between the transverse joints; however, the reinforcement is designed to hold these cracks tightly together so that they are only hairline cracks (< 0.5 mm wide).

1.2.3. Continuously reinforced concrete pavement (CRCP)

Continuously reinforced concrete pavements (CRCP) (see Figure 1.7) do not have regular transverse joints. Instead, regular cracks are allowed to form naturally because continuous steel reinforcement is incorporated which is designed to hold these cracks together. This ensures they are only very fine (hairline) cracks, small enough to maintain load transfer and to not allow water and other substances to enter the cracks. The reinforcement at pavement ends is anchored into ground beams or a universal steel beam anchorage to provide tension on the reinforcement.

CRCP is designed to develop a fine transverse crack pattern soon after the concrete is laid. Initially, the crack spacing is about 3 or 4 m. Over the following four years or so, further transverse cracks slowly develop between the wider spaced cracks, resulting in a typical crack spacing of 1 to 2 m. The crack pattern in CRCP is closely related to the sub-surface friction, the aggregate used, the strength of the concrete and the proportion and type of steel reinforcement used and its location in the slab.

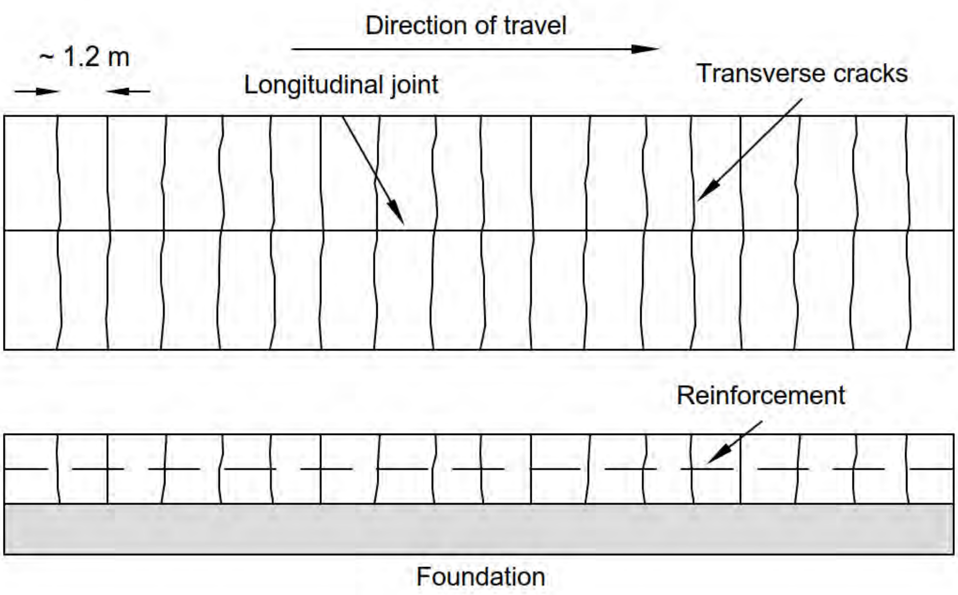


Figure 1.7 Typical CRCP construction

1.3. Structure of this manual

The CPMM has the following chapters:

- Chapter 2: Investigation techniques
- Chapter 3: Defects and features
- Chapter 4: Defect causation and diagnosis
- Chapter 5: Repair techniques
- Chapter 6: Repair materials
- Chapter 7: Restoration

Chapter 2 discusses the techniques which can be used to investigate the condition of concrete pavements.

Chapters 3 through 6 focus on the correct identification and diagnosis of localised defects using the investigation techniques discussed in Chapter 2, linking them with the appropriate repair techniques and materials dependent on the maintenance strategy.

Chapter 7 outlines restoration techniques for concrete pavements to address surface and structural deficiencies over extended sections of pavement.

Appendix A outlines innovations in concrete pavement maintenance. Appendix B summarises the treatment options available for the defects discussed in Chapter 3. Appendix C provides detail on Highways England’s 3D process specific to concrete pavements and Appendix D supplements the CPMM with a visual guide to concrete pavement distresses, containing additional photographs to aid identification.

1.4. Applicability of this manual

The CPMM has been developed for use on the Highways England SRN and the terminology used and guidance offered is specific for this application. However, the contents of the CPMM may also be applicable to other organisations in the UK and internationally.

The requirements for diagnosing the cause of defects and designing treatments to concrete pavements on the SRN are contained within the Design Manual for Roads and Bridges (DMRB). The performance requirements and techniques for maintenance of concrete pavements on the SRN are contained within the Manual of Contract Documents for Highway Works (MCHW).

The CPMM incorporates investigation techniques and treatments for concrete pavements identified from the 2020 UK SRN innovation trials that may be used on the SRN under a ‘departure from standard’ application. Techniques which require a departure from standard at the date of publication of the CPMM are identified.

The technology readiness levels 1 - 9, assigned as necessary to maintenance, repair, restoration at the date of publication of the CPMM, are described in Table 1.1 [9].

Table 1.1 Technology readiness levels

Technology readiness level	Description	Assessment implication & further work recommendation
1	Basic principles observed and reported.	(Further) Laboratory investigation and validation.
2	Technology concept and / or application formulated.	
3	Analytical and experimental critical function and / or characteristic proof-of-concept.	
4	Technology validation in a laboratory environment.	Demonstration / validation of concept trial (off SRN).
5	Technology basic validation in a relevant environment.	Trafficked demonstration / validation of concept trial (off SRN).
6	Technology model or prototype demonstration in a relevant environment.	Demonstration / validation of concept trial (on SRN).
7	Technology prototype demonstration in an operational environment.	Departure from standard authorised on project basis.
8	Actual technology completed and qualified through test and demonstration.	Departure from standard authorised on project basis. Standard / specification in development.
9	Actual technology qualified through successful mission operations.	Approved for network use. Standard / specification published.

1.5. Maintenance strategy and treatment selection

Pavements require a programme of maintenance to ensure that they:

- remain safe to use and are serviceable; and,
- continue to have the potential to achieve their original structural design life.

The need for maintenance depends on the pavement durability and its performance during the use phase of its asset lifecycle. These are affected by factors including the design, construction, traffic, drainage and the environment (e.g. temperature, severe weather and water). These factors, individually as well as combined cause and accelerate pavement deterioration. Maintenance is ideally targeted to prevent or reduce any unacceptable rates of deterioration.

The selection and timing of maintenance actions can greatly influence the cost and effectiveness of the action as well as the overall pavement life [13], and it should be targeted to prevent or reduce any unacceptable rates of deterioration. Maintenance options have their optimum ‘window of opportunity’ and in many cases, a less costly strategy adopted earlier can give significant life cycle benefits.

An example of neglecting targeted maintenance is not undertaking joint resealing until a point when water and material infiltration are allowed to cause and / or accelerate to occurrence of structural defects (e.g. poor slab support, joint spalling). At the point when structural defects exist which require treatment, maintenance costs could be dramatically higher. Not maintaining linear assets, such as drainage elements, can also drastically accelerate the deterioration of the pavement.

Maintenance decisions should be made with an understanding of the asset management strategy and the effectiveness of repair or restoration treatments in arresting / slowing down pavement deterioration or even improving condition.

Generally, continued preventative maintenance is important even for end of life pavements. It is often suggested that, once a pavement has deteriorated to a state where restoration such as overlaying is not practical, it may be preferable to delay reconstruction for as long as possible, only continuing with piecemeal repairs to ensure the pavement remains serviceable. However, significant delay before intervention may result in undesirable degradation of the foundation layers beneath the pavement, resulting eventually in the need for less economical, and far more disruptive full depth reconstruction, when the foundation condition may have once permitted partial depth reconstruction.

There are several different strategies that can be adopted ranging from planned preventative maintenance through to a wholly reactive approach, where maintenance is only undertaken to keep the pavement above a minimal serviceable and safe condition until reconstruction. The selection of a maintenance strategy for pavements depends on the asset management approach adopted by the owner / operator, considering factors such as budgets, user expectations and risk.

Life cycle cost analysis (LCCA) can be used to compare the long-term economic efficiency between alternative strategies for road pavement design and maintenance. LCCA can be applied to different types of assets and to a wide variety of investment-related decision levels [14].

LCCA of a concrete pavement asset should account for any costs that can be attributed to the use, care and maintenance of the pavement. These costs should be captured and considered in any treatment design decision process [15], in order to obtain optimum pavement life cycle costs.

For the planned maintenance of assets on the SRN, Highways England follow a standardised management process for the identification, development, design and delivery of all maintenance schemes. Known as 3D (Develop, Design, Deliver), the intention of the management process is to ensure that the right schemes are undertaken in the right way at the right time. The scheme identification process and maintenance strategy for concrete pavement assets is based on various factors including threshold limits, key performance indicators, scheme pipelines, budget availability, risk and user disruption.

Within 3D, the lifecycle of each maintenance scheme is governed by an end-to-end process, with seven stages (Stage 0 - 6). A defined set of cross-functional activities is undertaken in each stage and stage gates are used between each stage to review scheme readiness for next stage and take decisions as to whether to progress the scheme into the next stage.

Stage gates ensure risks and opportunities are identified at the appropriate points in the scheme lifecycle so that intervention is possible. Depending on scheme complexity, stages and stage gates may be combined.

Further detail on the application of this end-to-end scheme maintenance process can be found in Appendix C - Highways England 3D.

1.6. Reference documents and additional information

The CPMM is intended to supplement and complement the existing content in Highways England's requirement and advice documents for the investigation, repair and renewal of concrete pavements. Therefore, relevant documents are referenced throughout.

The assessment of concrete pavements is covered by CS 229 Data for pavement assessment [3] and CS 230 Pavement maintenance assessment procedure [4]. The assessment and maintenance considerations related the skidding resistance on concrete pavements is covered in CS 228 [2].

The maintenance design of concrete pavements is covered in CD 227 Design for pavement maintenance [1] and CM 231 Pavement surface repairs [5].

Requirements for construction of new pavements and maintenance of existing pavements are in MCHW Series 700 Road pavements - general [6]. Requirements for the construction materials and techniques for concrete road pavements are in MCHW Series 1000 Road pavements - concrete materials [7].

Documents cited throughout the CPMM provide useful additional or background detail for the reader. In particular, Britpave - the British cementitious paving association, an industry focal point for the development and promotion of concrete and in situ cementitious infrastructure solutions, offers free technical publications available on their website [16] that cover a wide range of topics including:

- introductions to concrete pavements;
- future innovations in concrete pavements; and,
- principles on the design and construction for various applications including roads, airfields and hardstandings.

Furthermore, a procedure for WLC analysis of treatment options is outlined in CD 226 NAA E, and guidance on LCCA of concrete pavements is well described in LCCA-related publications such as that by EUPAVE [14].

2. Investigation techniques

Undertaking the appropriate maintenance of concrete pavements is important to ensure that the pavement remains in a serviceable condition, does not fail prematurely and ultimately offers optimum whole-life cost. Before undertaking any significant repair and / or restoration treatment, a pavement investigation needs to be undertaken to determine the cause(s) and extent of any pavement deterioration. Without this investigation, inappropriate or unnecessary treatments may be undertaken; or defects which may increase in severity may go untreated. The investigation requires a systematic data collection effort and an analysis of the structural and serviceability condition of the pavement.

Asset owners and asset managers investigate and monitor the condition of their pavements in different ways. This depends on multiple factors including network size, budgets, the availability of technologies and the maturity of their asset management strategy and pavement management system (PMS).

On the SRN, standard procedures have been developed for the regular monitoring and assessment of concrete pavements. Surveys are carried out in two stages:

1. Network level surveys - routine full coverage surveys collecting data at traffic speed to identify sites for further investigation. Data collected for concrete pavements includes:
   - visual condition;
   - wet skidding resistance;
   - longitudinal profile variance; and,
   - construction thickness from ground penetrating radar (GPR) [4].
2. Scheme level surveys - targeted localised surveys using an array of investigation techniques where a treatment need is identified.

This chapter outlines the techniques that can be used to investigate the condition of a concrete pavement at a ‘scheme level’ once a treatment need has been established, to assist in the development of investigation plans based on the condition of the asset to correctly diagnose and appropriately maintain the pavement.

The guidance in this manual on the interpretation of data obtained by testing, including trigger values, supplements the existing requirements and advice contained in CD 227 [1] and CS 229 [3]. Trigger values define the point where pavement deficiencies are at a level that improvements may be required to restore or preserve pavement condition. However, these can vary for different asset owners.

The analysis and interpretation of the data collected, which is captured in an investigation report, will typically need to be undertaken by experienced pavement engineers and / or authorised by a certified pavement engineer [12]. This is particularly applicable to falling weight deflectometer (FWD) and GPR analysis and interpretation.

It is important that the results from the investigation techniques are not viewed in isolation when determining maintenance activities but are considered with all the other available information including asset condition data.

Other information that should be reviewed includes:

- safety inspections;
- historical routine inspections to identify the presence of pavement defects and their subsequent treatment(s); and,
- historical records of unplanned and / or reactive treatment(s).

Any history of rapid critical failure that has posed a safety risk, caused disruption to road users and / or required expensive / complicated treatments will be a major indicator of the appropriateness of any maintenance activities.

2.1. Visual condition surveys (VCS)

Visual condition surveys (VCS) will provide the most useful information in determining the causes of defects and deciding on the appropriate treatment. VCS are used to:

- record features and defects;
- determine any need for further investigations;
- provide an inspection record for future reference; and,
- allow the appropriateness and success of any treatment to be evaluated.

2.1.1. VCS methods

VCS data can be collected using various methods, which are broadly categorised as ‘traffic speed’ surveys and ‘walked’ surveys. Traffic speed surveys use various vehicle mounted methods to collect condition data, whilst walked surveys require a physical presence on the carriageway or adjacent footpath / verge to identify pavement defects and features. Table 2.1 outlines the benefits, limitations and opportunities of each general approach.

Where high quality traffic speed surveys can be procured, that are suitable for the options selection, preliminary design and potentially detailed design phase of a scheme, then on safety grounds these surveys are the standard default approach over walked surveys where no footway is present.

Walked surveys may be necessary where structural defects are identified (see Sections 3.2, 3.3 and 3.5), as traffic speed surveys rarely provide sufficient detail to enable informed treatments of these structural defects to be planned and undertaken.

Typically, the appropriate VCS method is dependent upon the:

- findings from previous investigations;
- scheme stage (i.e. options assessment, preliminary design, or detailed design);
- type and severity of defects; and,
- proposed maintenance activities.

The ideal solution is a multi-phase approach, identifying predominant defects with a traffic speed survey, then as necessary at the detailed design stage dependent on the presence of structural defects, a walkover survey to identify and appropriately assign maintenance treatments to these defects. These walkover surveys are ideally undertaken in conjunction with the contractor, who can support the options selection process, whilst other non-destructive and invasive testing activities are taking place.

Table 2.1 Benefits, limitations and opportunities of traffic speed and walked surveys

Survey type	Benefits	Limitations	Opportunities
Traffic speed	High output, cost efficient No closure required Reduced risk to personnel	Defects / severity may not be captured	Automated defect detection
Walked	Can capture defect severity	Increased risk to personnel Low output Closure required where no footpath Potential missed defects during night time surveys	Walk, talk and build at ECI stage Undertake concurrently with other surveys

2.1.1.1. Traffic speed VCS

VCS should, wherever possible, be undertaken at traffic speed. Traffic speed assessments not only increase safety and avoid the need for temporary traffic management lane closures, but also allow the collected data to be re-reviewed, as necessary. Surveys can be undertaken during daylight and good weather to capture high-quality imagery so that the defects can be observed most clearly.

Vehicle-mounted camera survey options are available, such as TRACS downward and forward facing images, or bespoke 4K resolution images taken at 4 m intervals from 6 mounted cameras (forward, left, right, down, reverse and vertical down) and accurate to ± 1 m in areas of good global positioning system (GPS) coverage (see Figure 2.1).

Traffic speed VCS can be collected from a multi-function vehicle (MFV) that utilises mobile mapping systems, which integrate sensors with navigation systems to collect georeferenced information of the surveyed area [17].

TRACS (Figure 2.2) is one example, other bespoke MFVs can also be fitted with devices including:

- ground-penetrating radar (GPR) to collect pavement construction, thickness, and voiding data (see Section 2.2.1.2);
- 3D laser triangulation to identify cracking, faulting, joint widths and other defects (see Figure 2.3); and,
- light detection and ranging (LiDAR) radar to collect geospatial data to give an understanding of carriageway profile and whether any structural defects including stepping, heave or settlement are present (see Figure 2.4 and Figure 2.5).

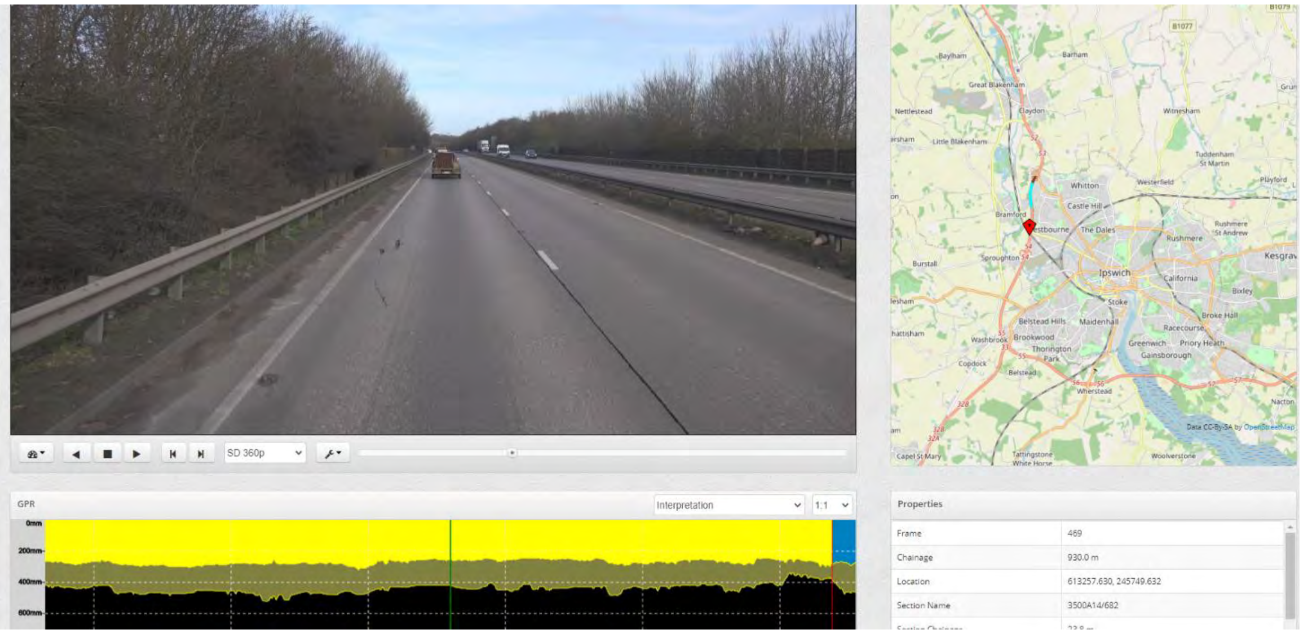


Figure 2.1 Traffic speed VCS output including high quality multiple angle imagery, location, chainage and indicative layer thickness from simultaneously collected GPR onboard the MFV

The typical data collection speed ranges from 40 mph to 60 mph [17]. The quality of data collection is dependent on several variables including environmental conditions, presence of vegetation, GPS obstructions and time of day [17]. The MFV output accurately ties the various data at the same location, further increases safety and reduces overall survey time on site.

Figure 2.2 TRACS vehicle

Processed pavement surface data can be further analysed in any geographic information system (GIS), computer-aided design (CAD) or data analytics software, thereby facilitating the automated detection of surface defects and / or the development of scheme treatment plans (see an example in Figure 2.3 & Figure 2.4).

3D laser triangulation technologies utilise lasers with a high resolution (up to 1 mm x 1 mm) and vertical accuracy (up to 0.1 mm) to measure the distance from a sensor to the surface, acquiring a 'range' (distance between sensor and ground) and an 'intensity' (amount of light absorbed) to create a 3D image [18]. Software algorithms can then be used to automatically identify defects such as cracks, joint stepping and joint spalling as well as establishing joint width [17].

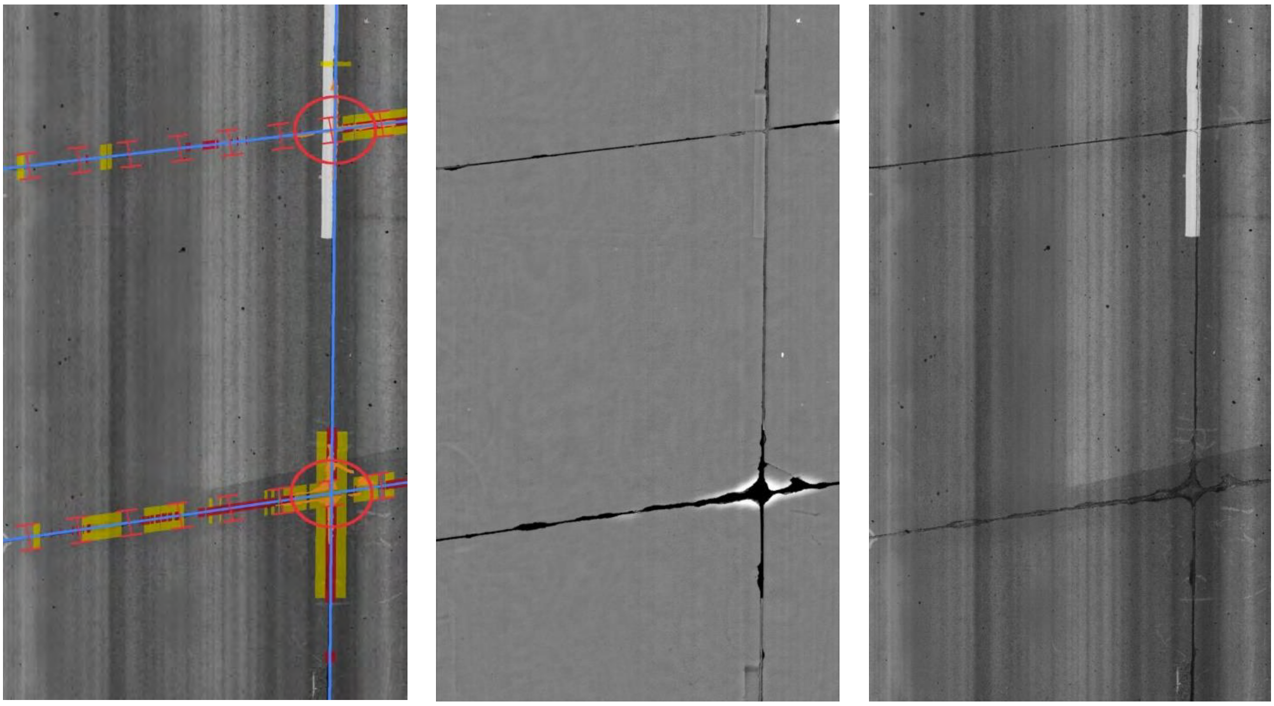


Figure 2.3 Inspection of joints and corners for spalling and defective joint sealant using 3D laser triangulation. Automated defect detection (left) based on intensity (centre) and range (right) [18].

LiDAR point cloud data can detect large pavement defects, with a depth greater than 25 mm. However, small defects, such as fine cracking, cannot be detected within the mobile LiDAR point cloud on its own but can be captured in conjunction with VCS imagery. This is typically due to the resolution (up to 20 mm x 20 mm) and level of noise present in the point cloud in individual drive lines [17].

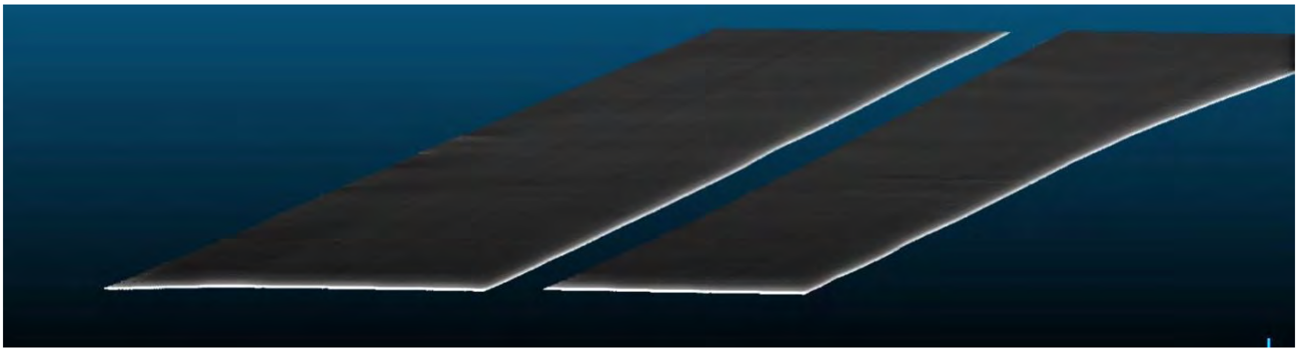


Figure 2.4 Example of LiDAR point cloud road surface [17]

Figure 2.5 shows two examples of defects detected by imagery and LiDAR point cloud during A12 Mountnessing and the A12 Chelmsford site trials [1].

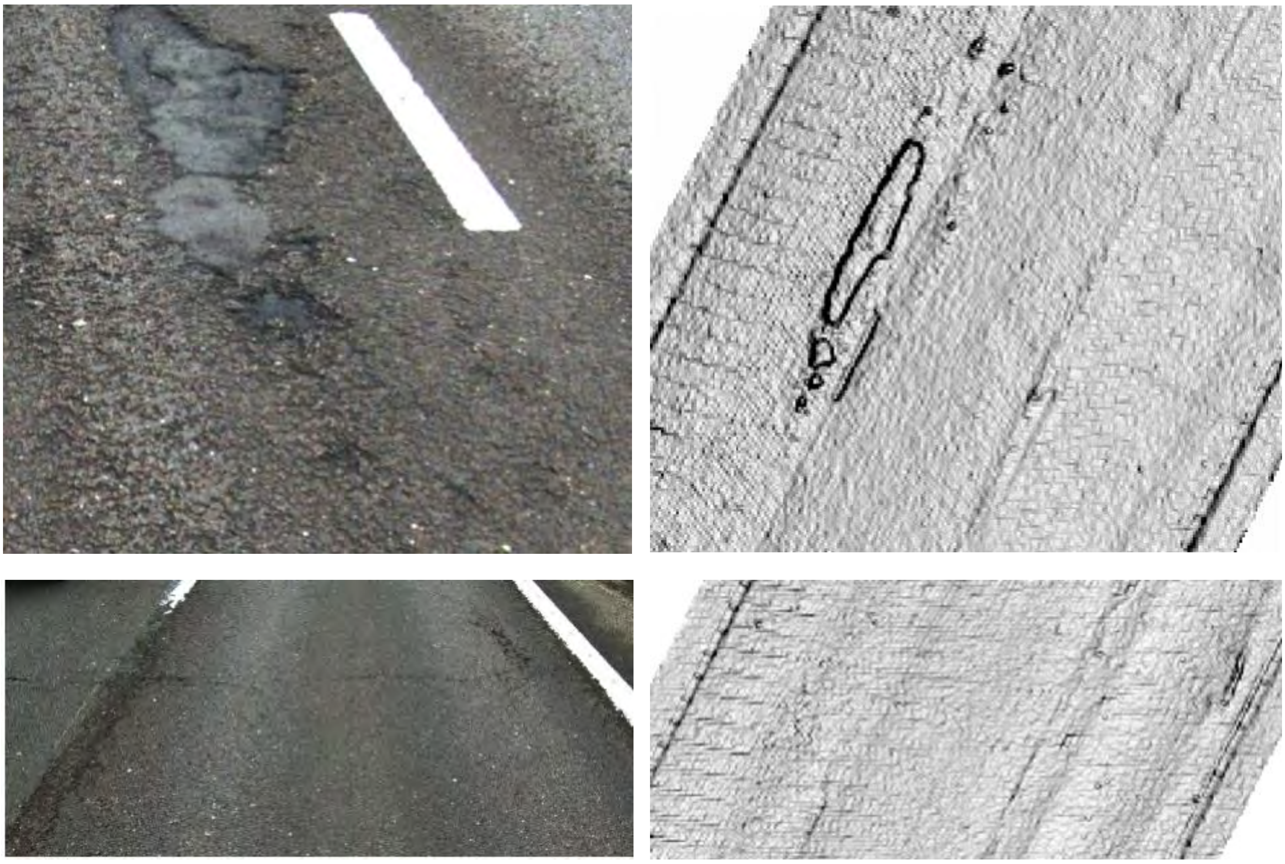


Figure 2.5 Examples of defect detected during A12 Mountnessing and the A12 Chelmsford site trials [3]: pothole visible in pavement image (top left) and in LiDAR (top right); cracking visible in pavement image (bottom left) and not visible in LiDAR (bottom right)

2.1.1.2. Walked VCS

Walked VCS (Figure 2.6) were historically preferred for measuring, recording and mapping all features and defects present on site. However, where walked surveys require traffic management lane closures to ensure the safety of the surveyors, this adversely impacts on road users in terms of safety and delay.

To mitigate travel disruption walked surveys are commonly undertaken at night, which can affect quality and create repeatability issues. There are challenges for surveyors to accurately record all defects and capture high-quality images under artificial lighting. In addition, the severity of defects observed and recorded can be affected by the lighting.

Figure 2.6 Walked VCS

The main advantage of walked VCS is the high probability of a surface defect being observed and recorded and the ability to more accurately judge or measure any crack widths.

Recent technology developments mean that mobile devices can be used to capture defect extents, descriptions and locations and upload the data to cloud-based systems. This allows for the visualisation of defects on mapping interfaces [17]. Use of this digital technology also improves the efficiency of receiving survey results and developing treatment or investigation plans.

2.1.2. Measurement requirements

Requirements for VCS are covered by CS 229 [3]. Standard symbols and definitions for the identification and characterisation of features and defect types, their severity and extent, are specified in CS 229 [3] and reproduced in Table 2.2, to be applied consistently across each scheme. Adopting this standard approach assists in grouping areas of concrete pavement exhibiting similar deterioration and defects, gaining insight into the probable causes, and hence, planning targeted further investigations.

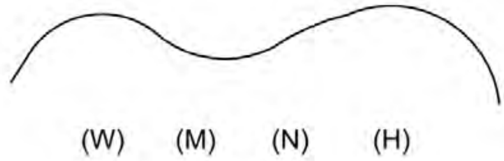
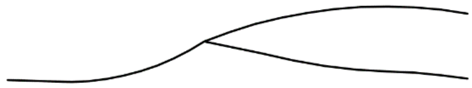
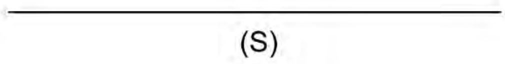
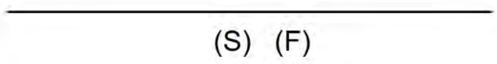
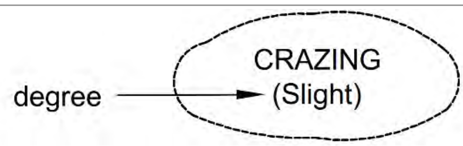
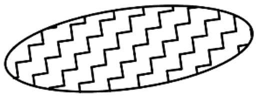
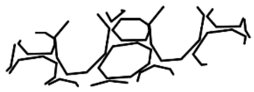
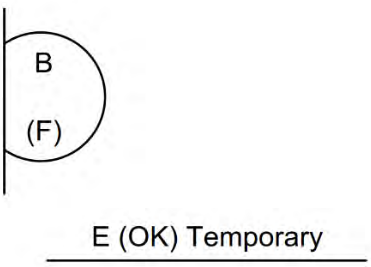
For jointed concrete pavements, referencing should be supplemented by bay numbering. This would provide a positive referencing system that can be applied on site and may also be used in contract preparation.

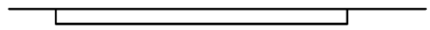
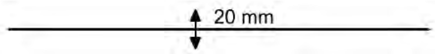
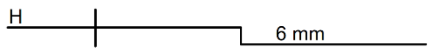
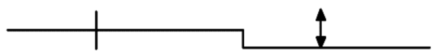
Continuously reinforced concrete pavements (CRCP) should be referenced by appropriate fixed assets such as marker posts.

For concrete pavements overlaid with asphalt, structural defects in the underlying concrete may not be visible. It may be necessary to look at historical surveys to inform structural condition, or to carry out an additional visual inspection during resurfacing works after removal of the asphalt surfacing.

Additionally, key features should be recorded on the VCS, including the extents of cut and fill, underbridges, the clearance to overbridges and any safety concerns.

Table 2.2 Standard symbols for recording condition of concrete pavements [3]

Condition	Symbol
Cracks **W - wide: wider than 1.5 mm** **M - medium: 0.5 to 1.5 mm wide** **N - narrow: narrower than 0.5 mm** **H - hairline: present but distinguishable with difficulty**	
Bifurcation	
Overbanded or sealed crack.	
Overbanded or sealed crack that has subsequently failed	
Plastic shrinkage cracks	
Surface crazing (showing the degree)	
Scaling	
Miscellaneous surface defects	
Surface texture worn	
Repairs: **Asphalt - B** **Cementitious - C** **Epoxy - E (OK) indicates sound** **(F) indicates failed**	

Condition	Symbol
Missing road stud (and cracks)	
Rust staining	'RUST'
Defective joint seals	
Shallow spalling at joints (or at cracks)	
Deep spalling at joints	
Opening of longitudinal joint (width in mm)	
Faulting (stepping) at joint or crack (with difference in level in mm)	
Vertical movement at joint or crack observable under passing traffic	
Evidence of pumping	'PUMPING' or 'STAINING'
Settlement	

Results from a VCS are typically expressed on an annotated field survey ‘strip map’ (see Figure 2.7) that is accurately (± 1 m) referenced to the site and includes a photograph of each condition identified by a standard symbol. The most helpful presentation of the VCS is on a CAD drawing of the scheme, which should also show the concrete bay layout and other key features, as necessary.

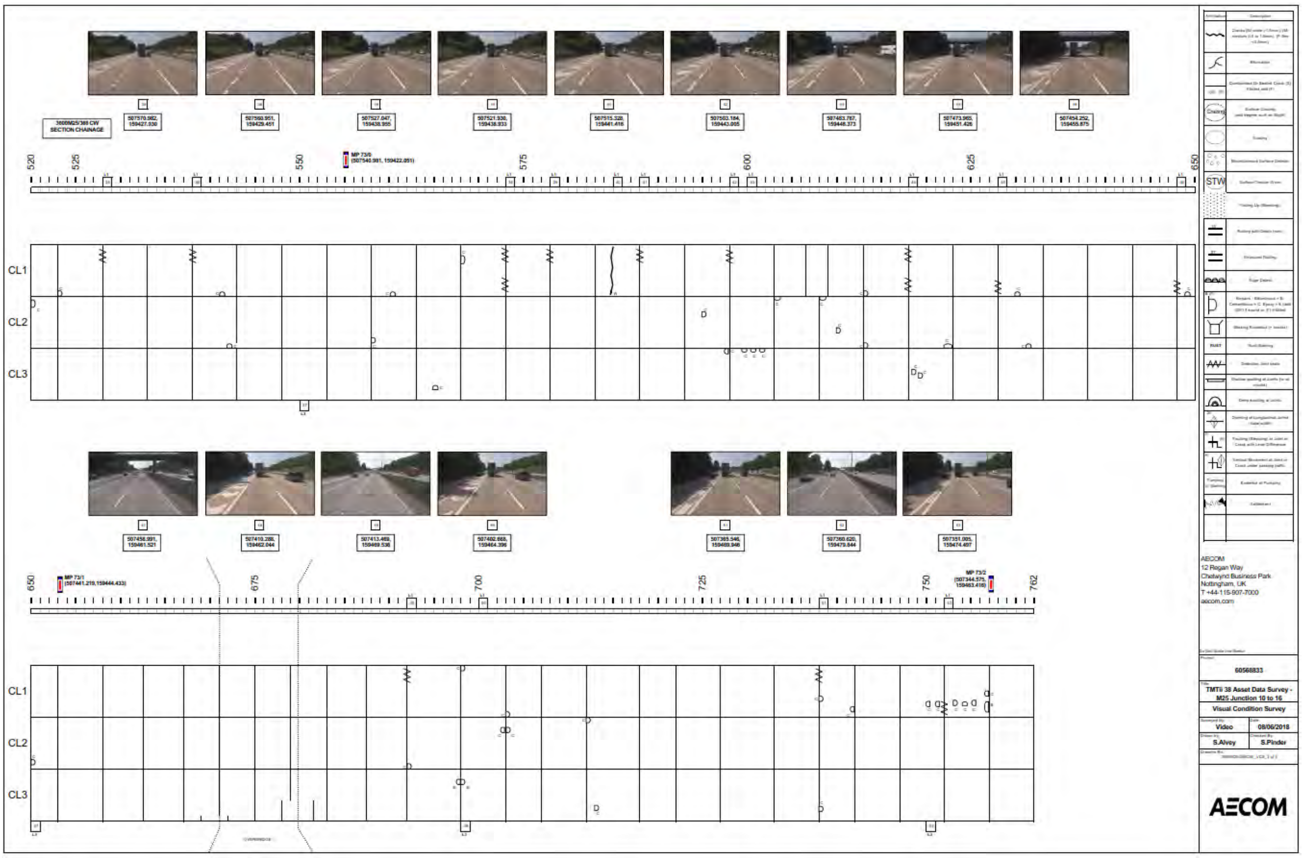


Figure 2.7 Example VCS strip plan for a jointed concrete pavement

2.1.2.1. Crack measurements

Cracks are characterised as follows, to ensure that the most appropriate treatment can be assigned:

- H - Hairline: present but distinguishable with difficulty.
- N - Narrow: < 0.5 mm wide.
- M - Medium: 0.5 to 1.5 mm wide.
- W - Wide: > 1.5 mm wide.

Traffic speed video VCS presents two current technology issues with this characterisation. Firstly, the width of a crack is difficult to approximate without direct measurement. Secondly, which is equally difficult for a walked VCS, the width is rarely consistent over the length of a crack and it will become wider at lower temperatures. Cracks are 3-dimensional and the width will vary with depth, especially when the slab is subject to curling or warping with daily temperature and moisture variations in the concrete (see Section 4.1.2). However, cracks are generally acknowledged to be widest at the surface [10].

As crack width directly influences the load transfer performance across the crack, they should be approximated when they are likely to be at their widest, typically at temperatures < 15 °C. An averaging process can be adopted for any cracks where the width is inconsistent. When there is any question over the characterisation of a crack, it is best assumed as the wider category crack.

The initial investigation should focus on identifying cracks which are medium or wide. In situ verification measurements can be undertaken as necessary at the detailed design stage when access is available during a lane or carriageway closure.

Note, narrow transverse cracks present at 1 - 2 m intervals are an inherent design feature of good quality CRCP. Hence, these cracks do not need to be identified except where the interval is closer or they are wider, bifurcated or spalled.

The presence of any areas of crazing (or map cracking) should be noted, including notes of any associated white or cream powdery material that streaks over the surface after heavy rain as such a deposit may indicate an alkali-silica reaction.

2.1.2.2. Spall measurements - width and depth

Spalls are a breakdown of the surface material at the slab edges or joints. Spalling can also occur at cracks. It is important to be aware of the depth of any spalling at joints as this has a significant impact on the maintenance options available. The relevant categories as outlined in Figure 2.8 are:

Shallow spalls:

(a) contained within the depth of the joint sealant groove;
(b) exceeding the depth of the joint sealant groove but less than one third slab depth; or,

Deep spalls:

(c) exceeding one-third slab depth.

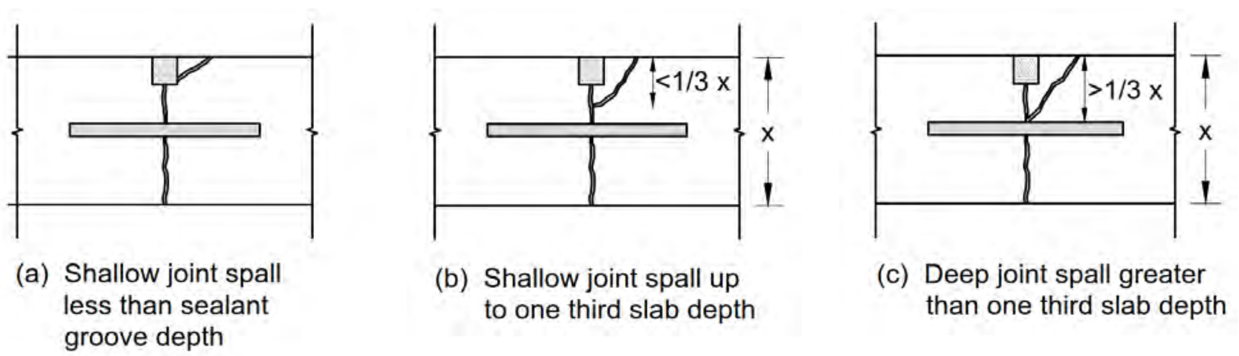


Figure 2.8 Joint spall categories

The initial investigation should focus on identifying each spall location and estimating shallow spalls (a) and (b) from deep spalls (c), as deep spalls will require more significant maintenance to rectify (see Section 3.3.1). In situ verification measurements may be undertaken when access is available during a lane or carriageway closure.

To distinguish between categories (a) and (b) it may help to remove the joint seal or prise out sample pieces of spalled concrete and measure the depth. Care should be taken to check by sounding (with a hammer or metal rod, or preferably, a Schmidt-type rebound hammer) that the underlying concrete is not further spalled.

Drilling cores at a sample of representative spalls will often be needed to distinguish between (b) and (c) (see Section 2.2.2.1) and to explore the possible causation such that the optimum treatment can be identified. Where the equipment is available, ultrasound tomography (see Section 2.2.1.4) can be used to non-invasively measure the depth of spalls.

2.2. Sampling and testing

Visual condition alone may not be sufficient to identify the severity and extent of defects. Failure to identify the extent and severity of defects can cause issues when it comes to undertaking repair works, particularly during short possession windows, where it may not be possible to increase the plan area or depth of the repair due to lack of materials or time, resulting in a potentially poor durability of the repair.

An example is shown in Figure 2.9, where deterioration of the concrete extending into the adjacent bay was not identified at the investigation stage. This would have been identified with targeted trial holes or test pits, or potentially ground penetrating radar investigations, in proximity of defects and at the boundaries of the area to be repaired to check the adjacent concrete was sound.

Figure 2.9 Deterioration of concrete extending into the adjacent bay not identified by the pavement investigation prior to repair works

The non-destructive testing, invasive sampling and testing and laboratory testing outlined in this section should be used in conjunction with the visual condition survey to plan maintenance works and identifying the treatment area and depth which encompasses the defect.

2.2.1. Non-destructive testing (NDT)

Non-destructive testing (NDT) can be used to assess the overall structural condition of the pavement without causing damage to the pavement. There is a suite of NDT techniques available within the UK market to assist in the evaluation of concrete pavements, which are described in the following sections. The capability of these techniques is outlined in Table 2.3 [17].

Typically, one or more of these NDT techniques is used for the detailed investigation of each scheme based on the findings of the initial investigation, the defects identified, and measurement capabilities required. They are often used in combination with invasive testing (see Section 2.2.2), the results of which are often used to calibrate the NDT outputs.

Table 2.3 Capabilities of non-destructive testing techniques available within the UK [19]

Measurement capability	**FWD**	**GPR**	**Magnetic imaging tomography**	**Ultrasonic tomography**	**Concrete sounding**
Load transfer efficiency	●				
Void detection	●	●		●	
Layer thickness		●	○	○	
Position of steel reinforcement, dowel bars and tie bars		●	●	●	
Structural assessment	●			○	●
Key: ● = main measurement ○ = secondary measurement [blank] = no capability					

2.2.1.1. Falling weight deflectometer (FWD)

The FWD is a dynamic plate test (DPT) device used for measuring the overall structural performance of a concrete pavement and any formed joints. The deflection response of the pavement surface is measured when a load pulse is applied that represents a moving heavy goods vehicle wheel load. The load pulse is generated by dropping a known mass from a calculated height onto a circular loading plate that is placed in contact with the pavement surface. This pulse generates a surface deflection that is measured at increasing radial distance from the loading plate by a series of geophones.

The FWD is the most common device used for concrete pavement surveys. It can be a trailer mounted device towed behind a vehicle (Figure 2.10) or mounted within the body of a vehicle. DPT variants include heavy weight and super heavy weight deflectometers [3], which are typically used on airfields and heavy duty pavements.

Whilst the FWD is identified in CD 227 [1] as an optional NDT technique, it can provide very informative data as part of a detailed investigation. Requirements for the use of an FWD and recommended arrangements for concrete testing using an FWD are outlined in CS 229 [3]. They require annual accreditations, which ensures the accurate, reliable and consistent assessment of the pavement condition.

Depending on how the FWD geophones are set up and where the loading plate is positioned, the resultant surface deflection measurements can be used to evaluate pavement characteristics. For concrete pavements, the primary use of the FWD is to assess the performance of any formed joints or cracks. The most common outputs are:

- load transfer efficiency (LTE); and,
- void intercept (VI).

FWD deflection data can be processed to provide important information about the existing pavement structure. These include:

- layer stiffness evaluation; and,
- absolute deflection (AD) and variability in deflections.

Figure 2.10 FWD undertaking concrete joint testing

Load transfer efficiency (LTE)

The load transfer efficiency (LTE) across any crack or joint contributes to the structural capacity of the concrete slab. LTE is a measure of the ability of a discontinuity (crack or joint) in a pavement to transfer a load applied to one side of the discontinuity across to the other side. Load transfer is further discussed in Sections 1.2 and 4.3.2.

One side of the slab is loaded during the FWD test, and surface deflections are measured on each side of the joint or crack. The more load that is transferred to the adjacent slab, the lower the stresses that are produced in the loaded slab. Pavements on the SRN with poor load transfer between slabs tend to deteriorate at a faster rate due to increased stresses. Therefore, an understanding of the LTE across joints and cracks in a concrete pavement will support a targeted, effective maintenance programme.

LTE of 100 % represents perfect load transfer between slabs and LTE of 0 % represents zero load transfer between slabs. CD 227 [1] indicates that < 50 % LTE is the point at which load transfer is unsatisfactory. However, the LTE values obtained should not be considered in isolation as there are many factors which affect LTE. Acceptable values will be highly dependent on the pavement thickness, concrete strength, foundation support and actual traffic loading. Poor load transfer can arise for various reasons, but the most common reason is that the dowel bars (across the transverse joints) or tie bars (across the longitudinal joints) are corroded, broken or loose. Identifying the causation of these issues is covered in detail in Chapter 4.

It is important to note that LTE is expressed as a percentage and is therefore more variable (less repeatable) when the absolute deflections are low, meaning that any corresponding low LTE might be considered satisfactory, when assessed in conjunction with other factors.

LTE values can be used in a relative assessment to identify the poorest performing joints or cracks across the scheme, in conjunction with AD and VI values, to establish a priority order for treatment. Hence, when LTE testing is undertaken, every joint should be assessed. Testing should be undertaken in the most heavily trafficked lane as a minimum, which is normally lane 1 [1].

Joints showing poor LTE values, despite an absence of visual defects (see Chapter 3), might be subject to further investigation. In the case that poor LTE is due to dowel bar issues, typically bay replacement (See Section 5.4.1) or full depth repairs are recommended (see Sections 5.4.3 and 5.4.4).

The recommended arrangement for testing LTE at a transverse joint is to position the loading plate (with a geophone positioned at the centre) on the leave slab 250 mm from the joint, with two further geophones at approximately 50 mm on either side of the joint [3], as shown in Figure 2.11.

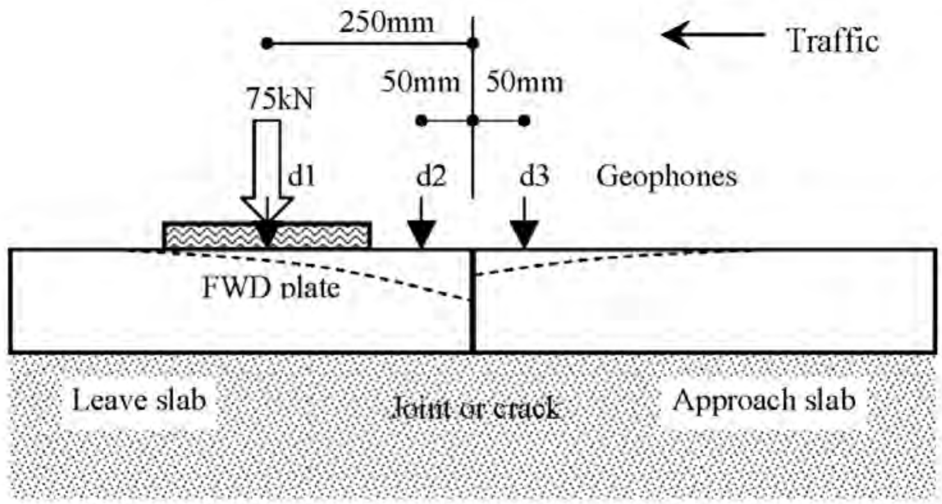


Figure 2.11 Recommended FWD arrangement for assessing joint / crack LTE and VI as detailed in CS 229 [3]

The slab temperature should be measured using an electronic thermometer placed in pre-drilled (i.e. cooled to ambient temperature) holes and recorded throughout the survey. Measurements should be made preferably between 5 °C and 15 °C [3]. At higher temperatures, the joints close and can be unrepresentative, i.e. falsely high LTE may be indicated. A significant temperature gradient through the slab may also affect the indicated LTE.

Requirements for undertaking LTE testing and further guidance on temperature limits and the analysis of FWD data is contained in CD 227 [1] and CS 229 [3].

It is possible to assess the LTE of joints in JRC and URC with an asphalt overlay provided the accurate location of the joint is known. However, this is typically only the case where a crack has reflected through the asphalt surfacing, from an underlying crack or joint. In some cases, the reflected cracks may be bifurcated or angled, which could reduce the accuracy of the testing.

Void intercept (VI)

Void intercept (VI) testing can be used to determine the possible presence of voids beneath the slab edge in-line with the wheel track. To optimise the FWD survey, this test should be undertaken at the same time as LTE testing. The arrangement for VI is the same as for LTE testing but requires three different target load levels to be applied so that the surface deflection readings (from the geophone at the centre of the loading plate only) can be plotted as shown in Figure 2.12.

The plotted FWD deflection data points can be extrapolated to intercept the zero applied load. If the extrapolated line does not intercept zero load at zero deflection, this indicates that a void may be present below the concrete slab.

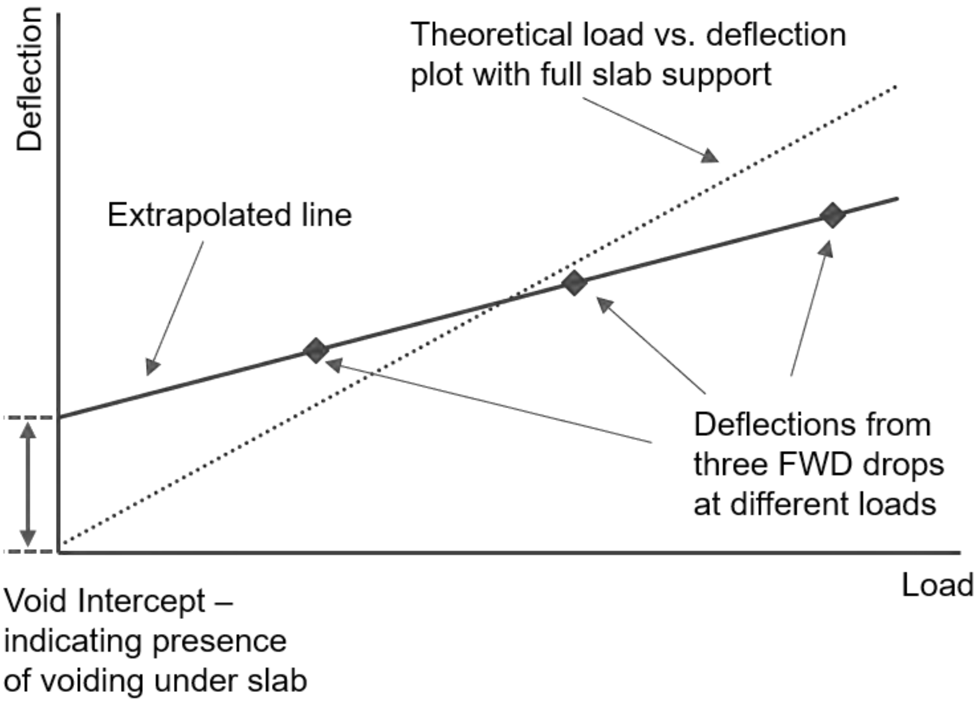


Figure 2.12 Load vs. deflection plot. Example of use of FWD load versus deflection graph to identify the presence of voids beneath the slab, though the Void Intercept (VI) method

A plot of the deflections in a load vs. deflection graph, at three target loads, 50 kN, 75 kN and 100 kN as recommended in CS 229 [3], can be used to identify potential voiding. A VI > 50 microns is an indicator of potential voiding according to CD 227 [1].

Any voiding below a concrete slab edge is a potential causation mechanism for further defects and the VI test should therefore (like the LTE testing) be undertaken at every joint and not just targeted at those concrete pavement areas currently exhibiting defects.

Poor load transfer between joints / cracks can indicate the presence of voids. However, as previously discussed in this section, slabs exhibiting a low level of LTE may also indicate an issue with dowel bar condition (see Section 4.3.2). Table 2.4 gives interpretations of level of slab support based on measured values of VI and LTE.

Table 2.4 Interpretation of FWD data from VI and LTE testing

Scenario	Void intercept (VI) (microns)	Load transfer efficiency (LTE) (%)	Interpretation
1	0 - 50	50 – 100	Likely adequate slab support
2	0 - 50	0 – 50	Likely adequate slab support, review dowel bar effectiveness (see Section 4.3.2)
3	> 50	50 – 100	Slab voiding likely, check if absolute deflection values are high
4	> 50	0 – 50	Slab voiding likely, also review dowel bar effectiveness (see Section 4.3.2)

Joints showing poor VI values in an absence of visual defects might be subject to further investigation. If a presence of voiding is ascertained, treatment options include under slab grouting (see Section 5.4.5). This way early treatment of the causation should prevent more costly repair of the defects that would otherwise develop.

Layer stiffness evaluation

The concrete pavement and foundation layer stiffnesses can be evaluated by the 'back-analysis' of the surface 'deflection bowl' data measured at each FWD 'slab centre' test location. These layer stiffnesses can be used together with laboratory test data (see Section 2.2.3) to inform the structural condition and integrity of the different layers within the pavement.

Measurements should be made away from discontinuities such as joints and cracks, as these will influence the result. For jointed concrete pavements, measurements are typically taken within the wheel track areas coinciding with the centre of slabs. For highways, the load level applied should be 75 kN ± 10 % [3].

The back-analysis should be undertaken by an experienced pavement engineer as the outputs are highly sensitive to the inputs and assumptions used including layer thickness. Proprietary software is also required.

The geophones used to record the deflection bowl response to loading at each slab centre FWD test are located at the standard radial distances shown in Table 2.5.

Table 2.6 provides reference values of layer stiffness that can be used as a guide when interpreting the condition of the pavement and foundation layers.

Table 2.5 Recommended FWD geophone positions [3]

Distance (mm) from centre of loading plate						
Geophone number						
d1	d2	d3	d4	d5	d6	d7
0	300	600	900	1200	1500	2100

Table 2.6 Condition related to layer stiffness in concrete pavements as outlined in CD 227 [1]

Material type	**Layer stiffness derived from FWD**		
	Poor integrity throughout	Some deterioration	Good integrity
Pavement quality concrete (PQC)	< 20 GPa	20 - 30 GPa	> 30 GPa
Hydraulically bound mixture (HBM)	< 8 GPa	8 - 15 GPa	> 15 GPa
Unbound foundation	< 0.1 GPa		≥ 0.1 GPa

Absolute deflection (AD)

The absolute deflection (AD) of a road pavement under a given load provides a direct means of assessing the pavement structural condition without further analysis. AD is typically reviewed in combination with the other parameters measured during FWD testing and other testing results to assist with gaining an overall view of pavement condition. AD provides additional context to LTE and VI results. Acceptable deflections may vary dependent on the type of subbase and subgrade. Generally, deflections of 0.2 mm or more are likely to indicate inadequate subbase support.

Variability in deflections can be associated with localised defects, and a stepped increase in the general deflection level can be associated with a more general area of distress (e.g. associated with a drainage deficiency at a cut / fill transition).

With reference to the geophone position numbers shown in Table 2.5, it is common to plot the following key deflection parameters to draw conclusions from the relative performance of the individual pavement layers:

• Central deflection (d_1)	Indicator of overall pavement response
• Deflection difference (d_2 - d_4)	Indicator of the response from the concrete slab
• Deflection difference (d_3 - d_5)	Indicator of the response from any HBM layer
• Outer deflection (d_6)	Indicator of foundation response

2.2.1.2. Ground penetrating radar (GPR)

GPR is an NDT technique which can be used for concrete pavements to interpret the:

- presence of voids beneath the concrete slab;
- position and condition of embedded steel reinforcement, dowel bars and tie bars; and,
- concrete slab and HBM layer thicknesses.

The quality of the information obtained from GPR depends mainly on three factors [3]:

- The electrical properties (dielectric constant and the conductivity) of the materials forming the pavement. When the dialectic constants of the pavement layers are not significantly different, cores may be required to aid interpretation of the data.
- The survey conditions, including the methodology and type of GPR equipment employed. Environmental conditions such as external sources of electromagnetic interference also impact readings.
- The processing software and analysis methodology, including calibration procedures employed.

Additionally, voids may be masked by reinforcing steel in the concrete or water within the voids.

Surface water affects the signal and readings, which makes interpretation difficult. Therefore, GPR surveys should not be carried out when it is raining or when standing water is present on the surface of the pavement. In addition, GPR surveys should not be carried out on recently salted (de-iced) roads [3].

The interpretation of raw GPR data from concrete pavements is a specialist skill and should only be undertaken by an experienced analyst. GPR surveys can be undertaken on a variety of surfaces with a multitude of antenna types and at different speeds [17]. They can be divided into two types:

- 2D GPR - undertaken with simple single frequency antenna equipment. Layer thicknesses are normally reported as 2D sections. 2D GPR survey can further split into:
  - walked GPR survey - providing a continuous cross section of the pavement; and,
  - traffic speed 2D GPR survey.
- 3D GPR - undertaken with complex equipment with closely spaced multi frequency antennae. 3D GPR allows data to be presented in such a way that patterns caused by structural features such as joints and reinforcement bar can be easily identified and defects such as voids and delamination become much clearer. When reported, layer thicknesses can be mapped as contours, a heat map or shown in a 3D model (see Figure 2.13).

Presence of voids

GPR surveys can be used to estimate the presence, position and size of air voids and water-filled voids below unreinforced concrete slabs.

In reinforced concrete, reflections from voids or wet patches may be masked by reflections from the reinforcement within the concrete slab. In this case, the scan spacing and the antenna footprint need to be sufficiently small to allow scans to pass between the mesh.

GPR surveys to detect and measure voids and wet areas can be considered when there are other indications of problems related to poor slab support. Such indications might be available from the results of an FWD survey or from inspection of the subbase condition through drilled core holes (see Section 2.2.2.1).

The accuracy of measurements can depend on the:

- scan spacing achievable;
- antenna frequency;
- size of the voids; and,
- number of longitudinal profiles collected.

Air voids less than 80 mm in height and water filled voids less than 25 mm in height are difficult to identify [3].

Figure 2.13 shows an example of voiding and moisture detection using 3D GPR traffic speed surveys (approximately 1000 m section from the A12 near Chelmsford [17]). The plot is generated using data from four horizontal cross sections within the pavement structure.

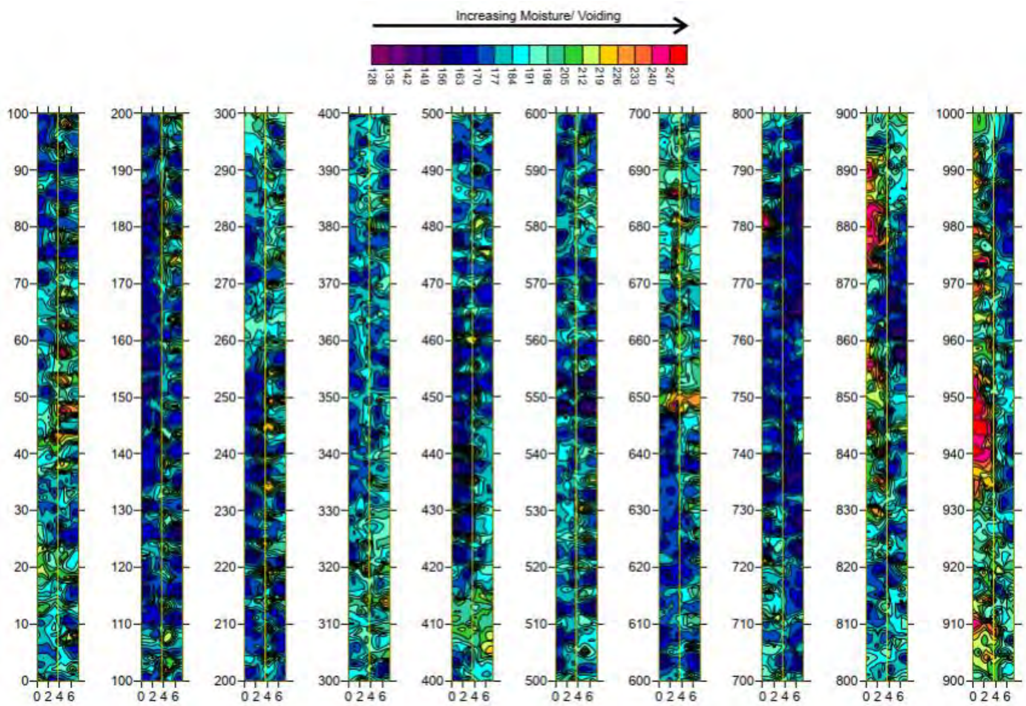


Figure 2.13 GPR survey results plot showing moisture / voiding presence beneath concrete slabs - A12 Chelmsford [17]

Position of steel reinforcement, dowel bars and tie bars

GPR surveys can provide details of the reinforcement in the pavement, such as depth and spacing. In this case, surveys should be carried out with a very small scan spacing (≤ 50 mm, ideally ≤ 10 mm). For this type of GPR survey, very precise location referencing techniques should be used to give high accuracy [3].

Whilst GPR is unlikely to directly indicate the condition of any steel, it can detect damage if the steel has corroded and damaged the surrounding concrete. Reinforcement investigations are carried out more easily and with greater confidence using a magnetic imaging tomography (MIT) device (see Section 2.2.1.3).

GPR can measure the depth of dowel bars and tie bars at joints and whether they are grossly misaligned. Transverse scans are required along the line of the joint to assess dowel and tie bars. One scan is required just beside the joint, and two further scans each side of the joint on a line just above the anticipated ends of the dowel or tie bars.

Layer thickness

GPR surveys can be used to determine the depth of concrete pavement layers, by detecting the interface between layers of contrasting material. For reliable layer thicknesses, readings should be calibrated with layer thicknesses obtained from cores (see Section 2.2.2.1) at regular intervals. When calibrated with cores, the confidence of results ranges from around ± 5 % for slow-speed surveys to around ± 9 % for traffic-speed surveys. For underlying bound and unbound material layers, the expected confidence in results is ± 15 % and ± 30 % of the real thicknesses, respectively [3]. In the case of reinforced concrete, scan spacing less than the spacing of the reinforcement bars should be used for the survey [3].

2.2.1.3. Magnetic imaging tomography (MIT)

MIT devices (also known as electromagnetic covermeters) are NDT technologies which can be used for concrete pavements to determine [19]:

- position of steel reinforcement, dowel bars and tie bars; and,
- layer thickness (in specific circumstances).

Position of steel reinforcement, dowel bars and tie bars

MIT devices provide the most accurate NDT method of determining the depth of steel reinforcement, and the depth and orientation of steel dowel bars and tie bars, and hence any horizontal or vertical misalignment.

The equipment for this survey consists of three components:

- a sensor unit;
- an onboard computer; and,
- a glass fibre-reinforced rail system [20].

Electromagnetic pulses are sent out from the sensor unit as it glides along the rail system over the joint and identifies the induced magnetic field (see Figure 2.14).

Figure 2.14 MIT device in use [20]

Layer thickness

To determine slab thickness using MIT devices, reflective disks have to be placed below the concrete prior to paving.

The MIT equipment makes use of pulse induction technology to detect the reflective disks, and then determines the pavement thickness. The advantage of this NDT survey (as opposed to GPR) is the high level of confidence in measurements without the need to cut cores. This is a common approach in the USA but has not been used in the UK.

2.2.1.4. Ultrasonic tomography

Ultrasonic tomography is an NDT technology with several applications for the investigation of concrete pavements [19, 20]:

- pavement thickness;
- position of steel reinforcement, dowel bars and tie bars;
- voiding presence; and,
- concrete strength.

In addition, this method can help to identify cracking, areas of debonding, joint deterioration and poor consolidation.

Ultrasonic tomography is often used on critical structures where invasive testing is not possible. It is a slow-speed investigation technique so lane or carriageway closures will be required. Engagement with technology suppliers will be critical to gaining a better understanding of measurement capabilities and accuracy.

The equipment typically includes a screen to access preliminary information and allows various operating modes (see Figure 2.15). This test evaluates the speed at which ultrasonic pulses travel through layers. These pulses are converted into electrical energy by a transducer; and the speed is calculated based on the transducer distance to the pulse application [20].

Figure 2.15 Ultrasonic tomography equipment [20]

2.2.1.5. Rebound (Schmidt) hammers

Rebound hammers (or Schmidt hammers) are commonly used to provide a rapid indication of whether a concrete repair has gained sufficient compressive strength to permit the pavement to be reopened to traffic. They consist of a spring-controlled mass that slides on a plunger within a tubular housing. The amount of rebound of the mass depends on the hardness of the concrete surface. Therefore, this measurement can be correlated to the compressive strength of the concrete [21] (see Figure 2.16).

The test itself is simple, instant and non-destructive, but mixture specific correlations with cube compressive strengths will be necessary to provide an estimate of in situ compressive strength.

This test is particularly useful to assess uniformity and quality of the concrete, for example, identifying the extents of defects before undertaking repairs. A minimum of 9 test positions is recommended spread over the area being assessed, with the result being the median of the readings.

Figure 2.16 Rebound hammer

2.2.1.6. Concrete sounding

Concrete sounding is a qualitative evaluation technique used to identify areas of deteriorated concrete close to the surface. Sounding is particularly useful for identifying the extents of defects before undertaking thin bonded or shallow repairs. Concrete sounding is typically undertaken by striking the concrete with a hammer or steel rod.

When good concrete is struck, there is a ringing sound; however, when deteriorated concrete is struck, a hollow or drum-like sound is produced.

A thin layer of dust can be applied over the surface to further aid identification of deteriorated concrete, the dust will jump when an area of deteriorated concrete is struck. Where defects are deep, generally greater than one third of the slab depth, sounding may not identify them.

2.2.2. Invasive sampling and testing

Invasive sampling and testing of pavements serve several purposes, including:

- identifying pavement foundation conditions;
- verifying pavement and foundation layer types and thicknesses;
- retrieving samples for laboratory testing and analysis;
- allowing intrusive soil testing, using the dynamic cone penetrometer (DCP);
- allowing foundation surface modulus testing using the light weight deflectometer (LWD); and,
- calibrating non-destructive testing results.

The types and amount of invasive testing required for an investigation is primarily dependent upon the:

- type, severity and extent of observed visual defects;
- road space and traffic management allocation restrictions for the detailed investigation; and,
- amount of historic information available.

The two main excavation techniques are:

- **coring**: can range from small (100 mm) to large (450 mm) diameter, or numerous cores can be combined in secant formation to create test pits (see Figure 2.17); and,
- **saw cutting**: can be scaled to suit the testing and sampling required, overcutting and the creation of butterfly corners can compromise structural integrity.

Figure 2.17 Extracted core (left) and multiple cores taken to create test pit (right)

The selection of excavation techniques and size will depend on the sampling and testing required for the investigation to evaluate the concrete pavement and the underlying foundation. Table 2.7 summarises the most common categories used within the UK for concrete pavement investigations, these are grouped into ‘trial hole’ and ‘test pit’ based on the reinstatement implications. Each of these technologies is described in the following sections.

Table 2.7 Comparison of excavation techniques

Technique	**Coring**			**Saw cutting**
	Trial hole	**Test pit**		
Excavation / **Capability**	**Small core 100 - 150 mm**	**Large core > 300 mm**	**Secant coring**	**Saw cutting**
Layer type, thickness and condition	●	●	●	●
Depth of defect	●	●	Can be aligned to show defect profile	Can be aligned to show defect profile
Position of steel reinforcement	If diameter is larger than steel spacing or aligned to encounter reinforcement	●	●	●
Concrete properties	Laboratory testing of core	Intact core can be sub sampled for laboratory testing or compositional analysis	Compositional analysis, initial core will have suitable dimensions for laboratory testing	Intact slab can be sub sampled for laboratory testing or compositional analysis
In situ foundation testing and sampling	DCP through core hole	DCP, LWD at base of core hole and material sampling	DCP and material sampling, LWD if large diameter core used	DCP, LWD at base of hole and material sampling
Reinstatement considerations	Small repair	Temporary reinstatement followed by full depth repair or bay replacement.		
Key: ● = measurement capability				

2.2.2.1. Coring

The most common invasive sampling method is coring, and there is a requirement in CD 227 for coring to be undertaken during all scheme level investigations [1]. Cores can be used to determine the:

- condition and thickness of each of the bound layers;
- presence and location of steel reinforcement;
- depth of defects (such as cracking and spalling);
- severity of defects and causation; and,
- the condition of joints and cracks (including condition of dowel and tie bars and degree of interlock).

The strategy for determining the optimum locations for coring depends on the specifics of each site and the findings of the initial investigation. Similarly, the diameter of the core will depend on the scope of the investigation. Coring can have structural implications on the pavement and permanent reinstatement is a particular challenge for concrete pavements, so the diameter of cores should be minimised where possible. Typically, 150 mm diameter cores are taken but 100 mm diameter cores can be extracted in sensitive areas; and ‘test pit’ cores up to 450 mm diameter can be extracted to enable collection of samples of the subgrade (see Section 2.2.3).

There is a requirement in CD 227 for at least one core per 200 m to be taken in lanes with visible defects. The use of test pits is not a requirement. Wherever the cause of the defect is unclear then cores should be taken both at the defect and in the adjacent area of intact pavement to gain an understanding of the relative change in pavement performance [1].

Following core extraction:

- laboratory testing can be undertaken on the recovered bound layers;
- direct in situ testing, LWD or dynamic cone penetrometer (DCP) testing (see Section 2.2.2.3) can be undertaken on the unbound foundation and subgrade layers; and,
- unbound foundation and subgrade layers can be sampled for laboratory testing from the larger diameter core holes or secant test pits.

After extraction, the cores should be carefully logged and photographed in the laboratory to record the full core details. Pictures should be taken of the core location on site, the core hole and the core as extracted for future reference (see Figure 2.18). Cores extracted where an asphalt surfacing material is present should be tested using a PAK marker to detect the possible presence of tar.

Figure 2.18 Example pavement material core log

2.2.2.2. Saw Cutting

Saw cut test pits are optional survey methods [1]. Test pits achieved by saw cutting through the concrete pavement layer are much slower and more disruptive than coring. Furthermore, the circular blade on the saw results in overcutting, beyond the limits of the test pit, creating butterfly corners that compromise the integrity of the surrounding surface. Large 450 mm diameter coring is a more straightforward means of gaining access to the foundation layers and subgrade.

Due to the size of test pits, a full depth repair (see Sections 5.4.2 to 5.4.4) or bay replacement (see Section 5.4.1) will be required shortly following any temporary reinstatement (typically with asphalt due to time constraints) to prevent rapid deterioration of the surrounding pavement. This requires an additional closure and significant expense; therefore, they should only be used to support investigations or scheme design where data cannot be obtained by any other means.

Test pits through the concrete pavement may be required to investigate [3]:

- sub-surface drainage issues;
- stepping / differential movement;
- poor LTE and / or the cause of pumping at joints; and,
- potentially suspect material beneath the concrete.

Test pits can also be used to:

- obtain bulk samples of the bound or unbound layers for laboratory testing; and,
- carry out detailed examination of the unbound layers or subgrade including in situ testing.

The plan dimensions of a test pit depend on the type of investigation and the required final depth. A typical plan size could be 0.6 m wide x 1.0 m long for a pit of 0.6 m depth [3]. The test pit should be logged and photographed by a suitably experienced engineer.

2.2.2.3. Invasive in situ testing

Dynamic cone penetrometer (DCP)

Dynamic cone penetrometer (DCP) testing is an invasive test that can be conducted at the base of a core hole when unbound material has been exposed. It is required for all scheme level investigations [1]. The DCP is used determining the in situ properties of unbound foundation materials and subgrade soils (see Figure 2.19).

The DCP has gained wide-spread popularity, largely because the testing is rapid and it easy to use, portable and robust. It economically provides a reliable estimate of the thickness and stiffness modulus of the unbound foundation and upper subgrade materials, as there is a direct correlation between the resistance to penetration and the strength of a soil.

The strength profile of the foundation can give an indication of any lack of support to the concrete slab through poor quality materials, inadequate compaction, or ingress of water (indicating possible drainage problems) [1].

Testing should be carried out through the core holes immediately following the core extraction and before recovering any surface sample of the exposed unbound foundation material. CD 227 requires that DCP testing is undertaken in at least one-third of the core holes [1].

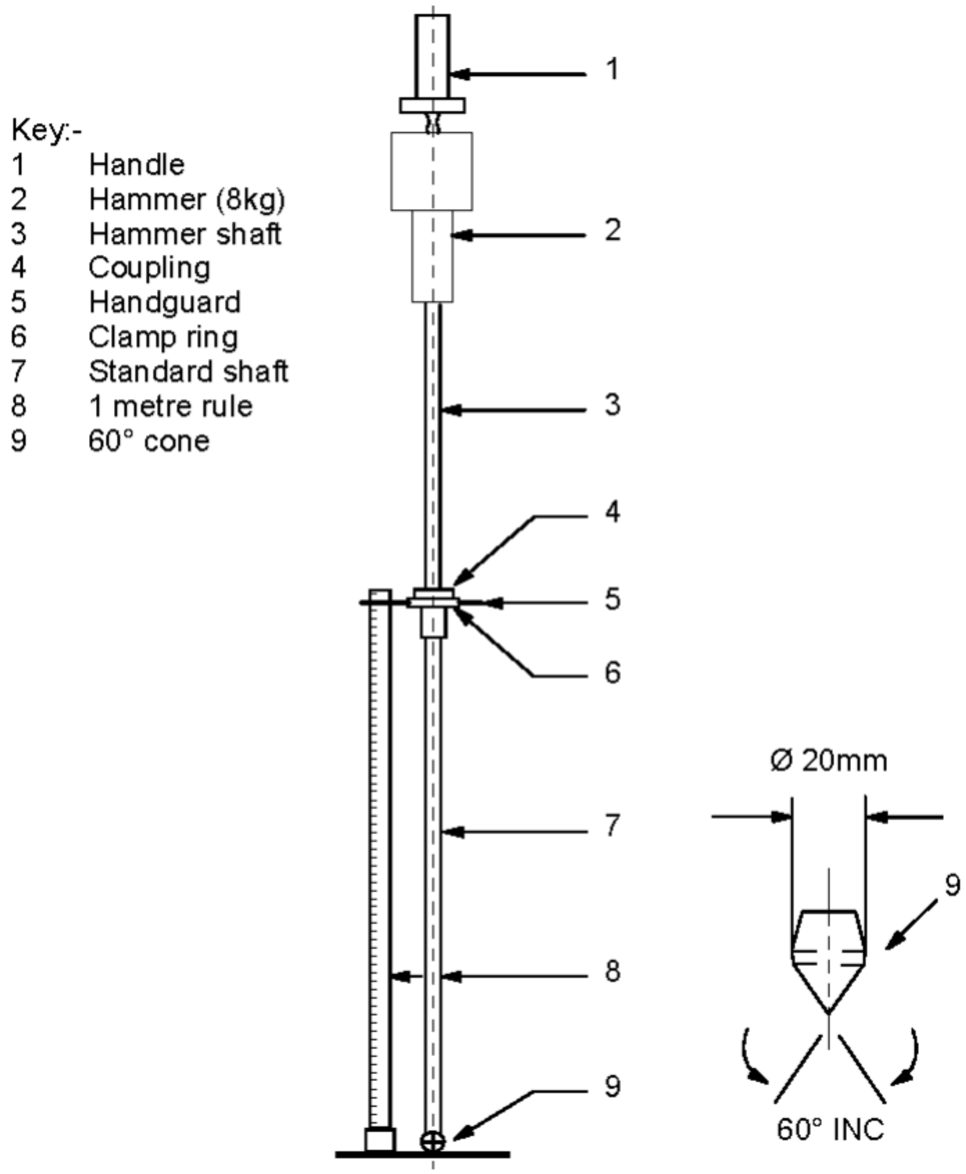


Figure 2.19 Illustration of DCP equipment [3]

The data obtained with a DCP is plotted as the cumulative number of blows against depth of penetration. A change in slope of the plotted data indicates a change of strength and / or material type. Although step-changes in the penetration resistance can indicate a new layer interface, shallow refusals can be related to the presence of large size aggregate and / or a particularly strong, or weakly bound, material.

The penetration rate can be converted to a stiffness modulus for each layer then used to estimate the surface modulus using the relationships outlined in Clause 882 [22]. Further information on the application of DCP can be found in CS 229 [3].

Light weight deflectometer (LWD)

The light weight deflectometer (LWD) is a portable dynamic plate test (DPT) device that is suitable for establishing the surface modulus of exposed pavement foundations and subgrades (see Figure 2.20). The device imparts a transient load (pulse) through a loading plate and measures (directly or indirectly) the movement under load of the ground (or the loading plate) [23].

The measurements are interpreted as the surface modulus, which is the response of the layers within the test's zone of influence to the applied load [23]. The zone of influence is dependent on several factors including the applied load, plate diameter and material response, specifically layer stiffness.

Figure 2.20 LWD in use

Where test pits are undertaken, an LWD may be used to give an indication of foundation conditions. The pit needs to be large enough to accommodate a 300 mm diameter loading plate. Information on the application of LWDs is given in MCHW 800 and BS 1924-2 [24].

2.2.3. Laboratory testing

Laboratory testing is not always required for scheme level investigations and should be targeted for specific design or investigation reasons. However, laboratory testing in conjunction with a comprehensive site investigation can be valuable in determining relevant material properties, undertaking failure investigations, and evaluating options for recycling the concrete pavement at end of life. The properties of the concrete are often considered when estimating pavement residual life.

Laboratory testing is typically used to:

- confirm the in situ concrete compressive strength of recovered core samples;
- confirm or clarify certain results from VCS or NDT surveys;
- provide additional insight into the mechanisms of defects; and,
- provide additional information needed to identify treatment alternatives.

When laboratory testing is needed, the number and type(s) of laboratory tests should be specified after an assessment of the field data. The testing itself should be in accordance with the European and British Standards and carried out by accredited laboratories [1].

The extent of the testing programme depends mostly on the types of observed defects or suspected failure mechanisms. This section presents some of the most common laboratory testing methods used in the evaluation of pavement layer materials. The types of tests discussed here are divided into the general categories of concrete testing and soil testing.

2.2.3.1. Concrete testing

Laboratory testing on concrete might be carried out to evaluate properties of the mixture or to understand the cause and extent of material-related defects and features. There are many laboratory tests that can be undertaken on concrete cores and samples.

Routine testing typically comprises compressive strength testing in accordance with BS EN 12390-3 on core samples to determine integrity; validate FWD results; and approximate design compressive and flexural strength for the purposes of estimating pavement residual life and as necessary potential overlay treatments.

For compressive strength testing, the preferred diameter of cores is 150 mm and the preferred length to diameter ratio is 2:1 if the strength result is to be compared to the cylinder strength, and 1:1 if the strength result is to be compared to the cube strength [11] [25]. The length to diameter ratio should not be lower than 1:1. The length to diameter ratio of the test specimen can be reduced by grinding as outlined in Annex A of BS EN 12390-3. Ideally, cores should be obtained free of reinforcement, but if testing is carried out on specimens containing reinforcement, it is necessary to apply correction factors to obtain the material strength [25].

Additional non-routine testing, which may support a targeted investigation, is outlined in Table 2.8.

Table 2.8 Non-routine testing on concrete cores and samples

Test / reference	Purpose of testing
Freeze-thaw resistance **PD CEN/TS 12390-9:2016**	Identify possible cause of surface defects including scaling, crazing and spalling.
Alkali-silica and alkali-carbonate reactivity **BS 812-123:1999**	Identify possible causes of surface defects including scaling and expansion related defects such as heave, spalling and compression failures (blow-ups) [19].
Coefficient of thermal expansion **MCHW Clause 871**	Evaluate compatibility of repair materials for thin bonded repairs and shallow repairs, and bonded concrete overlay materials.
Petrographic analysis **BS 1881-211:2016**	Investigate the composition and characteristics of the concrete including aggregate type, cement type, voids and water / cement ratio. Reasons include assessing suitability of existing concrete for texturing; compatibility of materials for thin bonded repairs and shallow repairs and identifying any potential durability issues such as sulfate attack.
pH **PD CEN/TR 17310:2019**	Investigate the level of carbonation of concrete (see Section 4.1.2) and aggressive ground conditions (e.g. acidic groundwater).
Water soluble sulfates **BS 1881-124:2015**	Investigate potential concrete durability issues related to propensity of concrete to sulfate attack.
Acid soluble sulfates **BS 1881-124:2015**	Investigate potential concrete durability issues related to propensity of concrete to sulfate attack.

2.2.3.2. Soil testing

Collected subgrade samples are often subjected to a series of standard laboratory tests. The most common laboratory soil testing methods include:

- soil classification;
- water content determination;
- particle size distribution (PSD);
- determination of liquid limit and plastic limit to give plasticity index (Atterberg limits); and,
- density determination.

These tests are primarily carried out to understand if the properties of the materials have changed since construction or to understand causation when subgrade related defects have occurred. Additional strength-related testing such as triaxial testing and resilient modulus testing, can be used to assess the ability of a pavement to adequately carry repeated traffic loadings when undertaking overlay or reconstruction designs.

Soil laboratory testing results should be used in conjunction with the results from the other investigation techniques for a correct diagnosis of defect causation. If the moisture content of the soils or unbound materials is being assessed, allowance should be made for their variation with time, for example between summer and winter or over shorter periods following rainfall, particularly for cracked pavements.

2.3. Drainage surveys

Water is often a major factor in pavement deterioration. Moisture sources are typically rainwater, runoff and high groundwater. Failure to drain water properly from a pavement will result in premature deterioration and failure.

Drainage surveys should be carried out regularly, as a minimum when scheme level surveys are taking place and analysed in conjunction with the rest of the investigation so that action can be taken.

CS 551 details requirements for undertaking drainage surveys [26].

A visual inspection of manholes, catch pits and gullies after rainfall, or a water test, will reveal whether water is standing in the system. Examination of the outfall pipes will confirm whether they are functioning correctly. If there is evidence of blockages within the system, a CCTV survey, with jetting as required, should be carried out.

Where the edge drains are of the combined filter drain type, the presence of excessive growth and detritus over the filter media may indicate that they have become contaminated and rendered ineffective or partially effective. Should there be any doubt, further examination is required which can be achieved by excavating a short length down to pipe level.

Current standards require sub-surface drains to be provided where subbase and capping terminate. In embankments, where sub-surface drains are not present, the subbase and capping should be extended to the side slopes. If this has not been done, the lower unbound layers of the pavement construction may be prone to holding water.

2.4. Monitoring techniques

The in service monitoring of pavements can provide crucial information on pavement condition to complement routine and targeted localised surveys and assist in determining the effectiveness of maintenance treatments. A variety of sensors and techniques are available to monitor the noise generated at the pavement surface and evaluate the impact on the neighbouring community and road users (Section 2.4.1).

Technologies are emerging to monitor the pavement condition during the in service life that will indicate the structural integrity of the pavement, which in turn allows for timely maintenance (Section 2.4.2).

2.4.1. Noise

The impact of road traffic noise can have a significant impact on the quality of life for residents close to major road networks. As vehicles traverse pavements, the interaction between the vehicle tyres and the pavement generates a level of noise that is dependent on the vehicle speed as well as the properties of the tyre and pavement [27].

Noise is one of the functional characteristics of a pavement and it is one of the most important characteristics of a concrete pavement, as perceived by road users [28]. The mechanisms of tyre / road noise generation can be traced to the following mechanisms: (i) tread impact, (ii) air pumping, (iii) stick-slip and (iv) stick-snap. A comprehensive description of these mechanisms can be found in [27] and [29].

One of the most effective measures for reducing the noise from road traffic, particularly on high-speed roads, is to ensure the use of a low noise road surface. The acoustic performance of concrete pavements is generally considered to be poor compared with negatively textured asphalt thin surface course systems. Brushed concrete for instance is associated with high levels of vehicle noise, around 1 - 2 dB louder than hot rolled asphalt (HRA) surface. This has led to a significant amount of concrete on the SRN being overlaid with a lower noise generating thin surface course system.

However, recent measurements of concrete pavements which have been subject to longitudinal diamond grinding (see Section 7.4.1.3) have been shown to provide around 4 - 5 dB lower noise than a HRA surface and give similar noise generation characteristics to asphalt thin surface course systems [30].

An understanding of the noise generation characteristics of surfaces is necessary to inform selection of a surfacing material or texturing technique. Field tests are vital for the assessment of noise generation characteristics of a pavement to avoid subjectivity. Measurement can either be taken by ‘moving’ on-board microphones or with non-moving microphones capturing vehicle passing by. However, field tests are subject to the variability of the in situ environment. The most relevant field tests include:

- statistical pass-by method;
- close proximity method; and,
- on-board sound intensity.

Research has shown that road user response to noise is variable and influenced by many factors such as perception of the noise source, physiological condition of the subject and time of day amongst others. The intensity, tonal and intermittent characteristics of the noise have been shown to affect user response to noise [28].

Subjective assessments have suggested that road users prefer to drive on asphalt thin surface course systems (TSCS) rather than concrete surfaces. Concrete surfaces which have been subject to longitudinal diamond grinding (see Section 7.4.1.3) were preferred over untreated and fine-milled concrete surfaces (see Section 7.4.1.2), and there was no overall preference between untreated and fine-milled concrete.

More information on acoustic measurements (in field and laboratory) can be found in [27] and [29].

2.4.1.1. Statistical pass-by method

Statistical pass-by (SPB) is probably the most frequently used procedure for assessing the acoustic performance of a pavement. The procedure is defined in BS EN ISO 11819-1 [31] and it involves taking kerbside measurements of the speed and maximum A-weighted sound pressure level (L_{Amax}) of a statistically representative number of vehicles [32] (see Figure 2.21).

The tachometer for speed measurements and microphone for sound level measurements are installed on the roadside at 7.5 m from the centre of the lane at a height of 1.2 m.

Figure 2.21 SPB setup

The measurement gives absolute levels of road traffic noise and is used to compare the effect of road surfaces on noise. Its main benefits are related to the portability and ease of use of the equipment required and the statistical significance of results.

However, there are limitations with the SPB method:

- The static nature of the equipment means any variability of performance characteristics across the section in the case of surface inconsistencies is not captured.
- Factors such as wind speed and temperature can affect results as the method relies on far field noise measurements.
- The method has stringent measurement condition criteria, including distance from the traffic and the presence of trees, embankments and verges, meaning it is not suitable or practicable for extensive sections of pavement on the SRN.

2.4.1.2. Close proximity method

The close proximity method (CPX) is another standard method for pavement noise assessment. As specified in BS EN ISO 11819-2 [33], this method is designed to assess the acoustic properties of a road surface by measuring the rolling noise of a set of standard reference tyres and two microphones (see Figure 2.22). These microphones are surrounded by a soundproof enclosure and mounted beneath a moving test vehicle-trailer having tyres with defined tread pattern [32].

Figure 2.22 CPX setup [27]

During testing the tyre is allowed to roll freely at a constant speed over the road surface. Results are averaged over 20 m road sections and across the two microphone positions.

The main advantages of CPX are:

- it is a direct noise measure, meaning lower influence of other extraneous noise sources;
- it can be used for assessing any length; and,
- there are minimal practical constraints on performing the test.

However, the limitations include:

- the method includes the estimation of a transfer function to account for the difference between the exterior tyre noise and interior noise;
- it can only measure noise accurately at certain constant speeds; and,
- the test requires a specialised, instrumented vehicle or trailer.

2.4.1.3. On-board sound intensity method

The on-board sound intensity (OBSI) method is a near-field technique that measures tyre / road noise in close proximity to the source [27]. The procedure is standardised as per AASHTO TP 76.

The OBSI method measures tyre / road noise using a phase-matched pair of microphones that are located in such a manner to isolate sound generated near the tyre / road contact patch. The standard vehicle speed is 60 mph.

2.4.2. In service pavement condition monitoring

Pavement condition monitoring techniques provide the necessary inputs to evaluate the structural integrity of the pavement, which in turn allows for timely maintenance. The deployment of embedded technologies that can monitor structures continuously is an emerging method of understanding real time deterioration throughout the life of asphalt and concrete pavements. Embedded technologies can also be used to monitor early life performance of newly constructed pavements and repairs, giving indications of when the pavement is suitable for opening.

This section outlines some of the emerging embedded technologies being used to monitor the early life and long term condition of concrete.

2.4.2.1. Fibre optics for monitoring concrete deterioration

Embedded fibre optic sensors are emerging technologies for quantitative non-destructive long-term monitoring of concrete pavements, capable of measuring changes in strain under load. They can be used to monitor parameters such as temperature, deformation, corrosion, vehicle speed and frequency. They are small and geometrically adaptable. However, they are brittle, so some form of protection, typically a metal sheathe, is required to ensure they do not break in service under load.

Fibre optic sensors can be divided into two distinct types:

- Fibre Bragg gratings (FBGs); and,
- Brillouin based sensors.

Fibre Bragg gratings (FBGs) are systems which detect changes in strain and temperature at a single point in the fibre length (see Figure 2.23). The FBG is a section of the fibre over which the refractive index is modified to a specific wavelength, known as the Bragg wavelength. As the section of fibre is strained, this wavelength will change. The change in wavelength can be measured and used to determine the strain that the fibre is experiencing. Cables are typically produced with FBGs at a given, constant, spacing to take measurements at regular intervals.

Figure 2.23 Fibre Bragg grating fibre optic sensor [34]

Brillouin based sensors are systems which use the principle of Brillouin scattering to detect changes in strain and temperature along the length of the fibre. Brillouin scattering occurs due to the interaction between the transmitted light through the fibre and the fibre along its length which causes light to be backscattered. The amount of backscattering is dependent on changes in the fibre's structure (due to temperature or strain).

Both types have their distinct advantages. FBG measurements can be pinpointed back to specific areas of pavement, meaning deterioration can be compared at joints and edges versus in the centre of bays. Whereas Brillouin based sensors can give an overall indication of the pavement condition along the length of the cable.

The challenge for all types of sensor is installation, ensuring they are placed in the correct location to be subject to a sufficient amount of strain to take notable measurements but not subject to so much strain that they break in service. The particular issue with jointed concrete pavements is how to span joints with the sensors, which will expand and contract, without the sensors breaking. One solution as shown in Figure 2.24 includes a sheathing system whereby the slack fibre is placed within tubing between the bays, reducing the risk of excessive movement breaking the fibre [35].

In order to position the fibres at the required depth within a concrete slab, the fibre may be attached to the steel reinforcement within the concrete section or alternatively, specifically selected chairs can be placed at the desired depth with connecting bars for the fibre to be attached.

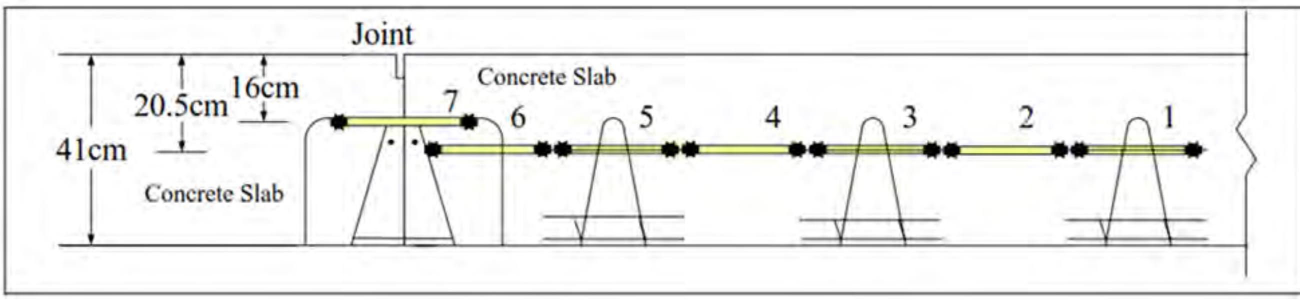


Figure 2.24 Cross section of fibre optic sensor installation in a concrete pavement [35]

Further information about embedded fibre optic sensor installation and performance in concrete structures can be found in [36].

2.4.2.2. Sensors for monitoring concrete curing

Cementitious materials require an initial period of curing (strength gain) before they can be trafficked to avoid damage. Depending on the type of concrete used, this initial curing can be as low as 3 hours for early strength concrete and as much as 28 days for conventional pavement concrete.

For time critical work, the strength gain needs to be monitored to ensure the concrete reaches adequate levels of compressive strength (typically 25 N/mm^2) to prevent the concrete from being opened to traffic too early while minimising closure durations.

Traditionally, monitoring curing has meant taking samples when the concrete is poured and undertaking compressive strength testing in a laboratory environment. However, this may result in:

- different rates of strength gain during the curing period between concrete samples and the concrete installed on site. This is due to the possible differences in curing and temperature conditions on and off site;
- delay in receiving testing results, increasing closure times; and,
- testing the samples either too early or too late.

An alternative to conventional compressive strength testing is the use of sensors within concrete that can provide real time measurements to a connected mobile device or cloud based system. They can be external or buried wireless devices (see Figure 2.25).

The parameters measured include temperature and strain, which can be correlated to the strength development of the concrete. This can reduce the need for laboratory testing and allow trafficking sooner, reducing disruption [37]. Correlation of the strength-maturity relationship with the temperature of the specific concrete mixture and cube compressive strength is required to provide useful readings.

Figure 2.25 External transmitter with thermocouple placed in slab (left) and wireless sensor being installed within slab (right)

The positioning of the sensors and thermocouples is important to retrieve accurate and reliable data. The optimum placement for thermocouples and strain gauges is within the 20 - 30 mm of the slab surface. The surface will cool the quickest so placement in this area will represent the most conservative scenario with respect to the curing / strength gain of the slab.

Buried devices are typically installed on reinforcing steel, chairs or steel pins where reinforcement is not present (see Figure 2.25) as data may not transmit if placed too deep in the slab. Some wireless sensors need to be within 50 mm of the concrete surface for readings to transmit to a mobile device. The battery life of the sensors is up to 4 months after installation, depending on the interval between readings.

3. Defects and features

Concrete pavements deteriorate due to multiple factors, mostly related to the in-service loading (both traffic and environmental), design and maintenance decisions, construction quality and the durability of the materials used. Deterioration would, subject to any pro-active maintenance, ultimately result in the occurrence of defects.

A defect is defined as a shortcoming, imperfection, or deficiency. In pavements, a defect refers to evidence of an undesirable condition affecting the serviceability, structural condition or appearance. Joint spalling is an example of a concrete pavement defect that affects appearance, accelerates pavement deterioration by permitting ingress of water, salts and incompressible materials into the pavement, and may also, if located in the wheel track, affect the serviceability. Defects in concrete pavements can be broadly categorised as being surface or structural [1]:

- Surface defects are present only within the upper third of the slab thickness.
- Structural defects are present at a depth and / or extend greater than one third slab thickness including the pavement foundation.

Concrete pavements can also have or develop distinct features, which are a function of the design and construction of the pavement. A feature, in pavements, is defined as a distinctive attribute that is expected under specific conditions; it may indicate good or poor design and / or construction. An example of a good feature is the regular spaced fine transverse cracking that develops in a well-designed and well-constructed CRCP.

This chapter focuses on the identification of defects and features in concrete pavements and provides guidance on the selection of the appropriate maintenance options based on causation, severity and the asset management strategy being used. Defects are grouped based on the pavement type (i.e. URC, JRC or CRCP) in which they occur, and their type (surface or structural). Photographs are included to aid the identification of defects and judge severity. Further guidance on the identification of defects is given in Appendix D.

An understanding of the underlying causation of defects is key to undertaking the appropriate repair (Chapter 5). If the underlying causation is not addressed, defects may rapidly reoccur. In this chapter, possible causation is discussed for each defect. Chapter 4 gives additional details of the general causation of deterioration of concrete pavements, which leads to defects and investigation techniques for identifying the underlying causation.

Depending on the defect severity, treatment may be needed, either in the short-term (perhaps even as an emergency) or in the longer term (allowing better planning), to ensure that the pavement remains serviceable and safe to use. For each type of defect, long-term repair options and, where it is appropriate, holding repair options, linked to defect severity and pavement type, are outlined:

- Holding repairs reduce the rate of deterioration of the pavement without necessarily rectifying the defect or addressing the underlying causation. Holding repairs typically have service lives of 3 to 7 years before additional repair works may be required, depending on various factors not limited to the defect location, repair type, pavement loading and pavement condition.

- Long-term repairs rectify defects and address the underlying causation and are expected to have service lives greater than 10 years.

The treatment options for each defect are summarised in Appendix B. Recommended treatment options are provided based on the pavement type and the characteristics (typically severity) of the defect.

Whilst treatment options are provided on a defect-by-defect basis, it is important that defects are not viewed in isolation, but in the context of the overall condition of the pavement. Where a pavement is in poor condition with frequent defects requiring costly maintenance, better value for money may be obtained by overlay or reconstruction of the pavement (see Chapter 7) as opposed to continued reactive repair of distinct defects.

As illustrated in Section 1.5, the selection of treatment will depend on factors including safety, WLC and network availability, as part of the maintenance strategy adopted by the asset manager.

3.1. Surface defects

Minor defects or imperfections, confined to the upper third of the slab, in a concrete pavement are often referred to as surface defects. These include:

- surface irregularities (see Section 3.1.1);
- surface scaling (see Section 3.1.2);
- crazing (see Section 3.1.3);
- pop-outs (see Section 3.1.4);
- defective joint seals (see Section 3.1.5); and,
- shallow joint spalls (see Section 3.1.6).

These defects typically do not significantly affect the structural integrity of the pavement, but they can have an impact on its aesthetic appearance and accelerate pavement deterioration by permitting ingress of water, salts and incompressible materials into the pavement, and potentially deteriorate into hazards that require treatment, impacting on the pavement serviceability.

Surface defects in concrete pavements with an asphalt overlay can include rutting, fretting (loss of aggregate), potholes, delamination and reflective cracking. This document provides guidance on asphalt overlay treatments in Chapter 7. However, defects within an asphalt overlay are not discussed in detail.

3.1.1. Surface irregularities (poor surface profile)

Surface irregularities are bumps or depressions in the pavement surface. These defects can affect the ride quality and at a ‘severe deterioration’ level (i.e. condition category 4 as defined in CS 230 [4]), they may pose a safety hazard to moving vehicles.

Table 3.1 gives an overview of the causation, treatment options and applicability for surface irregularities. They most commonly occur at transitions between rigid and flexible pavement types and at transitions with structures. They may be visible to the naked eye, or apparent from the level of linear assets i.e. kerbing, or rubber deposits left as a result of dynamic vehicle loading as shown in Figure 3.1. Surface irregularities are the result of the occurrence of other structural defects, including:

- depressions (settlement) (see Section 3.2.1);
- heave (see Section 3.2.2);
- joint stepping (see Section 3.3.7); and,
- compression failures (blow-ups) (see Section 3.3.9).

Table 3.1 Overview of surface irregularities

Considerations	Treatment options	Applicability
Caused by the occurrence of other defects, including: • **settlement;** • **heave;** • **compression failures; and,** • **stepping.** **Any treatment should be in consideration of the surface irregularity causation. Holding repair options are short-term solutions.**	**Holding repair**	
	Bump cutting (see Section 5.4.7).	Localised high areas. Check impact on carriageway profile and cover to any reinforcement.
	Thin bonded repair (see Section 5.3.2).	Localised low areas, to a maximum of 40 mm.
	Long-term repair	
	Full depth repair (see Sections 5.4.2 and 5.4.4).	JRC and CRCP
	Bay replacement (see Section 5.4.1).	URC

Figure 3.1 Surface irregularities evidenced by rubber deposits on road surface

3.1.1.1. Treatment options

For surface irregularities, where treatment has been determined to be necessary, the treatment options available will be dependent on the causation of the surface irregularity. The causations discussed previously in this section are typically as a result of foundation and / or drainage issues; therefore, further investigation is often necessary prior to treatment unless a reoccurrence of the surface irregularity is acceptable and / or if the repair is only needed temporarily.

Holding repair options for low severity irregularities include **bump cutting** (see Section 5.4.7) for localised high areas and **thin bonded repairs** (see Section 5.3.2) for localised low areas. These treatments can restore surface profile temporarily; however, the surface irregularities can be expected to reoccur as the underlying defect causation has not been addressed.

When undertaken on JRC and CRCP, bump cutting will reduce the cover to the steel reinforcement. The implications of this may be reduced pavement durability and accelerated deterioration of the pavement. Generally, a minimum of 50 mm cover is necessary to provide corrosion protection and prevent cracking and spalling from occurring directly above the reinforcement bars.

The area to receive a thin bonded repair will need to be prepared to receive a consistent nominal depth of repair, which may mean that removal of some of the existing concrete is required. This option is limited to a maximum of 40 mm depth, as greater depths risk curling and debonding failure.

Long-term repairs typically will consist either **full depth repairs** (see Sections 5.4.2 to 5.4.4) or **bay replacement** (see Section 5.4.1); foundation replacement and drainage renewal may be necessary dependent on defect causation.

3.1.2. Surface scaling

Surface scaling is the delamination or disintegration of the slab surface to the depth of the defect, occurring over a portion of a slab [38]. Examples of surface scaling are presented in Figure 3.2. Table 3.2 gives an overview of the causation, treatment options and applicability for surface scaling.

Figure 3.2 Surface scaling

Table 3.2 Overview of surface scaling

Considerations	Treatment options	Applicability
Causation includes: • **freeze-thaw, weathering and carbonation (see Section 4.1.2);** • **improper finishing and curing during construction (see Section 4.3.3);** • **inadequate cement content or insufficient entrained air (see Section 4.3.3);** • **damage from rain or other climate conditions during construction (see Section 4.3.3); or,** • **fuel or chemical spillages or vehicle fire (see Section 4.3.3).**	**No treatment advised**	
	Where a risk to safety or serviceability is not posed. Requires monitoring.	
	Holding repair	
	Retexturing (see Section 7.4.1).	Check impact on carriageway profile and cover to any reinforcement.
	Long-term repair	
	Thin bonded repair (see Section 5.3.2).	Small areas (typically up to 1 m^2). Up to 40 mm in depth.
	Shallow repair with cementitious or resin repair material (see Section 5.3.2).	Small areas (typically up to 1 m^2). 40 mm - one third slab depth.
	Full depth repair (see Sections 5.4.2 to 5.4.4).	JRC and CRCP in severe cases.
	Bay replacement (see Section 5.4.1).	URC in severe cases.
	Overlay or reconstruction (see Chapter 7).	Widespread across a section of carriageway.

3.1.2.1. Causation

Surface scaling is caused by exposure to freeze-thaw cycles (see Section 4.1.2), due to the expansion that occurs when water changes to ice. Shallow cracks develop as a result of the expansion of water as it freezes, in saturated concrete, which can propagate with further water ingress and subsequent freeze-thaw cycles. Although efforts are made to prevent ice formation on pavements, the use of chemical de-icing solutions can accelerate the deterioration if certain de-icing salts come into contact with unprotected reinforcement, as chlorides are corrosive to steel.

Carbonation through natural weathering is also a factor in the occurrence of scaling (see Section 4.1.2). Carbonation increases the porosity of the concrete, leaving the concrete more susceptible to freeze-thaw. The level of carbonation of concrete increases with age, so carbonation is unlikely to be the cause of scaling in pavements that were constructed relatively recently i.e. within the last 20 years.

In newer pavements, scaling can occur if the conditions during the placement or curing of the concrete were inappropriate, including low temperatures resulting in frost attack during placement and curing, or rain increasing the water / cement ratio at the surface, resulting in a weak layer.

Improper finishing and curing are also factors. Scaling can occur if the concrete is overworked and / or the surface is watered to achieve the desired finish. Finishing the surface while there is still bleed water can weaken the concrete surface significantly. If curing does not commence in a timely manner using recognised curing practices, excessive moisture loss can occur, resulting in a weak layer.

Other factors that can contribute to scaling include:

- an inadequate cement content or insufficient entrained air in the mixture;
- vehicle fires; and,
- acid attack due to spillages.

3.1.2.2. Treatment options

Minor surface scaling typically does not require treatment unless it poses a safety or serviceability risk.

Where deterioration is limited to the surface of the pavement it may be possible to restore surface smoothness and improve appearance by **retexturing** (see Section 7.4.1). However, the implications of a reduction in pavement thickness on pavement durability should be assessed. A reduction in thickness will reduce the structural life, and a reduction in cover to any reinforcement may accelerate the occurrence of structural defects due to reduced protection from external substances such as water and de-icing salts. As a point of reference, new concrete pavements require a minimum cover of 50 mm to reinforcement. Any carriageway profile implications resulting from retexturing should be considered. Chapter 2 outlines techniques for assessing reinforcement cover.

For small areas typically up to 1 m^2, a **thin bonded repair** or **shallow repair** using a cementitious or resin repair material (see Section 5.3.2) can be undertaken, dependent on treatment depth. However, if the surrounding concrete surface is similarly susceptible to the same causation then future scaling may occur around the area of the repair. Sounding (see Section 2.2.1.6) can be used to inform the area of deterioration. As shallow repairs are over 40 mm thick, there is a greater risk of curling and debonding failure of the repair, especially if it has not been adequately prepared and cementitious or resin repair materials are used.

In severe cases of scaling, then **bay replacement** (see Section 5.4.1) or **full depth repair** (see Section 5.4.2 to 5.4.4) may be appropriate.

For large areas, or in anticipation of severe scaling becoming widespread across a section of carriageway where the surrounding concrete surface is similarly susceptible to scaling, the options outlined in Chapter 7 should be explored.

Alternatively, surface impregnation coatings, as used on airfields and structures in the UK and on highways internationally, might be applied to inhibit the ingress of moisture and prevent further deterioration of the surface. This may arrest any surface scaling deterioration and could be considered as a ‘holding repair’. A departure from standard application is required for any use of surface impregnation coatings as at the time of publication surface impregnation coatings have not been used on the SRN and there is no specification governing their use. In addition, the effectiveness and the implication of these coatings on skidding resistance has not been confirmed.

3.1.3. Crazing

Crazing, also known as ‘map cracking’, comprises a network of shallow, narrow or hairline cracks which extend only through the upper surface of the concrete. The cracks tend to intersect at angles of 120 degrees (see Figure 3.3). Crazing may be localised or occur over the entire surface of the concrete slab. Crazing can lead to scaling (3.1.2) as moisture and de-icing salts can ingress through the fine cracks. Table 3.3 gives an overview of the causation and treatment options for crazing.

Figure 3.3 Crazing

Table 3.3 Overview of crazing

<table>
<tr><th>Considerations</th><th>Treatment options</th><th>Applicability</th></tr>
<tr><td rowspan="10">Causation includes:
overworking the concrete surface during finishing and / or improper curing (see Section 4.3.3);
excessive laitance (see Section 4.3.3) or,
alkali-silica reaction (see Section 4.1.2).</td><td colspan="2">No treatment advised</td></tr>
<tr><td colspan="2">Where surface layer is relatively intact and a risk to safety or serviceability is not posed. Requires monitoring.</td></tr>
<tr><td colspan="2">Holding repair</td></tr>
<tr><td>Retexturing (see Section 7.4.1).</td><td>Check impact on carriageway profile and cover to any reinforcement.</td></tr>
<tr><td>Thin bonded repair (see Section 5.3.2).</td><td>Small areas (typically up to 1 m²). Up to 40 mm in depth.</td></tr>
<tr><td>Shallow repairs with cementitious or resin repair material (see Section 5.3.2).</td><td>Small areas (typically up to 1 m²). 40 mm to one third slab depth.</td></tr>
<tr><td colspan="2">Long-term repair</td></tr>
<tr><td>Life extension or reconstruction (see Chapter 7).</td><td>Extensive crazing or where ASR is identified.</td></tr>
</table>

3.1.3.1. Causation

Crazing is usually caused by excessive laitance or improper curing during construction (see Section 4.3.3). Specifically, crazing can occur when a weak layer of cement and aggregate fines is allowed to develop at the concrete surface. This weak layer can be formed by:

- the concrete having too much water in the mix;
- the concrete continuing to be worked when bleed water is present;
- additional water being added during finishing, as entrained air can be displaced and the increased water / cement ratio at the surface results in a weak layer; or,
- curing being either delayed or inadequate, leading to excessive moisture loss that results in excessive drying shrinkage and poor strength gain.

However, if there are visual signs of expansion and / or gels or staining at the surface, this is an indication that the crazing is being caused by alkali-silica reaction (ASR) (see Section 4.1.2).

3.1.3.2. Treatment options

Minor crazing is not typically a major issue. If there is not a significant loss of the surface layer it does not require treatment unless it poses a risk to safety or serviceability. The area should be monitored regularly for signs of further deterioration.

Minor crazing limited to localised areas which may cause a future hazard can be treated with **retexturing** (see Section 7.4.1). However, the implications of a reduction in pavement thickness on pavement durability should be assessed. A reduction in thickness will reduce the structural life, and a reduction in cover to any reinforcement may accelerate the occurrence of structural defects due to reduced protection from external substances such as water and de-icing salts. As a point of reference, new concrete pavements require a minimum cover of 50 mm to reinforcement. Chapter 2 outlines techniques for assessing reinforcement cover. Any carriageway profile implications resulting from retexturing should also be considered.

Thin bonded repair or **shallow repairs** using cementitious or resin repair material (see Section 5.3.2) may be appropriate for small areas up to 1 m^2. As shallow repairs are over 40 mm thick, there is a greater risk of curling and debonding failure of the repair with this maintenance option.

If extensive crazing is observed or if ASR is identified, reconstruction as discussed in Chapter 7 should be explored.

3.1.4. Pop-outs

Pop-outs are isolated losses of surface material that typically range from 25 mm to 100 mm in diameter and 10 mm to 50 mm in depth, as shown in Figure 3.4 [38]. A fractured aggregate is often found at the bottom of the cavity where the pop-out has occurred. Table 3.4 gives an overview of the causation, treatment options and applicability for pop-outs.

Figure 3.4 Pop-outs and thin bonded repairs to pop-outs

Table 3.4 Overview of pop-outs

Considerations	Treatment options	Applicability
Causation includes: • **freeze-thaw (see Section 4.1.2);** • **alkali-silica reaction (see Section 4.1.2); or,** • **embedded foreign materials (see Section 4.3.3).**	**No treatment advised**	
	Pop-outs not posing a safety or serviceability risk.	
	Long-term repair	
	Single particle pop-out repair with resin mortar (see Section 5.3.1).	Isolated pop-outs to prevent deterioration and enlargement.
	Thin bonded repair (see Section 5.3.2).	For small areas with multiple pop-outs (typically up to 1 m^2). Up to 40 mm depth.
	Shallow repair with cementitious or resin repair material (see Section 5.3.2).	For extensive and severe instances only with multiple pop-outs. 40 mm - one third slab depth.

3.1.4.1. Causation

Pop-outs are most commonly caused by unsound aggregates being subjected to freeze-thaw action (see Section 4.1.2). Aggregates with high absorption values can hold water, which is problematic when used in concrete. The volumetric expansion of the freezing water can create enough pressure to either break the aggregate or separate the aggregate from the mortar, resulting in a pop-out. These defects will present themselves after periods of cold weather.

Some aggregates can be expansive under the right conditions. Physical or chemical reactions (including alkali-silica reaction), resulting in pop-outs, are associated with certain types of shale and siliceous aggregate, such as chert and flint.

Isolated pop-outs can be a result of the concrete containing clay, organic or friable materials. Pop-outs caused by this contamination tend to occur in early life. Retexturing may cause pop-outs depending on the strength of the aggregate in the concrete and the concrete itself.

3.1.4.2. Treatment options

Isolated pop-outs, particularly those outside the wheel track zones, often require no immediate treatment. The treatment for isolated single pop-outs is straightforward as they can be **plugged with a resin mortar** after minimal preparation to ensure that the cavity is clean (see Section 5.3.1). This will prevent enlargement of the pop-outs through traffic abrasion or deterioration of the surrounding pavement if water is allowed to sit and freeze in the cavity. For widespread pop-outs up to 40 mm depth, **thin bonded repairs** (see Section 5.3.2) can be undertaken. **Shallow repairs** using cementitious or resin repair material (see Section 5.3.2) can rectify pop-outs between 40 mm and one third slab depth.

3.1.5. Defective joint seals

As outlined in Chapter 1, concrete slabs expand and contract as the daily and seasonal temperature rises and falls. They also warp, or curl, when the upper surface temperature is substantially different from that of the underside of the slab. Within URC and JRC pavements longitudinal and transverse joints permit these movements to occur within acceptable limits. CRCP does not have regular transverse joints, but transverse joints are present at ground anchorages and longitudinal joints may be present between lanes.

Effective joint seals are vital to ensure that concrete pavements remain in a serviceable condition. They mitigate the risk of:

- salt ingress, which has the potential to corrode the steel (e.g. dowel bars, tie bars and reinforcement mesh);
- water infiltration into the concrete, which can cause deterioration with freeze-thaw cycles;
- water infiltration into the lower layers of the pavement, which can locally soften or increase the deterioration rate of these supporting layers; and,
- hard particles of road detritus entering and filling the joint space, which could inhibit expansion.

Most transverse joint seals do not last for the life of the pavement, the vast majority fail because of age, particularly if they are over five years old. Sealants are polymeric, including bitumen-based materials, that become brittle over time through exposure to oxygen and UV light. This embrittlement reduces the ability of the sealant to accommodate movement and ultimately results in adhesion or cohesion failure. Sealants that have been correctly applied in transverse joints are expected to last six to more than ten years, depending on the type used (see Section 5.2.1), unless called upon to accommodate movements outside their design range.

Joint seals in longitudinal joints typically only have to accommodate very small amounts of movement due to the tied nature of the joint; therefore, they often last significantly longer than joint seals in transverse joints.

'Defective joint seals' is an all-encompassing term for joint seals which are not performing their function (see Figure 3.5). This includes:

- absence of the joint seal, by vehicle pick up;
- stripping of the joint seal;
- debonding of the joint seal from the sides of the sealant groove (adhesion failure);
- cracking within the joint seal parallel or transverse to the joint groove (cohesion failure);
- joint seal pushed out of the sealant groove by compression (extrusion failure);
- hardening of the joint seal (oxidation); and,
- weed growth through the joint and joint seal.

Figure 3.5 Defective joint seals

Table 3.5 gives an overview of the causation, treatment options and applicability for defective joint seals.

Table 3.5 Overview of defective joint seals

Considerations	Treatment options	Applicability
Causation includes: • **sealant age, traffic and environmental loading (see Section 3.1.5.1 and Section 4.1);** • **poor construction practices (see Section 3.1.5.1 and Section 4.3.2.3);** • **excessive horizontal movement, typically as a result of nearby locked up joints (see Section 4.3.2.2); or,** • **excessive vertical movement as a result of poor slab support (see Section 4.3.1).** **Any premature failures of joint seals should be investigated further.**	**Long-term repair** Replace joint seal (see Section 5.2.1).	If the defective joint seal is because of excessive horizontal or vertical movement, any replaced joint seal is likely to have poor durability. Where excessive movement is identified, additional treatments are recommended to address movement.

Defective joint seals are most commonly found at transverse joints, as this is where the greatest range of movement occurs. Longitudinal joints are constructed as warping joints, i.e. the adjacent slabs are tied together, so the joint seal typically accommodates minimal movement. As such, correctly installed sealants at longitudinal joints often perform satisfactorily throughout the life of the pavement. The exception being where they are proximal to a wheel track; for example, due to realignment or widening works. Defective joint seals can result in the initiation or acceleration of formation of other defects, including:

- cracking (see Sections 3.3.2, 3.3.4, 3.3.5 and 3.3.6);
- compression failures (see Section 3.3.9);
- joint spalling (see Sections 3.1.6 and 3.3.1);
- stepping (see Section 3.3.7); and,
- slab rocking (see Section 3.3.8).

3.1.5.1. Causation

The main causes of defective joint seals can be split into four categories:

- age related in-service defects to be anticipated;
- poor construction practices;
- locked up joints and excessive horizontal movement at transverse joints only (see Section 4.3.2); or,
- poor slab support and excessive dynamic vertical movement (see Section 4.3.1).

Joint sealant failures occurring in the seasons following installation are likely to be a result of the construction practice, poor application or incorrect dimensions of either the sealant groove or the joint sealant. This includes extrusion failures where joint seals are squeezed proud of the pavement surface, resulting in vehicle pick up and removal of the seal.

Defective joint seals less than five years after installation is an indicator that excessive movement is occurring at the joint. An investigation should be undertaken to determine the appropriate maintenance option, and aligned with the maintenance strategy for the pavement, to avoid further premature joint seal failures and more significant structural defects arising such as compression 'blow-up' failures (Section 3.3.9).

Poor construction practices

Table 3.6 outlines typical causes during construction for each type of failure. To ensure good initial adhesion the sides of sealant grooves must be clean, dry and not too cold at the time of application. Scouring of the sealant groove by abrasive blasting is often undertaken and, depending on the type of joint sealant applied, a primer may be required.

The preparation of the sealing material is important, hot applied sealants can become brittle at the time of application if overheated and two-part, cold applied, sealants may perform poorly if components are not correctly proportioned.

Groove dimensions should be appropriate for the amount of movement expected and the type of sealing material used. Good practice procedures for replacing joint seals including sealant types and minimum and maximum dimensions for sealant grooves are discussed in more detail in Section 5.2.1.

Table 3.6 Causes of premature failures during preparation for and application of joint sealing

Type of failure	Causes	
Premature adhesion failure	Preparation	• sealant groove dirty • moisture in sealant groove • incorrect sealant groove dimensions
	Application	• sealing material faulty • sealing material inappropriate
	Weather conditions	• too cold • moisture in sealant groove
Premature cohesion failure	Preparation	• incorrect sealant groove dimensions • lack of bond breaking material beneath seal
	Application	• sealing material faulty • sealing material inappropriate
Extrusion failure	Preparation	• incorrect sealant groove dimensions • lack of bond breaking material beneath seal
	Application	• overfilled sealant groove

Excessive horizontal movement

With URC pavements the transverse joints are typically 4 - 5 m apart. Not all of these joints may be functioning, i.e. there is no horizontal movement, due to:

- movement joints having 'locked up' because of unintended bonding between concrete and dowel bars or ingress of hard particles between the joint faces; or,
- variations in slab / subbase friction that concentrate horizontal movements at a few joints.

The remaining functioning joints are then subject to excessive horizontal movements to compensate for those that are inoperative. As a result, the sealant in the operative joints is likely to fail through excessive extension and compression cycles. The inoperative joints will continue to allow the small rotations required for the joints to operate as warping joints.

Early indicators of locked up movement joints include spalling and transverse cracking in proximity to the joint. For identification of locked up movement joints, see Section 4.3.2. Excessive horizontal movement can also occur at longitudinal joints, where the slabs have separated (see Figure 3.6).

Figure 3.6 Adhesion failure of the joint seal in longitudinal joint as a result of the joint opening up

Excessive dynamic vertical movement

Load transfer between adjacent slabs with dowelled joints is normally such that differential vertical movements between the ends of adjacent slabs do not exceed the limits of the sealant in newly constructed URC and JRC pavements. However, repeated trafficking over the lifetime of the pavement slowly increases the range of dynamic vertical movement.

This dynamic vertical movement is itself accelerated through presence of defective joint seals resulting in infiltration of salts and water into the joints causing dowel bar corrosion and foundation erosion.

If the root cause is left untreated, it is likely to lead to repeat premature adhesion or cohesion failure of any replaced joint sealants at the affected joints.

3.1.5.2. Treatment options

Most defective joint seals can be replaced following the procedure outlined in Section 5.2.1. However, if the defective joint seal is a result of excessive horizontal or vertical movement, any replaced joint seal is likely to have limited durability. Where excessive movement is identified, additional treatments are recommended to address movement.

If there are several locked up joints causing excessive horizontal movements and extrusion failure or adhesion failure of the joint seal, it may be appropriate to undertake **bay replacement** (see

Section 5.4.1), including introducing expansion joints to reduce the risk of future compression failures from the locked up joints.

Where there is excessive vertical movement then treatment may comprise **under slab grouting** (see Sections 5.4.5) or **bay replacement** (see Sections 5.4.1), and treatment to address any drainage deficiency.

3.1.6. Shallow joint spalls

Joint spalls are a breakdown of the surface concrete material within 600 mm of the joint and typically intersecting the joint at an angle [38]. They are regularly the most common visible defect in a concrete pavement.

As shown in Section 2.1.2.2, joint spalls have different levels of severity and are classified as:

- shallow: the spall depth is less than one third of the slab depth (surface defect); or,
- deep: the spall is more than one third of the slab depth (structural defect).

Shallow joint spalls do not affect the load transfer devices in the slabs and as such, they are considered to be surface defects. Less extensive and expensive treatment options are suitable for this defect.

Deep joint spalls are structural defects with different causation and maintenance options and are discussed in the structural defects section (see Section 3.3.1). Table 3.7 gives an overview of the causation, treatment options and applicability for shallow joint spalls.

Examples of shallow joint spalls are shown in Figure 3.7.

Figure 3.7 Shallow joint spalls

Table 3.7 Overview of shallow joint spalls

Considerations	Treatment options	Applicability
Causation includes: • **traffic and environmental loading (see Section 4.1);** • **locked up joints (see Section 4.3.2); or,** • **poor construction practice (see Section 4.3.3).** **Further investigation is needed to select the appropriate maintenance option.**	**Holding repair**	
	Shallow repair with polymeric material (see Section 5.3.2).	40 mm to one third slab depth. For use outside wheel track zones. Use of polymeric materials may be beneficial where differential movement is expected.
	Long-term repair	
	Widen joint sealant groove (see Section 5.2.2).	The maximum depth of the joint spall is less than the sealant groove. Maximum width 40 mm for transverse joints and 25 mm for longitudinal.
	Thin bonded repair (see Section 5.3.2).	Shallow joint spalls up to the depth of the sealant groove or 40 mm.
	Shallow repair with cementitious or resin repair material (see Section 5.3.2).	40 mm to one third slab depth.

3.1.6.1. Causation

The main causes of shallow joint spalls are infiltration of incompressible detritus into the joint groove and damage due to traffic and environmental loading.

Spalling due to the ingress of incompressible material, such as filter drain media from adjacent verge drainage, into the joint groove is usually sudden and often in the form of wedge-shaped concrete fragments. Typically, these taper towards the sides of the spall and towards the edge remote from the joint face (see Figure 3.7).

If spalling is present at multiple joints early within the expected design life of the pavement, the likely cause is an inadequate concrete mixture including aggregates with poor durability or poor construction practice such as saw cutting joints when the concrete is still ‘green’.

3.1.6.2. Treatment options

Shallow joint spalls often require no immediate treatment when they do not pose a safety or serviceability risk. However, shallow joint spalls can accelerate pavement deterioration by permitting ingress of water and salts into the pavement. The spall may deteriorate rapidly if located in the wheel track. Therefore, it is recommended to treat them during the next routine maintenance cycle.

It is important that the extent of the spalling and the circumstances which have caused the spalling are understood in order to avoid inappropriate treatments being undertaken that will require further treatments in the near future. Visual indications alone are unlikely to be conclusive. Further information on the identification of the severity of joint spalls is provided in Section 2.1.2.2.

Joint spalls less than the depth of the sealant groove may be treated by vertical saw cutting to form a **widened sealant groove** (see Section 5.2.2) to remove the spalled edge up to a maximum width, including any chamfers, of 40 mm for transverse joints and 25 mm for longitudinal joints. It is good practice to continue this treatment for the entire length of the joint.

Spalls exceeding these limits have to be repaired with more intrusive treatments, at significantly greater cost. It is therefore important to establish the extent to which spalling falls within or outside these limits. It is unlikely to be practicable to examine and measure every spall other than within a representative length of carriageway.

Shallow joint spalls exceeding the depth of the sealant groove but less than one third slab depth should be treated with either **thin bonded repairs** or **shallow repairs** (see Section 5.3.2). The process is similar, but thin bonded repairs are undertaken up to 40 mm depth or the depth of the sealant groove if this is less, and shallow repairs are used for repairs greater than 40 mm to one third slab depth.

The choice between a thin bonded repair or a shallow repair and the repair material used is dependent on the depth of the spall and the cause of the spall, pavement condition and available working window. This is discussed further in Section 5.3.2. Use of polymeric materials to treat spalls should be avoided as they do not have equivalent properties to the surrounding concrete pavement. Consequently, they can have service lives of up to 5 years depending on treatment location, as this will be reduced if used within wheel tracks. Polymeric materials can accommodate movement, so may be appropriate as secondary repair options where rigid repair materials have historically failed rapidly, typically due to localised movement in slabs associated with cracks.

Re-establishing joints at the location of the treatment with the same dimensions as the adjacent joint is very important to prevent point loading and compression failures of the treatments. Some proprietary polymeric materials claim to not require the joint to be re-established due to their flexibility, these should be treated with caution to avoid the need for future repairs.

3.2. Structural defects common to URC, JRC and CRCP

Structural defects extend deeper into the concrete slab than surface defects and can significantly reduce the pavement structural capacity to carry the future design traffic. This section covers structural defects that are common to URC, JRC and CRCP. These include:

- depressions (settlement) (see Section 3.2.1);
- heave (see Section 3.2.2); and,
- punchouts (see Section 3.2.3).

Structural defects specific to URC and JRC are reported in Section 3.3 and CRCP structural defects are reported in Section 3.5.

3.2.1. Depression (settlement)

A depression, also referred to as settlement, often affects a localised pavement area as a result of a downward differential displacement in the underlying foundation or subgrade compared with the adjacent pavement (see Figure 3.8). The concrete slab is ground bearing and follows this downward displacement. Differential settlement rather than uniform settlement is of concern for concrete pavements, resulting in stepping at the concrete joints. CRCP is less affected initially, being able to bridge localised losses of support. Table 3.8 gives an overview of the causation, treatment options and applicability for settlement.

Figure 3.8 Settlement

Table 3.8 Overview of settlement

Considerations	Treatment options	Applicability
Causation includes: • **geotechnical factors;** • **environmental loading (see Section 4.1.2); or,** • **inadequate drainage (see Section 4.1.2 and 4.3.1).**	**Holding repair**	
	Under slab grouting and slab lifting (see Sections 5.4.5 and 5.4.6) followed by bump cutting as necessary (see Section 5.4.7).	May be ineffective if sub-surface drainage is inadequate. Intact pavements only.
	Long-term repair	
	Bay replacement (see Section 5.4.1) or full depth repair (see Sections 5.4.2 to 5.4.4). Drainage renewal and foundation replacement may be necessary.	Drainage renewal where inadequate drainage or pumping identified.
	Drainage renewal and reconstruction (see Chapter 7).	For extensive affected areas.

3.2.1.1. Causation

Depressions are mainly caused by geotechnical factors resulting in a downward movement of the ground below the concrete pavement. High embankment fill materials and backfill to bridge structures are particularly susceptible to differential settlement, between the ground bearing slab and the structure. Inadequate compaction of the subgrade fill during construction can result in secondary compaction during service. However, volumetric changes can occur if the subgrade or fill material used is sensitive to water content change. This volumetric change can be exacerbated when these materials are not uniform in their composition or thickness.

An inadequate or ineffective drainage system is a primary cause of depressions; surface water can enter through any poorly maintained joints in a URC or JRC. Saturated material may provide a significantly reduced support to the concrete slab. The movement of water can cause erosion of the subgrade and a reduction in volume below the concrete pavement. An inadequate drainage system is often evidenced by pumping at URC or JRC joints and is confirmed by inspection of the drainage system, to see whether it is functioning as designed.

3.2.1.2. Treatment options

Depressions caused by saturation and erosion of the subgrade due to a deficiency in the drainage system are likely to continue to deteriorate, whereas further movement is unlikely once secondary compaction is complete. The pavement drainage system should be checked and renewed as necessary before any of the following treatments are implemented.

For depressions where the pavement is still intact and has not cracked, then **under slab grouting** and **slab lifting** (see Sections 5.4.5 and 5.4.6) followed by **bump cutting** as necessary (see Section 5.4.7) can restore the pavement surface profile. Slab lifting is typically limited to a maximum length of 15 m (or three URC bays) in one operation.

Severe depressions may require the foundation to be replaced, meaning that **full depth repair** (see Sections 5.4.2 to 5.4.4) or **bay replacement** (see Section 5.4.1) is required. **Reconstruction** (see Chapter 7) should be considered when depressions are severe and affect extensive areas.

3.2.2. Heave

Heave is the result of an upward differential displacement in the underlying foundation or subgrade compared with the adjacent pavement (see Figure 3.9). The concrete slab is ground bearing and follows this upward displacement. Heave may result in stepping at the concrete joints. Table 3.9 gives an overview of the causation, treatment options and applicability for heave.

Figure 3.9 Heave

Table 3.9 Overview of heave

Considerations	Treatment options	Applicability
Causation includes: • **expansive soils (typically clays) or treated soils (e.g. lime stabilised capping); or,** • **environmental loading, frost heave (see Section 4.1.2).**	**Holding repair**	
	Bump cutting (see Section 5.4.7).	Localised high areas. Further heave may occur. Check impact on cover to any reinforcement.
	Long-term repair	
	Foundation replacement or improvement followed by bay replacement for URC and JRC (see Section 5.4.1) and full depth repair for CRCP (see Section 5.4.2).	
	Full depth reconstruction (see Section 7.4.9)	For severe cases.

3.2.2.1. Causation

Heave is caused by expansive soils (typically clays) that are present in the subgrade, or the foundation incorporating treated soil layers (such as lime stabilised capping) that are sulfate rich.

Upward heave should not be confused with localised upward movement and / or shattering of a concrete slab at transverse joint or cracks, as a result of lack of joint space for the concrete to expand in to. These compression failures or 'blow-ups' are discussed in Section 3.3.9.

Soils with clay mineralogy are susceptible to shrink and swell with changing water content. The removal of vegetation can change the amount of available water that results in a large volume increase when absorbed into the soil. The stress relief associated with the removal of overburden from over consolidated soils can result in heave, which should be accounted for during the design stage. Susceptible foundation materials can also temporarily expand as a result of the formation of ice lenses (frost heave), this phenomenon is not common in the UK where the frost index depth is generally 450 mm.

Where calcium based additives (such as lime) are used to treat soils with high concentrations of sulfates as part of foundation preparation works, the calcium addition can raise the pH of the soil, and in the presence of water, this results in an expansive hydration reaction which forms ettringite, a water rich crystalline mineral that is approximately twice the volume of the original minerals. Where sulfates are present in sufficient quantities, this reaction can result in heave of the overlying pavement.

3.2.2.2. Treatment options

For localised areas of minor heave in the foundation then **bump cutting** (see Section 5.4.7) can be considered as a temporary treatment, although further heave will likely develop.

To rectify heave issues, depending on the causation, either foundation replacement or stabilisation will be necessary together with **bay replacement** (see Section 5.4.1) or **full depth repair** (see Sections 5.4.2 to 5.4.4), or **full depth reconstruction** (see Section 7.4.9) of the pavement section is heave is widespread.

3.2.3. Punchouts

Punchouts are localised defects in which intersecting transverse and longitudinal cracks or joints create loose fragments within the concrete slab, normally occurring at the pavement edge or joint, that are gradually 'punched out' by the action of traffic and may be driven down into any underlying subbase layer (see Figure 3.10). They mainly arise in CRCP but can occasionally be a defect in JRC, as the concrete normally ruptures at the depth of the reinforcement.

Punchouts are generally associated with a poor or non-uniform foundation support. Specifically, this type of defect usually occurs at locations where severe differential settlement causes a loss of support to the concrete slab, or where localised deterioration of the foundation is evident. Deterioration of the foundation is often caused by erosion or softening of the subgrade due to the ingress of water. Table 3.10 gives an overview of the causation, treatment options and applicability for punchouts.

Table 3.10 Overview of punchouts

Considerations	Treatment options	Applicability
Causation includes: • **closely spaced transverse cracks, traffic loading (see Section 4.1) and poor or non-uniform support (see Section 4.3.1);** • **water ingress and corroded reinforcement;** • **reinforcement too high in the concrete; and,** • **inadequate thickness, concrete strength or steel reinforcement (design life exceeded).**	**Long-term repair**	
	Full depth repair (see Sections 5.4.2 and 5.4.4). Foundation replacement may be necessary.	


Figure 3.10 Punchouts

3.2.3.1. Causation

Punchouts fundamentally require the intersection of closely spaced transverse cracks with longitudinal cracks, in the presence of steel reinforcement. Once these cracks intersect, repeated loading of the fragmented area causes progressive deterioration. The longitudinal steel either yields or breaks as a result of the concrete displacement, culminating in the punchout.

Punchouts can occur where the steel reinforcement has corroded. Corrosion of the reinforcement typically results in excessive yielding or fracturing of the reinforcement which leads to widened cracks and a loss of load transfer. This is often exacerbated by harsh wet winters, the pavement holding water (i.e. an asphalt surfacing or at the interface) and traffic loading. These widened cracks within the pavement allow ingress of water which can pool between the bound foundation and CRCP layers, initiating reinforcement corrosion.

Insufficient cover to reinforcement whilst impacting on corrosion protection can exacerbate thermal heating and cooling of the reinforcement resulting in different expansion rates between the steel and concrete. This can cause micro-cracking to occur which can lead to, albeit generally smaller, punchouts.

Generally, punchouts occur where the concrete pavement has:

- poor or non-uniform support (may be evidenced by depressions) (see Section 4.3.1) as a result of:
  - inadequate drainage;
  - secondary compaction of the subgrade; or
  - use of non-uniform soils; and,
- inadequate thickness, concrete strength or steel reinforcement resulting in load-induced cracking (see Section 4.2) due to:
  - insufficient design (including over assumption of design factors);
  - poor construction; or,
  - overloading or exceeding the design loading.

3.2.3.2. Treatment options

Punchouts will require maintenance as they pose an immediate safety and serviceability risk. However, ideally the areas where punchouts may arise (i.e. intersecting cracks) would be identified and treated by planned maintenance before the punchout occurs to avoid emergency repairs to punchouts which can cause significant unplanned disruption to road users.

Full depth repair (see Sections 5.4.2 and 5.4.4) is the preferred maintenance option for punchouts that occur in CRCP; or individual **bay replacement** (see Section 5.4.1) for JRC. Depending on the condition of the foundation, it may be necessary to also replace the foundation and / or address any drainage deficiency.

Even where a punchout is limited to the reinforcement depth and the reinforcement remains intact, shallow repairs are not recommended, and full depth repairs should be undertaken. Shallow repairs to punchouts are likely to have a poor lifespan because the original cause of the punchout, i.e. poor or non-uniform support or load induced cracking, has not been treated.

Where frequent punchouts have occurred in a section of pavement, it is an indication that the pavement has reached the end of its serviceable life. Therefore, reconstruction should be considered (see Chapter 7).

3.3. Structural defects specific to URC and JRC pavements

This section covers structural defects specific to URC and JRC pavements. These include:

- deep joint spalls (see Section 3.3.1);
- corner cracks (see Section 3.3.2);
- cracks around ironwork (see Section 3.3.3);
- diagonal cracks (see Section 3.3.4);
- transverse cracks (see Section 3.3.5);
- longitudinal cracks (see Section 3.3.6);
- stepping (see Section 3.3.7);
- slab rocking (see Section 3.3.8); and,
- compression failures (blow-ups) (see Section 3.3.9).

3.3.1. Deep joint spalls

Joint spalls are a breakdown of the concrete material within 600 mm of the joint and typically intersecting the joint at an angle [38]. In contrast to the shallow joint spall surface defects discussed in Section 3.1.6, deep joint spalls are structural defects that extend more than one third of the slab depth. Examples of deep joint spalls are shown in Figure 3.11. Table 3.11 gives an overview of the causation, treatment options and applicability for deep joint spalls.

Figure 3.11 Deep joint spalls

Table 3.11 Overview of deep joint spalls

<table>
<tr><th>Considerations</th><th>Treatment options</th><th>Applicability</th></tr>
<tr><td rowspan="3">Causation includes:
traffic and environmental loading (see Section 4.1);
locked up joints (see Section 4.3.2);
poor slab support (see Section 4.3.1); and,
dowel bar issues (see Section 4.3.2.1).</td><td colspan="2">Long-term repair</td></tr>
<tr><td>Full depth repair (see Section 5.4.4).</td><td>More practical and cost effective repair option for JRC versus bay replacement.</td></tr>
<tr><td>Bay replacement (see Section 5.4.1).</td><td>Recommended treatment option for URC.
JRC with defects widespread across a bay.</td></tr>
</table>

3.3.1.1. Causation

Spalling may develop predominantly in the top part of the slab or at a greater depth below the surface, depending on the environmental conditions, eventually reaching full pavement depth.

Understanding the causation of deep joint spalling is critical in informing whether such defects are likely to occur elsewhere in the pavement. Causes of deep joint spalling include:

- traffic and environmental loading (see Section 4.1);
- dowel bar / tie bar issues including corrosion and restraint (see Section 4.3.2);
- ingress of solid particles into the joint space (see Section 4.3.2); and,
- loss of subbase support at the joint causing excessive load transfer stresses in the concrete around the dowel bars that can result in shearing from the surrounding concrete causing cracking and spalling. This is more common in pavements with an unbound subbase and is likely to be present with signs of dynamic vertical movement (see Section 4.3.1).

Deep joint spalls, often, present themselves in a similar manner at the surface as shallow spalls, and it is many years before the point where the spall is readily identified as a deep spall i.e. deteriorating to the extent whereby the spall emanates far from the joint groove. For this reason, it is important to investigate the possibility that joint spalls are deep joint spalls as early as possible. Failure to do so is likely to result in abortive expenditure on ineffective shallow joint spall repairs and may mean it is too late to prevent escalation of the same defect to other joints.

Where there is doubt over the severity of joint spalls it is recommended initially to use non-destructive techniques such as GPR (see Section 2.2.1.2) and ultrasound (see Section 2.2.1.4) to identify whether the spall coincides with the position of a dowel bar and give an indication of whether any dowel bar present is corroded or misaligned.

3.3.1.2. Treatment options

Where deep joint spalls are present then a **full depth repair** (see Sections 5.4.3 and 5.4.4) or **bay replacement** (see Section 5.4.1) is necessary. For URC, bay replacement is generally preferred to full depth repair. As discussed further in Section 5.4.3, bay replacement in URC can be completed within a similar timescale and at a similar cost due to the limited size of the bay (up to 7 m), and can reduce the risk of accelerated deterioration of both the repair and the surrounding pavement linked to the:

- introduction of additional joints, which are an area of weakness;
- propensity of short 'bays' (< 2 m) to be liable to dynamic vertical movement, particularly where an unbound subbase is present; and,
- repair dimensions potentially being susceptible to drying shrinkage / sympathetic cracking where the width / length ratio of the repair is greater than approximately 1.3:1.

However, the bay sizes of JRC (up to 25 m) means full depth repair is generally most practicable when working within an overnight possession window. Bay replacement should be undertaken for JRC where other types of deterioration or defect exist either in or below the slab.

Whilst temporary measures such as shallow repairs with polymeric materials can be used on deep joint spalls to retain short-term serviceability and allow for the appropriate investigation and treatment planning, they will inevitably result in further deterioration of the pavement and will require re-attention within months of the treatment. Furthermore, if poor slab support or corrosion of dowel or tie bars is the cause of the defect, then cracking and spalling is likely to occur elsewhere across the width of the joint (see Section 4.3).

3.3.2. Corner cracks

Corner cracks are cracks between 0.3 m and 2 m in length across the corner of the concrete bay. If larger than 2 m, they are considered as diagonal cracks (see Section 3.3.4). Figure 3.12 shows examples of corner cracks.

Figure 3.12 Corner cracks

A corner crack differs from a joint spall in that the crack extends vertically through the entire slab thickness, whilst a joint spall intersects the joint at an angle. If the crack length is less than 0.3 m, the crack is likely intersecting the joint at an angle, so can be considered a shallow or deep joint spall depending on severity (see Sections 3.1.6 and 3.3.1, respectively).

Corner cracking often starts as a single crack that quickly deteriorates further, with the corner segment breaking into two or more pieces. If not repaired, corner cracks can lead to further localised deterioration of the subbase and perhaps pumping. Table 3.12 gives an overview of the causation, treatment options and applicability for corner cracks.

Table 3.12 Overview of corner cracks

Considerations	Treatment options	Applicability
Causation includes: • **traffic and environmental loading (see Section 4.1);** • **poor slab support (see Section 4.3.1);** • **design issue - acute angles (see Section 4.2.1);** • **dowel bar issues (see Section 4.3.2.1); and,** • **locked up joints (see Section 4.3.2.2).** **The presence of any vertical movement will impact on maintenance options.**	**No treatment advised**	
	Cracks < 0.5 mm wide.	
	Holding repair	
	Inlaid crack repair (see Section 5.3.3).	For static cracks < 1.5 mm wide outside wheel track zones. Holding measure to prevent ingress of water, de-icing salts and solids, reduce effects of freeze-thaw cycle and reduce risk of crack spalling.
	Long-term repair	
	Full depth repair (see Sections 5.4.3 and 5.4.4).	Cracks in URC ≥ 0.5 mm wide. Cracks in JRC ≥ 1.5 mm wide. More practical and cost effective repair option for JRC versus bay replacement. Corner repair may be considered.
	Bay replacement (see Section 5.4.1).	Cracks in URC ≥ 0.5 mm wide. Cracks in JRC ≥ 1.5 mm wide. Recommended treatment option for URC. JRC with defects widespread across a bay.

3.3.2.1. Causation

Corner cracking can be caused by localised restraint causing a build-up of stresses. This is typically as a result of:

- the ingress of incompressible materials into the joint through defective joint seals; or,
- dowel bars being locked up, i.e. where the dowel cannot move freely as the joint expands and contracts (see Section 4.3.2.2).

Another common mechanism for the occurrence of corner cracking is top-down cracking as a result of repeated loading of corners that have poor or non-uniform support and / or ineffective load transfer devices. Environmental loading may contribute to corner crack development as slabs curl / warp upwards with temperature / moisture gradients in the slab, reducing underlying support and increasing stresses on the slab when loaded.

Ingress of material can occur during construction or maintenance treatments. In this case, cracking typically occurs during the next period of very warm weather when the closure of the joints is restrained by the infiltrated material. If the amount of infiltrated material is small, spalling may occur; but if the amount and depth of infiltrated material is great, then a small corner crack may develop.

If slabs are non-rectangular, corner cracks can be expected where there are acute angles due to accumulated stress (see Section 4.2).

3.3.2.2. Treatment options

For corner cracks smaller than 0.5 mm in width then no immediate action is necessary as the ingress of solids or water into cracks this narrow is unlikely, but it is important to keep monitoring the crack width. Alternatively, an **inlaid crack repair** (see Section 5.3.3) may be undertaken as a holding repair to reduce the rate of deterioration of cracks less than 1.5 mm wide. However, if a corner crack is propagating as a result of poor slab support, the crack is likely to be dynamic and further deterioration should be expected even with an inlaid crack repair.

To rectify corner cracks a **full depth repair** (see Sections 5.4.3 and 5.4.4) or **bay replacement** (see Section 5.4.1) is necessary. Bay replacement is typically preferred for URC (see Section 5.4.3 for more details), or JRC bays that also exhibit other defects, or where poor slab support has been identified and the foundation requires treatment.

An alternative treatment option is a **full depth corner repair** (see Section 5.4.9), which involves creating another joint chamfered at a 45 ° angle to the existing joints. However, experience has shown that, although sometimes successful, corner repairs cannot be relied upon to achieve the life expected of a bay replacement or full depth repair with joints following the existing joint layout. Therefore, a departure from standard is required for this alternative treatment technique. Circular corner repairs using precast units have been used successfully internationally, with the circular shape helping to distribute stresses throughout the treatment.

3.3.3. Cracks around ironwork

It is not uncommon for cracks to emanate from ironwork (such as surface water gullies and manholes) penetrating through a concrete pavement (see Figure 3.13). Table 3.13 gives an overview of the causation, treatment options and applicability for cracks around ironwork.

Figure 3.13 Cracks around ironwork

Table 3.13 Overview of cracks around ironwork

Considerations	Treatment options	Applicability
Causation: • **design issues including lack of isolation around ironwork (see Section 4.2.2).**	**No treatment advised**	
	Cracks < 0.5 mm wide.	
	Holding repair	
	Inlaid crack repair (see Section 5.3.3).	Holding measure to prevent ingress of water, de-icing salts and solids, reduce effects of freeze-thaw cycle and reduce risk of crack spalling.
	Long-term repair	
	Bay replacement or full depth repair and reinstate ironwork (see Sections 5.4.1 to 5.4.4).	For severe cracking and to reduce risk of future serviceability issues.
	Relocate ironwork in verge and bay replacement or full depth repair (see Sections 5.4.1 to 5.4.4).	Where ironwork relocation offers improved WLC value.

3.3.3.1. Causation

If ironwork (such as surface water gullies, manholes, etc.) is housed in recesses within the length of the bay then cracking is likely to extend out from the corners. This cracking can occur under environmental loading alone, noting that most ironwork is placed in the relatively untrafficked areas. However, if the bay containing the ironwork is subjected to traffic loading, then cracking is likely to be more frequent and more severe. This design issue is covered in more detail in Section 4.2.2.

3.3.3.2. Treatment options

For cracks smaller than 0.5 mm in width then no immediate action is necessary, but it is important to keep monitoring the width evolution, as ingress of water, de-icing salts and incompressible materials can accelerate deterioration. An **inlaid crack repair** treatment (see Section 5.3.2.4) is an option for cracks emanating from ironwork, to prevent ingress of external substances.

Where cracking is more severe, it may be necessary to undertake a **bay replacement** (Section 5.4.1) or **full depth repair** (see Sections 5.4.2 to 5.4.4) and reinstate the ironwork. This can be undertaken following the details outlined in Volume 3, C Series of the highway construction details in the MCHW [39]. The detail includes reinforcement with the intention of keeping any cracking that does occur held tightly together so that external substances cannot enter the pavement.

In some circumstances, such as planned extended trafficking of a hard shoulder containing ironwork, it may be most appropriate to relocate the ironwork to the verge whilst undertaking a bay replacement.

3.3.4. Diagonal cracks

A diagonal crack traverses two perpendicular joints in a bay and is greater than 2 m in length. Any crack traversing perpendicular joints with a length of less than 2 m is considered to be a corner crack (see Section 3.3.2). Figure 3.14 shows examples of diagonal cracks. Table 3.14 gives an overview of the causation, treatment options and applicability for diagonal cracks.

Figure 3.14 Diagonal cracks

Table 3.14 Overview of diagonal cracks

<table>
<tr><th>Considerations</th><th>Treatment options</th><th>Applicability</th></tr>
<tr><td rowspan="9">Causation includes:
traffic and environmental loading (see Section 4.1);
poor slab support (see Section 4.3.1); and,
slab dimensions (see Section 4.2).</td><td colspan="2">No treatment advised</td></tr>
<tr><td colspan="2">Cracks < 0.5 mm wide.</td></tr>
<tr><td colspan="2">Holding repair</td></tr>
<tr><td>Inlaid crack repair (see Section 5.3.3).</td><td>Holding measures for cracks < 1.5 mm wide to prevent ingress of water, de-icing salts and solids, reduce effects of freeze-thaw cycle and reduce risk of crack spalling.</td></tr>
<tr><td colspan="2">Long-term repair</td></tr>
<tr><td>Full depth repair incorporating expansion joints (see Sections 5.4.3 and 5.4.4). Foundation replacement may be necessary.</td><td>Cracks in URC ≥ 0.5 mm wide.

Cracks in JRC ≥ 1.5 mm wide.

More practical and cost effective repair option for JRC versus bay replacement.</td></tr>
<tr><td>Bay replacement incorporating expansion joints (see Section 5.4.1). Foundation replacement may be necessary.</td><td>Cracks in URC ≥ 0.5 mm wide.

Cracks in JRC ≥ 1.5 mm wide.

Recommended treatment option for URC.

JRC with defects widespread across a bay.</td></tr>
</table>

3.3.4.1. Causation

Diagonal cracks are generally caused by different factors to corner cracks; their causation is more closely associated with that of a transverse crack (Section 3.3.5). Poor or non-uniform support from the foundation, including those factors that can further result in a localised depression (settlement), is the most likely cause of diagonal cracks.

Poor support conditions can result from factors including poor compaction, pumping and erosion. Generally, bound subbases are less susceptible to erosion than unbound subbases (see Section 4.3.1).

The action of repeated traffic and environmental loading can induce diagonal 'fatigue' cracking. Environmental conditions can affect long-term concrete behaviour and influence the development of diagonal cracking; thermal curling and moisture warping can produce stresses in the slab that together with traffic loading contribute to the development of diagonal cracking (see Section 4.1).

Diagonal cracks can also be caused by excessive slab length. The layout and spacing of joints in relation to slab thickness can have a great influence on the development of diagonal cracking in concrete pavements (see Section 4.2). However, due to a standardised design methodology, this is unlikely to be the cause of diagonal cracks on the SRN.

3.3.4.2. Treatment options

For narrow (< 0.5 mm wide) diagonal cracks, no immediate action is necessary, but it is important to keep monitoring the width evolution, as if the crack grows beyond this width, it may permit the ingress of water, de-icing salts and incompressible materials into the crack, which can accelerate slab deterioration.

Diagonal cracks in JRC

Where diagonal cracks in JRC are between 0.5 and 1.5 mm wide, the reinforcement is likely to be intact and offering some load transfer. However, cracks of this width may allow the ingress of external substances into the crack. This may result in accelerated deterioration of the bay due to corrosion of the reinforcement and spalling of the crack, at which point a full depth repair or bay replacement would be required. Therefore, it is recommended that these cracks are treated with an **inlaid crack repair** (see Section 5.3.3).

Where cracks are above 1.5 mm wide in JRC, it is probable that the reinforcement has yielded and there will be no load transfer across the crack. Therefore, **full depth repair** (see Section 5.4.4) is recommended, along with replacement of the foundation as necessary. However, where defects are widespread across the bay, a **bay replacement** (see Section 5.4.1) may be more appropriate than a full depth repair.

Diagonal cracks in URC

Cracks greater than 0.5 mm in width in URC will offer little or no load transfer across the crack and may permit the ingress of external substances, so the bay is likely to deteriorate further. Therefore, **bay replacement** (see Section 5.4.1), including replacement of the foundation as necessary will be needed to rectify the issue. **Full depth repair** (see Section 5.4.3) is not recommended even if the diagonal crack only spans half the bay (see Figure 3.14) in order to avoid the:

- introduction of further joints, which could be a future maintenance liability and cause sympathetic cracking in adjacent bays; and,
- creation of new bays with an excessive width / length ratio, which may be susceptible to drying shrinkage / sympathetic cracking.

In some circumstances, an **inlaid crack repair** (see Section 5.3.3) may be appropriate for cracks in URC as a holding repair dependent on the asset management strategy and serviceability requirements. This will not improve load transfer, and the life expectancy of the repair may be short (i.e. less than 5 years) depending on the width of the crack, but the inlaid crack repair may slow deterioration by preventing the ingress external substances into the crack, reducing the effects of freeze-thaw cycles and reducing the risk of the crack spalling.

3.3.5. Transverse cracks

Transverse cracks (similar to the function of formed transverse joints) will divide a concrete slab perpendicular across the width of the slab to create multiple bays (see Figure 3.15). Their presence is unplanned in URC given the short, typical 4 - 5 m bay lengths; but the presence of narrow transverse cracks is expected in JRC at typically greater than 4 m intervals in between the formed transverse joints with the reinforcement holding these transverse cracks tightly together. Table 3.15 gives an overview of the causation, treatment options and applicability for transverse cracking.

Figure 3.15 Transverse crack

Table 3.15 Overview of transverse cracks

Considerations	Treatment options	Applicability
Causation includes: • **locked up joints (see Section 4.3.2);** • **traffic and environmental loading (see Section 4.1);** • **poor slab support (see Section 4.3.1);** • **misalignment between top and bottom crack inducers or sympathetic cracking (see Section 4.2.3);** • **local causes (see Section 3.3.5.1); and,** • **end of pavement life.**	**No treatment advised**	
	Cracks < 0.5 mm wide.	
	Holding repair	
	Inlaid crack repair (see Section 5.3.3).	Cracks in JRC < 1.5 mm wide. Holding measures for other cracks to prevent ingress of water, de-icing salts and solids, reduce effects of freeze-thaw cycle and reduce risk of crack spalling.
	Long-term repair	
	Full depth repair incorporating expansion joints (see Sections 5.4.3 and 5.4.4). Foundation replacement may be necessary.	Cracks in URC ≥ 0.5 mm wide. Cracks in JRC ≥ 1.5 mm wide. More practical and cost effective repair option for JRC versus bay replacement for cracks in proximity of joints. Not recommended for mid-bay cracks in JRC (see Section 3.3.5.2).
	Bay replacement incorporating expansion joints (see Section 5.4.1). Foundation replacement may be necessary.	Cracks in URC ≥ 0.5 mm wide Cracks in JRC ≥ 1.5 mm wide Recommended treatment option for URC and JRC with mid-bay cracks. JRC bays with widespread defects.

3.3.5.1. Causation

A transverse crack will generally appear either in close proximity to a formed transverse joint or in the middle between two formed transverse joints i.e. mid-bay. Depending on where the crack occurs, the causation is typically different. The cause of transverse cracks occurring in close proximity to joints (see Figure 3.16) can be broken down into those that occur shortly after construction (or repair) and those that occur during the pavement's life cycle.

Construction issues causing transverse cracks in close proximity to joints include:

- misaligned dowel bars resulting in restraint, followed by cracking when the slab contracts (see Section 4.3.2.1); or,
- misalignment between top and bottom crack inducers and sympathetic cracking (see Section 4.3.3).

In service, transverse cracks forming in proximity to joints are caused by:

- locked up joints preventing the joints from opening and closing (see Section 4.3.2);
- traffic and environmental loading (see Section 4.1); and,
- non-uniform or poor slab support (see Section 4.3.1).

Figure 3.16 Transverse cracks in URC in proximity of a joint

Unplanned transverse cracks (or widened cracks in the case of JRC) occurring in the middle of a concrete bay (mid-bay) can be due to specific local causes or may indicate the onset of failure in a concrete pavement nearing the end of its design life. It is important to establish which of these causes is most likely before considering options for remedial works, as undertaking treatments to cracked bays in pavements close to the end of life may not offer the best WLC value.

The following are some specific local causes that may account for unplanned mid-bay cracking:

- erosion / wash out of the subbase or subgrade due to ineffective drainage;
- poor slab support above a cross-carriageway duct either installed by trenching during the original construction or by a post-construction trenchless technique;
- differential foundation support caused by different subgrade materials and / or thickness of fill material, for example, a soft area encountered during construction or backfill to an underbridge;
- a position in the pavement where thermal contraction has been constrained by locked up joints, frictional restraint or other local effects; and,
- sympathetic cracking caused by a crack or new joint formed in the slab of an adjacent lane.

If these and any other contract specific causes can be eliminated, then the possibility that the pavement is nearing the end of its serviceable life needs be considered.

3.3.5.2. Treatment options

The recommended maintenance options for transverse cracks is dependent on the crack width, defect causation and type of pavement. For transverse cracks in proximity of the transverse joints caused by issues with construction or the load transfer devices, further deterioration should be anticipated and **full depth repair** (see Sections 5.4.3 and 5.4.4) or **bay replacement** (see Section 5.4.1) will be necessary.

Where mid-bay cracks have occurred, the maintenance option is dependent on the type of pavement and the crack width.

Mid-bay transverse cracks in JRC

Narrow transverse cracks (< 0.5 mm wide) are a normal feature of JRC. They are considered to be structurally insignificant, are too narrow to allow the ingress of water and other materials into the pavement and are not expected to deteriorate any further under normal conditions, so no action will be necessary.

Where transverse cracks in JRC are between 0.5 and 1.5 mm wide, the reinforcement is likely to be intact and offering some load transfer. However, cracks of this width may allow external substances into the crack. This may result in accelerated deterioration of the bay due to corrosion of the reinforcement and spalling of the crack, at which point a full depth repair or bay replacement would be required. Therefore, it is recommended that these cracks are treated with an **inlaid crack repair** (see Section 5.3.3) to slow deterioration of the bay.

Where cracks are above 1.5 mm wide in JRC, it is probable that the reinforcement has yielded and there will be no load transfer across the crack. Evidence from full-scale experimental roads shows that when loading induced wide (> 1.5 mm) transverse cracks form in JRC, at which point the reinforcement has likely yielded, at least four more cracks develop in the same bay within a year [40]. Therefore, **bay replacement** (see Section 5.4.1) is recommended, along with replacement of the foundation as necessary.

Mid-bay transverse cracks in URC

Transverse cracking is not an expected feature in URC. Whilst narrow cracks (< 0.5 mm wide) do not need immediate attention as it is likely that aggregate interlock is providing adequate load transfer and the crack will be too narrow to allow external substances into the crack, they are likely to widen rapidly, at which point they will require attention.

Cracks greater than 0.5 mm in width in URC will offer little or no load transfer across the crack and may permit the ingress of external substances into the crack, so the bay is likely to deteriorate further without attention. Therefore, **bay replacement** (see Section 5.4.1) and replacement of the foundation as necessary will be needed to rectify the issue.

However, in some circumstances, an **inlaid crack repair** (see Section 5.3.3) may be appropriate for cracks in URC dependent on the asset management strategy and serviceability requirements. This will not improve load transfer, and the life expectancy of the repair may be short (i.e. less than 5 years) but will prevent ingress of external substances into the crack, reduce the effects of freeze-thaw cycles and reduce the risk of spalling.

Dowel bars have been retrofitted across transverse cracks in URC internationally in lieu of bay replacements or full depth repairs. The insertion of dowels provides load transfer and essentially turns the crack into a working joint. The process is explained in Section 5.4.10; however, there is little history of using this technique in the UK and no specification has been developed for the process; therefore, a departure from standard is required for this treatment option. Specialist equipment is needed to create the slots, which may not be readily available. The presence of reinforcement may limit the application of this technique within JRC.

3.3.6. Longitudinal cracks

Longitudinal cracks (similar to the function of formed longitudinal joints) will divide the slab roughly parallel to the pavement centreline to create two narrower slabs (see Figure 3.17). Table 3.16 gives an overview of the causation, treatment options and applicability for longitudinal cracks.

Figure 3.17 Longitudinal crack

Table 3.16 Overview of longitudinal cracks

Considerations	Treatment options	Applicability
Causation includes: • **poor slab support (see Section 4.3.1);** • **traffic and environmental loading (see Section 4.1);** • **locked up joints (see Section 4.3.2);** • **dowel bar issues (see Section 4.3.2);** • **design issues: slab dimensions (see Section 4.2);** • **other construction issues (see Section 4.3.3).**	**No treatment advised**	
	Cracks in JRC < 0.5 mm wide	
	Holding repair	
	Inlaid crack repair (see Section 5.3.3).	Holding measure to prevent ingress of water, de-icing salts and solids, reduce effects of freeze-thaw cycle and reduce risk of crack spalling.
	Crack stitching (see Section 5.4.8).	Minimises further growth in crack width. Suitable for cracks up to 0.5 mm wide in URC and cracks up to 1.5 mm wide in JRC.
	Long-term repair	
	Full depth repair incorporating expansion joints (see Sections 5.4.3 and 5.4.4). Foundation replacement may be necessary.	Cracks in JRC ≥ 1.5 mm wide. Potentially more practical and cost effective repair option for JRC versus bay replacement. However, longitudinal cracks may extend beyond extents of repair post-treatment.
	Bay replacement incorporating expansion joints (see Section 5.4.1). Foundation replacement may be necessary.	Cracks in URC ≥ 0.5 mm wide. Cracks in JRC ≥ 1.5 mm wide.

3.3.6.1. Causation

Poor or non-uniform support from the foundation at the pavement edge combined with repeated loading cycles is the likely cause of longitudinal cracking. Poor or non-uniform support may occur at the slab edges caused by saturation of an unbound subbase or subgrade as a result of ineffective drainage, but this is less likely to happen with a bound subbase. Longitudinal cut / fill transitions may also result in non-uniform support conditions. This type of longitudinal cracking typically occurs along the nearside wheel track of Lane 1, particularly where a hard strip or tied shoulder is not present. If longitudinal cracking is present throughout the concrete pavement it can be regarded as having reached the end of its service life.

However, environmental loading (thermal expansion and warping) (see Section 4.1) in the presence of locked up joints (see Section 4.3.2.2) or corroded dowel bars (see Section 4.3.2) can cause excessive compressive stress, and as a consequence critical tensile stresses by the Poisson's effect, which leads to longitudinal cracking.

Other design or construction factors that can result in longitudinal cracks, that appear soon after construction, are:

- Inappropriate design of the slab width dimension or thickness. Magnitudes of temperature, shrinkage, curling and warping stresses increase with increasing bay dimensions; therefore, excessively wide or thin slabs may develop longitudinal cracks.
- Inappropriate construction. Shallow cover to reinforcement, late or shallow depth saw cutting of joints, late or ineffective curing and embedded features (see Section 4.3.3) can trigger longitudinal cracking.

3.3.6.2. Treatment options

The recommended maintenance treatment options for longitudinal cracks are dependent on the type of pavement and the width of the crack. However, if longitudinal wheel track cracking is widespread across a section of pavement, the pavement has likely reached the end of its service life and better value may be obtained from reconstruction (see Chapter 7).

Longitudinal cracks in JRC

Narrow longitudinal cracks (< 0.5 mm wide) in JRC will have some load transfer through aggregate interlock and the presence of the reinforcement. Also, they will be too narrow to allow the ingress of external substances into the pavement. Therefore, no action is necessary, but they should be monitored regularly for crack width evolution.

Longitudinal cracks between 0.5 and 1.5 mm wide in JRC will offer reduced load transfer, relying solely on the reinforcement, and may permit the ingress of external substances into the crack, so the bay is likely to deteriorate further without attention. **Crack stitching** (see Section 5.4.8) is recommended to hold the crack at this width and potentially enhance load transfer. The subsequent sealing of the crack following the crack stitching process will prevent ingress external substances into the crack.

An **inlaid crack repair** (see Section 5.3.3) will not prevent the crack from widening further but can prevent ingress of external substances into the crack, reduce effects of the freeze-thaw cycle and reduce risk of spalling.

Where cracks are above 1.5 mm wide in JRC, the reinforcement has probably yielded and there will be no load transfer across the crack. Therefore, **full depth repair** (see Sections 5.4.3 and 5.4.4) or **bay replacement** (see Section 5.4.1) is recommended along with replacement of the foundation as necessary.

Longitudinal cracks in URC

URC does not contain any reinforcement to provide load transfer across cracks. Some load transfer across narrow cracks (< 0.5 mm wide) is likely to be provided by aggregate interlock, but beyond this there is likely to be inadequate load transfer. Longitudinal cracks in URC will continue to increase in width so it is important that narrow cracks are treated as soon as possible.

An **inlaid crack repair** (see Section 5.3.3) can prevent ingress of external substances into the crack, reduce effects of freeze-thaw cycles and reduce risk of spalling. However, this maintenance option will not prevent the crack from widening further.

To reduce the tendency for a narrow crack (< 0.5 mm) to widen to an extent where there is no aggregate interlock, **crack stitching** (see Section 5.4.8) may be undertaken. Crack stitching can also enhance load transfer across these cracks.

For longitudinal cracks in URC wider than 0.5 mm, there is unlikely to be any load transfer across the crack. Therefore, **bay replacement** (see Section 5.4.1) is recommended along with replacement of the foundation as necessary.

3.3.7. Stepping

Stepping (also known as faulting) is a difference in level across a joint or crack that is greater than 3 mm (see Figure 3.18). Table 3.17 gives an overview of the causation, treatment options and applicability for stepping.

Figure 3.18 Stepping

Table 3.17 Overview of stepping

Considerations	Treatment options	Applicability
Causation includes: • **traffic and environmental loading (see Section 4.1);** • **poor slab support through water ingress into the foundation and inadequate drainage (see Section 4.3.1); and,** • **dowel bar / tie bar issues (see Section 4.3.2).**	**Holding repair**	
	Under slab grouting and slab lifting (see Sections 5.4.5 and 5.4.6) with bump cutting as necessary (see Section 5.4.7).	May be ineffective if sub-surface drainage is inadequate. Intact bays only.
	Long-term repair	
	Drainage renewal and bay replacement or full depth repair (see Sections 5.4.1, 5.4.3 and 5.4.4). Foundation replacement may be necessary.	Where bays are cracked and / or where pumping is identified.

3.3.7.1. Causation

Stepping is most commonly a symptom of a loss of subbase support to the slab and requires the load transfer devices to have yielded before it occurs. Stepping normally occurs in proximity to other vertical movement such as depressions (settlement) and slab rocking. Stepping may also be apparent at a location of heave (Section 3.2.2).

After initiation, stepping typically becomes more severe with time under repeated dynamic loading and continues to degrade the ride quality. This can result in spalling at the concrete joint and cracking.

Poor slab support can occur for several reasons, which are further explained in Section 4.3.1. A typical mechanism is the ingress of water through defective joint seals and / or ineffective drainage followed by repeated traffic and environmental loading creating a void under the slab, reducing the load transfer efficiency and increasing the stresses applied to load transfer devices which eventually leads to the shearing of these load transfer devices.

Evidence of pumping indicates that the stepping is being caused by a loss of support due to the ingress of water and loss of fines creating a void in the subgrade. When water is trapped below the slab, the action of heavy traffic can initiate pumping and continued loss of the fine material will increase the severity of the stepping.

3.3.7.2. Treatment options

Stepping as a consequence of voiding in the absence of any pumping suggests that it has been caused by repeated loading, and treatment might comprise **under slab grouting** and **slab lifting** (see Sections 5.4.5 and 5.4.6) followed by **bump cutting** as necessary (see Section 5.4.7). There is a risk of reduced load transfer versus the rest of the pavement following this treatment where load transfer devices have yielded.

Under slab grouting is not recommended for joints where there is pumping as, whilst the immediate void will be filled, voiding is likely to continue around the filled void unless the root cause, water ingress and ineffective drainage, is addressed.

Severe stepping at joints and cracks may require major action. Where pumping has been identified, **full depth repair** (see Sections 5.4.3 and 5.4.4) or **bay replacement** (see Section 5.4.1) with drainage renewal and foundation replacement will be necessary. Depending on the severity and frequency of the stepping and other defects, reconstruction (see Chapter 7) may be appropriate.

3.3.8. Slab rocking

Slab rocking is the vertical movement observed at joints (or cracks) that occurs under the action of traffic loading (see Figure 3.19). Slab rocking can be identified audibly, visually or from pumping stains from a joint. Slab rocking can lead to more safety critical defects including stepping and joint spalling. Table 3.18 gives an overview of the causation, treatment options and applicability for slab rocking.

Figure 3.19 Slab rocking (evidenced by pumping)

Table 3.18 Overview of slab rocking

<table>
<tr><th>Considerations</th><th>Treatment options</th><th>Applicability</th></tr>
<tr><td rowspan="4">Causation includes:
erosion of foundation under the slab ends by water penetration and accumulation. Often identified by pumping stains at the joint (see Section 4.3.1); and,
dowel bar / tie bar issues (see Section 4.3.2).</td><td colspan="2">Holding repair</td></tr>
<tr><td>Under slab grouting (see Section 5.4.5).</td><td>Likely to be ineffective if sub-surface drainage is inadequate.
Intact bays only.</td></tr>
<tr><td colspan="2">Long-term repair</td></tr>
<tr><td>Drainage renewal and bay replacement (see Section 5.4.1). Foundation replacement may be necessary.</td><td>Where bays are cracked; drainage renewal is necessary; and / or pumping is identified.</td></tr>
</table>

3.3.8.1. Causation

Slab rocking is the symptom of a loss of uniform support and, unless occurring at cracks, normally requires any load transfer devices, at both ends of the slab, to have yielded before it occurs. Slab rocking cannot occur unless there are voids under the slab or there is poor slab support (Section 4.3.1) localised at the joints / cracks, to permit the vertical movements.

Slab rocking is often accompanied by the pumping of water with suspended subbase or subgrade fines at the joints / cracks. Pumping will over time increase the severity of the slab rocking and potentially result in other defects such as stepping, cracking or joint spalling.

3.3.8.2. Treatment options

For an intact slab, no immediate treatment is required in the absence of any safety or serviceability risk, but it is important to keep monitoring the defect evolution. **Under slab grouting** (see Section 5.4.5) can be undertaken as a holding repair, although this might not address the original causation. Additional measures such as renewing sub-surface drainage may be necessary to prevent further deterioration.

Where slab rocking is severe or where a long-term solution is required, it is recommended to undertake a **bay replacement** (see Section 5.4.1) and replace the foundation, as well as restoring the surface and sub-surface drainage.

3.3.9. Compression failures

Compression failures, also known as 'blow-ups', are the localised upward movement and / or shattering of a concrete slab at transverse joints or cracks. They generally occur when a crack or joint is not wide enough to permit the thermal expansion of a concrete bay (see Figure 3.20).

Although it is most commonly associated with JRC and URC, it has been known to occur infrequently in CRCP at terminations or construction joints.

Figure 3.20 Compression failures

Table 3.19 gives an overview of the causation, treatment options and applicability for compression failures.

Table 3.19 Overview of compression failures

Considerations	Treatment options	Applicability
Causation includes: • **environmental (high temperature loading) and / or narrow joints (less than 5 - 10 mm) (see Section 4.1.2);** • **design issues - slab dimensions (see Section 4.2);** • **locked up joints in preceding bays (see Section 4.3.2); and,** • **a lack of bond between upper and lower layers of concrete (in two-layer slab construction) and poorly compacted concrete (see Section 4.3.3).** **Immediate treatments are required to retain serviceability of carriageway.**	**Long-term repair** Full depth repair or bay replacement incorporating expansion joints (see Sections 5.4.1 to 5.4.4). Foundation replacement may be necessary. Further treatments may be necessary to mitigate the risk of future compression failures.	Emergency temporary repair is often required to enable continued use of the pavement. Full depth repair or bay replacement with concrete is required to prevent further deterioration of the pavement. Additional bay replacements, full depth repairs or introduction of expansion joints may be necessary at other bays in proximity of the defect to prevent further compression failures.

3.3.9.1. Causation

Where compression failures occur, it is important to identify the causation to understand whether the failure is isolated due to local issues with that joint or likely to reoccur on other sections of the pavement. The mechanism is typically a volumetric expansion in the concrete pavement that cannot be accommodated by the available joint space. These are likely to occur during hot weather, and the effect may be accentuated by a high moisture content in the concrete. Compression failures often occur after heavy rainfall followed by high temperatures, the pressure built-up is suddenly released as the pavement thrusts upwards and / or shatters.

Many factors can contribute to this mechanism, such as:

- locked up joints in a length of preceding concrete bays to effectively create a long bay. Typically, this is due to the ingress of incompressible material into the joint space through defective joint seals, misaligned or badly corroded dowel bars (see Section 4.3.2);
- design issues - long bays are more susceptible to compression failures (see Section 4.2); or an absence of expansion joints;
- construction issues - thin joints formed less than 5 - 10 mm in width; or a lack of bond between upper and lower layers of concrete (in two-layer slab construction) and / or poorly compacted concrete (see Section 4.3) to create a thin and / or weak slab;
- over expansion of the slab due to extreme high temperature (see Section 4.1.2); and,
- ground anchorage issues in CRCP (see Section 5.2.4).

3.3.9.2. Treatment options

Compression failures normally extend over the width of at least one traffic lane and require an immediate emergency temporary repair to enable traffic to use the carriageway. Permanent repair involves **bay replacement** (see Section 5.4.1) or **full depth repair** (see Sections 5.4.3 and 5.4.4) across the full carriageway width incorporating an expansion joint to cope with potential future expansion. Bay replacement should take place together with full depth repair or **retrofitting dowel bars** (see Section 5.4.10) at other joints if they are identified to be locked up, to reduce risk of future compression failures. Retrofitting dowel bars is a non-standard technique, therefore, a departure from standard is required for this option.

Once a section of road has failed in compression, the likelihood of further similar failures can be reduced by installing new expansion joints. Spacings of 250 m and 300 m have been reported as successful although closer spacing may be necessary to limit seasonal movements within the capability of joint sealants. Installation should be carried out during the spring or autumn, avoiding hot weather.

Where it is identified there is a risk of compression failures, either due to a history of compression failures, locked up joints or narrow joint spacings, saw cutting the full depth of locked up joints to ease strain has been reported as a successful temporary measure. However, as this method effectively creates a scenario whereby there is zero load transfer between bays, accelerated deterioration of the pavement at these locations is likely.

3.4. Features specific to CRCP

This section covers features specific to CRCP, namely, inherent (transverse) cracks.

3.4.1. Inherent (transverse) cracks

CRCP does not contain regular contraction and expansion joints. Instead, thermal stresses are relieved by regular closely spaced cracks that form within a well-designed and well-constructed CRCP slab. Inherent cracks in CRCPs are exclusively transverse with no spalling or bifurcations, they are ≤ 1 mm in width and are spaced at least 1 m apart (see Figure 3.21). These cracks are held closely together by the continuous longitudinal reinforcement, thereby ensuring good aggregate interlock and load transfer.

Figure 3.21 Inherent (transverse) cracks in CRCP

It is normal for fine transverse thermal contraction / shrinkage cracks to develop every 1 - 4 m soon after construction. Over the following 4 years or so, further transverse cracks slowly develop between the wider spaced cracks, resulting in a typical crack spacing of 1 - 2 m. These regular fine width cracks are an inherent feature of good quality CRCP and essential to the long-term performance; they do not indicate weakness.

3.5. Defects specific to CRCP

This section covers defects specific to CRCP. These include:

- significant transverse cracks; and,
- longitudinal cracks.

Repairs to CRCP are generally more difficult and costly than to other types of concrete pavement because of the large quantity of heavy steel reinforcement in the slab and the high levels of stress that are generated within it. The most appropriate time of the year to carry out work requiring part of the slab to be demolished (i.e. full depth repairs), is during the spring and autumn months, which avoids working during particularly hot weather when compressive stresses are high and the slab may buckle, or during cold weather when the slab is in tension.

Preventive maintenance to prolong the structural life of the slab is highly desirable in terms of WLC. Measures such as the sealing of medium to wide cracks, treatment of spalled cracks, and grouting to stabilise vertical slab movement may all be necessary.

3.5.1. Significant transverse cracks

Significant transverse cracks are those exhibiting different behaviour to inherent transverse cracks in CRCP. Table 3.20 gives an overview of the causation, treatment options and applicability for significant transverse cracks in CRCP. This includes transverse cracks that are:

- greater than 1 mm wide;
- spaced less than 1 m from another transverse crack (see Figure 3.22);
- bifurcated;
- polygonal; and / or,
- spalled.

Transverse crack width and spacing is the most critical factor affecting punchout development.

Figure 3.22 Significant transverse cracks in CRCP

Table 3.20 Overview of significant transverse cracks in CRCP

<table>
<tr><th>Considerations</th><th>Treatment options</th><th>Applicability</th></tr>
<tr><td rowspan="7">Causation includes:
• external restraint from the foundation;
• internal restraint from the reinforcement and concrete coefficient of thermal expansion;
• construction issues (insufficient reinforcement, poor quality concrete, inadequate curing); and,
• poor slab support (see Section 4.3.1).</td><td colspan="2">No treatment advised</td></tr>
<tr><td colspan="2">Inherent cracks (see Section 3.4.1).</td></tr>
<tr><td colspan="2">Holding repair</td></tr>
<tr><td>Inlaid crack repair (see Section 5.3.3).</td><td>Isolated cracks > 1.0 mm wide.</td></tr>
<tr><td colspan="2">Long-term repair</td></tr>
<tr><td>Full depth repair (see Section 5.4.2). Foundation replacement may be necessary.</td><td>For high severity transverse cracks.
Foundation may need to be replaced if poor slab support or inadequate drainage is found.</td></tr>
</table>

3.5.1.1. Causation

The transverse cracks that form in CRCP are designed to remain tight. However, where these cracks widen, the resultant infiltration of water, de-icing salts and incompressible materials, could lead to the corrosion of the reinforcing steel and / or further exacerbation of the defect including crack spalling.

All pavements are designed to carry a number of vehicle loads over their service life. End of service pavements may show signs of fatigue cracking due to the exceeded number of vehicle loads and increased stresses. This will be accelerated by higher traffic levels and heavier loading than expected. However, several design, construction and maintenance factors can cause significant transverse cracking in CRCP:

- external restraint of the concrete from the foundation;
- internal restraints on the concrete;
- insufficient reinforcement;
- poor or non-uniform slab support; and,
- poor quality control of concrete.

External restraint of the concrete from the foundation. CRCP requires sufficient and uniform friction from the underlying foundation to produce desirable crack patterns. Materials with low friction (i.e. asphalt) will produce larger crack spacings with larger crack widths versus hydraulically bound materials. Where restraint is non-uniform, variable and potentially undesirable initial crack patterns can occur. This can be exacerbated if the total drying shrinkage is not minimised during curing through effective curing practices.

Internal restraints on the concrete from reinforcement and aggregate linked to the coefficient of thermal expansion. The crack pattern of CRCP using siliceous gravels is generally less desirable (cracks are wider with increased spacings) than CRCP using limestone aggregates. The crack pattern of CRCP using siliceous gravels can be improved by the placement of reinforcement at a third of the slab depth rather than at mid-depth. Irregular crack patterns will be worsened if curing conditions are poor and excessive drying shrinkage is permitted.

Insufficient reinforcement. The transverse crack pattern in CRCP is also largely controlled by the percentage of cross-sectional longitudinal steel. Insufficient reinforcement can result in longer transverse crack spacing and wider cracks.

Poor or non-uniform slab support due to inadequate compaction of the subgrade or subbase (where unbound) during construction or subsequent erosion due to lack of adequate drainage (see Section 4.3.1); or lack of uniform formation level control to create anchor points for the CRCP slab above. However, this is unlikely to be a major contributor to transverse cracking as CRCP is designed to bridge and flex over localised areas of poor support.

Poor quality control of concrete. Failure to achieve uniform strength of concrete around the targeted design values can result in different crack spacings than intended in the design. A properly calibrated concrete plant, good aggregate stockpile management, and the ability to make adjustments to the concrete as a function of ambient conditions are important factors in maintaining uniform properties.

3.5.1.2. Treatment options

For isolated significant transverse cracks that are greater than 1 mm in width, it is recommended that they are treated with an **inlaid crack repair** (see Section 5.3.3) to prevent ingress of water, salt and incompressible materials into the crack space, which will reduce the effect of freeze-thaw cycles and reduce the risk of corrosion of the reinforcement.

Full depth repair (see Section 5.4.2) is recommended in the following situations to prevent the occurrence of punchouts:

- at transverse cracks wider than 1.5 mm, where the reinforcement is likely to have yielded;
- where crack spacing is < 1 m; and,
- where cracks are bifurcated or polygonal.

3.5.2. Longitudinal cracks

Longitudinal cracks run approximately parallel to the centreline of the pavement. They can be designed to develop, using crack inducers below breaks in the transverse reinforcement, at the lane edge or centreline in a wide slab. Where they are not designed and not expected they may deteriorate further if no action is undertaken.

Where both longitudinal cracks and, either the inherent or significant, transverse cracks intersect there is a risk of spalling and potentially the formation of punchouts. Depending on the width, longitudinal cracks can provide a route for water and de-icing salts into the pavement, potentially initiating corrosion of the reinforcement. If incompressible materials enter the cracks, spalling may occur. Figure 3.23 shows an example of a longitudinal crack in CRCP.

Figure 3.23 Longitudinal crack in CRCP

Table 3.21 gives an overview of the causation, treatment options and applicability for longitudinal cracks in CRCP.

Table 3.21 Overview of unplanned longitudinal cracks

<table>
<tr><th>Considerations</th><th>Treatment options</th><th>Applicability</th></tr>
<tr><td rowspan="7">Causation includes:
closely spaced transverse cracks and traffic loading;
poor slab support (see Section 4.3.1);
shallow cover to reinforcement (see Section 3.5.2.1);
late longitudinal saw cuts (see Section 4.3.3); or,
design / construction issues - inadequate slab thickness, improper longitudinal joint layout.</td><td colspan="2">No treatment advised</td></tr>
<tr><td colspan="2">For narrow cracks (< 0.5 mm).</td></tr>
<tr><td colspan="2">Holding repair</td></tr>
<tr><td>Inlaid crack repair (see Section 5.3.3).</td><td>Isolated cracks between 0.5 and 1.5 mm wide.</td></tr>
<tr><td colspan="2">Long-term repair</td></tr>
<tr><td>Full depth repairs (see Section 5.4.2). Foundation replacement may be necessary.</td><td>For high severity longitudinal cracks.
Subgrade and foundation may need to be removed and replaced if poor slab support or inadequate drainage is found.</td></tr>
</table>

3.5.2.1. Causation

Where longitudinal cracking occurs in CRCP, it is generally caused by either poor design, poor construction or poor maintenance. Short longitudinal cracks can form between closely spaced transverse cracks, typically in the nearside wheel track, to create short ‘segments’ in the CRCP, particularly in the presence of localised poor support, where stresses are much higher. The end result of this cracking is likely a punchout (see Section 3.2.3).

More significant lengths of longitudinal cracking may form as a result of poor or non-uniform support conditions due to poor construction techniques and / or poor maintenance.

The performance of CRCP is highly dependent on an adequate and uniform support from the underlying foundation. Poor or non-uniform slab support can arise for multiple reasons including:

- inadequate preparation of the foundation during construction;
- use of non-uniform soils with different moisture sensitivities; or,
- erosion of the subgrade due to inadequate or ineffective sub-surface drainage.

Load induced longitudinal cracking under poor or non-uniform support conditions is often recognised by the close proximity of the cracking to the pavement edge.

Other design and construction related factors can cause or contribute to the development of unplanned longitudinal cracking, examples include:

- Insufficient slab thickness as a result of under-design or construction practices (see Section 4.2).
- Shallow cover to reinforcement, which can result in cracking and spalling directly above the bars.
- Inadequate joint layout. Where multiple CRCP lanes are constructed adjacent to each other and ‘tied’ together to prevent lane migration or opening of the, induced or formed, longitudinal joint, depending on the level of friction or restraint below the slab, tying more than 3 to 4 lanes together may result in the development of random longitudinal cracks, generally in one of the inside lanes.
- Late longitudinal saw cuts (see Section 4.3.3). Saw cutting too late or too shallow can result in the development of a random longitudinal crack.
- Inadequate curing (see Section 4.3.3). Adequate and timely curing is necessary to promote strength development in the concrete, minimise moisture and temperature differentials within the slab during early age strength development, and minimise plastic shrinkage cracking.

3.5.2.2. Treatment options

Unplanned longitudinal cracks are likely to deteriorate further once formed, and where they intersect with transverse cracks, they are likely to eventually culminate in the formation of punchouts. Dependent on the severity of the crack, proximity to significant transverse cracks and the asset management strategy, a **full depth repair** (see Section 5.4.2) may be appropriate to prevent the formation of punchouts.

Otherwise, for unplanned longitudinal cracks that are greater than 0.5 mm in width, it is recommended they are treated with an **inlaid crack repair** (see Section 5.3.3) to prevent ingress of water, salt and incompressible materials into the crack space, which will reduce the effect of freeze-thaw cycles and reduce the risk of corrosion of the reinforcement.

4. Defect causation and diagnosis

Deterioration mechanisms in concrete pavements are very different to those of other pavement types. The combined effect of traffic and environmental (particularly the daily cycle of thermal expansion and contraction) loading to concrete pavements generates horizontal tensile stresses and warping stresses. If these stresses are high enough, relative to the strength of the concrete slab, fatigue cracking will develop in the concrete, which can ultimately lead to a structural failure condition. See Section 1.2 for further details.

Chapter 3 has outlined the various defect types that can arise in a concrete pavement and introduced the possible causations, along with the available maintenance options to support either a short-term or long-term asset management strategy.

Different defects that occur in a concrete pavement can be the result of the same causation factor. It is the extent, location and severity of the causation factor that influences the type of defect that might occur. An example of this is poor slab support; where this occurs at isolated joint locations it may result in corner cracking (Section 3.3.2.1), whereas, when it is more widespread it can lead to depressions (settlement) (Section 3.2.1.1). In addition to loading, the most relevant factors contributing to occurrence of defects and / or accelerating deterioration of a concrete pavement, are related to deficiencies in the design, construction and / or maintenance.

The cause of a defect may not be immediately apparent, in which case, an investigation is necessary to determine the most likely causation(s). Choosing from a suite of proven and practical investigation techniques, the possible causations can be confirmed or ruled out so that the appropriate maintenance option can be selected to rectify the defect(s) and ideally address the causation to prevent reoccurrence. This may be repair (Chapter 5) or reconstruction (Chapter 7).

This chapter expands on Chapter 3 to discuss the primary causations in more detail and to prescribe investigation techniques linked to Chapter 2 for correctly diagnosing the underlying causation(s).

4.1. Loading

Concrete pavements that are well designed, constructed and maintained typically fail by some form of tensile fatigue cracking under repetitive heavy traffic loading and / or environmental loading after many years in service.

4.1.1. Traffic loading

A primary function of a concrete pavement and its foundation is to distribute the design traffic loading in order to reduce the vertical compressive strains experienced in the subgrade to an acceptable level under drained equilibrium moisture conditions. The concrete slab should be designed to achieve distribution of the design traffic loading without cracking, unless it is designed with steel reinforcement to hold the crack tightly together.

Cracking of the concrete slab through repeated low stress traffic loading applications is known as fatigue cracking. Limiting fatigue cracking to an acceptable level by minimising the magnitude of stresses over the desired number of traffic loading repetitions through the design of a suitable concrete pavement structure (including thickness, strength and foundation support) is the basis of most mechanistic-based design procedures.

In order to design a pavement, the expected traffic loading in terms of tyre pressure, axle and wheel configurations, repetition and channelisation, and vehicle speed should be estimated for the required design life (or design period), which is typically taken to be 40 years.

Most pavement design methods relate real traffic to an equivalent number of standardised loads, in the form of a ‘power law’ to link the magnitude of a real axle load to a standard design axle. The damage produced by different axle loads is then the sum of the damage, expressed as a number of standard axles, and this represents the pavement design traffic [11].

The UK approach uses a fourth power law to convert the typical range of 2 - 6 axle rigid and articulated commercial vehicle traffic loadings to an equivalent number of passes by an 80 kN standard axle. This is expressed in millions of standard axles (msa). CD 224 details the procedure to be used for traffic calculations in the UK [41].

The design traffic (in msa) is used:

- for designing a minimum of three ‘standard’ CD 226 pavement options for a new carriageway [12], to include flexible with an asphalt base, flexible with an HBGM base and at least one type of rigid concrete pavement, for comparative analysis to determine the optimum pavement construction; and,
- for comparison with the pavement residual life of an existing carriageway, to identify and select the most appropriate maintenance techniques (see Chapter 5 and Chapter 7) to suit the preferred maintenance strategy.

Once a pavement approaches or exceeds the msa loading cycles for which it was originally designed (i.e. the design traffic) then continued traffic loading will result in a higher risk that deterioration of the pavement will occur.

4.1.2. Environmental loading

Environmental loading can have a significant impact on the pavement materials and / or the underlying subgrade material. It can lead to differential movement and / or loss of strength which in turn can drastically reduce the performance of these materials under traffic loading within the design life of the pavement. The main environmental factors that affect concrete pavement materials are:

- thermal movements;
- moisture variations;
- frost, frost heave and freeze-thaw cycles; and,
- carbonation.

Environmental loading therefore should be considered during the design of a new pavement in terms of material selection, and throughout the service life of the pavement to ensure timely and appropriate maintenance, in order to prevent unwanted expansion, shrinkage or loss of strength from occurring.

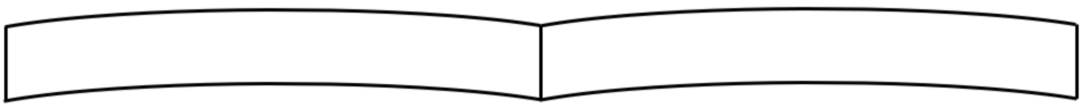
4.1.2.1. Thermal movements

Thermal movement in a concrete slab occurs during:

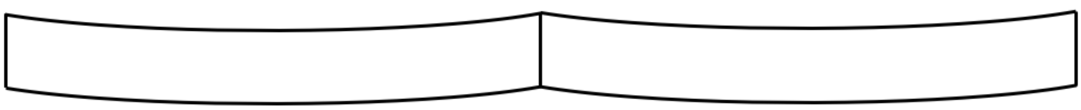
- early life, which is influenced by factors including the:
    - temperature differential through the slab;
    - coefficient of thermal expansion of the concrete;
    - restraint to movement offered either by adjacent elements or by differential strain within the concrete; and,
    - ability of the concrete to resist tensile strain [42]; and,
- service life, due to the variations in the temperature of the environment surrounding the matured concrete slab where the temperature change is:
    - gradual and the slab is unrestrained at the edges, then it will simply expand and contract in plan area, and;
    - more rapid then slab curling can occur.

Slab curling movements are caused by a temperature gradient through the depth of the concrete slab. During daylight hours the pavement surface absorbs solar radiation, therefore, the surface temperature increases and the top portion of the slab expands more rapidly than the lower portions of the slab, which are increasingly insulated from the solar radiation. This results in a negative slab curvature and can result in a loss of contact support below the centre of the slab, particularly when the underlying subbase is bound see a) in Figure 4.1.

A positive slab curvature describes the opposite curling movement that occurs with the reverse temperature gradient during the night that can result in a loss of contact support below the edges of the slab see b) in Figure 4.1. Slab curling is more severe for longer and / or wider slabs but is not usually observable.



a) Negative slab curvature (curling or warping) where concrete slab surface temperature is higher than the lower portion of concrete slab. Usually occurring during daytime.



b) Positive slab curvature (curling or warping) lower portion of concrete slab is higher than the concrete slab surface temperature. Usually occurring during night time.

Figure 4.1 Illustration of a) negative curvature, and b) positive curvature.

The early life thermal movement is controlled by effective curing practices. For a matured concrete pavement subject to daily and seasonal temperature cycles that induce thermal shrinkage and expansion movements, these are accommodated in URC and JRC design by the use of regularly spaced contraction joints and less frequent expansion joints. In CRCP these thermal movements are accommodated by closely spaced narrow transverse cracks (Section 3.4.1).

However, when design issues (see Section 4.2) or construction and / or maintenance issues (see Section 4.3) result in a restraint to this thermal movement then the internal stresses that are generated may build up to exceed the concrete strength.

Where the slab is restrained from contracting, the result will be cracking to relieve any restrained shrinkage stress. However, where expansion is restrained, typically as a combination of multiple factors, the result can be shoving (horizontal movement) (see Figure 4.2) and in extreme circumstances during a particularly hot summer when the temperature variations are high, compression ‘blow-up’ (vertical movement) to relieve any restrained expansion stress (see Chapter 3). Contraction restraint can be caused by factors including:

- locked up joints; and,
- the underlying layer.

Expansion restraint, in addition to the above factors, can be caused by factors including:

- excessive temperatures resulting in slab expansion greater than the allocated joint space; and,
- incompressible foreign materials in the joint space.

Figure 4.2 Shoving (left) and compression failure (right)

4.1.2.2. Moisture variations

A moisture gradient through the depth of the concrete slab can result in the same negative and positive slab curvatures as described above for thermal movements. They may be differentiated as moisture ‘warping’ movements rather than thermal ‘curling’ movements, but in practice the cause may not be known and the terms warping and curling are often used interchangeably.

Negative movement may occur as a response to surface water that penetrates the upper portion of the concrete slab after rainfall, while positive movement may occur when the surface water dries out more quickly than the saturated lower portions of the concrete slab. It may also be a consequence of water vapour, from the water table, that reaches and penetrates the underside of the concrete slab, which may occur if there is no barrier to this movement.

In addition, pavements could be subjected to considerable moisture variations throughout the year. A good understanding of moisture variation effects on materials and pavement performance are important in both the design and maintenance of a pavement. The pavement structure needs to be protected from surface water ingress following periods of rain or the melting of snow, and from rising groundwater levels, by the provision of adequate drainage. In order to account for a particular local climate, drainage condition and subgrade type then weather factors and drainage coefficients or safety factors should be accounted for in the design of initial construction and then maintenance.

In subgrade materials, especially in those with a high fines content such as a clay subgrade, variations in water content can significantly affect the material’s stiffness and permanent deformation characteristics thereby affecting the support provided to the overlying layers [43]. This effect becomes critical in expansive subgrade soils, which are rich in clay materials with a high plastic index and liquid limit, along with a low shrinkage limit. These soils can exhibit significant volume changes (swell and shrinkage) in response to increasing and decreasing water content.

For all pavement types this potential is typically addressed at the design and construction stages, including adequate drainage provision and geotechnical risk mitigation (such as mechanical or chemical stabilisation, or ground improvement). Concrete pavements may be more resilient than flexible pavements, in terms of resisting the effects of ground movements if they do occur. However, concrete pavements are designed to be predominantly ground bearing; therefore, it is important to mitigate any geotechnical risk, rather than trust that the pavement design can accommodate the risk. Consistent across all pavement types, deterioration in the form of cracking, settlement or heave can occur when the water content in the subgrade is allowed to vary.

The water content in a subgrade typically returns to its level of equilibrium approximately 10 years post-construction. At this point, a well-maintained pavement will not experience significant volume changes in the foundation as a result of changing water content. An increase in the amount of water allowed to enter into the pavement is needed to bring about any changes in subgrade condition. This increase may be from a change in environmental conditions, or joint seals or drainage becoming ineffective at preventing water ingress into the pavement due to lack of maintenance.

Locally trapped water at joints can also be pumped at high pressure under moving traffic loads (see Figure 4.3), resulting in erosion at the slab edges in the supporting unbound layers, which may be followed by cracking, slab rocking and stepping.

Figure 4.3 Pumping

Where investigation is necessary, non-destructive techniques, such as GPR (see Section 2.2.1.2), can be used for in situ real-time measurements of the groundwater table level as well as subgrade water content. Deflection measurements from FWD surveys (see Section 2.2.1.1) can be used as indicators for a preliminary assessment of the pavement structure, evaluating seasonal climatic effects on overall pavement response and material stiffness. More invasive techniques and testing should be used to determine soil properties and their susceptibility to moisture (see Section 2.2.2). Specific care should be taken where problematic soils are known to be present. These should be identified as a geotechnical risk as part of subgrade assessment.

As illustrated in Section 2.3 and specified in CS 551 [26], drainage surveys should be carried out regularly and analysed in conjunction with data coming from other investigation steps.

4.1.2.3. Frost, frost heave and freeze-thaw cycles

In cold regions, frost action can result in defects such as surface scaling (Section 3.1.2) and pop-outs (Section 3.1.4). The basic mechanism of frost damage is a product of the freeze-thaw action of entrapped water within the structure of the concrete matrix. This issue can be prevented by reducing the size of voids, adding air-entraining agents or ensuring a high-strength and dense concrete slab [11].

Frost heave can occur in certain frost susceptible soil types in areas with sustained freezing temperatures. Frost heave occurs as a result of capillary gradients that are created as water passes through porous media toward a freezing plane. Ice lenses and layers of ice accumulate as more water is drawn toward the freeze plane leading to heave of the pavement above.

Differential frost heave can occur where there is non-uniformity of the subgrade soil and frost susceptible soil pockets are present. Although the heave itself can present as a surface profile defect (Section 3.2.2), the subsequent thawing and differential settlement (Section 3.2.1) of the concrete slab can raise even greater concern when the surface profile defect is accompanied by pumping and / or a cracking defect.

Frost susceptible soils have a low plasticity index, meaning a low percentage of water can change the soil from a plastic condition to a liquid condition. In this regard, it is important to assess soil properties through invasive testing (see Section 2.2.2), especially where the risk of sustained freezing temperatures is reasonably high.

4.1.2.4. Carbonation

Carbonation is the physical and chemical transformation of cementitious material under the influence of prolonged exposure to the carbon dioxide present in air. Carbon dioxide penetrates into pores in the concrete and reacts with calcium hydroxide present in the concrete to form calcium carbonate. Carbonation has the following effects on the concrete:

- The alkali nature of the concrete (> pH 11), and the calcium hydroxide present, results in the creation of a 'passivating layer' around steel reinforcement, protecting the reinforcement from corrosion. However, carbonation reduces the pH of concrete, leaving the reinforcement susceptible to corrosion.
- Calcium carbonate molecules are smaller than calcium hydroxide molecules; therefore, the porosity increases in the affected portion of concrete, leaving the concrete more susceptible to further carbonation and freeze-thaw cycles through water ingress. This can also result in shrinkage and microcracking and potentially crazing and delamination (scaling) of the carbonated area.

Carbonation is mainly affected by time; however, it is a complex process where many other factors are involved related to the design and the construction of the concrete. The rate of carbonation is principally affected by the porosity of the concrete, the lower the porosity the less carbon dioxide can penetrate into the concrete. Therefore, using low porosity concretes i.e. those with a low water / cement ratio that have been compacted adequately, can reduce the rate of carbonation.

As carbonation reduces the pH of the concrete, simple assessments using pH indicators (such as phenolphthalein) can give an indication of the rate level of carbonation of an area of concrete (see Section 2.2.3.1). Concrete with lower pH (≤ 8) will likely be carbonated, so any reinforcement present is at greater risk of corrosion.

4.2. Design issues

Concrete pavements on the SRN have been designed based on empirical data derived from the observed performance of full-scale experimental pavements. This well proven approach has long been established as the design basis for both unreinforced and reinforced (jointed and continuous) concrete pavements, as specified in CD 226 [12]. The principles and equations for undertaking concrete pavement designs in the UK are set out in TRL RR 87 [44] (for URC and JRC pavements) and TRL 630 [45] (for CRCP).

These design equations carry assumptions and approximations. The major limitation to developing more economic designs for sustainable long-life concrete pavements is the available empirical performance data when considering variables such as different concrete mixtures, aggregate thermal properties, moisture gradients, climate, joint spacing and performance. Nevertheless, most UK concrete pavements designed using the equations provided in these two TRL documents have significantly outlived their intended (designed) service life and traffic loadings. This indicates that concrete pavements have been designed to a very high level of reliability.

It is beyond the scope of this CPMM to consider the possible implications of a concrete pavement design deviating from the standard design methodologies outlined in CD 226 [12] and detailing outlined MCHW 1000 [7] and MCHW Volume 3 [39], in terms of its ability to cope with traffic and environmental loading. However, there are a few frequently occurring lesser regarded design aspects related to detailing that often lead to the formation of defects in concrete pavements in the UK. This section therefore discusses:

- acute slab corner angles;
- isolation around ironwork;
- joint misalignment; and,
- ground anchorages in CRCP.

4.2.1. Acute slab corner angles

Acute angles are those less than 90 ° and their detailing, to create a non-rectangular slab plan dimension, may be necessary where a concrete pavement is specified at junctions or bends. However, they create higher stress concentrations that can lead to the development of cracking. Typically, slab corner angles that are more acute than 85 ° are at risk of corner cracking (Section 3.3.2) and / or deep spalling (Section 3.3.1).

They can often be designed out by careful planning of a joint layout pattern that avoids them. However, if the use of acute angles cannot be avoided, additional reinforcement should be installed at these corners to hold tightly together any cracks that do occur.

4.2.2. Isolation around ironwork

Ironwork in pavements can include gullies, manholes, inspection chamber covers and stop-tap covers. Studies of defects in concrete pavements on the SRN have identified that many defects, predominantly cracks, emanate from ironwork that is placed within concrete slabs that are subject to heavy and / or direct traffic loading. If ironwork penetrates through a slab, transverse sympathy cracking is likely. Similarly, cracking is likely if the slab is ‘propped up’ or resting on the gully or manhole construction.

To prevent this, the ironwork should be located in the verge, ideally, by the original design. However, where it has to be located within a concrete slab, the ironwork should be isolated, to give the best chance of avoiding cracking, as shown in Volume 3, C Series of the highway construction details in the MCHW [39] and additional reinforcement included to hold tightly together any cracking that may occur. The opening for the ironwork either should be made large enough to encompass the shaft of the structure and any concrete surround to it, or the top of the shaft and surround should not be closer than 200 mm to the underside of the subbase.

If necessary, to reduce the risk of such cracking, the recesses should be in the comers of the bays, either astride or alongside a transverse joint, with chamfered corners. When possible, an extra warping joint should be constructed from the centre of the recess across the slab to the nearest longitudinal joint and / or edge of the slab. A ‘sympathetic crack’ is likely to form in the adjacent lanes unless this additional joint is extended across the adjoining bays.

4.2.3. Joint misalignment

For concrete pavements comprising parallel bays separated by longitudinal joints, as originally designed or as a subsequent widening design, it is important that the transverse joints are not staggered but are aligned to form a single joint passing through all of the concrete bays across the full width of the carriageway. Failure to do this will almost inevitably result in a ‘sympathetic crack’ forming across the width of the adjacent bays, emanating from the misaligned end of the joint.

The same situation can occur where previous remedial work has included a bay replacement or full depth repair (Sections 5.4.1 to 5.4.4) but without following the good practice guidance that corresponding joints are provided in the adjacent slabs in order to design out the risk of a sympathetic crack occurring.

It is important to correctly diagnose sympathetic cracks, and not to attribute their occurrence to some other cause such as poor slab support (Section 3.3.5).

4.2.4. Ground anchorages in CRCP

Longitudinal thermal movements accumulating at the ends of a CRCP slab can be considerable and have the potential to damage adjacent pavements or structures. The central part of the CRCP slab is more restrained and internal thermal stresses are relieved by the many small longitudinal movements that are facilitated by the transverse cracks, which are normally present at 1 - 2 m spacings (Section 3.4.1).

This end-movement can be partly restrained by one of two established systems in the UK, a ground beam anchorage (GBA) or a wide-flange steel beam (WFB); or in some circumstances accommodated by a special joint [1]. A potential innovation is developing a more economic termination system based on a bridge-type joint [45].

The design concept of the anchorage system is to transfer the pavement expansion forces to the soil mass through the passive bearing and shear resistance of the subgrade. In design, the critical elements are the bearing area of the beams and the shear plane along the bottom of the beams.

When ground anchorages do fail, the results are often catastrophic. Failure mechanisms include partial extraction of the anchoring system from the adjacent concrete and separation of the end of the hook from the compression sleeve.

Standard detail (Drawing number C18) has been developed for GBA on the SRN [39]; however, GBA is not recommended where the subgrade strength is poor, or on high embankments where consolidation may be insufficient to restrain movement of the beam downstands. The main problems associated with the performance of WFB terminations, reported in TRL 630 [45], are debonding of the joint seal and fatigue of the beam. Recommendations are given to reduce the amount of thermal movement at terminations by locally increasing the subbase friction and / or reducing the coefficient of thermal expansion of the concrete.

4.3. Construction, maintenance and use issues

Several elements of construction operations can impact the overall quality of the pavement. The weather conditions, site preparation, the equipment used, the availability of experienced and skilled staff and the quality of materials are a few examples.

For CRCP, evidence of pavement failure in the UK suggest that this is likely to be as a result of poor construction or maintenance, rather than issues in design and that the failure mechanism can be complicated and a result of a combination of more than one element.

In general, to ensure that a pavement has the desired quality and durability then considerable care, good practice and expertise are required at all stages of construction from the preparation and protection of the subgrade to finishing and curing of the concrete slab. There should be specific focus on the placement of any steel reinforcement, concrete compaction and curing, which, together with the actual concrete strength achieved in situ, are the construction variables that have been associated with the largest impacts on the long-term performance of concrete pavements in the UK.

To assist in quality control, innovative non-destructive systems are increasingly becoming the key to success, providing continuous monitoring of the pavement properties during construction and rapid feedback (see Section 2.4.2).

Further to loading as discussed in Section 4.1, the following section discusses the primary causes of the defects in concrete pavements that are related to construction, maintenance or use of the pavement.

4.3.1. Poor slab support

All ground bearing concrete pavements are reliant on uniform support from the foundation. Non-uniform support can result in a negative curvature of the slab across the location of reduced support or void (see Figure 4.1), putting areas of the slab into tension that, under repeated use by vehicle loads, may lead to structural cracking defects or shearing of the load transfer devices followed by stepping or slab rocking. Loss of support to a concrete slab is more common in those pavements with an unbound subbase linked to water penetration and softening / erosion.

A local reduction of support can be linked to water penetrating through joints and softening the subbase or subgrade layers where the joint sealant has been defective and not replaced. Repeated loading of these affected joints by traffic can result in voids being formed, reducing the support to the slab (see Figure 4.4).

Reduced slab support over a wider area is characterised by defects which include an element of settlement. Settlement typically occurs as a result of poor construction that permits secondary compaction of fill, or as a result of poor maintenance that allows the sub-surface drainage to become ineffective.

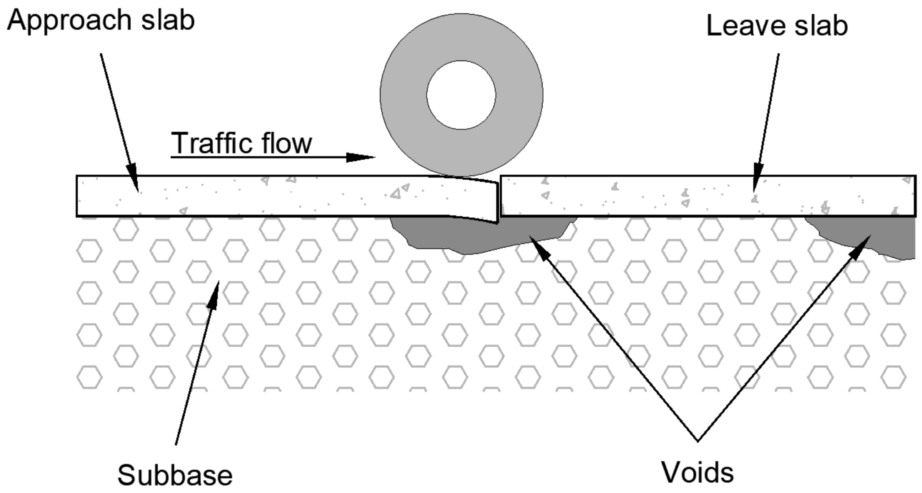


Figure 4.4 Voiding caused by water penetration into the subbase through joints resulting in localised inadequate slab support.

In order to effectively identify areas of inadequate slab support as well as correctly diagnose and treat the associated defects, an appropriate investigation needs to be undertaken. Without such an investigation any subsequent maintenance treatment carries the risk that it may not address the root cause of the issue to prevent continued deterioration.

Different techniques, beyond visual assessments, for the detection of voids and loss of foundation support to a concrete slab are outlined in Section 2.2.1; these include FWD, GPR and ultrasonic tomography surveys [22].

4.3.2. Load transfer devices and joints

A wheel load applied to an unsupported edge or corner of an individual concrete bay can result in a tensile stress that is up to double that induced by the same wheel load applied to the centre of the slab. Therefore, load transfer devices (dowel bars and tie bars) are used to significantly reduce the increased wheel load stresses away from the slab centre by transferring a portion of the load to the adjacent bay, providing a degree of support to the edge and removing corner loading as a design condition.

For highways, dowel bars transfer wheel loads across transverse joints and tie bars transfer wheel loads across longitudinal joints. Historically, these load transfer devices have been made from steel and the main issues during construction and use are:

- restrained dowel bars; and,
- corrosion.

Construction and maintenance issues at joints include:

- locked up joints; and,
- defective joint sealant.

GPR surveys and ultrasound can be used to determine the position and orientation of the load transfer devices non-destructively; ultrasound can give an indication of bar condition including corrosion, or physical examination of drilled core samples can be undertaken.

Any initial core sample should be drilled at a location away from the wheel track zones. Only if this reveals significant corrosion should a core be extracted from the wheel track zones. This reduces the possibility of causing an unintended reduction of LTE at a suspect joint.

4.3.2.1. Restrained dowel bars

Restrained dowel bars can be a consequence of poor construction, specifically the dowels can become 'locked' due to misalignment, which prevents movement as discussed in Section 4.3.2.2. Dowel bars need to be parallel to the pavement surface to enable free, uninhibited opening and closing of the joints.

Dowel bar assemblies between new bays need to be fixed in place and sufficiently rigid to resist deformation during placement of the concrete. Where dowel bars are being fixed into an existing joint face, significant care is required to ensure dowel bar alignment is correct. This requires careful drilling practices, then alignment, ensuring that the line and level of the dowel bars is checked once dowel bars have been inserted into the drilled holes prior to the concrete being placed (see Section 5.4.1.1).

Restraint of dowel bars may be apparent shortly after construction from the occurrence of defects such as deep joint spalling, corner cracks, transverse and longitudinal cracks, and compression failures which can develop due to the high stresses in the concrete slab as a result of the restrained movement.

Poor installation of load transfer devices might be diagnosed immediately following construction by measuring the load transfer efficiency (LTE), see Section 2.2.1.1. The same diagnosis can be made in service where there are either visible or audible indications of relative vertical movement between the slabs each side of a joint when a heavy vehicle passes (i.e. slab rocking, see Section 3.3.8).

The LTE across a well-constructed new joint that includes a load transfer device should be around 80 - 90 % initially, and normally more than 75 % in a well maintained condition during service. LTE significantly below 50 % should be regarded as indicating unsatisfactory load transfer requiring further investigation. For long-term performance, it is almost inevitable that maintenance will be needed.

Over the past few decades, there have been variations in the construction methods adopted in the UK to ensure that one half of a dowel bar slides freely. From 1969 to 1986, the bond-breaking compound applied to half of each dowel bar often comprised an ordinary bitumen emulsion, but subsequent laboratory tests demonstrated that the bitumen emulsion resulted in a bond between the dowel bar and the concrete, which could be slightly greater than that for a plain uncoated bar.

From 1991, bond breaking was achieved by covering the dowel bars with a thin plastic (no greater than 1.25 mm) sheath over 67 % of the length of the bar. This sheath permitted free sliding of the dowel bars and provided corrosion protection to the vulnerable central portion of the bars where they passed through the joint. The thickness of these plastic sheaths, and the fact that they extended into both bays adjoining each joint, was such that some loss of load transfer efficiency at the joint was to be expected.

From 1998, a flexible polymeric coating of minimum thickness 0.3 mm replaced the plastic sheath to provide better durability and improved load transfer. These possible variations in load transfer device installation should be considered during any diagnosis.

Corrosion of load transfer devices

Sodium, calcium, and magnesium chlorides can be introduced into the concrete from the application of de-icing salts during winter maintenance. These chlorides will (albeit slowly) penetrate into the pavement surface and will, in the presence of water, lead to corrosion of steel present within the pavement, including reinforcement, dowel bars and tie bars.

The expansive products of corrosion can cause seizure of what was previously a freely moving dowel bar, resulting in locked up joints. Further, if there is sufficient expansion, cracking can occur above steel, resulting in spalling and in the case of CRCP, punchouts. Corroded steel is also more susceptible to shear failures if the cross-sectional area of the bar is reduced by corrosion.

Therefore, measures are taken to prevent water and chlorides from penetrating into the pavement and minimise corrosion:

- dowel bars and tie bars are coated with corrosion resistant materials;
- pavement concrete with low permeability (low water / cement ratio) is used to slow the process of water and chloride penetration;
- steel reinforcement has a minimum cover requirement to reduce the risk of chlorides penetrating to the depth of the reinforcement;
- joint seals are used to prevent water and chlorides from penetrating through joints to load transfer devices;
- alternatives to steel for load transfer devices are emerging; and,
- alternatives to chlorides are being explored for the winter maintenance of pavements.

Corrosion will inevitably occur more rapidly and the life of a pavement designed on the basis of effective reinforcement, dowel bars and tie bars is likely to be shortened where:

- reinforcement is placed too high in the pavement;
- medium to wide cracks that occur are not sealed; or,
- poorly maintained joint seals allow the ingress of water and chlorides through the joint space.

4.3.2.2. Locked up joints

In URC and JRC pavements, joints are used to accommodate thermal movement. When joints become 'locked up' these horizontal movements are restrained. Joints can lock up for different reasons:

- construction issues resulting in misalignment of the dowel bars (see previous Section 4.3.2.1);
- construction and maintenance issues leading to unintended bonding of the dowel bars to the concrete (see previous Section 4.3.2.1);
- construction issues leading to improper cleaning of the joint reservoir prior sealing; or late saw cutting of the joint after the concrete has been poured (recommended between 12 and 24 hours after casting); or saw cuts being not deep enough to form the joint [10]; and,
- maintenance issues leading to the ingress of solid particles into the joint space through defective joint seals.

Locked up joints can result in microcracking when slabs come together as they expand, which can develop into spalling with repeated traffic loading and compressive stresses at the joint. In severe circumstances, when excessively high compressive stresses develop during hot weather, the presence of transverse and longitudinal cracks can result in compression 'blow-up' failures (Section 3.3.9).

Generally, if there is no measurable horizontal movement at more than three consecutive joints in a URC pavement, remedial action is required. Failure to respond will lead to cracks forming roughly parallel to the transverse joints.

Historically, demountable mechanical (demec) strain gauges have been used to diagnose where movement is not occurring at joints [46]. With comparative measurements taken when the slab temperature is high and when the slab temperature is low. However, it is recognised that this physical type of measurement is unlikely to be practicable in most circumstances on the SRN outside of planned closures. Alternative remote or embedded sensing technologies are available and some of the automated survey techniques discussed in Chapter 2 can also record joint widths.

4.3.2.3. Joint sealant

As discussed in Section 3.1.5, sealants in longitudinal joints can have a significant lifespan; however, the service performance is strongly influenced by the method used to install the joints during construction. The two common methods are:

- method A: a continuous strip inserted through a hollow vibrating plough in machine-laid concrete; and,
- method B: glued to the vertical face of the previously laid slab before the second pass of the paving equipment, where the carriageway is too wide to be laid in one pass of the paving machine.

Longitudinal joint sealant correctly installed by method A will often perform satisfactorily throughout the life of the pavement. It acts as a crack inducer, either alone or in conjunction with some form of additional crack inducer at the base of the slab. The total width of practically all carriageways is such that longitudinal joints act as warping joints and opening and closing of the joints is minimal.

The steel tie bars are well bonded to the concrete and aggregate interlock is created by the induced longitudinal fracture of the slab. The result is that the joint usually remains sealed and the tie bars therefore suffer little corrosion. Because the tie bars retain their effectiveness, aggregate interlock is maintained along with the load transfer efficiency of the longitudinal joint.

The long-term performance of longitudinal joints constructed by method B is often less satisfactory and formed longitudinal joints tend to require more maintenance than induced longitudinal joints. Unless more than one set of paving machines were in use (and this is unusual), the second strip (or 'rip') of paving is likely to have been placed some weeks after the first. Tie bars were probably bent through a right-angle at the joint so that they could be installed within fixed forms used for placing the first 'rip'. The cranked ends of the bars would later have been broken out of the hardened concrete and re-bent to form tie bars that were only approximately straight. Although tie bars are usually inserted directly into the edges of slipformed 'rips', they are also often cranked to avoid obstructing operations on the unpaved adjacent strip and subsequently re-bent. In contrast to the joints formed by method A, these joints would have the following characteristics:

- The kink in the tie bars caused by bending and re-bending permits some initial opening of the joint under tension as the kink straightens.
- Some early tension across the joint is almost inevitable as the second 'rip' concrete contracts or cools from its hydration temperature.
- There is little aggregate interlock because the slab placed first has a vertical formed face; therefore, load transfer across the joint is provided solely by the shearing resistance of the tie bars and the stiffness and continuity of the subbase.
- Any applied corrosion protection is likely to be damaged, caused by the subsequent bending.

4.3.3. Other issues

Chapter 3 and the previous sections in this chapter have illustrated the main causes of concrete pavement defects through construction, maintenance and use. Other issues that occur less frequently to cause defects include:

- poor construction or treatment practice;
- improper mixture constituents;
- improper repair materials;
- foreign matter within the concrete;
- shallow cover to reinforcement;
- misalignment between the top and bottom crack inducers;
- vehicle fires and spillages; and,
- alkali-silica reaction.

Poor construction or treatment practice: inadequate compaction, curing, improper finishing or overworking of the concrete can create a weak concrete surface more prone to surface defects. Specifically, overworking can occur if the concrete continues to be worked when bleed water is present or additional water is added (or present due to weather conditions) during finishing, as entrained air can be displaced and the increased water / cement ratio at the surface results in a weak layer.

Improper mixture constituents: concrete with excessive water / cement ratio due to poor batching practices, further to potentially reducing the strength of the concrete, will increase the amount of bleed water and produce a weakened surface layer with a greater susceptibility to freeze-thaw, scaling and crazing.

Improper repair materials: patching and bay replacements using asphalt materials, whilst an effective temporary solution for emergency repairs, inevitably results in increased deterioration of the surrounding pavement if not replaced with a suitable repair as outlined in Chapter 5. When used as a partial depth repair, the reduced material stiffness of the asphalt material increases the stress imparted on the surrounding pavement. When used as a full depth repair, there will be no load transfer the discontinuity, causing significantly increased stress on the surrounding pavement.

Foreign matter within the concrete: typically identified as clay, mud or other friable materials. However, within older pavements, materials such as timber may be present within the slab having been used to position formwork.

Shallow cover to reinforcement: a primary causation of early life longitudinal cracks; it increases the likelihood of corrosion during service and if placed at variable depths can be subject to shear failure and punchouts. Where reinforcement is fixed prior to concrete installation, it should be fixed on chairs at the required depth and not supported on bricks or concrete spots. Steel reinforcement positions and spacings and details of minimum cover to reinforcement are specified in Series 1000 of the MCHW [11].

Misalignment between the top and bottom crack inducers: can result in transverse cracking appearing close to transverse joints. It is important to check whether the original construction procedure included the use of bottom crack inducers; if not, then this cause can be eliminated. If it is not practicable to inspect the slab edge, an exploratory core hole would need to be drilled.

Vehicle fires and spillages: although concrete surfacing is reasonably resistant to fire and chemical spillages, such incidents can contribute to weaken the concrete surface and eventually promote scaling.

Alkali-silica reaction: can produce severe surface performance defects in a concrete slab. ASR is a chemical reaction in the concrete between reactive silica in the aggregate and the alkalis present in the hydrated cementitious paste. The result of ASR is the formation of a gel that expands in the presence of moisture and cracks the concrete matrix. ASR can result in crazing (Section 3.1.3) or pop-outs (see Section 3.1.4). It can also initiate deeper cracking and spalling. If ASR is suspected, by the visible presence of gel or staining at the surface of the concrete, then a detailed investigation should be planned, including material sampling and laboratory testing for ascertaining the presence of ASR in accordance with BS 812-123 [47].

5. Repair techniques

This chapter provides guidance on the range of proven repair techniques that are available to address the defects identified in Chapter 3; and ideally to address a specific causation diagnosis guided by Chapter 3 and Chapter 4 based on the targeted investigations discussed in Chapter 2. The objective is to restore serviceability and reduce the rate of overall deterioration of the concrete pavement consistent with the preferred maintenance strategy.

The applicability of the repair techniques outlined in this chapter to address specific surface and structural defects is discussed in Chapter 3 and Appendix B. The surface and structural repairs outlined in this chapter can be generally categorised as:

- **holding repairs** to reduce rate of deterioration of the pavement without necessarily rectifying the defect or addressing the underlying causation. Holding repairs typically have service lives of 3 to 7 years before additional repair works may be required, depending on various factors not limited to the repair type, pavement loading and pavement condition; and,
- **long-term repairs** to rectify defects and address the underlying causation and are expected to have service lives greater than 10 years.

As outlined in Chapter 3 and Appendix B, often, there are several treatment options available to address a particular defect. The options may have different lifespans. The decision as to which treatment and repair material is preferred is dependent on multiple factors, which include but are not limited to the:

- defect severity and location (i.e. within wheel track zones);
- maintenance strategy;
- serviceability requirements;
- LCCA;
- user disruption / network availability; and,
- risk management.

Furthermore, there are variables within each repair option that will affect the repair lifespan, including the:

- repair materials selected;
- quality of the construction; and,
- extent to which the defect root cause is addressed.

As discussed throughout this manual, it is important that the repair technique is appropriate to address the root cause of the defect so that an intended long-term repair does not fail prematurely. For example, when a surface repair is mistakenly identified as suitable to address a defect that has a structural causation. This often means significant work is required at the investigation and design stage. Further common examples of inappropriate repairs are included within this chapter for guidance.

The surface repairs and structural repairs outlined in this chapter are intended to be used over localised areas to arrest deterioration and improve the pavement condition. However, if applied over larger sections, they may become uneconomical. Furthermore, they will not significantly increase the capacity of the pavement to carry traffic beyond its original design life. Therefore, where repairs are impractical or uneconomical due to the frequency of defects or the high deterioration rate of the pavement, restoration (see Chapter 7) should be considered in order to extend service life of the pavement.

Before providing guidance on surface repairs (Section 5.3) and structural repairs (Section 5.4), this chapter discusses emergency repairs (see Section 5.1), and routine joint maintenance works (Section 5.2), to remove and replace joint seals.

5.1. Emergency repairs

Sometimes, and often due to a sudden deterioration of the pavement serviceability, localised defects may arise that pose an immediate or imminent hazard. Temporary emergency repairs are used to restore a safe running surface to allow continued trafficking of the pavement with minimal user disruption and delay whilst long-term cementitious repairs are arranged.

These temporary emergency repairs have historically consisted of removing a section of the pavement and replacing it with bituminous bound material in order to quickly restore safety and serviceability of the pavement. Bituminous bound materials were typically used because historically concrete has needed an extended curing period prior to opening to traffic, but with the developments in early strength concretes, rapid rigid reinstatements are now available.

Bituminous material is not an effective long-term solution for repairs to concrete pavements. Bituminous bound materials will always be inadequate for the traffic loading compared with the equivalent thickness of concrete as the repair can only be laid to the same thickness of the existing concrete slab. Furthermore, the surrounding pavement will have accelerated deterioration when bituminous bound materials are used. The lower stiffness will impart additional stress on the surrounding concrete and where full depth repairs are necessary, the lack of load transfer between the repair and the adjacent pavement will increase the stress on the adjacent pavement.

A long-term repair should be undertaken as soon as possible following the appropriate materials and procedures outlined in this Chapter to limit the potential for further degradation of the surrounding concrete. Surface repair techniques to defects confined to the upper third of the pavement and structural repair techniques to replace the full depth of the pavement are outlined Sections 5.3 and 5.4 respectively.

A range of polymeric materials, specifically developed for use on concrete pavements, resin mortars, and proprietary cement mortars can be used for rapid long-term surface repairs. A range of early strength concretes are now available for structural repairs which can reach trafficking strength in as little as 3 hours post installation. Provided the guidance outlined later in this Chapter is followed for undertaking the repairs, including installation of load transfer devices, then a durable, and effective ‘right first time’ repair can be delivered with minimal disruption and without the need for a subsequent later closure to affect a long-term repair.

5.2. Joint maintenance

Joints are designed and constructed to facilitate movement and prevent excessive stresses occurring in the concrete. These joints are sealed to prevent foreign materials entering the void space and water from reaching the lower pavement layers.

However, these joint seals have finite lifespans and require replacement within the life cycle of the pavement. Where the joint is not functioning properly or the joint face is damaged, the joint may have to be widened and the seal replaced.

5.2.1. Replacing joint seals

Transverse joint seals are not expected to last for the duration of the designed structural life of a new concrete pavement, which is typically 40 years. As outlined in Section 3.1.5, the vast majority fail because they tend to harden and become brittle over time. This embrittlement reduces the ability of the sealant to accommodate the slab movements and ultimately results in an adhesion or cohesion failure of the joint seal. Longitudinal joint seals are generally subject to less movement and may remain serviceable for many more years than their counterparts.

It is essential to the continued serviceability of the pavement that joint seals are replaced regularly, ideally as part of a routine maintenance operation in advance of them becoming defective (see Section 3.1.5), to prevent moisture ingress into the lower pavement layers and prevent debris entering and lodging in the joint cavity and restraining the slab movements. The traffic management arrangements required for carrying out other repairs are such that it is usually possible to replace joint seals at the same time.

5.2.1.1. Joint sealant materials

Different classifications of materials are available for joint sealing. The classification and type of joint sealant used will be dependent on:

- joint groove dimensions;
- whether fuel or chemical resistance is required;
- compatibility with surrounding materials;
- closure windows (trafficability);
- weather conditions;
- durability requirements; and,
- initial and life cycle cost.

Table 5.1 gives a guide to the different types of joint sealant material. The service life expectancy of sealants will depend primarily on the joint dimensions, climatic conditions and type of sealant used. A general life expectancy is given for a quality controlled installation of each of the three material classifications at a transverse joint. Preformed sealants are expected to last the longest and some have reported design lives of over 15 years. However, they are less readily available, can have increased cost due to manufacturing and their preformed nature means they are specific to joint grooves with certain dimensions, meaning they may not be compatible with all joint grooves across a maintenance scheme.

Table 5.1 Types of joint sealant material

Classification	Type for use on highways	Service life expectancy	Performance requirements	Conformity requirements
Hot applied	N1, F1 or F2	6 - 10 years	BS EN 14188-1	BS EN 13880 Parts 1 - 13
Cold applied	Class A, B, C or D self levelling (sl) or non sag (ns)	7 - 12 years	BS EN 14188-2	BS EN 14187 Parts 1 - 9
Preformed	N/A	> 10 years	BS EN 14188-3	EN 14840

The sealant used needs to be appropriate for the joint groove dimensions to prevent premature failure. Fuel-resistant sealants should be specified where fuel spillage and leakage are expected, such as laybys and hardstandings. Furthermore, it is important that the sealant used is compatible with both the surface to be adhered to and any adjacent joint sealant. The specific characteristics and suitability of the three classifications of joint sealant material are discussed below.

Hot applied sealants

Hot applied sealants are thermoplastic or thermosetting materials that are heated to the recommended pouring temperature prior to placement. They require accurate temperature control to achieve the desired properties and can become brittle if overheated. BS EN 14188-1 covers the requirements for hot applied sealants.

There are four hot applied sealant types:

- N1 - Elastic - high extension
- N2 - Normal - low extension
- F1 - High extension fuel-resistant
- F2 - Low extension fuel-resistant

On the SRN, Type N1, Type F1 and Type F2 are suitable for resealing joints in concrete pavements. Type N2 has lower elasticity than Type N1, so is generally not used for movement joints.

Type F1 or Type F2 should be used where additional fuel resistance is required, such as a layby or hardstanding. Type N1 should be used for joints between concrete and bituminous surfacing.

Cold applied sealants

Cold applied sealants are either single component materials or two-component materials, containing a base component and a curing component which are mixed at ambient temperature prior to placement. BS EN 14188-2 covers requirements for cold applied sealants. There are four classes according to chemical resistance:

- Class A - no requirements of chemical resistance
- Class B - used in contact with jet fuel and de-icing chemicals
- Class C - used in contact with petrol, diesel and de-icing chemicals
- Class D - liquid chemicals as required

For general highway applications, Class A is considered suitable. Class C or Class D should be used for lay-bys and hardstandings where additional fuel resistance is required.

Cold applied sealants are split into two types according to their rheology:

- self levelling type (sl); and,
- non sag type (ns).

Self levelling cold applied sealants are flowable to allow them to be poured into the joint groove. No additional working is needed to ensure adhesion to joint faces, but additional measures to limit flow may be necessary when there is a significant fall in the pavement (> 2.5 %).

Non sag cold applied sealants are gun applied and require more careful application and working to ensure adhesion to joint faces and that the sealant does not extrude above the top surface of the concrete pavement. Gun-grade cold applied sealant materials are generally the most appropriate choice where only small quantities of resealing is being undertaken.

Preformed (compression) seals

Preformed (compression) seals can be pre-compressed neoprene impregnated expanding foam sealing strips, with product acceptance scheme certification, or preformed and vulcanised elastic rubber compression seals complying with BS EN 14188-3.

Preformed seals are installed in a compressed state and need to remain compressed throughout their service life to be effective. The width of the preformed seal used is selected in relation to the width of the sealant groove, the bay lengths and manufacturer’s recommendations, see Figure 5.2.

A lubricant-adhesive to adhere the seal to the joint faces is normally used that is compatible with the preformed seal and concrete materials and resistant to abrasion, oxidization, fuels and salts.

Preformed seals are installed in a single operation by a bespoke machine that pre-compresses the sealant, coats the sealant wall with the lubricant-adhesive, and inserts the sealant into the joint.

5.2.1.2. Backing materials

Backing materials are typically joint fillers, i.e. space filling elements; otherwise they are debonding tapes that comprise thin films or strips of material applied to the base of the joint sealant groove or the top of the joint filler.

Backing materials are used to:

- achieve the required sealant dimensions;
- keep the sealant from flowing down and out of the sealant groove reservoir; and,
- prevent bonding to the base of the joint sealant groove, which may otherwise cause local stress concentrations as the joint opens and closes and adhesion or cohesion failure.

Preformed (compression) joint seals do not require a backing material.

Joint fillers generally comprise a ‘backer rod’ (Figure 5.1) or other compressible caulking material. Backer rods are made up of closed-cell polyethylene foam having a diameter greater than the joint width. Open-cell backer rods should not be used as they can retain moisture.

Joint grooves are often formed deeper than the required sealant depth, so a joint filler is needed towards the bottom of the joint groove to form a depth / width that is consistent with the sealant manufacturer’s stated tolerances. Some types of joint filler can act as a debonding layer, otherwise a bond breaker tape should also be used at the base.

Backer rods are typically installed with a backer rod inserter as shown in Figure 5.1, which has an insertion wheel that can be height adjusted to ensure the backer rod is installed at the correct depth.

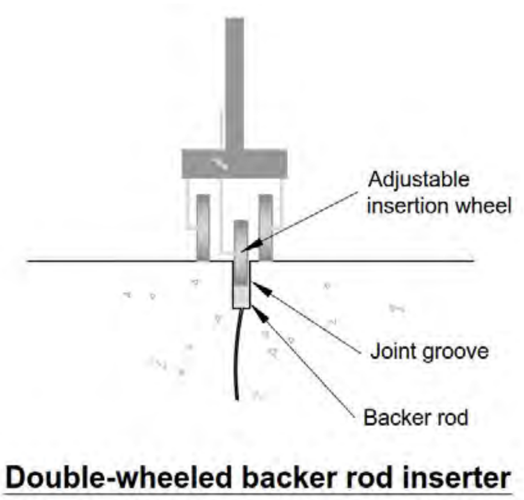


Figure 5.1 Backer rod [48] (left) and backer rod inserter (right)

5.2.1.3. Installation of joint seals

BS 10948 provides the code of practice for the application and use of hot applied and cold applied joint sealant systems for concrete pavements. The installation of preformed (compression) seals should be in accordance with BS EN 14188-3 Annex C and the manufacturer's recommendations. Site testing requirements can be found in BS 10948. The important considerations for the installation of joint sealants are outlined in below.

Where possible, joint sealing should be undertaken in spring or autumn to avoid the potential excessive compression (if applied in winter) or extension (if applied in summer) of the sealant. In hot weather, sealants that have been applied at a cooler time of the year may be extruded from the joint groove due to compression of the joint space. If this happens, they may be damaged by traffic and can be lost altogether.

Joint groove dimensions

The groove dimensions need to be appropriate both for the expected amount of movement at the joints and the type of sealing material used. To allow the sealant to function correctly, the joint sealant reservoir needs to match the requirements of the joint sealant being used in terms of:

- depth;
- width; and,
- depth / width ratio.

The stresses induced in the sealant by thermal and other movements of the slabs can become excessive if the minimum and maximum depths and widths are exceeded or if the depth / width ratio is incorrect, resulting in adhesion or cohesion failure.

The minimum and maximum depth and width and depth / width ratio of joint seals is product specific. Typically, a depth / width ratio of between 0.8:1 and 1:1.5 is required with a minimum depth of 15 mm for cold applied sealants and 20 mm for hot applied sealants.

The sealant groove needs to be at least 10 mm in width to allow for groove preparation, inspection and sealant application. The minimum dimensions for applied joint sealants is dependent on the type of joint and the sealant used. More detail can be found in Clause 1016 [11]. For new construction, the maximum width of joint grooves is limited to 30 mm.

See Section 5.2.1.2 for controlling the sealant depth. The depth and width of the sealant groove can be increased by diamond sawing, but should only be done where this is essential. For example, where an existing joint groove requires widening during maintenance to remove any damage to the joint face; and this should be kept to a minimum to reduce the risk of debris lodging in the joint cavity, see next Section 5.2.2.

Where joint grooves are beyond the maximum width for sealants, the joint may need to be reformed by undertaking a thin bonded repair (Section 5.3.2).

The uncompressed width of preformed (compression) seals and the initial width of the sealant grooves are related to the distance between joints and should to be in accordance with the manufacturer's recommendations to ensure that sealants remain permanently in compression in the groove, see Figure 5.2.

Sealant groove preparation

When existing joints are being resealed, the joints must be properly prepared to ensure performance.

Compression seals can often be pulled out manually. Other types of sealant may be removed by mechanical or hydraulic means, such as using a diamond-bladed saw or a metal plough attached to construction equipment. Any damage to the joint face during this process should be avoided.

Residues from certain sealant types may cause adhesion failure or other compatibility issues such as plasticiser migration, particularly for cold applied materials. Therefore, all traces of the old sealant should be removed.

Debris should be removed by water-jetting or other means from the joint groove as soon as the sawing is completed to remove slurry remaining. The existing joint groove surface should then be roughened using dry abrasive blasting that will remove laitance and provide a key. Any debris should then be removed and the joint groove cleaned using oil-free compressed air prior to priming (where this is recommended by the manufacturer) in advance of receiving the hot or cold applied sealant or a preformed (compression) seal.

Caution is required with spray-applied primers at high ambient temperatures as they can vaporise before adhering to the concrete. Inadequate primer adhesion to the joint faces is a significant cause of premature failure of joint seals.

For newly constructed concrete slabs, the concrete needs to have gained sufficient strength for sawing and dry abrasive blasting before joint sealing can be undertaken without damaging the joint grooves.

Sealant application

It is critical that the ambient conditions are suitable for application of joint sealants. Hot applied sealants can fail to adhere to the sides of the sealant groove if they applied at lower ambient temperatures due to rapid cooling / hardening.

The temperature at which cold applied sealants can be installed is limited by the time it takes to become tack-free. At lower temperatures, cold applied sealants cure less quickly, meaning they remain tacky for longer and can be damaged by any inclement weather. Preformed seals do not need to cure, but the adhesive used to bond them to the joint face will have minimum temperature requirements.

Cold applied sealants need to be protected during their curing period to prevent damage by any inclement weather. The curing time for cold applied sealant can vary according to ambient temperature. In colder temperatures, automatic mixer / dispenser machines may be used for two-component cold applied sealants to facilitate a shorter curing time. Table 5.2 contains the typical application conditions and trafficability for the three classifications of joint sealant materials.

Table 5.2 Typical application conditions and trafficability for joint sealant materials

Sealant classification	Typical ambient / surface installation conditions	Trafficability
Hot applied	>8 °C and rising Frost and dew free	Upon sealant cooling
Cold applied	>8 °C and rising Frost and dew free	When sealant is tack free
Preformed	>4 °C (dictated by adhesive not seal)	Immediately

The manufacturer's recommendations should be followed for installation conditions as well as any requirements in Clause 1016. Local heating of the concrete to meet installation conditions is not recommended because the effect will be temporary and the concrete will quickly cool, causing moisture to condense on the surface.

Nevertheless, applying hot air through a lance to remove any moisture from within the sealant groove can be undertaken before the application of the primer and joint sealant if the temperature conditions are acceptable.

Where the road crossfall is steeper than 2.5 %, additional measures should be taken for using self levelling (Section 5.2.1.1) cold applied sealants, such as one or more of the following:

- placing in thin layers and allowing each layer to stiffen before placing the next one;
- forming dams at intervals along the joints to restrict the flow; or,
- using automatic mixer / dispenser machines and a cold applied sealant with a shorter curing time.

Alternatively, non sag sealant or compression seals may be considered.

Recess depth below surface

Joint sealants need to be recessed within the joint groove to prevent extrusion during hot weather and subsequent damage from traffic. For transverse joints this recess should be at least 5 mm below the slab surface for normal vehicle traffic. For joints wider than 25 mm, the recess should be 7 mm. If seals are applied in cold weather, this value should be increased to 10 mm. The tolerance of the recess depth is ± 2 mm. If the pavement surface is textured, the recess depth should be measured from the lowest point of the texture. These dimensions are shown in Figure 5.2.

Figure 5.3 shows self levelling joint sealant being installed using a specialist pressure applicator with the nozzle within the joint groove to avoid over filling and spillage of the joint sealant. This is the preferred method for achieving a uniform recess when installing self levelling joint sealants.

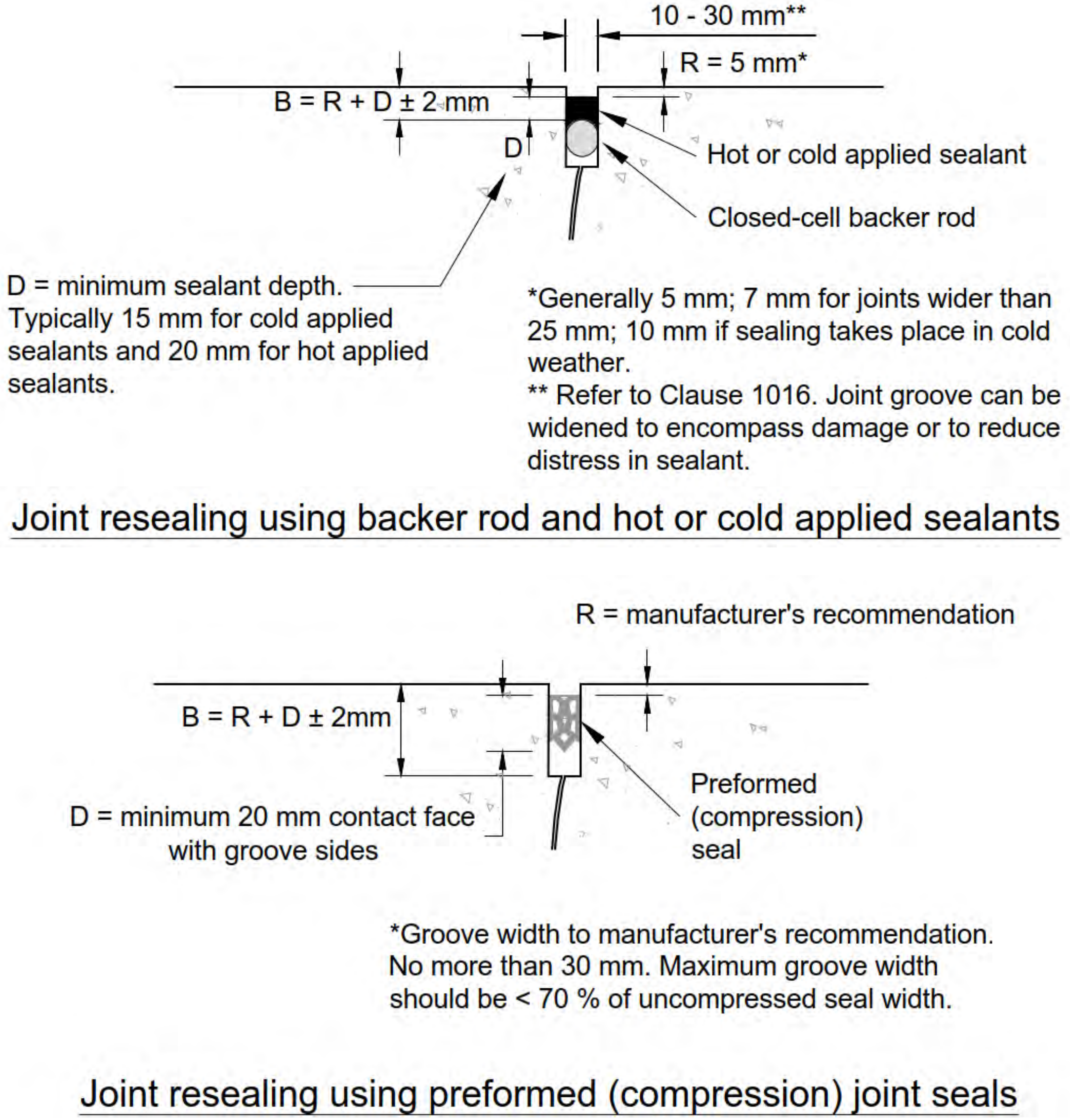


Figure 5.2 Joint sealant dimensions

Figure 5.3 Self levelling joint sealant being applied through a nozzle to achieve a uniform recess below the pavement surface

5.2.2. Widening joint sealant grooves

Joint groove widening may be undertaken for several reasons, including:

- where shallow joint spalls are confined to the depth of the sealant groove;
- where the joint face is damaged; and,
- where an existing joint sealant appears to have failed through overstressing.

Sealant groove widening is undertaken by diamond blade sawing, which is a common method for removing existing joint seals. The saw cut should be perpendicular to the pavement and the entire length of transverse joints should be treated.

The maximum width of the widened joint sealant groove, including any chamfers, should be 40 mm for transverse joints and 25 mm for longitudinal joints. Where the sealant groove would exceed this width, a thin bonded repair (see Section 5.3.2) can be undertaken. However, some joint sealants can be used in joints greater than 40 mm in width.

Consideration needs to be given to the proposed joint sealant material being used following the joint sealant widening, as it may be necessary to increase the depth of the joint to accommodate any depth / width ratio requirements outlined by the joint sealant manufacturer.

5.3. Surface repairs

Surface defects (see Section 3.1) typically do not significantly affect the structural integrity of the pavement, but they can have an impact on its aesthetic appearance and accelerate pavement deterioration by permitting ingress of water, de-icing salts and incompressible materials into the pavement. Repair techniques for these minor defects or imperfections include:

- single particle pop-out repairs;
- thin bonded repairs and shallow repairs; and,
- inlaid crack repairs.

5.3.1. Single particle pop-out repairs

Pop-outs are cavities in the pavement caused by an isolated loss of surface material. They typically range from 25 mm to 100 mm in diameter and from 10 mm to 50 mm in depth. Where small cavities are present, often as a result of aggregate fracturing, then where necessary they can be repaired by plugging the area with a resin mortar described in Section 6.4. Irregular or larger cavities may require a thin bonded or shallow repair, see next Section 5.3.2.

Where the sides of the cavity left by the pop-out is smooth, the area may need to be roughened, this is normally undertaken with a wire brush. The cavity should then be thoroughly cleaned by brushing and blowing out any loose particles with compressed air. The application of the resin mortar should follow the manufacturer's instructions, including any requirements for a primer and minimum and maximum thicknesses.

5.3.2. Thin bonded repairs and shallow repairs

Thin bonded repairs (up to 40 mm thick) and shallow repairs (over 40 mm thick up to one third slab thickness) are repairs that involve the removal and replacement of a partial depth of deteriorated concrete with a suitable repair material. They are commonly used at joints to repair shallow spalls (Section 3.1.6) and can be used for areas of deterioration away from joints. At joints they are classified as thin bonded repairs for repairs undertaken up to the depth of the joint sealant groove. Repairs greater than the depth of the sealant groove are classified as shallow repairs.

Thin bonded repairs and shallow repairs can be more cost effective alternatives to full depth repairs where:

- the slab deterioration is confined to the upper third of the slab,
- any exposed reinforcement is corroded; and
- the load transfer devices are functional.

In addition to thickness limitations, thin bonded repairs and shallow repairs differ in that:

- different repair materials are used; and,
- thin bonded repairs (up to 40 mm thick) have been used successfully for many years, whereas shallow repairs (over 40 mm thick) have proven to be more susceptible to failure; i.e. increased susceptibility to failure appears to be linked to the repair thickness.

Notwithstanding this, a study produced by the USA highway authorities showed that, for appropriately installed shallow repairs with effective quality control, 80 - 100 % performed well after 3 - 10 years and less than 2 % failed within 18 months [19].

Both repair types need to fully bond to, and become monolithic with, the existing adjacent concrete. Lack of bond development or loss of bond between the repair and the adjacent concrete in service for various reasons can result in the repair breaking up under loading.

5.3.2.1. Thin bonded repair materials

Thin bonded repairs are surface repairs undertaken up to 40 mm thick using a cement or resin mortar. They are considered to be long-term repairs.

They are generally used to repair spalled or damaged joints (Section 3.1.6); and they can also be used to rectify surface defects away from joints, such as scaling, crazing or pop-outs (Section 3.1), provided the repair depth does not exceed 40 mm.

Thin bonded repairs are undertaken with:

- proprietary cement mortars (see Section 6.3); and,
- resin mortars (see Section 6.4).

Where possible, repairs should be carried out using cementitious mortars. The practicable minimum depth is about 10 mm. Resin mortars, which have equivalent bond strength and compressive strength properties to the existing concrete, can be used; however, they may possess different thermal properties, meaning that further debonding or cracking of the existing concrete or repair can follow. Some resin mortars can be feathered to as little as 5 mm thick.

5.3.2.2. Shallow repair materials

Shallow repairs can be used to repair spalling, crazing, pop-outs and other surface defects that exceed the maximum depth for which thin bonded repairs may be used (i.e. greater than the depth of the sealant groove at joints, or greater than 40 mm away from the joint), up to a maximum of one third slab thickness. However, full depth repairs will be required instead of shallow repairs when:

- deterioration is deeper than one third slab thickness;
- reinforcement is exposed and corroded; or,
- load transfer devices are the cause of the defect (corrosion, lock up, misalignment).

Shallow repairs can be undertaken with various materials:

- rigid repair materials
  - pavement concrete (see Section 6.1);
  - early strength concrete (see Section 6.2);
  - proprietary cement mortars (see Section 6.3);
  - resin mortars (see Section 6.4); and,
- flexible polymeric materials (see Section 6.5).

The optimum material is dependent on the repair depth, curing time permitted before opening, placement conditions and required properties including bond strength, drying shrinkage, expansion and any requirement for flexibility. Polymeric materials are not long-term repair solutions due to their low strength; quoted service lives on product acceptance scheme certificates can be 5 years, and their service life will be reduced when used within the wheel track zones. However, they may be able to absorb some movement without failing, so may perform well in the short-term. The properties of these materials are further outlined in Chapter 6 to aid material selection.

5.3.2.3. Installation procedure for thin bonded repairs and shallow repairs

Figure 5.4 provides a schematic of the permitted repair dimensions of thin bonded repairs and shallow repairs.

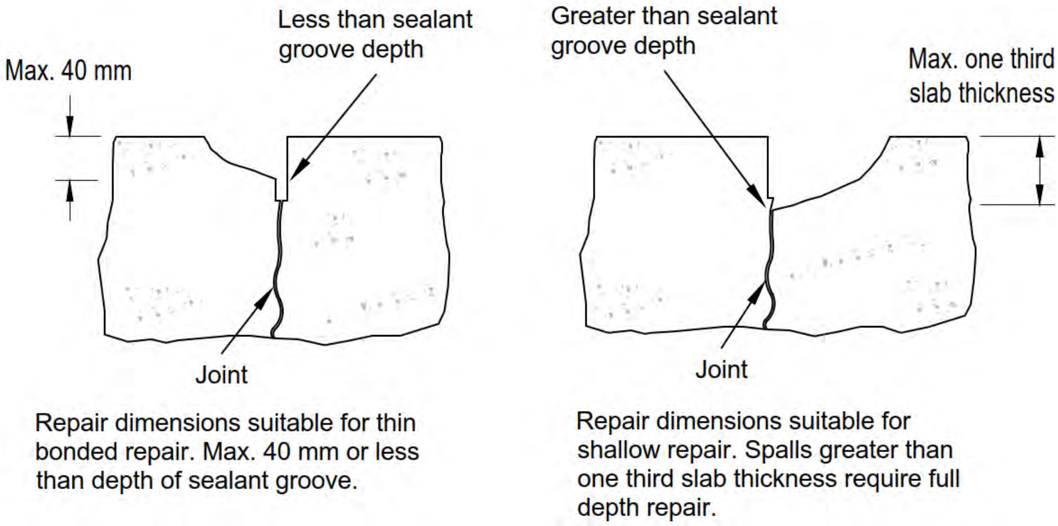


Figure 5.4 Dimensions for thin bonded repairs (left) and shallow repairs (right)

Thorough preparation and good workmanship are essential to the performance of thin bonded repairs and shallow repairs. The general thin bonded repair and shallow repair process is outlined in the steps below.

Figure 5.5 provides an illustration of the process for single and double joint repairs. This process is applicable for repairs away from joints, the difference being that joints do not need to be re-established, so a compressible joint insert is not needed.

Whilst repairs can be identified and the deteriorated concrete removed in adverse weather conditions, the preparation of the repair area and placement for the repair material needs to be undertaken in weather conditions acceptable for the repair materials used. The repair area will need to be recleaned if there is a delay between preparing the repair area and placing the repair material.

Weather condition requirements for the placement of cementitious materials are outlined in Clause 1045 [7]. For proprietary products, the manufacturer's instructions should be followed. Additional information on appropriate weather conditions for the installation of repair materials is outlined in Chapter 6.

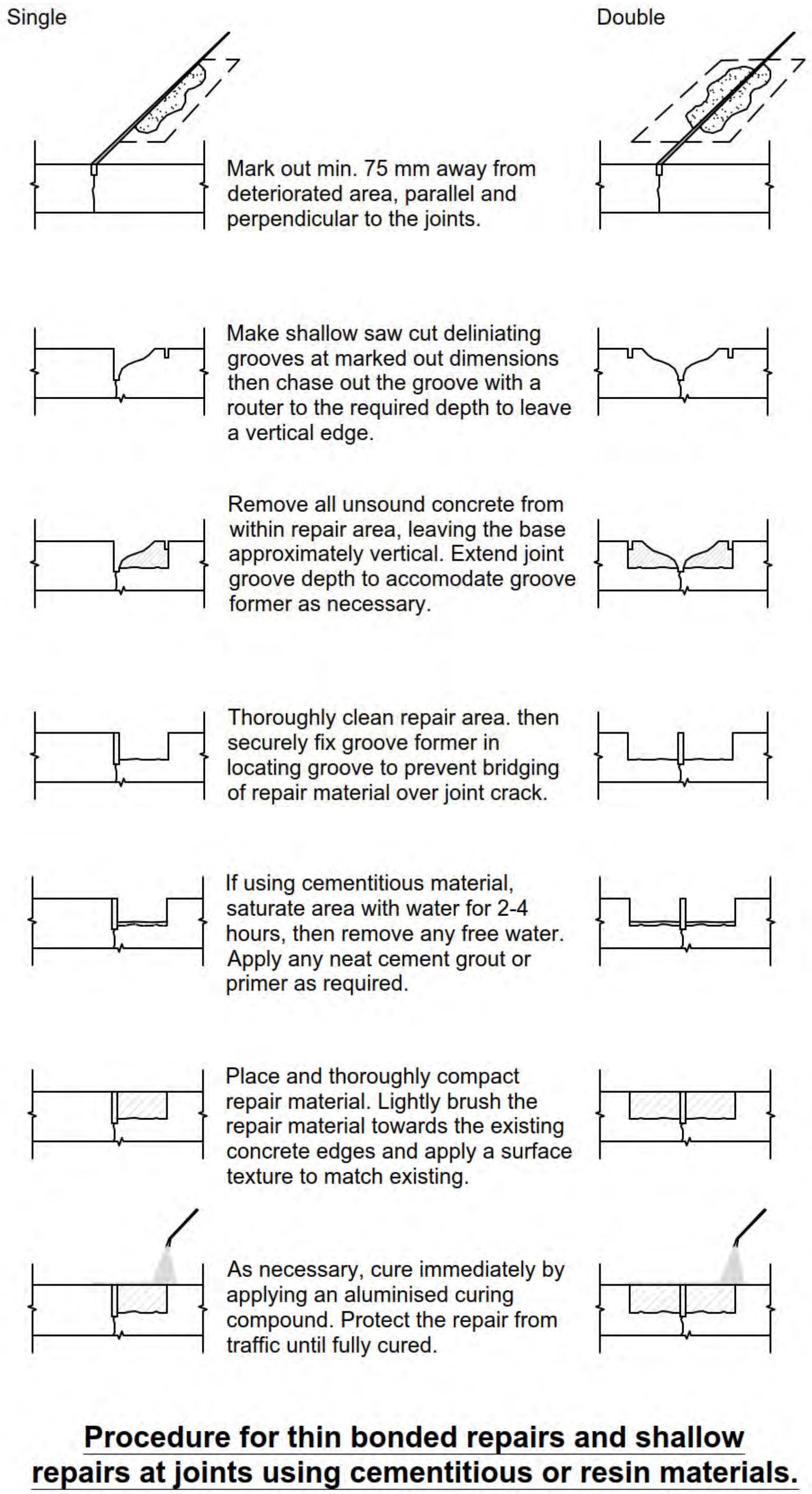


Figure 5.5 Thin bonded repair and shallow repair process

Step 1. Identify repair boundaries

Whilst the defective area may be clear, it is important that the full extent of deteriorated concrete is identified around the defective area. This is generally best accomplished by sounding the concrete (Section 2.2.1.6). Areas of concrete emitting a dull or hollow sound when tapped with steel rod, hammer or when a chain is dragged along the surface should be identified for removal.

Generally, the repair boundaries should extend at least 75 mm beyond where any visibly or audibly deteriorated areas are identified and be clearly marked. The repair area should be square or rectangular and follow the line of any existing joints to avoid cracking of the repair (see Figure 5.9 and Figure 5.10). Rectangular repairs should use repair materials with good drying shrinkage properties unless the aspect ratio of the repair is less than 1.3:1 to prevent the occurrence of non-structural cracking during the initial curing period.

It may be appropriate to combine repairs where they are within 500 mm of each other. Circular repairs may also be suitable. At joints where the repair depth is not known, this might be quickly identified visually following removal of the joint sealant and groove former. Where the repair depth extends beyond the joint sealant groove, a shallow repair should be undertaken.

Where the depth of deterioration is not known, the concrete can be removed to a nominal target depth to suit the proposed repair treatment and sounding undertaken again to identify whether the deterioration extends deeper.

Step 2. Remove deteriorated concrete

Once the repair boundaries have been identified, the deteriorated concrete needs to be removed.

For small repairs, saw cutting is often undertaken, followed by breaking out the interior of the concrete with a pneumatic drill. This removal method is likely to be required where steel reinforcement is present within the pavement. However, as saw cutting produces a polished surface / edge that inhibits good bonding, this is not the preferred method. Additionally, sawn grooves penetrate into the slab beyond the limits of the repair creating butterfly corners, which can be problematic.

Ideally a groove should be chased out using a milling machine to provide a roughened vertical edge around the repair, against which the repair material can be properly bonded (Figure 5.5). A shallow delineating groove may be sawn to start with, which is subsequently chased out to the full depth. The sides of the repair should be approximately vertical.

Where existing joints are being replaced as part of the repair, the joint groove needs to be made 20 - 30 mm deeper than the repair depth so that a joint groove former can be securely placed along the line of the existing joint (see Figure 5.6).

After breaking out the deteriorated concrete, the exposed concrete should be sounded again to check that all areas of deteriorated concrete have been removed. Where deteriorated concrete is found to extend further than one third slab thickness or where corroded reinforcement is exposed, a bay replacement (see Section 5.4.1) or full depth repair (see Sections 5.4.2 to 5.4.4) should be undertaken.

Step 3. Prepare repair area and any joints

Once the deteriorated concrete has been fully removed, the next step is dependent on the condition of the sides of the repair. Repair sides need to be rough to promote bonding. Therefore, where repair sides have been saw cut and are smooth, dry abrasive blasting is necessary to roughen the sides of the repair.

The sides and bottom of the repair area should then be thoroughly cleaned. This is normally undertaken using a hot air lance with the hot compressed air blowing out any loose material and drying the repair area simultaneously. The clean surface needs to be inspected immediately prior to placement of materials, including primer or bonding agent. If dust is still present on the surface, it should be cleaned again.

Any existing joint enveloped within the repair area has to be prepared. A compressible joint groove former should be placed and securely fixed in-line with the existing joint (see Figure 5.6).

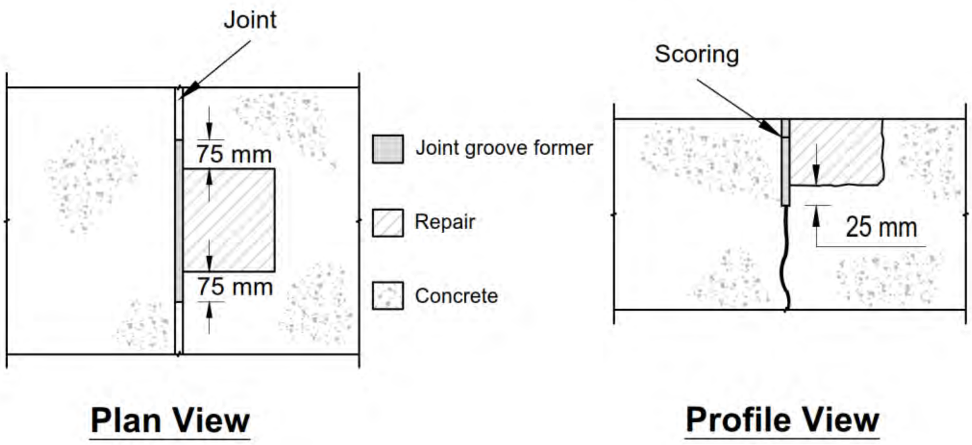


Figure 5.6 Placement dimensions of joint groove formers in joints for thin bonded repairs and shallow repairs

Joint groove formers prevent cracking or compression failures occurring between the repair and existing concrete before joint grooves can be sawn and stop the repair material from infiltrating the joint space, which could result in spalling or compression failures as slabs expand. The surface of the joint groove former should be treated before the repair material is placed to prevent adhesion.

Some flexible polymeric materials may be able to span joints, and therefore joint groove formers may not be required where these materials are being used.

Step 4. Apply primer or bonding agent

Immediately prior to the placement of the repair material, depending on the repair product used, a primer or bonding agent should be applied to promote bonding of the repair material to the surrounding concrete.

For cementitious materials, a cement grout can be used. However, where rapid curing is required to allow the repair to be opened to traffic, resins or resin grouts may be necessary. Where reinforcement has been exposed, resin primers are available which have corrosion inhibiting properties.

Where a cement grout is being used as a bonding agent, the surfaces of the repair should first be saturated with water, ideally for several hours prior so that they are in a saturated condition, then all free water should be removed so that the repair area is in a touch-dry condition immediately prior to the application of the cement or resin grout.

For proprietary materials, any primers or bonding agents used should be those recommended by the manufacturer.

Step 5. Place repair material

Repair materials should be placed immediately after any primer or bonding agent has been applied. Any pre-bagged repair products should be mixed at the recommended proportions and used without delay. Some early strength concretes, polymer modified mortars and resin mortars have short workability windows (less than 20 minutes), so it is important that placement is quick and methodical.

Material should be worked from the centre of the repair outward to prevent pulling material away from edges (Figure 5.7). Tamping will be necessary for all repairs, internal vibration may be necessary with deeper repairs to remove any trapped air from the mixture and to ensure bonding to the edges of the repair. The material should be screeded flush with the existing pavement and any extra repair material removed quickly from the adjacent concrete to prevent drying and bonding to the concrete.

Figure 5.7 Installation of a shallow repair

Step 6. Finishing

Whilst thin bonded repairs and shallow repairs are typically only undertaken over small areas, the repair surface still needs to be textured. For cementitious and resin-based materials this is achieved by brushing the surface prior to curing (see Figure 5.8). For polymeric materials, this involves the application of pre-coated or calcined bauxite chippings whilst the material is still hot and plastic to provide skidding resistance (Figure 5.8).

Figure 5.8 Brushing shallow repair (right) and application of bauxite to polymeric material (right) to provide texture and skidding resistance

Step 7. Curing

For cementitious repairs, curing will be essential to prevent rapid moisture loss and shrinkage cracking. This is generally undertaken with a white-pigmented aluminised curing compound shortly after finishing once any water has evaporated from the surface of the repair. Requirements for curing of cementitious materials are outlined in Clause 1027 [7]. Proprietary products should be cured by following the manufacturer's instructions.

Additional protection such as polythene sheets or insulation blankets may be required in particularly wet, hot or cold weather. Insulation blankets and tenting may accelerate curing (see Section 5.4.1 for more details).

Step 8. Saw-cut and seal any joints

The final step in the procedure for thin bonded repairs and shallow repairs undertaken at joints is to re-establish the joint. This may not be necessary where polymeric materials are used for shallow repairs (see Section 6.5).

When the repair material has gained sufficient strength to support plant, the compressive inserts are generally sawn out to the required joint dimensions and the joints resealed following the procedure outlined in Section 5.2.1.

Step 9. Opening to traffic

Repairs need to have cooled and / or attained sufficient strength prior to trafficking. Chapter 6 outlines typical curing periods required for repair materials before they can be opened to traffic. For repairs made with concrete, the in situ strength should be a minimum of 25 N/mm^2 before opening to traffic, as determined by cube compressive strength, Schmidt hammer or alternative (see Section 5.4.1 for more details).

5.3.2.4. Lessons learnt

As shallow repairs are thicker than thin bonded repairs, they are more prone to debonding failures caused by curling and warping due to temperature and moisture gradients through the repair thickness (see environmental loading in Section 4.1.2). The success of both repair types is highly dependent on the surface preparation, material selection and workmanship. Table 5.3 outlines some of the common causes of early failures of thin bonded repairs and shallow repairs.

Figure 5.9 and Figure 5.10 are examples of shallow repairs that have failed rapidly (within months) due to inappropriate use of a rigid repair material in large irregular shapes and / or spanning a working joint. Failure to re-establish joints and drying shrinkage cracking as a result of the irregular repair shape are often the cause of early failure, but fundamentally, defects of this size require bay replacement (see Section 5.4.1) or full depth repair (see Sections 5.4.3 and 5.4.4).

Where possible, shallow repairs should not exceed an aspect ratio of greater than 1.3:1, unless a repair material with good drying shrinkage characteristics is used. The associated increased risk of non-structural cracking due to increased early life drying shrinkage during curing is associated with increased water / cement ratio.

Non-structural cracking may evolve into structural cracking under trafficking and this is a common issue in repairs that are not square or circular as greater stress concentrations are generated. Good drying shrinkage characteristics can be achieved by reducing the water / cement ratio of cementitious mixtures using superplasticisers or other shrinkage reducing admixtures, or by using a an alternative cement-based concretes or non-cementitious repair material (see Chapter 6).

Figure 5.11 is an example of a shallow repair that was undertaken with a rigid repair material using a very high aspect ratio across a working crack. The continued movement at the crack combined with differential drying shrinkage due to the aspect ratio resulted in the repair failing rapidly. A shallow repair with a flexible polymeric material (see Section 6.5) may have offered an improved interim performance; however, bay replacement (see Section 5.4.1) or full depth repair (see Sections 5.4.3 and 5.4.4) would still have been necessary to provide a long-term repair to this particular defect.

Figure 5.9 Failure of a shallow repair undertaken with a rigid repair material using an irregular shape and spanning a joint without re-establishing the joint

Figure 5.10 Failure of an irregular shaped shallow repair spanning a joint without re-establishing the joint

Figure 5.11 Failure of an irregular shape, high aspect ratio shallow repair undertaken over a working crack

Where thin bonded and shallow repairs are being proposed, it is critical that the repair is appropriate, mindful of the valuable past experience summarised in Table 5.3; and it is essential that the repair is undertaken using the optimum material and following the good practice guidance provided in the remainder of this section.

Table 5.3 Common causes of failure of thin bonded repairs and shallow repairs.

No.	Causation	Commentary
1	Inappropriate use of the repair techniques.	Thin bonded repairs and shallow repairs should not be used where: • deterioration is deeper than one third slab thickness; • exposed reinforcement is corroded; • load transfer devices are the cause of the defect (corrosion, lock up, misalignment); or, • dynamic movement is occurring (except when polymeric materials are used).
2	Incompatibility of thermal expansion characteristics between the repair material and the original slab.	The drying shrinkage of repair materials is generally greater than normal concrete. Some concretes and mortars may have high shrinkage and high coefficient of thermal expansion, depending on factors including aggregate type and water / cement ratio. Resin mortars can have high initial shrinkage and a high coefficient of thermal expansion, which may result in debonding.

No.	Causation	Commentary
3	Improper use of repair materials.	See causation 1 & 2. Manufacturer's installation instructions should be followed for proprietary materials. Rigid repairs where dynamic movement is anticipated may fail quickly. Polymeric materials are not long-term repair solutions due to their low strength, quoted service lives can be 5 years, less when situated in wheel tracks. However, they are flexible, so they may be able to absorb some dynamic movement without failing.
4	Insufficient bond and non-structural cracking due to preparation and workmanship.	Repair boundaries need to be confirmed by sounding (Section 2.2.1.6). Repair areas need to be square, rectangular or circular to avoid differential early life shrinkage cracking (see Figure 5.9, Figure 5.10 and Figure 5.11). The aspect ratio of repairs using cementitious materials prone to early life shrinkage cracking should be limited to 1.3:1. Surfaces should be irregular and rough but clean to promote bonding. A bonding agent or primer needs to be applied for all repair materials, unless where recommended by the manufacturer. Material should be worked from the centre of the repair outward to prevent pulling material away from edges. Compaction using a vibratory poker will be necessary for deeper repairs with concrete to remove any unwanted entrapped air. Any curing compound, where required, needs to be applied immediately after finishing; and trafficking avoided until the repair has cured.
5	Cracking and compression failures due to failure to re-establish the joint See Figure 5.9 and Figure 5.10.	Compressible joint inserts need to be placed into existing joints and positioned to prevent repair material flowing into the joint space. Joint inserts should then be sawn out and the joint sealant groove established and sealed following Section 5.2.1.

5.3.3. Inlaid crack repairs

Inlaid crack repairs are holding repairs that are undertaken over static or working cracks to seal them against the ingress of water, de-icing salts and incompressible materials that might otherwise cause further and accelerated deterioration of the pavement at the locality of the crack. Where installed correctly at single (non-bifurcated) cracks, they can be expected to be last 3 to 5 years, with the lower end of the expected life likely at wider full depth cracks and longitudinal cracks in wheel tracks. Overbanding should not be used as a method to seal cracks in concrete pavements.

Clause 1090 outlines the requirements for inlaid crack repairs [7]. Inlaid crack repairs are undertaken with proprietary polymeric materials; these materials are discussed in more detail in Section 6.5. Some polymeric materials may only be appropriate for use on flexible pavements, so it is important to check product acceptance scheme certificates to ensure the repair products are suitable for repairs to concrete surfaces.

The inlaid crack repair procedure involves the milling of a recess centred around the crack to a nominal depth dependent on the crack location and orientation. The recess and crack are then cleaned, normally using a hot air lance. The recess is then filled with the polymeric material in one or multiple layers until flush with the surface. Pre-coated chippings or calcined bauxite chippings are then applied (and rolled in if necessary) whilst the material is still plastic to provide skidding resistance (see Figure 5.12). Primers are not generally necessary but where they are required see Step 4 of Section 5.3.2.3.

Figure 5.12 Inlaid crack repair process. Milling recess (top left), cleaning / drying recess (top right), filling recess (bottom left), applying chippings (bottom right).

The recess width and depth should follow the manufacturer's instructions. This typically varies dependent on the size, location and the orientation of the crack being treated:

- 20 mm deep inlaid crack repairs are only used for transverse cracks;
- 40 mm deep (or greater) inlaid crack repairs are used for longitudinal cracks and cracking in wheel tracks.

200 mm wide recesses are recommended for most applications. Wider recesses may be necessary for wandering cracks (i.e. those that do not follow a relatively straight line). The crack should, where practicable, be centred in the recess.

Some inlaid crack repairs have suffered early failure through loss of adhesion at the ends of the milled-out recesses. Often these have been left in the as-milled form as shown in (a) of Figure 5.13, i.e. effectively with a feathered edge. As the polymeric materials are highly flexible, this can result in the entire length of the repair being stripped out of the inlaid repair. It is therefore recommended that, after milling, the ends of the recesses are cut by hand or light tools to leave a uniformly deep recess with vertical ends as shown in (b) of Figure 5.13.

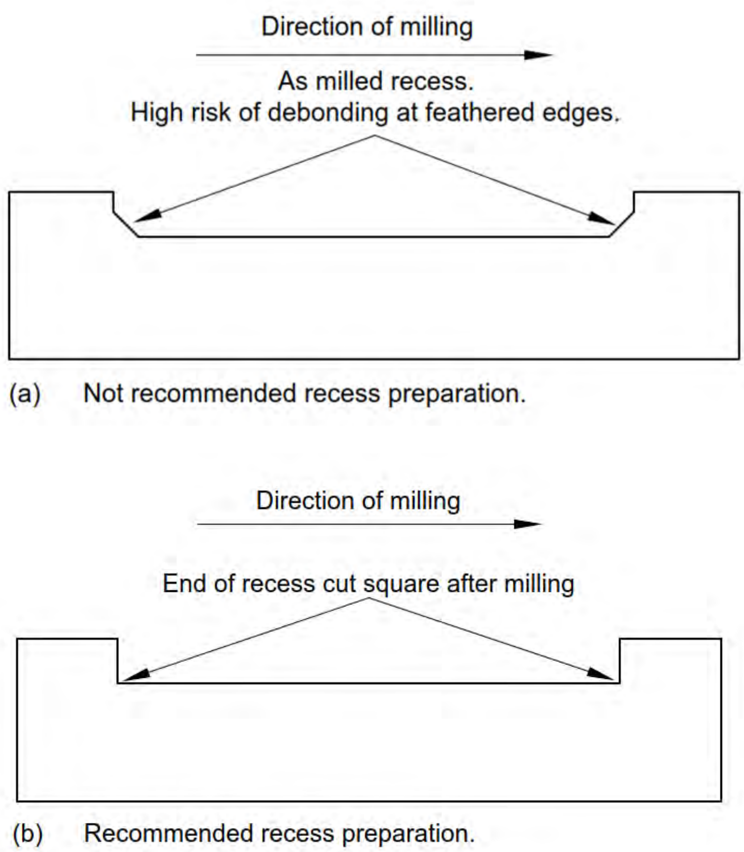


Figure 5.13 Inlaid crack repair recess preparation. (a) As milled form (not recommended) and (b) recommended preparation with square cut transverse edges.

Polymeric materials can be sensitive to poor adhesion caused by moisture in the recess, resulting in stripping of the repair. The recess has to be thoroughly dried before the application of a polymeric material.

5.4. Structural repairs

Structural defects (detailed in Sections 3.2, 3.3, and 3.5) can significantly reduce the pavement residual life to carry the future design traffic. Repair techniques to maintain or restore structural integrity include:

- bay replacement;
- full depth repairs;
- slab levelling (under slab grouting, slab lifting and bump cutting);
- crack stitching; and,
- retrofitting dowel bars.

5.4.1. Bay replacement

Bay replacement is a common long-term full depth repair technique for URC and JRC pavements. It involves the replacement of the full depth of concrete across single or multiple bays with joints reinstated at the same locations as the original joints. The requirements for bay replacements are outlined in Clause 1081 [7].

Bay replacements are generally preferred for URC pavements rather than a ‘full depth repair’ (see Section 5.4.3) to avoid the introduction of additional joints over short lengths. However, bay replacements in JRC are typically reserved for bays with extensive defects owing to the practicability of replacing long bays up to 25 m in what is often a short closure window. Therefore, for JRC a ‘full depth repair’ (see Section 5.4.4) may be preferred.

The standard approach for a bay replacement is to cast the concrete in place. Precast concrete slab systems can be used to undertake bay replacements; however, at the time of publication of this manual, whilst trials are progressing, precast concrete slabs have not been used on the SRN for bay replacements. This section provides guidance on the implementation of both approaches.

The selection of the approach is generally dependent on:

- closure window;
- availability of materials; and,
- the number of consecutive bays to be replaced.

Cast-in-place bay replacements can be undertaken using pavement concrete (see Section 6.1) or early strength concrete (see Section 6.2). The process is the same, but early strength concretes have a short curing (or strength gain) period and so they can be trafficked after a period of hours, rather than days for pavement concrete, meaning that bay replacements can be undertaken in short closures.

The use of precast concrete slabs (see Section 5.4.1.2) removes the need for an in situ curing period, required for cast-in-place bay replacements. This would allow for bay replacements to be undertaken in short overnight closures, as may be encountered on the SRN.

5.4.1.1. Rapid cast-in-place bay replacement

This section describes the process for a rapid cast-in-place bay replacement. Non-rapid cast-in-place bay replacements follow the same key steps, but with more time required to ensure the quality of the concrete placement and finishing owing to the longer setting time of an ordinary pavement concrete. The key steps are:

1. Preparation and removal of existing bays.
2. Preparation of joints, foundation and any reinforcement.
3. Concrete placement and finishing.
4. Curing and opening to traffic.
5. Joint sealing.

Preparation and programming

High levels of competency are required across the supply chain to undertake this type of repair. It should be executed by skilled personnel with a good understanding of not only their responsibilities but also of those around them; the timing and sequencing required to assure quality and safe execution of these in situ works is critical.

A mixture trial is essential to assure the strength gain characteristics of the proposed mixture and to calibrate the cube compressive strength, rebound (Schmidt) hammer or alternative regime being used to monitor the strength gain. Furthermore, careful batching control of the mixture is necessary to ensure the workability and strength gain characteristics, with particular attention to the control of water addition, which needs to consider the water content of the aggregates to prevent any over or under addition of water.

Whilst the bay itself can be replaced in an overnight closure, the full bay replacement process including preparatory works is often undertaken over multiple over-night closures. Depending on the bay removal method, there can be an initial shift when preparations are made so that the bay can be quickly removed, including saw cutting the slab into liftable pieces and drilling holes for any lifting eyes to be inserted. A final shift often follows the replacement of the bay to seal the joints.

The process is generally only achievable in overnight closures where replacement of the foundation is not required. Only minor repairs to the existing foundation such as levelling with a rapid setting mortar to receive the concrete and prevent foundation restraint will be possible in an overnight closure.

Where foundation replacement is required (i.e. for a HBM subbase that exhibits ‘poor integrity’ (see

Table 2.6) as defined by CD 227, and this is supported by visible secondary cracking or disintegration; or unbound subbase that exhibits low surface modulus), the works can take considerably longer. It may be more practicable to place asphalt as the subbase material instead of a hydraulically bound material.

Any new foundation material needs to be fully compacted, with particular attention paid to corners. For an existing granular subbase, where it is difficult to compact adequately within small repairs, replacement with a bound material may be considered as an alternative. However, consideration of any potential negative effect on pavement performance, from the creation of a discontinuity in the sub-surface drainage or non-uniform slab support conditions, is required.

Weather conditions

Rapid cast-in-place bay replacements and other full depth repair procedures are highly weather susceptible, notably to low temperatures and rain. Acceptable weather conditions will be critical to a successful outcome within the closure window. Protection is required against any heavy rain otherwise it will increase the water / cement ratio at the surface, resulting in a weak surface layer.

Whilst there are different methods available to improve curing rate, including the use of an alternative concrete mixture design (see Section 6.2) curing is still highly temperature dependent.

Any rapid cast-in-place bay replacement should be avoided where the weather conditions are approaching those outlined in Clause 1045 [7]. Works should be postponed where there is a lack of certainty around whether the conditions will be acceptable.

The key steps for rapid cast-in-place bay replacement are detailed below.

Step 1. Preparation and removal of existing bay

The existing concrete bay can be removed using several methods. However, the only practicable option to assure swift removal of the bay is by lifting, often in multiple pieces. This method is not only quicker than using a breakout method, it also generally results in less disturbance to the adjacent bays and the underlying subbase.

Removing the bay by lifting requires the use of heavy-duty lifting equipment, but different methods can be used to lift out the concrete including:

- lifting lugs;
- vacuum lifting equipment; and,
- bespoke excavator buckets.

The lifting method used may be dependent on the intactness of the bay being removed. Vacuum lifting requires minimal preparation but lifting lugs are typically more suitable for lifting out broken up pieces of varying sizes (see Figure 5.14).

Intermediate steps are required to prepare the concrete for removal. These are all generally undertaken in a separate shift prior to the bay replacement operation to maximise the available curing time. Initially, full depth saw cuts using diamond saw blades are made at the existing joints to cut the load transfer devices and remove any aggregate interlock which may hinder removal.

It may be appropriate to overcut up to 50 mm past existing transverse joints where complications related to non-vertical induced cracks are identified during the pavement investigation so that joint faces are vertical. This is further discussed in Step 2.

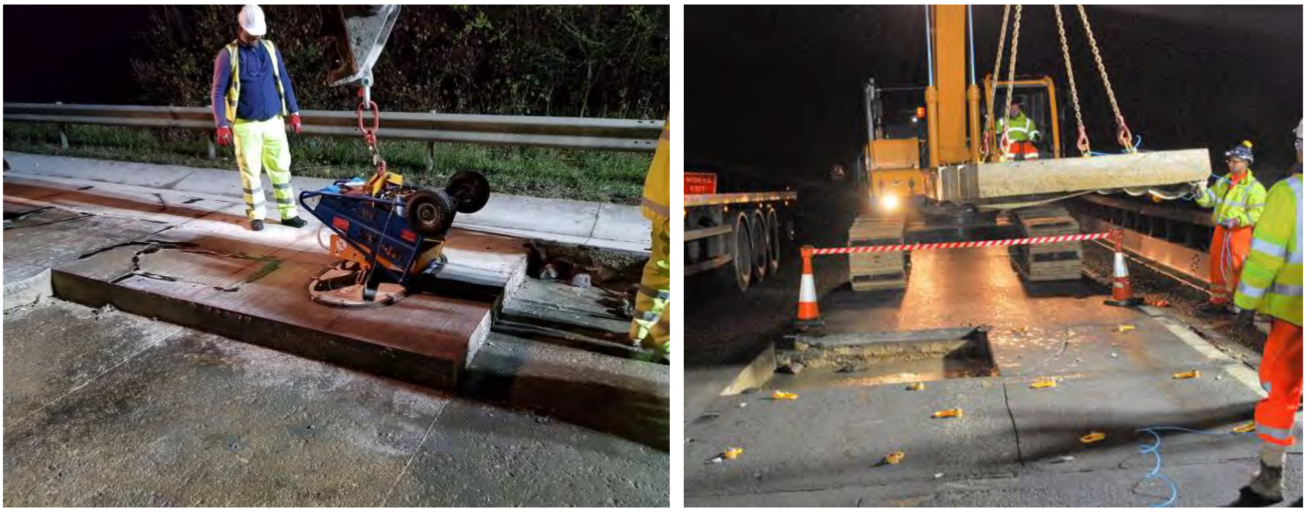


Figure 5.14 Concrete removal by vacuum lift (left), and using lifting lugs (right)

During transverse sawing, slab compression may cause the saw to seize, particularly in warmer weather. If so, stress relief cuts can be made with a 'rock wheel' saw. Where a ‘rock wheel’ saw is used, diamond saw cuts should be made just outside the stress-relief cuts to avoid having a rough-faced joint produced by the ‘rock wheel’ saw, which could introduce a higher risk of joint spalling. Any saw cut intrusion into the adjacent bays and the underlying subbase should be limited. It is essential to prevent slurry from the sawing operation, which could solidify and block drains, from getting into the drainage system.

Dependent on the size of the bay being removed and the equipment available, the bay might have to be sawn into smaller pieces that can be safety lifted out and transported away for recycling. Cores can be taken out of the slab at corners and reinstated with instant fill material so that the slab can be more easily removed (see Figure 5.15). Setting the saw blade at a slight angle of 3 to 5 degrees will create a wedge shaped section which can help to prevent the slab jamming as it is lifted.

Figure 5.15 Cutting cores at slab corners (left) and placing lifting lugs (right)

Once the bay has been prepared then, immediately prior to removal, which is usually the first operation at the start of a new shift for the bay replacement operation, any lifting elements that are needed to remove the bay or bay pieces are placed. The concrete slab can then be removed.

Step 2. Preparation of joints, foundation and reinforcement

Once the existing concrete has been removed, the foundation and the joints should be inspected. It may be necessary to level the foundation with a rapid setting grout to avoid restraint (see Figure 5.16). Where additional deterioration of the foundation is identified, repairs may be possible dependent on the closure window.

Joint faces need to be sound and vertical. Where there is deterioration of the joints in the form of undercutting a repair with a proprietary mortar may be required (see Figure 5.17), particularly at expansion joints where compressible joint filler boards are being placed.

Where joint faces are unsound i.e. exhibiting significant undercutting, deterioration or cracking, it will be necessary to increase the repair area (see Figure 2.9). If spalling or deterioration is present in the upper portion of the slab at the existing joints, the area should be broken out and treated as a shallow repair (see Figure 5.16 and Section 5.3.2.2).

Once the foundation has been prepared and any additional areas of concrete removed, then a bituminous spray complying with Clause 920 [7] and Clause 1007 [7] should be applied to the prepared foundation and the vertical joint faces (except where a compressible filler board is being used). This prevents water being drawn out of the concrete from the foundation and repair sides during curing, which may result in shrinkage and / or a weakened area of concrete. Historically, plastic sheeting has been used; however, this can introduce a slip hazard.

Figure 5.16 Foundation being levelled with rapid setting grout prior to bay replacement (left) and additional breakout of spalled area (right)

New dowel bars and tie bars should be installed in accordance with Clause 1011 and 1012 [7]; at mid-depth at 300 mm and 600 mm centres at the transverse and longitudinal joints, respectively, to provide load transfer between the existing and new bays.

Where multiple continuous bays are being replaced, dowel bars and tie bars between bays need to be supported on cradles in prefabricated joint assemblies to ensure that their optimum horizontal and vertical alignment and orientation is maintained throughout the concrete pour. Assemblies require careful design and placement, they need to be sufficiently rigid to maintain the alignment of the load transfer devices during placement, and sufficiently fixed to not move during concrete placement, but not so fixed that future thermal contraction and expansion movements are inhibited.

The installation of dowel bars and tie bars into the adjacent existing concrete requires a sound vertical face. If the face is not vertical or sound, installation of load transfer devices may not be appropriate or possible. Figure 5.17 includes two examples of unsuitable joint faces to receive load transfer devices.

The joint face in the left-hand image in Figure 5.17 is common at joints as induced cracking is often non-vertical, this may be addressed by overcutting into the adjacent bay by approximately 50 mm. This can be rectified with a proprietary mortar prior to the placement of any compressible joint filler board.

The right-hand image in Figure 5.17 has significant voiding at the corners but also there is a mid-depth crack across existing dowel bar positions. Dowel bars installed in this joint are likely to perform poorly and even accelerate the cracks development and deterioration of the pavement. This may have been identified by targeted coring at the investigation stage. In such instances, the repair should be extended into the adjacent bay.

Figure 5.17 Unsuitable joint faces to receive load transfer devices

Gang drills (see Figure 5.18) are the preferred option for drilling the holes to receive dowel bars and tie bars in existing adjacent slabs. Compared with a handheld drilling operation, they are a safer alternative and can provide better horizontal and vertical alignment of the drill holes, which is critical to the performance of dowel bars and prevents locking up of the transverse joints.

Once the holes have been drilled, the following steps are necessary to achieve proper anchoring of the dowels or tie bars:

1. Clean drill holes with compressed air to remove debris and dust.
2. Place suitable quick-setting non-shrinkage cement grout or resin in the back of drill holes.
3. Insert dowels or tie bars and twist to ensure they are encapsulated by the grout or resin.
4. Remove excessive resin or grout material from the joint face.
5. Check vertical and horizontal alignment of dowel bars and tie bars and adjust as necessary.

Figure 5.18 Gang drills being used to drill dowel and tie bar holes

Compressible joint filler board, ideally incorporating a tear-off strip to facilitate joint sealing, is then placed at the location of any expansion joints and any isolation joints to other linear assets such as drainage channels. The use of expansion or contraction joints in bay replacements is dependent on:

- the type of joint being replaced;
- a history of locked up joints in the pavement; and,
- the coefficient of thermal expansion of the concrete being used versus the existing pavement.

As a minimum, any expansion joint being removed should be replaced with a new expansion joint. However, it is common practice to use at least one expansion joint in a bay replacement even when contraction joints are being replaced. This is because the replaced bay may have greater expansion characteristics than the surrounding pavement, which could result in spalling or compression failures (blow-ups). Concrete containing siliceous aggregates tend to have a higher coefficient of thermal expansion than concretes containing limestone aggregates. Section 6.1 contains more information on the thermal expansion properties of concrete mixtures.

Throughout these operations, it is important to prevent debris, including the repair materials, from entering the joint space. Joint spaces and grooves should be thoroughly cleaned using compressed air and taped over with adhesive masking tape prior to placement of the concrete.

Following this, any reinforcement is placed on chairs following the requirements of Clause 1008 [7]. Similar to dowel bar assemblies, reinforcement needs to be fixed in place and not move during installation of the concrete.

Figure 5.19 Prepared bays

Step 3. Concrete placement and finishing

The rapid setting properties of early strength concrete means that limited time is available for concrete placement and finishing before it hardens. The working time can be as little as 20 minutes, depending on the air temperature. Experienced contractors are needed for this type of work to implement seamless workflow and ensure a good standard of finish. A significantly increased number of personnel is needed to complete rapid bay replacement works versus conventional bay replacement works using an ordinary pavement concrete mix, specifically to complete the placement and finishing process within the working time.

It is important to avoid disturbing the position of any reinforcement during the placement, compaction and finishing stages. Adequate and uniform cover to reinforcement is essential to ensure bond strength and the transfer of stresses from the concrete to the steel and mitigate the risk of corrosion of the reinforcement. Insufficient depth of cover to reinforcement is likely to result in spalling and cracking of the concrete above the reinforcement within the intended service life of the bay replacement.

Mixing

Mobile volumetric mixers are typically used to produce the early strength concrete for rapid cast-in-place bay replacements. Batch volumes for bay replacements and other full depth repairs are usually low and most works are undertaken during short closures, often overnight. Onsite mixing generally reduces potential wastage and avoids potential delivery complications. It also increases the in situ workability window of the concrete, which can be as little as 20 minutes for some early strength concrete mixtures.

Operator experience of the volumetric mixer is important. Whilst mixtures can be selected from a computer and automatically batched, some operator input is required including adjusting the mixture to account for the water content of the aggregate and the air temperature. The operator also needs to be able to recognise when the mixer might not be operating correctly. It is standard practice to have a backup mixer on site should there be an issue with the operation of the first mixer. Manual batching is prone to error and should not be undertaken.

Where volumetric mixers are being used, before placement commences, a small amount of mixture should be produced to check the consistency of the mix and operation of the mixer. This ‘first run’ should be disposed of in a small skip or excavator bucket and not placed in the repair area.

Placement

Once the mixture has been checked, placement of the concrete can commence. Placement requirements for concrete mixtures are outlined in Clause 1024 and 1025 [7]. As outlined previously, for early strength concrete it is important to work quickly and methodically. Adequate concrete compaction is critical to the strength gain and this can be problematic at the corners and edges of the bay replacement. Some mixtures may appear dry on discharge, but this can improve upon vibration. At this stage, cubes should be taken for quality assurance testing (see next Step 4 for curing).

Compaction

Compaction is normally undertaken with a vibratory poker (Figure 5.20). Compaction is complete when only a few small bubbles are rising to the surface and when the sound emitted by the vibrating poker is consistent.

Finishing

For rapid bay replacement using early strength concretes, the little working time available can impose a challenge on concrete finishing. This may be overcome by adjusting the mix design to allow more setting time. However, it is standard practice with these types of repair to begin finishing the surface before the entire slab has been poured. This reduces the risk of the first portion of the slab becoming unfinishable.

Attention should be paid to not overwork the new concrete. Overworking concrete will create a weakened concrete matrix that is more susceptible to shrinkage and spalling. The repair surface should be struck off flush with the adjacent concrete, this is normally undertaken with a vibrating screed (see Figure 5.21) with sufficient vibration to bring the cement paste to the surface to allow texturing to be undertaken without causing dragging of the coarse aggregate.

Figure 5.20 Compaction with vibratory poker

Once finishing is complete, surface texture is applied. Finished surface requirements for concrete repairs are outlined Clause 1026 [7]. A brushed surface finish is common for bay replacements and full depth repairs. The texture should be applied evenly with no ridges being formed, particularly where the brush overlaps with previous passes. The pressure of the brush on the surface of the concrete should be uniform and consistent, and the bristles should be kept clean, to produce a satisfactory finish (see Figure 5.21).

Figure 5.21 Finishing with paddles and a vibrating screed (left) and brushing to give surface texture (right)

Step 4. Curing and opening to traffic

Moisture retention and temperature control for concrete in its early life is key to minimising plastic shrinkage, achieving adequate strength gain in both the short term (to permit reopening as planned) and in the long term, ensuring surface durability. Curing requirements for cementitious materials are outlined in Clause 1027 [7].

Curing should start immediately after finishing and the evaporation of any surface water to control moisture loss. Typically, a white resin-based aluminised curing compound is spray applied (see Figure 5.22).

After the application of the curing compound, depending on the type of concrete used, the weather conditions and closure window, thermal insulation blankets or heated blankets or tents (see Figure 5.22) may be used to help accelerate the process to meet the specified strength in the required time. They offer the benefit of:

- preventing moisture loss from the concrete, increasing the rate of hydration and reducing shrinkage;
- promoting a uniform temperature gradient through the slab, reducing the risk of thermal cracking;
- inducing a higher temperature within the slab, by retaining the heat of hydration, and accelerating the chemical reactions that lead to strength development; and,
- protecting the concrete from the effects of inclement weather where it is anticipated.

With propane heating and an appropriately insulated tent or blanket, a high air temperature is achievable. An ambient temperature of between 50 °C and 60 °C is targeted to ensure that the concrete is not over heated. Heaters should be angled upwards to avoid direct heating of the concrete surface.

Any insulation blankets and tents should remain in place until the required short term concrete strength of 25 N/mm^2 for trafficking is achieved. In cold weather when there is a large variation between the concrete and ambient temperatures, any insulation blankets placed on the concrete should be loosened and removed gradually at the end of curing to avoid a sudden temperature drop that could lead to cracking in the new slab.

The strength of the concrete and rate of strength gain should be assessed using the temperature of the concrete and temperature / strength gain correlations developed during the mixture trial, which should include the curing of specimens at ambient temperatures similar to that expected during the works.

If thermocouples are not placed in the concrete during placement (see Section 2.4.2.2), a temperature probe should be inserted (see Figure 5.22) into the concrete to measure the temperature in the slab. Thermocouples or probes should be placed a maximum depth of 30 mm below the surface to provide conservative temperature readings.

The ambient temperature at the time of placing should be compared with the temperature at which the strength development curve(s) were obtained. For any temperature drop greater than 5 °C, extra time should be allowed, as strength development slows in cooler weather.

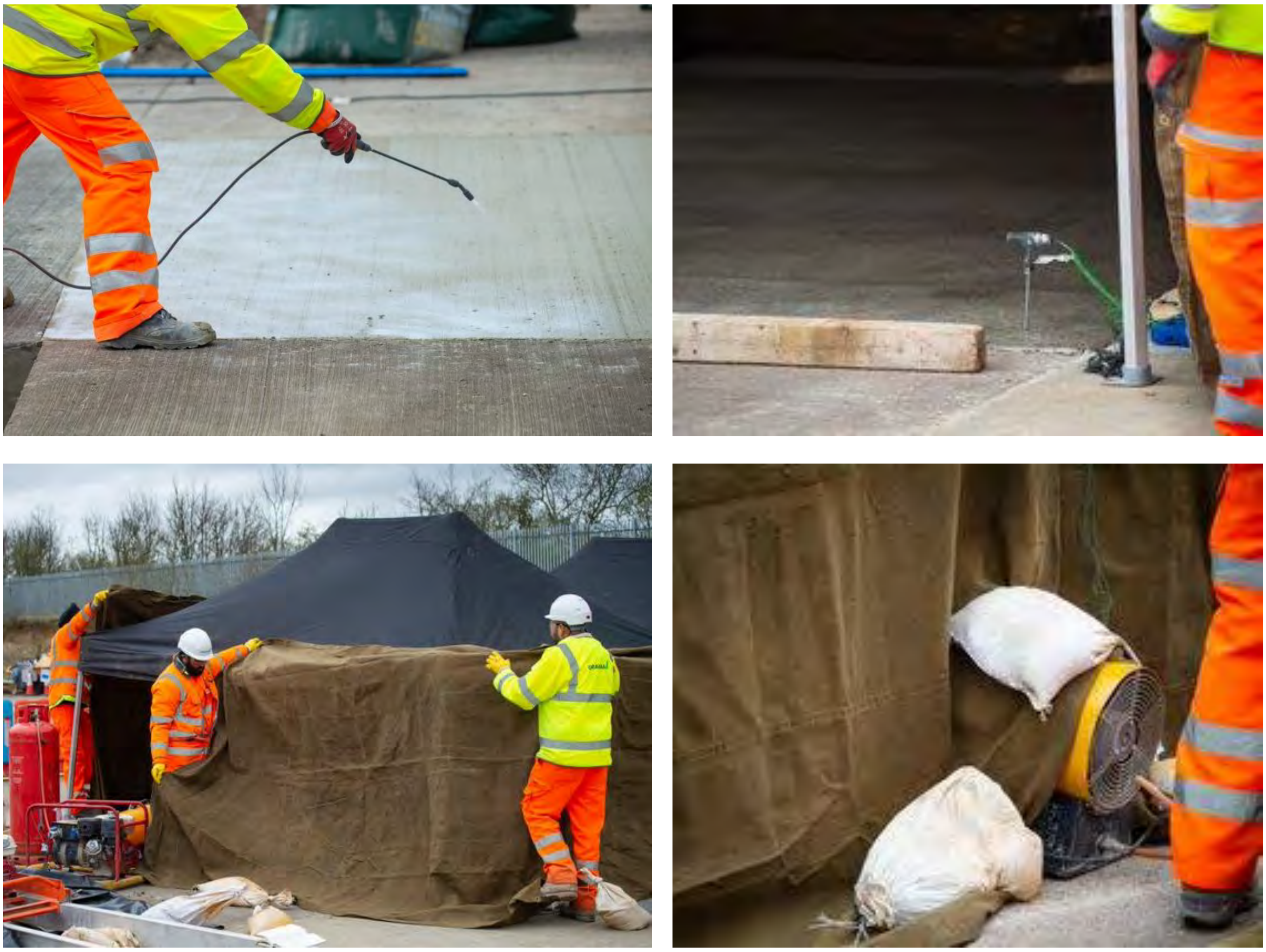


Figure 5.22 Aluminised curing compound being applied (top left), probe insertion to monitor curing rate by temperature-matched curing (TMC) (top right), tents being set up over bay (bottom left), propane heater being used to increase ambient temperature and curing rate (bottom right).

Once the temperature / strength gain assessment indicates that the concrete has gained sufficient strength, a final assessment of compressive strength prior to opening to traffic is typically undertaken by:

- using a correlated rebound hammer (or Schmidt hammer) (see Section 2.2.1.5 and Figure 5.23); or,
- crushing cubes taken during batching that are subject to temperature-matched curing (see BS 1881-130 for more details).

Further cubes are taken to test for strength, typically at 3 days and at 28 days. They should be made, stored and tested in accordance with BS EN 12390.

Embedded sensors can be used to estimate strength gain in real time (see Figure 5.23), the situ strength gain can be based on the relationship of compressive strength to maturity. Both rebound hammers and embedded sensors require correlation with the mixture design and cube compressive strengths obtained during the mixture trial to give reliable results. Some wireless embedded sensors may only provide readings if placed within 50 mm of the concrete surface. More details on embedded sensors is provided in Section 2.4.2.2.

Figure 5.23 Schmidt hammer (left), embedded curing sensors (top right), readout from embedded sensors (bottom right).

Step 5. Joint sealing

Due to time constraints with overnight closures, joint sealing is typically undertaken on a subsequent shift. Requirements for joint preparation and sealing is covered in Clauses 1013 to 1017 [7] and is further discussed in Section 5.2.1.

However, contraction joints between multiple bay replacements may need to be sawn within the same shift to avoid shrinkage cracking (see Figure 5.24) or, where this is not possible e.g. due to the greenness of the concrete or otherwise, joints should be wet-formed using a groove former.

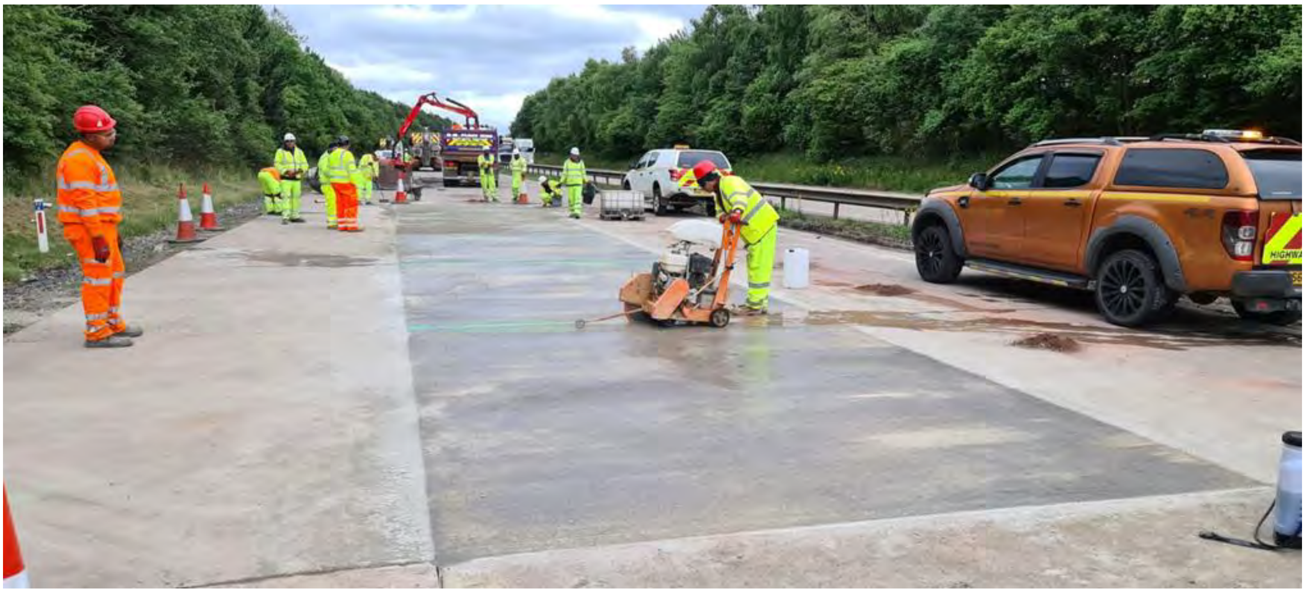


Figure 5.24 Sawing transverse joint grooves between bays

5.4.1.2. Precast concrete slabs for bay replacement

Precast concrete slabs have a history of successful use internationally for replacing bays in URC pavements. However, they have not been used for bay replacements in the UK on the SRN. At the time of publication of this manual, work is ongoing to develop a specification for precast concrete slabs suitable for bay replacements and full depth repairs and the procedure for installing them on the SRN. This work is expected to be completed in 2021.

JRC bays are generally too long for a bay replacement to be practicably undertaken with a single precast concrete slab. Multiple precast concrete slabs may be a suitable option for a JRC bay replacement; however, consideration is required as to how to prevent sympathetic cracking in the adjacent bay at the location of any new transverse joints. Precast concrete slabs may be appropriate as a full depth repair to a portion of a JRC bay.

A bay replacement using precast concrete slabs offers the following advantages versus a cast-in-place bay replacement:

- improved quality control;
- minimal weather restrictions on placement;
- reduced delay before opening to traffic;
- reduced risk of joint deterioration; and,
- reduced risk of in-service deterioration.

Improved quality control: precast concrete slabs are manufactured in a controlled environment. Therefore, there is a reduced risk of variable quality concrete. Issues that may arise during in situ and time-pressured mixing, compaction, finishing and curing are essentially eliminated.

Minimal weather restrictions on placement: precast concrete slabs are not susceptible to damage by cold or wet weather during placement, so can be placed in a wider range of weather conditions.

Reduced delay before opening to traffic: precast concrete slabs do not require a curing period after placement and can be trafficked immediately.

Reduced risk of joint deterioration: transverse and longitudinal joints do not need to be saw cut following installation to allow for joint sealing. Therefore, there is a reduced risk of early deterioration of joints through sawing action.

Reduced risk of in-service deterioration: precast concrete slabs incorporate reinforcement to mitigate risk of cracking during lifting; therefore, any cracking that occurs in-service as a result of traffic loading is likely to be held tight by the reinforcement [49].

However, bay replacements using precast concrete slabs present the following key considerations and challenges versus cast-in-place bay replacements:

- repair dimensions;
- support conditions;
- vertical alignment; and,
- provision of adequate load transfer.

Repair dimensions: precast concrete slabs are manufactured to specific length, width and thickness dimensions. Therefore, the precast concrete slab dimensions need to match the specific dimensions of the bay that is to be replaced. A repair area that is made too large may result in unacceptably wide joint widths, whilst a repair area that is too small will not be able to accommodate the new slab. Furthermore, a precast slab that is too thick for the adjacent concrete will require removal of the existing foundation, which could be time consuming. This generally means following a template of the precast slab to mark and remove the correct repair area, but it may be possible to make minor alterations to the dimensions of the precast slab on site by saw cutting.

Support conditions: precast concrete slabs are manufactured with a flat base, so the foundation to receive the slab, even after reworking, is often not sufficiently flat to offer the uniform support necessary for pavement performance. Therefore, a bedding material is needed that ensures full contact. This normally takes the form of a flowable grout that is injected under the slab to fill any voids after the slab has been placed and aligned with the adjacent pavement.

Vertical alignment: the precast concrete slab has to be placed in such a way that it aligns vertically with both the adjacent longitudinal and transverse joints without exceeding the permitted step tolerances. Where a flowable grout is being used, the standard approach is to set the slabs at the necessary vertical alignment using strongback beams spanning the repair area, then the remaining voids under the slab can be filled with the flowable grout to support the slab. For this reason, pavements with irregular falls are unlikely to be suitable for repair with precast concrete slabs.

Provision of adequate load transfer: the performance of the precast concrete slab and the surrounding pavement will be highly dependent on ensuring adequate load transfer across joints. As the joint faces of the precast concrete slab are smooth, there will be no aggregate interlock to provide load transfer. Therefore, load transfer devices (dowel bars and tie bars) need to be installed across all of the joints.

Tie bars are typically installed at longitudinal joints by cross stitching following the procedure outlined in Section 5.4.8. Dowel bars are installed at transverse joints by one of the following methods:

- slots are manufactured in the underside of the precast concrete slab and load transfer devices are installed in the existing joints following the standard bay replacement procedure. After placement, the slots are filled with grout;
- slots are cut in the adjacent concrete and dowels fabricated in the precast concrete slab are placed in the slots. The slots are then filled with an appropriate repair material; or,
- slots are manufactured in the top side of the slab then at the same location in the adjacent concrete. Dowel bars are retrofitted following the general procedure outlined in Section 5.4.10.

At the time of publication of this manual, some of the key design elements for the overall precast concrete slab system have not been trialled on the SRN including dowel slot systems, providing uniform support with bedding grout, and a single sided method of cross stitching tie bars. Each element contributes to the overall performance of the precast concrete pavement. Precast concrete slab systems need to be subject to a series of off-network non-destructive and invasive performance tests to ensure the system is deemed safe and reliable for installation on the SRN to withstand high traffic loading.

5.4.2. Full depth repairs to CRCP

Whilst the principle of full depth repairs to CRCP is similar to full depth repairs to other types of concrete pavement (i.e. removing the concrete full depth and casting the concrete repair in situ), the procedure is generally more difficult and costly because of the large quantity of reinforcement and the high levels of stress generated in it.

When undertaking full depth repairs to CRCP there is a need to maintain the continuity of the reinforcement as it is critical to the structural integrity of the retained pavement. This means either:

- the concrete surrounding the repair needs to be carefully cut back beyond the repair area to expose a sufficient amount of the existing reinforcement so that the new reinforcement can be connected to it; or,
- new reinforcement needs to be grouted into the adjacent existing concrete faces then connected to the new reinforcement bars used in the repair area.

Full depth repair to CRCP is complicated by the presence of inherent regular transverse cracks. These cracks may deteriorate if the transverse joints are too close. When both of these factors are considered, the resultant repair area often ends up being much larger than the defective area of pavement.

Furthermore, the repairs can only be carried out in certain temperature conditions. The daily temperature variations should be minimal at the time of the repair so that the surrounding slab movement, which could otherwise damage the repair through compression, is minimised. Saw cutting and removing the area in hot weather whilst the slab is in compression may be difficult as the slabs will compress into the saw cut, causing the saw blade to bind. This generally means full depth repairs can only be undertaken in spring and autumn.

This section describes the procedural variations and additional steps required when undertaking full depth repairs to CRCP compared with a cast-in-place bay replacement in URC and JRC as detailed in Section 5.4.1.1.

Reinforcement

As outlined above, reinforcement continuity should be maintained between the repair and the existing adjacent pavement. This is achieved by:

- exposing and connecting to the existing reinforcement through:
  - mechanical coupling (Figure 5.25 & Figure 5.26)
  - tied splicing (Figure 5.25)
- drilling and grouting reinforcement bars into the existing concrete (Figure 5.25).

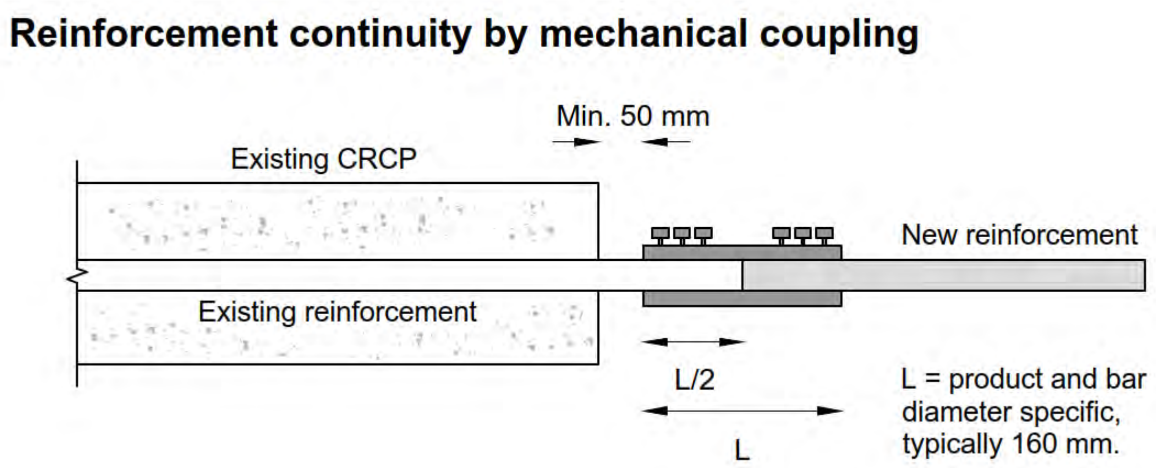
Mechanical couplers provide coupling by the use of lockshear bolts that are embedded into the reinforcement. Tied splicing uses a reinforcement overlap and a minimum of two tied steel wires along the length. The drill-and-grout method is used in conjunction with mechanical coupling or tied splicing to connect the reinforcement in the centre of the repair.

The reinforcement continuity option used will be dependent on the:

- repair dimensions;
- closure time; and,
- cost.

Figure 5.25 outlines the minimum dimensions for the different reinforcement continuity options. Mechanical coupling generally requires less reinforcement to be exposed than splicing; therefore, less preparation is required, and works can be completed more quickly. However, the additional cost of the coupling devices means tied splicing can be a more cost effective option.

The drill-and-grout method may take longer to undertake as the holes need to be drilled and cleaned, and the resin grout needs to be left to cure before additional reinforcement is coupled.



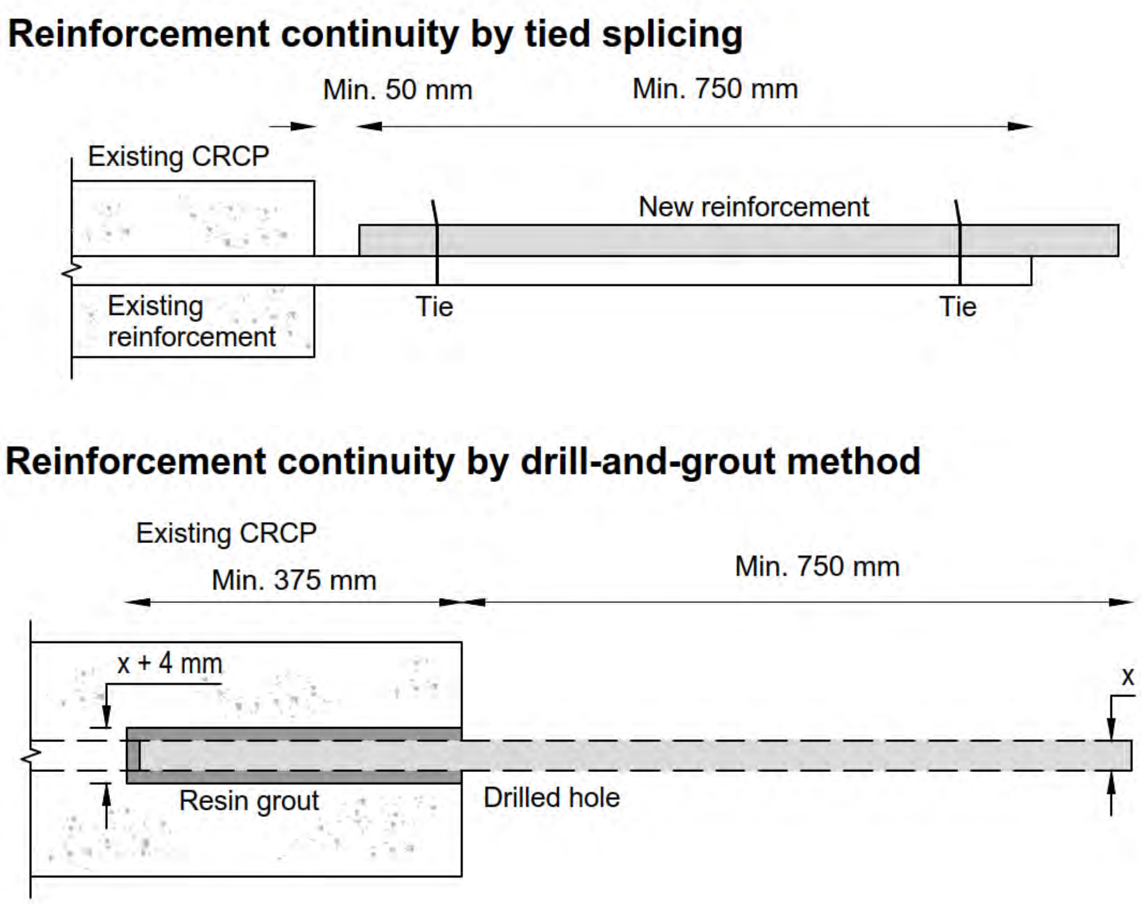


Figure 5.25 Reinforcement continuity by mechanical coupling (top), tied splicing (middle) and drill-and-grout method (bottom).

Where the existing reinforcement is being exposed to give reinforcement continuity, this has to be done carefully to avoid damaging reinforcing steel. Only hand tools or light percussion tools should be used.

The faces of the original concrete should be left roughened but sound and vertical without undercutting. Any concrete debris accumulated below the reinforcement bars should be reduced to a fragment size such that it may be lifted out and removed through the spaces between the bars. With all debris and dust removed, any damage to the subbase needs to be made good. This is normally undertaken with the same rapid setting grout used to level other areas of foundation (see Section 5.4.1.1 and Figure 5.16).

The pavement investigation, prior to the repair works being carried out should include an assessment of the condition of the reinforcement within the repair area, noting that it may not be practical to increase the repair area if poor condition reinforcement is identified only during the repair stage. If any reinforcement bars are identified to be in poor condition i.e. heavily corroded, or damaged or severed during the exposure process, they will either need to be replaced using the drill-and-grout method, or the repair area enlarged further to expose areas of reinforcement in good condition on to which the new reinforcement can be fixed.

When mechanical couplers are used, it is critical that the new reinforcement bars are the correct length. If bars are too short, the connection may be poor. If bars are too long, the reinforcement bar will sag or bow in the middle of the repair, which may cause premature failure of the repair. Typically, bars are cut approximately 3 mm short; however, depending on the coupling device used, reinforcement bars up to 12 mm shorter than the repair length may be acceptable.

Figure 5.26 Exposed reinforcement (left) and mechanical couplers being used to provide reinforcement continuity (right)

The reinforcement bars used in the repair should match the existing reinforcement's grade, quality, and size; however, mechanical couplers are available that can connect bars of different diameters.

The longitudinal reinforcement should be placed on chairs positioned across the full width of the repair and set to match the height of the existing reinforcement. The longitudinal joints should match the original construction, whether they are tied, or the reinforcement is made continuous, see details outlined in Volume 3, C Series of the highway construction details in the MCHW [39]. Where tie bars are used, the transverse reinforcement should be tied to the longitudinal reinforcement at 600 mm spacings with a 50 - 75 mm gap from the edge of the repair area (see Figure 5.27).

Figure 5.27 Prepared reinforcement and foundation

Repair dimensions

The repair dimensions for a full depth repair in CRCP will be dependent on the:

- size of the defect,
- reinforcement continuity method being used; and,
- location of transverse cracks in the proximity of the defect.

Full depth repairs should encompass the full lane width. Where multiple lanes require full depth repair, because of the high stresses that can develop in such a pavement, full depth repairs need to be carried out one lane width at a time. The minimum practicable length is 1.5 m except where tied splicing is used to restore reinforcement continuity where the minimum practicable length is 3 m.

As outlined in Clause 1082 [7], the edge of the repair needs to be more than 0.5 m from the nearest transverse crack. The typical approach is to create transverse joints close to the locations of existing cracks. Transverse joints also need to be 3 m away from end-of-day construction joints and anchorages.

Figure 5.28 outlines the typical repair dimensions for a CRCP full depth repair related to the size of the defect to avoid introducing transverse joints in proximity to existing transverse cracks.

Figure 5.28 Typical CRCP full depth repair dimensions. Full depth saw-cuts in orange and shallow saw-cuts to expose reinforcement for mechanical coupling in white.

5.4.3. Full depth repairs to URC pavements

The general procedure for undertaking a full depth repair to URC is the same as for a cast-in-place bay replacement. The key difference is that a full depth repair introduces additional transverse joints into the pavement, essentially creating additional bays. The requirements for full depth repairs are outlined in Clause 1082 [7].

Bay replacements (Section 5.4.1) are generally preferred to full depth repairs in URC. This is because bay replacements can be undertaken within a similar closure time, at a similar cost and are considered to offer reduced risk of deterioration of both the repair and the surrounding pavement linked to the introduction of additional joints and the other considerations discussed below.

When full depth repairs are undertaken in URC pavements, they should be full bay width. Part bay width repairs are not considered to be any more economical than a full bay width repair and bring additional durability risks linked to the need to provide load transfer across the new longitudinal joint whilst preventing potential sympathetic cracking adjacent to the new transverse joint.

Full depth repairs to URC pavements have the following additional considerations to cast-in-place bay replacements:

- preventing drying shrinkage cracking as a result of a high aspect ratio of the repair;
- minimum repair dimensions; and,
- preventing sympathetic cracking in an adjacent concrete bay at the new transverse joint.

The dimensions of full depth repairs to URC pavements will inevitably exceed a 1.3:1 aspect ratio, meaning reinforcement is necessary to prevent the occurrence of drying shrinkage cracking. A393 square or B785 long mesh reinforcement in accordance with Clause 1008 [7] is generally appropriate. Where long mesh is used, the main bars should be parallel to the longest dimension.

The minimum practicable length for a full depth repair is generally 1.5 m to permit drilling of holes to receive dowel bars and tie bars using a gang drill. However, historically full depth repairs 1.5 m long constructed over a granular subbase have often proven unsatisfactory. The subbase may deteriorate at the joints due to inadequate compaction of the subbase at edges, the subsequent traffic loading over this area of reduced foundation surface modulus can cause dynamic movement and deterioration of the repair. Therefore, it is recommended that where there is a granular subbase, the repair length be increased to at least 2 m so that subbase re-compaction is easier and the traffic load is spread over a greater length, to minimise any dynamic movement that may occur with a short repair.

For all full depth repairs, it is recommended that a minimum of one expansion joint is introduced regardless of the existing joints being replaced. This is to reduce the risk of spalling or compression failures occurring in instances where the:

- joints in the vicinity of the repair are locked up; and / or,
- repair material has different thermal expansion characteristics than the adjacent pavement.

The arrangement of joints for full depth repairs to URC pavements is dependent on whether the:

- full depth repair is required to one or both sides of an existing transverse joint; and,
- repair spans the full carriageway width.

Figure 5.29 details the longitudinal section of a full depth repair to URC where one side of an existing transverse joint is being repaired.

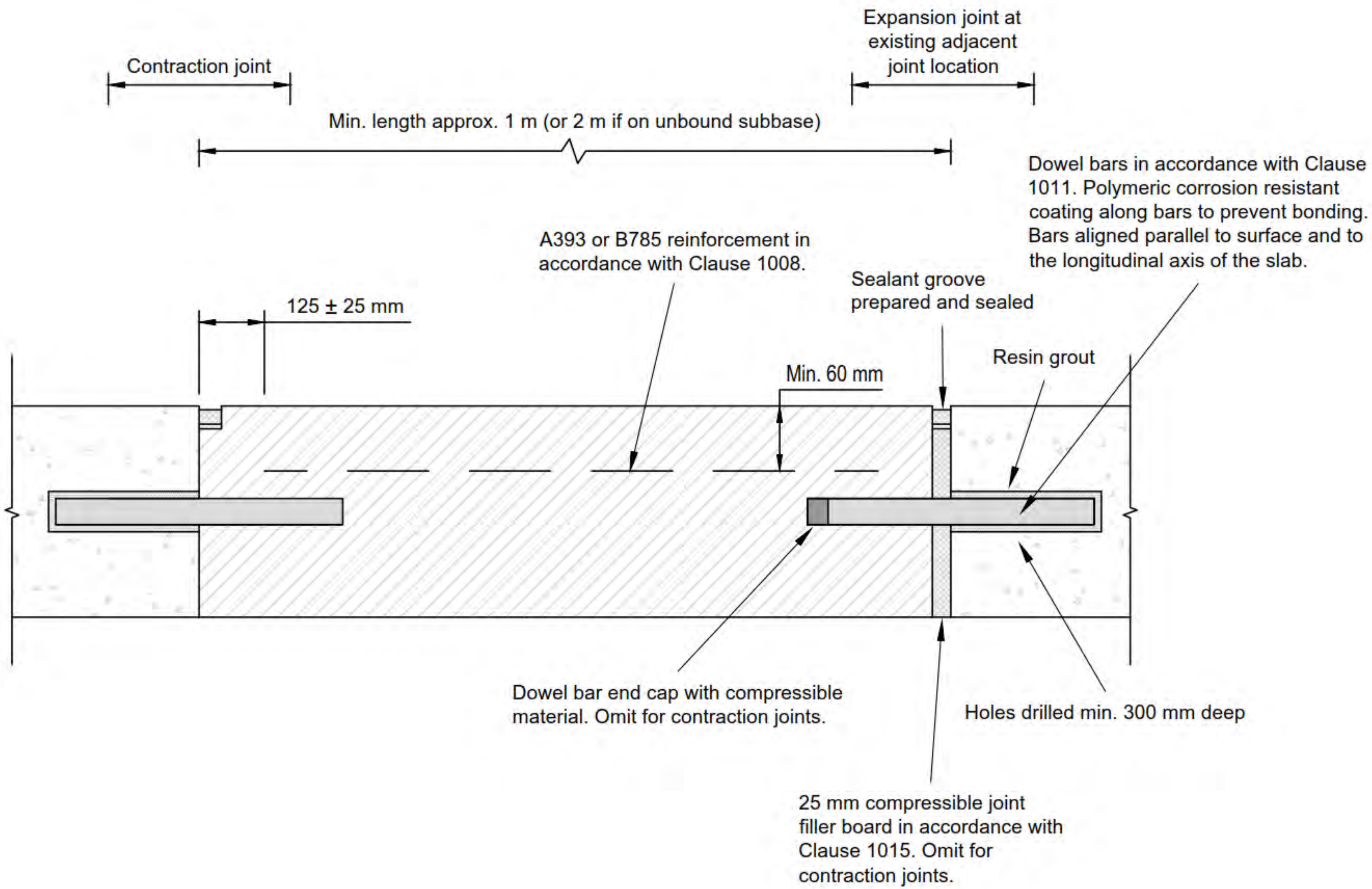


Figure 5.29 Longitudinal section of a full depth repair to URC adjacent to an existing transverse joint

Repairs not spanning the carriageway width

Where a full depth repair to URC is not being undertaken across the full carriageway width, the preferred transverse joint arrangement, consistent with Figure 5.29 and (a) in Figure 5.30, is to use an expansion joint aligned with the existing adjacent transverse joint, a contraction joint at the new joint, and a longitudinal joint that is tied. Where the full depth repair is required to both sides of the joint, then a second contraction joint is required, shown as (a) in Figure 5.30, and both longitudinal joints are tied. This reduces the risk of sympathetic cracking either in the repair or in the adjacent existing concrete at the location of the transverse joint.

An alternative option to (a) is shown in (b) in Figure 5.30. An expansion joint is used at the transverse joint closest to the existing adjacent transverse joint and tie bars are omitted for the portion of longitudinal joint between the expansion joint and the existing adjacent joint. The joint furthest from the existing adjacent transverse joint is a contraction joint and the longitudinal joint is tied up to the existing adjacent joint.

This alternative carries additional risk of sympathetic cracking at the location of the existing adjacent transverse joint or corner / diagonal cracking due to the omission of tie bars along a portion of the longitudinal joint, which would otherwise restrict movement. Depending on the anticipated risk of sympathetic cracking occurring, it may be appropriate to saw cut a joint into the adjacent lane as a risk mitigation response.

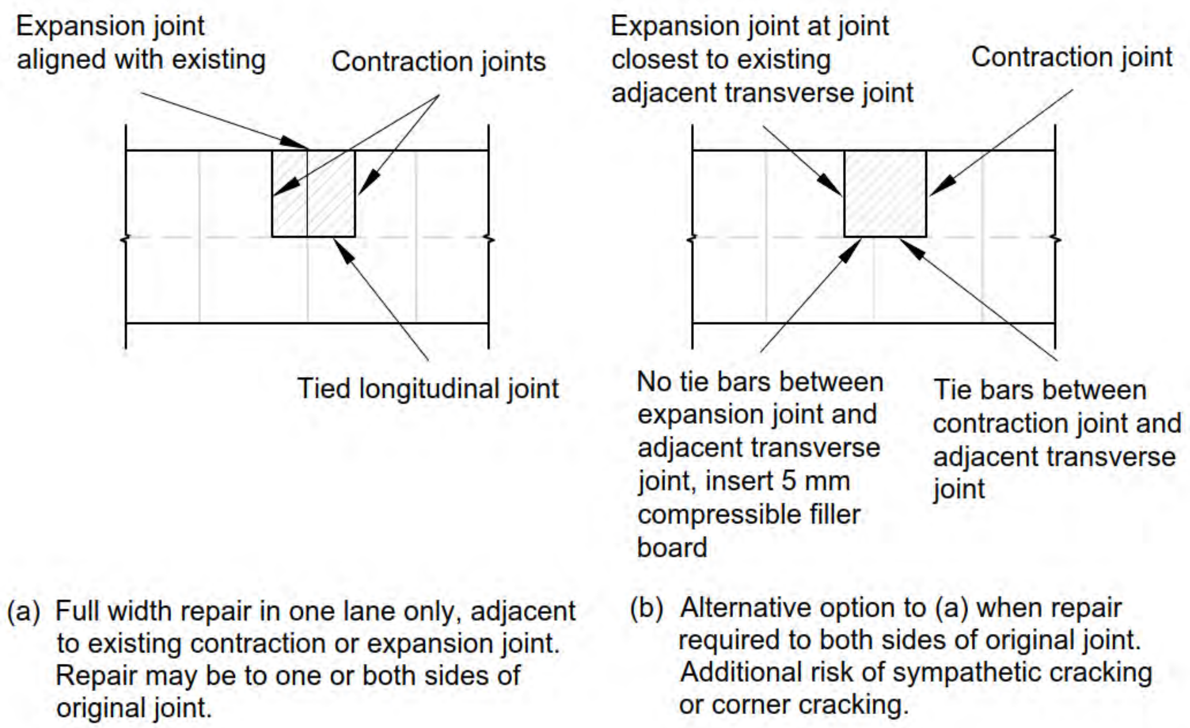


Figure 5.30 Single lane width full depth repair joint arrangements for URC

Repairs spanning the carriageway width

Figure 5.31 outlines joint arrangements for full depth repairs to URC being undertaken across the full carriageway width. Where only one side of the transverse joint is affected, the preferred joint arrangement, as shown in (c) in Figure 5.31, is to replace the original transverse joint with an expansion joint, then the new transverse joint can be a contraction joint.

Where both sides of the transverse joint are affected, the preferred practice, as shown in (d) of Figure 5.31, is to omit the original transverse joint altogether and introduce new transverse joints at each end of the repair across the full carriageway width, incorporating at least one expansion joint. It is common practice to incorporate two expansion joints where there is an increased risk of defects associated with insufficient joint space for thermal expansion within a particular section of the pavement based on the maintenance history.

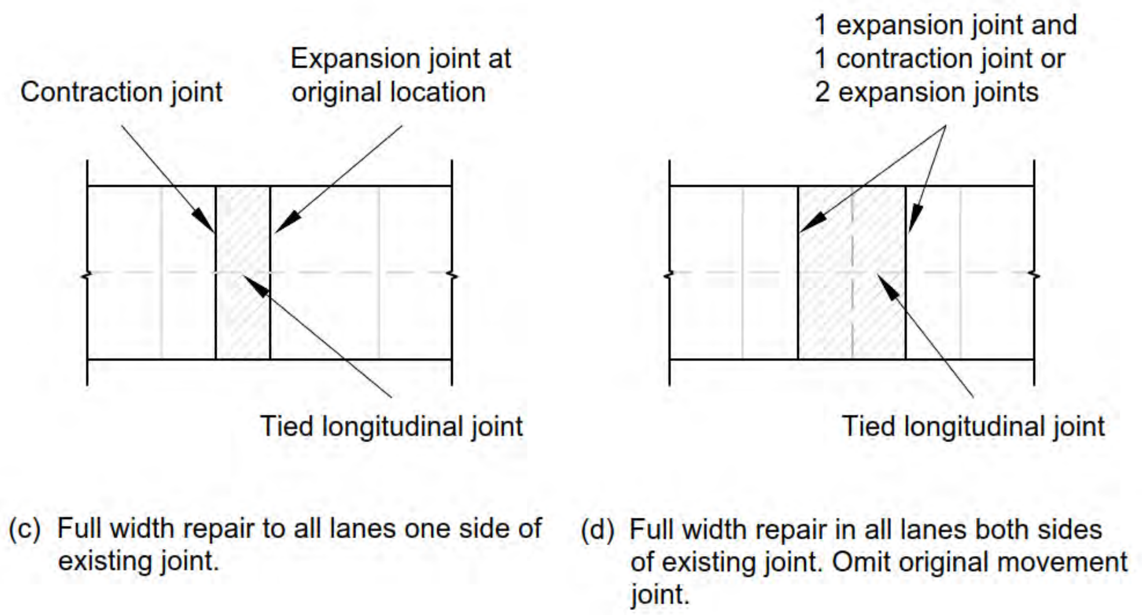


Figure 5.31 Carriageway width full depth repair joint arrangements for URC

5.4.4. Full depth repairs to JRC pavements

Full depth repairs to JRC are much more common than full depth repairs to URC. This is because it is often only necessary and much more economical to undertake a short full depth repair rather than a bay replacement that might be up to 25 m in length. Furthermore, a JRC bay replacement is often not possible in an overnight closure.

Where existing joints are being replaced during full depth repairs to JRC, many of the recommendations for bay replacements (Section 5.4.1) and full depth repairs to URC (Section 5.4.3) also apply to JRC. However, full depth repairs to JRC pavements may be limited to the central portion of the slab such that the repair does include replacement of an existing transverse joint.

When existing transverse joints are not replaced, then there is the problem of how to permit the inevitable initial drying shrinkage contraction in the fresh concrete. If the repair is to just one lane and existing transverse joints are not being replaced, there is no ideal solution. The best practicable arrangement is shown in (a) in Figure 5.32, where tie bars are omitted in the repair and a 5 mm compressible filler board included along the joint so the concrete is free to contract.

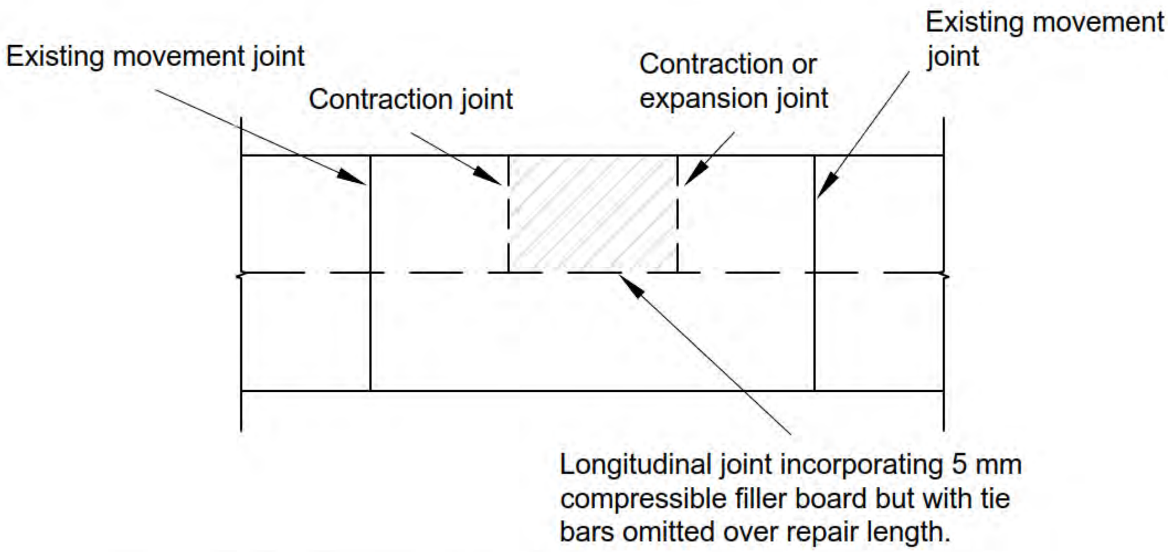
The disadvantages are:

- the edge stiffening effect from a tied longitudinal joint is lost for both the new concrete and the adjacent lane over the length of the repair; and,
- new transverse joints confined to the one lane are liable to result in a sympathetic crack in the adjacent retained lane.

Where existing transverse joints are not being replaced during a full depth repair to URC, the new transverse joints are typically contraction joints. However, the incorporation of one expansion joint, following the URC full depth repair detail in Figure 5.29, is recommended if works are being undertaken in the winter to permit any subsequent expansion that may occur at higher temperatures.

An alternative option to (a) in Figure 5.32, is exposing and connecting the existing reinforcement following the general procedure for full depth repairs to CRCP outlined in Section 5.4.2; with a tied longitudinal joint. However, this procedure is generally more expensive and time consuming.

The problem of sympathetic cracking can be avoided if the repair concrete spans all lanes, as shown in (b) in Figure 5.32. Transverse joints should be placed as detailed in the URC full depth repair in Figure 5.29 and tie bars can be installed at longitudinal joints with the compressible foam omitted. In this scenario, incorporation of an expansion joint is recommended to permit any future expansion of the repair.



(a) Full depth repair to JRC in one lane only, remote from movement joints.

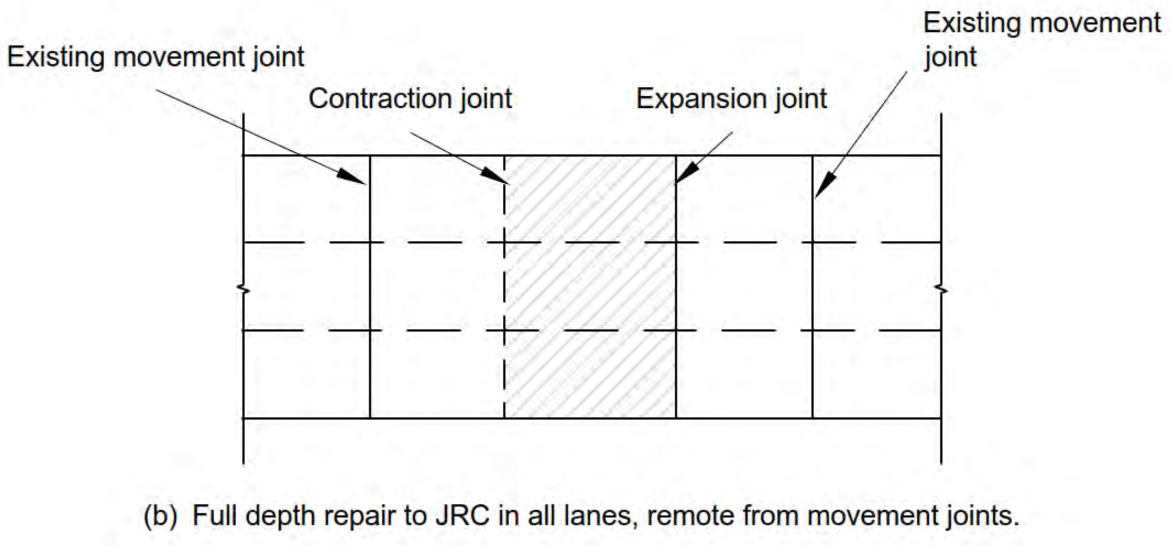


Figure 5.32 Full depth repairs to JRC

Part width, full depth repairs to JRC are not recommended. Whilst they may be more economical to construct, compared with a full width repair to a JRC bay, they offer poorer LCCA due to the much greater risk of poor long-term durability and performance. In particular, the longitudinal joint would need to be tied to provide load transfer across the repair and prevent slab wander, but this would limit the ability of the repair to contract and expand freely, which can often result in early cracking.

5.4.5. Under slab grouting

Under slab grouting involves filling voids below slabs with a cementitious or resin grout to restore slab support and slow the deterioration of the pavement. When combined with slab lifting, under slab grouting can be used to restore the carriageway profile where settlement or stepping has occurred.

Requirements for under slab grouting detailed in Clause 1088 [7] are based on end performance, meaning various techniques can be used provided that the end performance requirements are met.

The process of restoring support to an existing slab by under slab grouting is undertaken by a specialist contractor. The specialist contractor normally has the responsibility for recommending the most suitable type of grouting material and grout insertion method. The choice between options, once the available working window has been established, depends on several factors, as described below. The key steps for under slab grouting are:

1. Selecting the grouting method.
2. Selecting the grouting material.
3. Selecting the hole pattern for injection.
4. Complying with end performance requirements.

Selecting the grouting method

Details of under slab grouting methods vary, but they can be broadly grouped into:

- pressure grouting; and,
- vacuum grouting.

Pressure grouting (Figure 5.33): this technique can use cementitious and resin grout as well as expansive polymer grout depending on the application. Initially, grout is injected through holes in the slab at a set flow rate for a set time based on the assumed void size; this is followed by a slower injection of grout until real time monitoring equipment identifies that the void is filled and the slab is supported by the grout. Uniform injection into voids is critical to ensuring the slab does not flex excessively and crack.

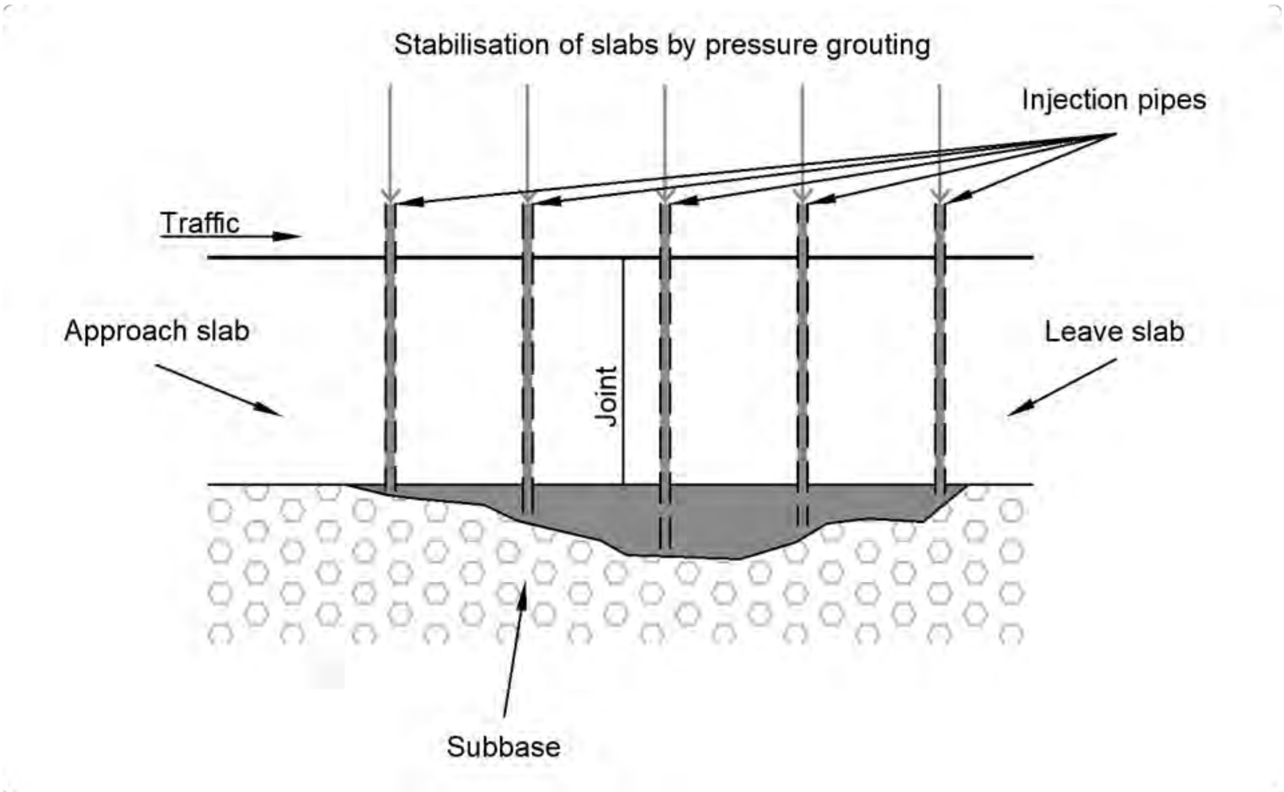


Figure 5.33 Stabilisation of slabs by pressure grouting

Vacuum grouting (Figure 5.34 and Figure 5.35): this technique uses a low-viscosity resin grout that is induced to flow into voids beneath the slab by creating a vacuum. Any water beneath the slab is drawn off before the grout is injected and the induced vacuum combined with the low-viscosity of the grout enables small voids to be penetrated.

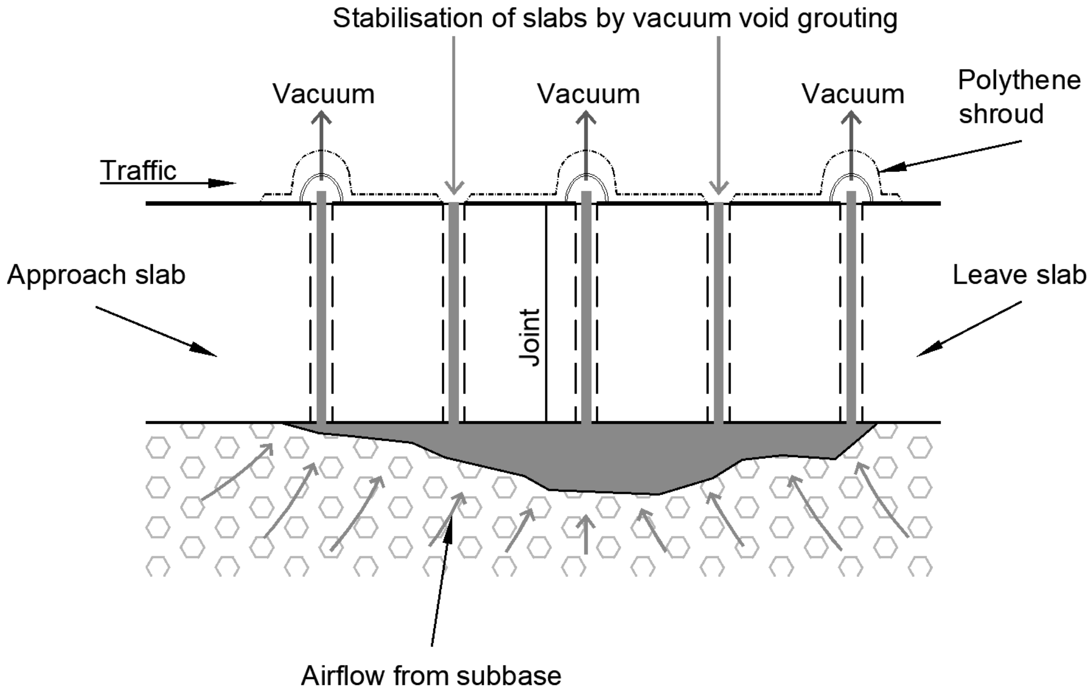


Figure 5.34 Stabilisation of slabs by vacuum grouting

Figure 5.35 Vacuum grouting

Both methods are suitable for filling voids immediately below slabs, but pressure grouting is more suitable for areas where deep voids exist within or below the pavement foundation.

Grouting in the vicinity of service ducts requires careful consideration to reduce the risk of filling service ducts with grout. Vacuum grouting may be more appropriate in this scenario.

The most appropriate grouting method may be limited by the available working window. Other determining factors include the location of the voids and, where slab lifting is required, how this would be achieved and the management of associated risks (see Section 5.4.6).

Selecting grouting material

Under slab grouting is undertaken with polymeric or cement-based grouts. The decision as to whether a polymeric or cement-based grout is required is likely to be dependent on void size and shape and curing needs, and is ultimately a decision for the specialist contractor, who will often use different grouts to meet the application's specific requirements.

The main considerations for selecting the grouting material are:

- **flowability**: the material should be able to penetrate into very thin voids;
- **compressive strength**: the material should be able to withstand stresses caused by traffic and environmental factors;
- **curing time and available working window**: the material may be required to cure during an overnight closure with the area reopened to traffic shortly after the works are complete;
- **durability**: the material should have adequate resistance to fatigue; and,
- **WLC considerations**.

Unless the identified voids are large, cement grouts for under slab grouting often need to be much finer than traditional cement grouts to enable penetration of the grout into very thin voids.

Typically, polymeric-based grouts are used. Whilst they have a lower compressive strength compared to cement-based grouts, they offer advantages, including:

- ability to be used in all weather conditions;
- rapid curing;
- relatively high tensile strength;
- lightweight;
- low expansivity; and,
- insensitivity to moisture.

During the grouting process, the contractor may add an amount of inert filler to the grout as packing, dependent on the void size and product requirements.

Selecting the drill hole locations for grout access

Selection of the optimal locations of the grout insertion holes are based on the:

- size and shape of the area requiring treatment;
- pavement type;
- slab condition;
- transverse joint spacing; and,
- flowability of the material to be injected.

When a multiple hole pattern is used, the distance between holes is determined based on the estimated distance the grout will flow after entering each hole to ensure that the void is completely filled. The contractor may need to undertake additional onsite checks to confirm the presence of the voids during the slab drilling process; and hole pattern might be adjusted and / or additional holes may be drilled in order to effectively fill each void.

End performance

The end performance requirements and compliance with the specification should be determined by an FWD survey over the treated slabs / joints (see Section 2.2.1.1). This is unlikely to be practicable in the same shift, so the works programme should allow for the completion of this testing within a subsequent closure.

5.4.6. Slab lifting

Where stepping (Section 3.3.7) or depressions / settlement (Section 3.2.1) of the pavement has occurred, slab lifting carried out in conjunction with under slab grouting (see previous Section 5.4.5) is a potential treatment option to lift the level of individual or multiple slabs to realign the edges with the adjacent unaffected slabs. The slab lifting method used is dependent on the under slab grouting technique.

Pressure grouting uses the lifting force created by injection of material underneath the slab to lift the slab. This fundamentally carries a risk of over lifting occurring. Pressure grouting should always be undertaken using real time monitoring equipment to measure levels. Experienced contractors will lift in increments to achieve finished level requirements without over lifting the slab. Over lifting, depending on the magnitude of the resultant step, may require a bay replacement (Section 5.4.1) treatment.

When vacuum grouting is used, the slab is pre lifted by a lifting frame spanning the repair area (see Figure 5.36 and Figure 5.37).

Figure 5.36 Slab lifting apparatus used in conjunction with vacuum grouting

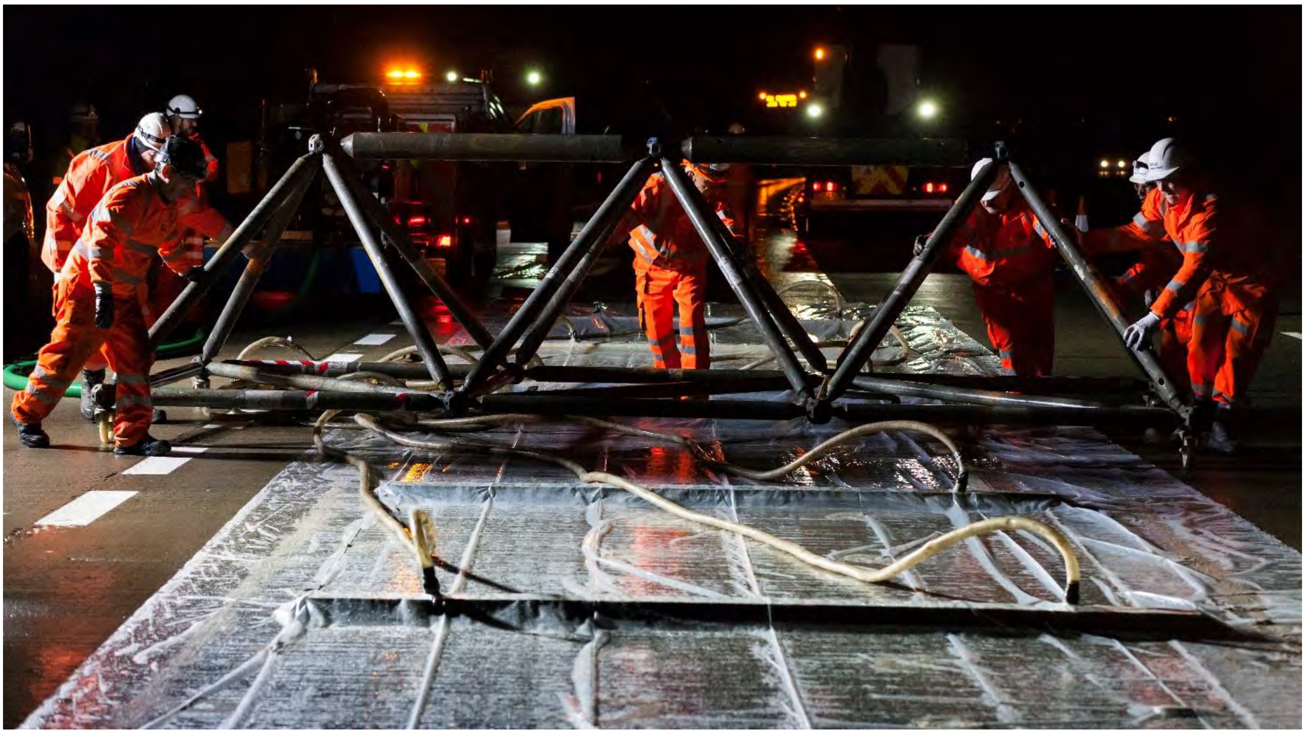


Figure 5.37 Slab lifting apparatus

Vacuum grouting using a lifting frame carries less risk when working in short closures. The grout does not provide any lifting force, meaning that over lifting cannot occur. The slab pre lifting is achieved by connecting the slab to a frame straddling the bay and hydraulically jacking it to the required level a few millimetres at a time. Whilst the slab is still connected to the lifting frame, the void created underneath is filled with the grout. However, pre lifting slabs with frames may not be physically possible in limited working areas.

When a substantial length of slab is lifted, it may be necessary to stitch tie bars across the longitudinal joint to stop it from opening subsequently using the same principle as crack stitching discussed in Section 5.4.8.

Bump cutting (see next Section 5.4.7) to restore regularity of the surface may be needed after lifting, as a minor step between slabs could be created following the lifting operation. Provision for bump cutting should be made with any slab lifting works, with a straight edge used to check regularity and identify joints for bump cutting.

5.4.7. Bump cutting

Bump cutting is the process of removing high spots, typically at joints, in jointed concrete pavements to improve the surface regularity and ride quality. This may be necessary to address stepping (Section 3.3.7), or as a finishing treatment following a thin bonded repair (Section 5.3.2) or where slab lifting followed by under slab grouting (see previous section) has been undertaken.

Bump cutting does not necessarily improve skidding resistance or surface texture of a pavement, but it is typically carried out with diamond grinding or bush hammering equipment.

5.4.8. Crack stitching

Longitudinal cracks can be repaired by crack stitching. Crack stitching is a holding repair to slow the rate of deterioration of the pavement; it essentially converts a crack into a tied warping joint that will allow the crack to 'hinge' at that point, whilst maintaining load transfer through aggregate interlock and minimising further growth of the crack. However, the success of the repair will be dependent on the causation of the longitudinal crack (see Section 3.3.6), particularly if there remains poor or non-uniform support from the foundation.

Crack stitching is suitable for longitudinal cracks in:

- URC that still have a degree of aggregate interlock (typically < 0.5 mm wide); and,
- JRC where the reinforcement is intact (typically < 1.5 mm wide).

Crack stitching does not allow for movement, so the technique is not suitable for transverse (working) cracks. If used on transverse cracks, a new crack will likely develop near the stitched crack repair.

The same ‘crack stitching’ technique can be used for tie bar ‘retrofitting’ at longitudinal joints to restore load transfer in the event of existing tie bars having yielded, or to potentially improve load transfer.

There are two crack stitching techniques available:

- Type 1 repair - staple tie bar repair (or slot stitching) (see Figure 5.38); and,
- Type 2 repair - diagonal tie bar repair (or cross stitching) (see Figure 5.39).

Type 1 repair - staple tie bar repair (slot stitching)

Staple crack stitching (see Figure 5.38) involves creating 25 - 30 mm wide by 470 mm long slots at 600 mm centres at right angles to the line of the crack, with a slot depth such that the tie bars lie between one third and one-half of the slab depth below the surface when bedded. Holes of 25 - 30 mm diameter and 50 mm deep are drilled at each end of the slot and the slots are then cleaned with oil-free compressed air.

Once dry, the slots are primed and staple tie bars are placed onto beds of resin mortar and covered with the same material to a minimum depth of 30 mm. The sides of the slots are then cleaned of any loose material and filled with resin or cementitious mortar.

After the repair material has cured, a groove is sawn or routed along the line of the former crack and sealed in the same manner as a joint (see Section 5.2.1). The tie bar alignment should be approximately perpendicular to the joint or crack.

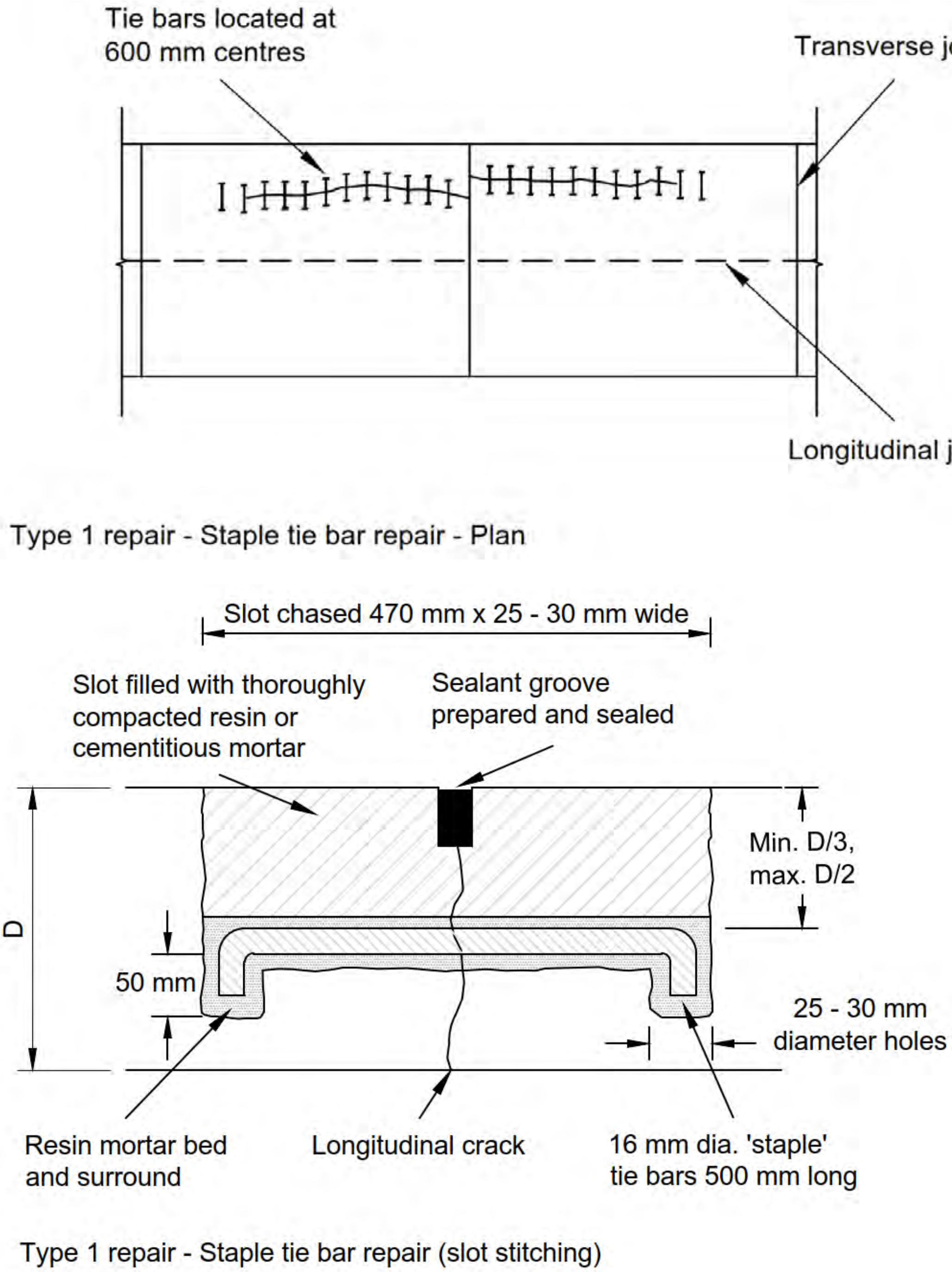


Figure 5.38 Type 1 repair - Staple tie bar repair (slot stitching)

Type 2 repair - diagonal tie bar repair (cross stitching)

Diagonal tie bar repairs (see Figure 5.39) involve the drilling of diagonal holes across the crack to intersect at mid-depth of the slab and about 26 ° to the slab surface. These holes are typically spaced every 600 mm along the crack with alternate entry points on opposite sides of the crack. The entry point of the repair should be at a distance from the crack equal to the slab depth (see dimension D in Figure 5.39).

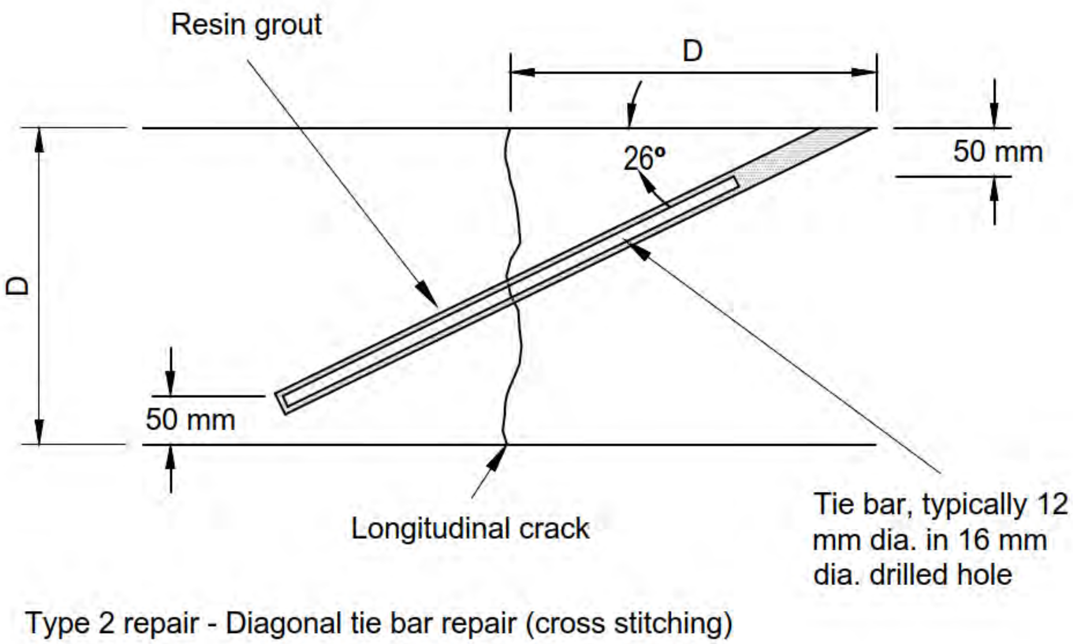


Figure 5.39 Type 2 repair - Diagonal tie bar repair (cross stitching)

The length of tie bar used depends on the slab depth. It should be enough to retain 50 mm cover at the bottom of the slab. Deformed 12 mm diameter tie bars are generally used in 16 mm holes; however, larger diameter tie bars may be appropriate dependent on trafficking conditions (including level of traffic and proximity to wheel tracks) and slab thickness. The USDoT FHWA Concrete Pavement Presentation Guide recommends up to 25 mm diameter tie bars depending on slab thickness [19].

The tie bars are notched at a point that will be 50 mm below the slab surface when the bars are fully inserted so they can be broken off after fixing. Resin grout is used to fix the tie bars as the material needs to harden quickly, before any crack movement can disrupt the repair. Each hole is filled with a sufficient amount of resin grout so that, with the tie bar inserted, the grout level reaches a level of 25 mm below the hole. Once the grout has set, the length of tie bar above the notch is broken off by twisting. Any bars that rotate after the grout should have hardened need to be withdrawn and the hole re-drilled.

5.4.9. Full depth corner repairs

Full depth corner repairs have been used historically as a cost effective method of repairing smaller corner cracks and corner spalls that have deteriorated beyond the upper third of the slab. However, experience indicates that a significant proportion of full depth corner repairs fail through separation from the original concrete or local settlement well in advance of any failure in the adjacent existing pavement.

Therefore, full depth repairs (see Sections 5.4.3 and 5.4.4) or bay replacements (see Section 5.4.1) are preferred on the basis of durability and LCCA. As such, it is considered a non-standard repair option and requires a departure from standard prior to it being used on the SRN.

Figure 5.40 outlines the minimum and maximum dimensions for full depth corner repairs. As large a chamfer as possible should be provided across the corner as shown in Figure 5.40 to reduce the risk of a crack subsequently developing across the slab from that point. It may not be possible to extend the saw cuts around the corners of the repair through the full slab depth because of the chamfer. This may mean using other methods to carefully break out the concrete to achieve the vertical face required in the corners. Particular care should be taken to avoid damaging the remaining top edges of the slab.

It is important that the initial drying shrinkage movements of the repair concrete are not restricted in any way. The repair should not inhibit contraction or expansion movement in the existing slab. Therefore, it is recommended, as shown in Figure 5.40, that:

- dowel or tie bars are omitted in the chamfered edge of the repair;
- dowel or tie bars are omitted in the longitudinal edge of the repair; and,
- a 5 mm compressible joint filler board is installed around the perimeter of the repair.

A variation on the detail in Figure 5.40 involves a diamond-shaped repair on plan, i.e. the chamfers at the corners of the repair are extended to intersect at the transverse and longitudinal joints respectively. This eliminates repair edges perpendicular to the joints. It is reported that such corner repairs can last over 14 years under heavy traffic. Precast circular corner repairs have also been reported as successful [50].

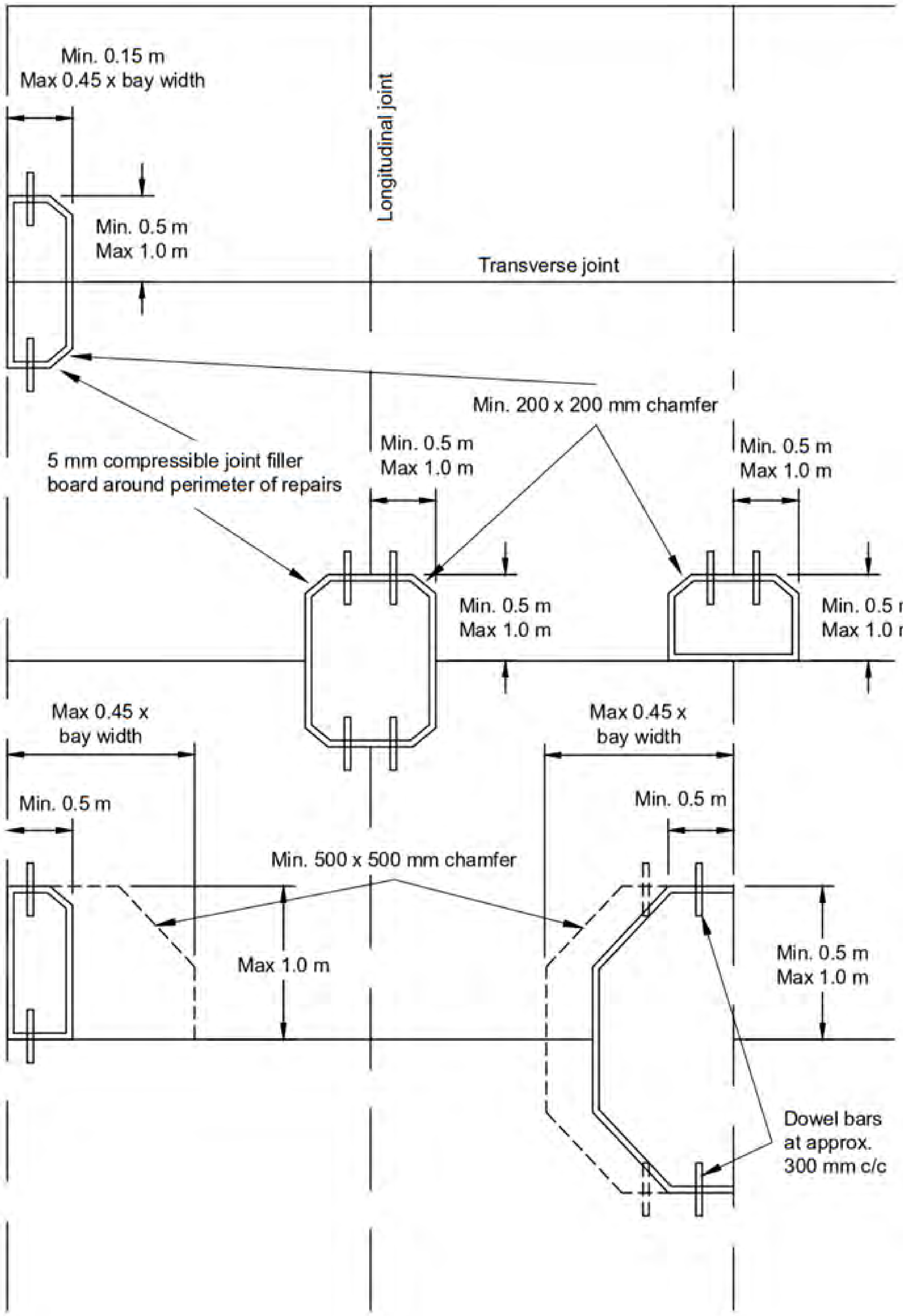


Figure 5.40 Full depth corner repairs

5.4.10. Retrofitting dowel bars

Dowel bar retrofitting involves installing new dowel bars as a holding repair to slow the rate of deterioration of the pavement by restoring load transfer at transverse joints after the original bars have either yielded or corroded. This is not a common technique used on the SRN, a full depth repair or bay replacement is preferred (see Sections 5.4.1, 5.4.3 and 5.4.4). Therefore, it requires a departure from standard.

Retrofitted dowel bars have been shown to last for 10 to 20 years [19]. However, close attention to the quality of construction procedures and material handling during construction is required.

The key steps for retrofitting dowel bars are:

1. Cutting slots across the joints
2. Fitting new dowel bars at mid depth of the slab
3. Backfilling slots with suitable materials
4. Re-establishing joints and joint sealing

Figure.5.41 outlines the typical arrangement for retrofitting dowel bars. The slot depth should be half of the slab thickness plus 20 - 30 mm so that the dowels can be positioned at the mid-depth of the slab.

Dowel bars in accordance with Clause 1011 are used with extension caps fitted at both ends of the dowels to allow for horizontal movement of at least 6 mm at each end. It is critical to ensure that the new dowel bars are aligned horizontally and parallel to the pavement centreline. To achieve this, they are mounted on chairs so that the infill material can flow around and fully support the dowel. Three dowel bars are typically used per wheel track.

A compressible joint filler board is used to keep the joint continuous at the existing joint groove and to allow horizontal expansion movement. The insert is extended into the slot sides and bottom to avoid mortar entering the joints.

Most failures of dowel bar retrofitting appear related to the backfill materials used in the slots [19]. The repair may debond from the original concrete, crack or spall within the dowel bar slots. In general, material suitable for shallow repairs (see Section 5.3.2.2), such as low shrinkage cementitious mortars, have been reported as acceptable. Material testing and modification should help to reduce unfavourable shrinkage. Proprietary materials including resin mortars may be considered. Primers are often necessary to promote a bond with the concrete.

FWD, GPR and ultrasound (see Section 2.2.1) and coring surveys (see Section 2.2.2.1) can be performed to help identify joints that require treatment, with repeat FWD surveys to confirm the success of the retrofitted dowel bars.

Bay replacements using precast concrete slabs can use a similar technique for retrofitting dowel bars at transverse joints. Alternative techniques for partial dowel bar retrofit in precast concrete slabs are also available, which include fixing dowel bars either to the existing or the precast slabs before panel placement and creating slots in the counterpart only. Section 5.4.1.2 contains more information on bay replacements using precast concrete slabs.

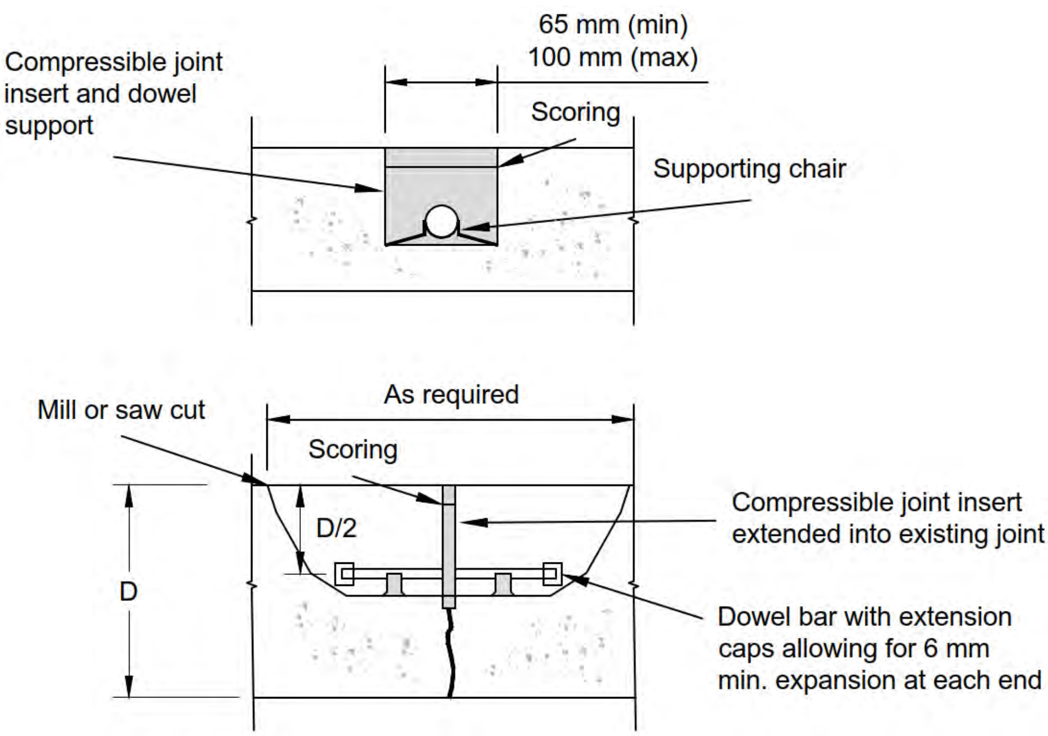


Figure.5.41 Typical arrangement for retrofitting dowel bars

6. Repair materials

The use of concrete in maintenance, traditionally, has been limited by the need for a long period of curing (the process of strength gain) in advance of opening to traffic. Trafficking concrete before it has gained sufficient strength (25 N/mm^2 in accordance with Series 1000 of the MCHW [11]) can result in permanent damage and poor long-term durability. A period of strength gain of between 7 and 28 days historically meant that extended lane or carriageway closures were required when undertaking repairs with concrete materials.

Increases in traffic on major roads and the need to keep lane closure durations to a minimum has driven advancements in the mixture design of concrete to gain sufficient strength in hours rather than days. In addition, the need to open roads to traffic has led to alternative materials to concrete being considered for maintenance works in some cases.

The available repair materials for concrete pavements generally fall under the following classifications:

- pavement concretes (see 6.1);
- early strength concretes (see 6.2);
- proprietary cement mortars (see 6.3);
- resin mortars (see 6.4); and,
- polymeric materials (see 6.5).

Repair materials that have specific applications, such as joint sealants and under slab grouts, are covered in Chapter 4. This chapter gives an overview of the properties of those repair materials that can be utilised by more than one type of repair to concrete pavements.

Whilst different repair material classifications can be used for the same repair technique, it is important that the properties of the repair material are chosen to be compatible with the type of defect being repaired. Refer to Chapters 3 and 4 for defect types, causation and diagnosis. The premature failure of repairs can often be linked to inappropriate selection from the available choice of repair materials, particularly thermal incompatibilities leading to differential thermal movement of the parent concrete and the repair at the location of shallow repairs and thin bonded repairs.

Concrete repair materials are often proprietary, so their properties will be product specific and may be unique. To aid in the decision process to identify the optimum repair material for the specific project conditions, the typical properties and general guidance are given in the following sections of this chapter specific to each classification of repair material.

Specialist material suppliers and contractors should be approached directly for product specific performance properties and evidence of previously successful applications of their proprietary products. Care is required as performance can vary between products from different suppliers. Where proprietary products are used, it is important that the installation can be (and is then documented to have been) undertaken in accordance with any product specific requirements.

The selection of an optimum repair material is based on characteristic factors, including:

- defect type and location (i.e. within wheel track zones);
- repair dimensions;
- material properties:
    - **trafficability (curing time):** time required for the material to develop sufficient strength before opening to traffic. This is often the primary factor in choosing the repair material;
    - **strength:** additional to compressive strength, tensile strength and bond strength are critical factors;
    - **flexibility:** rigid materials have good compressive and tensile strength properties but are brittle. Flexible materials do not offer equivalent strength to rigid materials, but may resist cracking if subject to dynamic movement, so may be appropriate for spanning cracks or where movement is expected;
    - **placement conditions:** ambient temperature and moisture condition;
    - **thermal compatibility:** ensures the repair material expands and contracts at a similar rate as the existing pavement;
    - **shrinkage:** early life shrinkage during curing can cause cracking and be detrimental to bond development. Factors affecting shrinkage are further discussed in this chapter. This is particularly relevant to thin bonded and shallow repairs;
    - **bond:** repair materials have to maintain bond to the substrate to prevent water ingress and to prevent the repair delaminating then breaking up; and,
    - **surface characteristics:** properties to achieve skid resistance can be a safety critical factor in highway applications;
- material performance (durability);
- initial and whole-life cost;
- environmental factors such as carbon footprint; and,
- project specific conditions:
    - closure time;
    - project size; and,
    - availability of experienced contractors for installation.

Typical characteristics for each of the repair material classifications are summarised in Table 6.1.

Table 6.1 Typical characteristics for each of the repair material classifications

Material Classifications	Pavement concrete	Early strength concrete	Proprietary cement mortar	Resin mortar	Polymeric materials
Type	Rigid	Rigid	Rigid	Rigid	Flexible
Trafficability (curing time)	2 - 7 days	3 - 18 hours*	2 - 24 hours*	3 - 16 hours*	10 - 120 mins*
Compressive strength (N/mm^2)	Min. 40	55 - 85*	20 - 60*	55 - 100*	N/A
Laying temperature	5 - 30 °C	5 - 30 °C*	5 - 40 °C*	10 - 30 °C*	0 - 50 °C*
Thickness	> 40 mm	> 40 mm	10 - 100 mm*	5 - 50 mm*	20 - 100 mm*
Drying shrinkage	Moderate	Moderate*	Moderate*	Low-moderate*	Low
Material handling and workability	Easy	Experienced contractor required	Moderate	Moderate	Easy
Material cost	Low	High	High	Moderate	High
Notes: *Product specific					

6.1. Pavement concrete

Pavement concrete is designated or designed concrete that is specified and produced in accordance with British and European standards.

The constituents of pavement concrete should comply with the relevant clauses of BS EN 206, BS 8500-1, BS 8500-2 and BS EN 13877-1 and the relevant requirements in Series 1000 of the MCHW [11]. The requirements for specification, performance, production and conformity are given in BS EN 206. BS 8500 is the complementary British standard to BS EN 206, containing additional UK provisions. For cast-in-place concrete pavements, reference can be made to BS EN 13877.

A range of cement types can be used in pavement concrete. Common cement types that are in accordance with BS EN 197 include:

- Portland cement CEMI;
- Portland-slag cement CEM II/A-S or CEM II/B-S;
- Portland-fly ash cement CEM II/A-V or CEM II/B-V;
- blast furnace cement CEM III/A or CEM III/B; and,
- pozzolanic cement CEM IV/A.

Cement combinations may be used in accordance with BS 8500-1, including:

- Portland cement CEMI to BS EN 197 with 6 - 35 % ground granulated blast furnace slag (ggbs) to BS EN 15167-1 & -2; and,
- Portland cement CEMI to BS EN 197 with 6 - 35 % fly ash to BS EN 450-1 & 2.

Portland cement CEM I with pozzolanic additive, having product acceptance scheme certification, is permitted [11]. Alternative cements may be used to meet specific project needs, such as the requirements for high early strength, chemical resistance, and / or to reduce the risk of drying shrinkage cracking, but a departure from standard may be required for their use on the SRN for those not complying with the requirements of Clause 1080 [7].

Replacing CEM I partially with fly ash and ggbs can reduce temperature rise during hydration and minimise the risk of drying shrinkage cracking. This may be beneficial for hot weather but requires careful consideration in cold weather due to the slower stiffening and strength development. Cements or combinations containing more than 55 % ggbs might not be suitable for use in surface slabs due to the possibility of scaling in the top few millimetres. The risk of scaling is linked to potential for bleeding to occur during construction. Bleeding is a form of segregation where some of the water in the concrete rises to the surface of freshly placed material which can result in a surface weakness in the form of laitance. This can be mitigated through mix design optimisation (e.g. an increased very fine aggregate content or use of polypropylene fibres). This phenomenon does not occur with drier mixes such as RCC.

Aggregates for paving concrete should comply with BS EN 12620 and be non-frost susceptible. The maximum nominal size of coarse aggregate should not exceed 40 mm. For JRC and CRCP, where the spacing between the longitudinal reinforcing bars is less than 90 mm, the maximum aggregate size should not exceed 20 mm. Limestone aggregate has a lower coefficient of thermal expansion than other natural aggregates. Hence wider joint spacings are allowed for concrete using limestone coarse aggregate.

Most external pavements and hardstandings are subject to exposure class XF4 for freezing and thawing resistance [7]. Therefore, concrete in at least the top 50 mm of surface should incorporate an air-entraining admixture complying with BS EN 934-2, with the exceptions of:

- C40/50 concrete surface slabs;
- C32/40 concrete slabs with at least a 30 mm TSCS overlay;
- C35/45 concrete slabs with at least a 20 mm TSCS overlay; and,
- proprietary cement mortars.

BS 8500 provides recommended limiting values for composition and properties of concrete to resist freezing and thawing, including air content requirements to achieve XF4.

For reinforced concrete pavements, the following exposure classes generally apply in addition to exposure Class XF4:

- XC3/4 for carbonation-induced corrosion; and,
- XD3 for chlorides-induced corrosion other than sea water.

The relevant durability recommendations are in BS 8500 Table A.4. Considering that pavements and hardstandings often have a design life less than 50 years and the reinforcement usually serves as a crack control measure rather than for structural purposes, the relevant requirements may be relaxed according to BS 8500 A.4.4. However, as reported in TR66 [64], where reinforcement is being used for structural purposes and / or where a high percentage of reinforcement is used, such as in CRCP, designed concrete meeting XD3 and XF4 requirements should be used. For reinforced concrete slabs that are exposed, or have thin asphalt overlay, galvanic anodes are available as an additional protective measure to delay reinforcement corrosion.

Where a high level of sulfates is identified in the ground assessment, concrete composition can be specified, such as limiting the water / cement ratios and controlling the types and quantities of cements and combinations used, to produce concrete with suitable sulfate resistance. See BRE special digest 1 [51] and BS 8500-1 & -2 for more details.

6.1.1. Applicability

Pavement concretes can be used for:

- shallow repairs (40 mm up to one third slab thickness, see Section 5.3.2.2);
- full depth repairs (see Sections 5.4.2, 5.4.3 and 5.4.4); and,
- bay replacements (see Section 5.4.1).

Pavement concrete is the standard material used for both constructing and repairing concrete pavements. Curing time is the biggest limitation for using pavement concretes. Typically, concrete has to be isolated from traffic for two to seven days until a compressive strength of 25 N/mm^2 is achieved. When concrete is being overlaid, a minimum 20 N/mm^2 strength is recommended.

6.1.2. Admixture technology

Concretes with low water / cement ratios will typically gain strength more rapidly and have higher overall strengths versus concretes with higher water / cement ratios. However, the practical issue with a low water / cement ratio is reduced workability. Water-reducing admixtures (plasticisers and superplasticisers) can be used to maintain the workability of concrete mixtures with a low water / cement ratio. The admixtures should conform to BS EN 934.

Accelerating admixtures can be used to increase the rate of strength gain of concrete mixtures. Where accelerating admixtures are used, calcium chloride or chloride-based admixtures must be avoided where the concrete contains steel reinforcement, dowel bars or tie bars as these admixtures can increase the possibility of the steel corroding.

6.1.3. Considerations

Provided that adequate curing time is available, pavement concretes can be used for full depth repairs and bay replacements, where it will likely be the most cost-effective long-term repair material. Pavement concretes are compatible with all types of joint filling materials and the physical properties will be similar to the surrounding concrete.

Where repairs are being undertaken with pavement concrete, the following aspects should be considered:

- assessment of the extent of unsound concrete by sounding techniques (Section 2.2.1.6);
- design and installation of suitable dowel bars or tie bars across formed joints;
- suitable concrete mix design; and,
- adequate curing versus site temperatures.

Cubes made, stored and tested in accordance with BS EN 12390 will not necessarily reflect the in situ material strength. The strength gain in the pavement will vary with temperature and climatic conditions and is usually around 15 % less than the strength of the cubes stored under laboratory conditions [52]. Temperature-matched curing (TMC) of the cube specimens prior to testing would give more representative results if time allows.

Pavement concretes can be used for shallow patch repairs in concrete pavements, but considerations should be given to the technical challenges, such as:

- short-term risk of debonding caused by the initial drying shrinkage of the repair material;
- long-term risk of debonding caused by differential daily and / or seasonal thermal expansion between the repair material and the substrate concrete; and,
- adequate preparation of repair area to ensure full bonding is achieved at the interface.

6.2. Early strength concrete

As outlined in Section 6.1, pavement concretes used in repairs may take 2 to 7 days to achieve the minimum recommended strength prior to trafficking. Early strength concretes enable repairs to be undertaken within night time or weekend closures. They can be used towards the end of longer schemes, where re-opening of the road would otherwise be delayed by a period of days while the final concrete pour cures.

Various proprietary early strength concretes are available with different performance. Proprietary products combined with an accelerated curing regime can achieve a strength of 25 N/mm^2 in as little as 3 hours.

Early strength can be achieved in two ways:

- pavement concretes using admixture technology, see Section 6.1.2. Standard rapid-hardening Portland cement conforming to BS EN 197 with a cement content of 400 kg/m^3 or more can achieve 25 N/mm^2 in less than 18 hours, and with accelerated curing this can be reduced further; and,
- using alternative cement technologies (see Section 6.2.2).

For early strength concretes, angular aggregates from crushed rock are preferable for added early stability, although some experience suggests that round flint aggregates are preferred in certain UK regions for their reduced absorption and availability. As some alternative cements may have high alkali contents, the requirements for the prevention of alkali-silica reaction (ASR) will apply and non-reactive aggregates should be specified.

Pozzolans in the form of pulverised fuel ash (PFA) and microsilica can be incorporated, as they produce a denser concrete with fewer voids. Microsilica increases early strength, while PFA improves workability and allows a lower water / cement ratio, increasing the long-term concrete strength.

Air entrainment is not required for early strength concrete provided they achieve a minimum strength class of C40/50, which means that the concrete provides the required resistance to freezing and thawing. Air entraining admixtures are likely to reduce concrete strength and increase setting and stiffening time.

6.2.1. Applicability

Early strength concretes can be used for:

- shallow repairs (40 mm up to one third slab thickness, see Section 5.3.2.2);
- full depth repairs (see Sections 5.4.2, 5.4.3 and 5.4.4); and,
- bay replacement (see Section 5.4.1).

6.2.2. Alternative cement types for early strength concrete

Cement types other than those covered by EN 197 may be used to achieve high early strength in concrete. Some are not covered by the common cement types in BS EN 197 or BS 8500; however, they can be used on the SRN with product acceptance scheme certification in accordance with Clause 1080 or under a departure from standard. They are not typically compatible with other cement replacements, such as PFA, as the combination can be detrimental to the early strength gain. Examples of alternative cement-based concretes include:

- calcium sulfo-aluminate cement-based concrete;
- calcium aluminate cement-based concrete; and,
- calcium sulfate (gypsum) based concrete.

Other alternative cement-based concretes may be available.

6.2.2.1. Calcium sulfo-aluminate cement-based concrete

Calcium sulfo-aluminate (CSA) cement-based concrete has been used on the SRN for bay replacements and full depth repairs. CSA cement-based concrete is usually produced at site using volumetric mixers due to its rapid setting time. It can develop a trafficable strength of 25 N/mm^2 in approximately 2 hours with an approximate 20 minutes working time before concrete starts to harden; however, this depends on the water / cement ratio used.

The rate of strength gain is sensitive to the ambient temperature. Hydration occurs more rapidly when ambient temperature is 20 to 40 ˚C and is expected to be slower at low temperatures. It is important to engage with the supplier to understand the anticipated strength gain profile of CSA based concrete if site temperatures are expected to be lower than the manufacturer's recommendations.

There are two types of CSA - calumex and belitic. The former replaces European standard cement partially at a ratio of approximately 1:2. The latter replaces European standard cement entirely.

The production of CSA produces less CO_2 compared to Portland cement and other alternatives, such as calcium aluminate cement (see below). Hence, it offers an additional environmental benefit in terms of reduced carbon footprint that may add value to its selection on a particular project.

6.2.2.2. Calcium aluminate cement-based concrete

Calcium aluminate cement-based concrete (also known as high alumina cement (HAC) concrete) has not been used on the SRN at the time of this CPMM publication. Historic case studies have evidenced occasions of structural failure. The use of calcium aluminate cement conforming to BS EN 14647 can produce concretes with good sulfate and acid resistance, resistant to high temperatures and exceptionally rapid hardening.

The hydration of calcium aluminate cement undergoes a process called "conversion" where some of the rapid early strength gained at low to normal ambient temperature (less than 40 °C) is lost after several days or many years depending on the temperature, before stable long-term hydrates develop. The process is inevitable and the minimum strength after the conversion process should be used for the long-term design. After an initial rapid strength gain, calcium aluminate cements exhibit a normal (i.e. similar to pavement concrete) long-term curing rate. They exhibit low shrinkage and maintain their initial curing rate in colder weather.

6.2.2.3. Calcium sulfate cement (gypsum) based concrete

Calcium sulfate (typically in the form of gypsum) is added to the other constituents of cement to provide setting times of 20 to 40 minutes meaning it could allow for traffic opening in as little as 1 hour. However, these concretes are not recommended for use in pavements with reinforcement, dowels or tie bars because the presence of free sulfates in a typical gypsum mixture may promote steel corrosion. For this reason, calcium sulfate cement is not suitable for use on the SRN.

6.2.3. Considerations

Where repairs are being undertaken with early strength concretes, the following aspects should be considered (the first four bullet points below are consistent with 6.1.3 for the use of pavement concretes):

- assessment of the extent of unsound concrete by sounding techniques (see Section 2.2.1.6);
- design and installation of suitable dowel or tie bars across formed joints;
- suitable concrete mix design;
- adequate curing versus site temperatures;
- cost versus lane closure time;
- batching (mixing) plant, personnel (labour intensive, experience needed), quality control; and,
- environmental impact including carbon footprint.

The need for early strength concretes is often driven by tight construction programmes and / or short time windows for traffic management to minimise travel disruption. However, the material cost can be up to six times that of pavement concretes.

The use of mobile volumetric type mixers for onsite batching is common for early strength concretes (see Figure 6.1). Batch volumes for repairs are usually low and most works are undertaken at night with tight programme durations. Onsite mixing generally reduces potential wastage and avoids delivery complications. It also removes the transportation period of the concrete to site, ensuring the concrete is delivered and installed within the workability window of the concrete, which can be just a few hours for some early strength concrete mixtures. In contrast, offsite batching carries an increased risk of the early strength concrete setting in transit or being unworkable when it arrives on site.

Early strength concrete is placed and finished in a similar way to pavement concrete. However, due to the quick-setting nature of the material, the work needs careful planning. Placing early strength concrete is a labour intensive process, and all operatives on site need to be aware of their responsibilities to be able to finish concrete installation within the workability window of the concrete. Experienced operatives are a necessity.

The increased hydration rate in early strength concretes can release greater heat on hydration and therefore increases the risk of developing early life thermal cracking in concrete. This is not normally associated with pavement slabs but should be considered for large thick pours. Refer to CIRIA report C660 [65] for details. The risk can be mitigated by:

- proper selection of materials and mix design;
- planning construction sequences to avoid extreme temperatures and weather conditions;
- use of insulation to reduce thermal gradients, such as using curing blankets; and,
- incorporation of reinforcement and appropriate movement joints.

Figure 6.1 Placement of early strength concrete for bay replacements

Consideration of the ambient temperature and weather (rain, wind, snow) conditions anticipated during the works is essential as the rate of strength development can decrease significantly with cold, wet and / or windy conditions. Early strength concrete to be placed during a cold night would require a different mixture design to one being placed in warm daytime or summer temperatures.

Care should be taken to encourage uniform curing through the depth of the slab. As an alternative measure to a last-minute change to the mixture design, mitigation of an unplanned low temperature risk may be achieved by the provision of insulated blankets or tenting and heating, to reach the minimum curing temperature (Figure 6.2).

Figure 6.2 Tenting and heating of bay replacements with early strength concrete to increase strength development rate

In hot weather, it may be difficult to place some quick setting materials as they harden rapidly. Retarders can help in delaying the setting time, but slow setting concrete may be more suitable in this case.

Strength gain assessments are critical to ensure the concrete has reached sufficient strength prior to re-opening. It is critical that any non-destructive test method used to assess strength is correlated with the mix through trials where curing is undertaken in representative temperature conditions. If the compressive strength is estimated using cubes taken from site at the time of placement, these should be subject to temperature-matched curing (TMC). Curing regimes and strength assessments for concrete are further discussed in Section 5.4.1.1.

6.3. Proprietary cement mortars

Ordinary cement mortars, made up of cement, sand and water, are not typically suitable for use as surface repair materials as they are unlikely (without a coarse aggregate content) to achieve the required strength and durability. Proprietary cement mortars are available which may be fit for purpose; they require less skilled installation when compared with resin mortars owing to their workability.

Proprietary rapid curing cement mortars can be subject to trafficking within as little as 2 hours following placement. They are often pre-bagged (Figure 6.3), incorporating additives and admixtures to offer improved strength gain and higher compressive strength characteristics versus conventional cement mortars. They require the addition of water and dependent on the product used, can be applied at a thickness from 10 mm up to 100 mm. Some products may be bulked out with aggregate to be installed up to 300 mm thick.

6.3.1. Applicability

Proprietary cement mortars can be used for:

- thin bonded repairs (up to 40 mm thick, see Section 5.3.2.1); and
- shallow repairs (40 mm up to one third slab thickness, see Section 5.3.2.2);

The mortar used is dependent on the dimensions of the repair and the trafficking requirements.

6.3.2. Properties

Before applying these products, surfaces should be clean and in saturated surface dry condition. Preparation, mixing and installation should follow the manufacturer's recommendations to prevent poor durability.

Figure 6.3 Polymer modified cement mortar

6.3.3. Considerations

Where repairs are being undertaken with proprietary cement mortars, the following aspects need to be considered:

- assessment of the extent of unsound concrete by sounding techniques (Section 2.2.1.6);
- cost versus lane closure time;
- location relative to the wheel tracks and formed joints; and,
- potential for increased drying shrinkage.

The long-term success of thin bonded repairs and shallow repairs is largely dependent on the bond strength established with the substrate. There is a potential risk of increased drying shrinkage, with an associated potential loss of bond with the substrate, for cement mortars with high:

- water / cement ratio;
- cement content; and / or,
- paste / aggregate ratio.

Typically with proprietary cement mortars, admixture technologies are used to reduce the water / cement ratio of the mortar and careful specification and mixture designs can target low shrinkage materials which reduces risk of drying shrinkage cracking and potential for loss of bond to the substrate.

Proprietary cement mortars are rigid repair products, so they are unsuitable for areas subject to movements. Polymeric materials (6.5) should be used where movement is anticipated.

These proprietary cement mortars often set quickly, within 30 minutes of mixing, so they need to be placed promptly as soon as mixing is complete. The constituents should be mixed together using a forced action mixer. The amount of water and additional aggregates need to be added with precision to ensure minimal drying shrinkage and hence a good and durable bond to the substrate.

6.4. Resin mortars

Resin mortars are alternatives to cement mortars. Depending on the product used, they can be installed at a variable thickness from 5 to 50 mm in single and multiple layers. Various products are available, which are formulated for different environmental conditions.

Resin mortars can be used for thin bonded repairs and shallow repairs and are often used for stitched crack repairs where rapid hardening is critical before any crack movement can disrupt the repair. Due to their ability to be feathered to as little as 5 mm, they can be used to plug holes where single particle pop-outs have occurred.

6.4.1. Applicability

Resin mortars can be used for:

- single particle pop-out repairs (see Section 5.3.1);
- thin bonded repairs (up to 40 mm thick, see Section 5.3.2.1);
- shallow repairs (40 mm up to one third slab thickness, see Section 5.3.2.2); and,
- crack stitching (see Section 5.4.8).

6.4.2. Properties

Resin mortars consist of a blend of reactive resins and aggregate. They harden by exothermic reaction which takes place when the resin blend and curing agent (hardener) components are mixed together. These two-part mixtures are available bagged in pre-weighted quantities that are mixed together on site immediately before use. Some products may be able to be bulked out by aggregates for deeper repairs.

They are rigid materials which offer equivalent or higher compressive, flexural and tensile strengths to pavement concrete. They also provide high bond strength, rapid strength development and good chemical resistance.

6.4.3. Considerations

Where repairs are being undertaken with resin mortars, the following aspects need to be considered:

- assessment of the extent of unsound concrete by sounding techniques (Section 2.2.1.6);
- ambient temperature and weather;
- cost versus lane closure time;
- location relative to cracks and formed joints;
- repair dimensions; and,
- quality control.

Generally, resin mortars require the surface temperature of adjacent concrete to be between 10 °C and 30 °C. If the ambient temperature is 20 °C some resin mortars can cure within three hours. However, curing will be very slow or even arrested when the ambient temperature is less than 10 °C. Therefore, if rapid curing is required, ambient temperatures need to be higher than 10 °C. This can restrict overnight or winter usage. However, cold setting products are available.

It is critical that the substrate surface is dry as well as clean and free of any loose material before the application as moisture can disrupt the ability of a resin mortar to bond, and a primer may also be required. The exothermic nature of the reaction means some contraction can occur when the material cools. This can induce stresses that may result in failure, so it is important to follow the manufacturer's requirements for the volume, repair depth and temperature.

Resin mortars can be formed to the joint edges and sealed with normal joint sealing compounds.

They can have a higher coefficient of thermal expansion and strength compared to pavement concrete, so their use is not generally recommended for thin bonded repairs and shallow repairs over large areas (> 1 m^2) since debonding or cracking of the existing concrete often follows.

6.5. Polymeric materials

There are various proprietary hot applied and cold applied polymeric materials available for sealing cracks and repairing surface defects. They are registered under a product acceptance scheme and their performance is monitored through a system installation performance trial (SIPT).

Polymeric materials are flexible materials that are capable of sustaining strains significantly greater than conventional repairs using pavement concretes or rigid resin repair products. Depending on the particular formulation and repair depth, the surface may be opened to traffic between 10 minutes and 2 hours after installation.

6.5.1. Applicability

Polymeric materials can be used for:

- shallow repairs (40 mm up to one third slab thickness, see Section 5.3.2.2); and
- inlaid crack repairs (see Section 5.3.3).

6.5.2. Properties

Most products use resins and / or polymer modified bitumen (PMB) as the binder. In addition to aggregate, they may also incorporate glass fibres and rubber granules.

The advantages over rigid cementitious or resin repair products include:

- application at lower ambient temperatures;
- short curing / setting period before re-opening to traffic;
- increased ductility enables the repair to withstand some substrate movement; and,
- low shrinkage characteristics of some products.

However, polymeric materials do not have equivalent structural properties to cementitious or resin repair products. They are low strength materials, which means additional stresses may be imparted on the surrounding concrete. Therefore, when used for shallow repairs the repair size should be small (< 1 m^2).

6.5.3. Considerations

At present, UK experience with polymeric materials under heavy traffic for long periods is limited. Their use should be considered as a holding repair, but further experience may justify longer-term applications. Polymeric materials that are used should have evidence of performance when used on concrete pavements, rather than flexible pavements.

Quoted service lives for polymeric materials on product acceptance scheme certificates can be 5 years, but the service life will be reduced when used within wheel track zones. For inlaid crack repairs, service life may depend on orientation. However, the ability of polymeric materials to absorb some dynamic movement without failing means they can have better short-term performance than rigid repair materials, for example when used over working (i.e. non static) cracks.

Where repairs are being undertaken with polymeric materials, the following aspects need to be considered:

- assessment of the extent of unsound concrete by sounding techniques (Section 2.2.1.6);
- durability and future retexturing activities;
- ambient temperature and weather;
- location relative to the wheel tracks and formed joints;
- repair dimensions;
- plant (for any heating), personnel (experience needed), quality control; and,
- environmental impact including carbon footprint.

A major consideration for the use of polymeric materials versus other rigid repair materials is compatibility with any future retexturing treatments. It is not recommended to use polymeric materials for repairs where retexturing using longitudinal diamond grinding or fine milling is planned as part of routine maintenance. They tend to perform in a similar manner to joint seals, when impacted by the milling or grinding they tend to shear, resulting in surface deformation / damage or plucking of the repair entirely.

Additional site plant is required to heat the hot applied products and there can be a greater amount of waste generated so these materials can incur a higher installation cost than cold applied polymeric materials or pre-bagged, cold-applied cementitious or resin repair products.

In all cases, it is important to follow the manufacturer's installation method statement, including the width and depth of the recess, which can vary based on material characteristics and repair location; the width and depth of the repair is generally required to be constant. The substrate surface is should be appropriately clean, sound and dry prior to application; the permitted surface temperature is product specific, typically between 0 °C and 50 °C. Further guidance is given in Section 5.3.3.

Some polymeric materials may be capable of bridging joints, but it is always good practice to retain joints by sawing them into the repair where possible.

Specialist contractors are normally needed to install the material by floating or pouring into a box applicator in one or more layers depending on the repair depth. Pre-coated chippings or calcined bauxite chippings are then applied (and rolled in if necessary) whilst the material is still plastic to give skidding resistance (see Figure 6.4). From a cost perspective, it is beneficial to undertake multiple repairs using polymeric materials in the same shift.

Figure 6.4 Inlaid crack repair using polymeric material

7. Restoration

The previous Chapters 5 and 6 focussed on options for routine maintenance and the localised repair of concrete pavements where defects have arisen. However, it may become necessary to undertake other, more widespread pavement restoration treatments for one or more of the following reasons:

- as a preventative maintenance measure;
- to improve surface characteristics including wet skidding resistance and noise generation;
- to improve ride quality;
- where safety critical defects are occurring on an increasingly regular basis;
- where an increase in pavement life is required to meet predicted future trafficking demands;
- where projected continued deterioration results in the pavement soon becoming unserviceable; or,
- it has become too expensive and / or disruptive to undertake ongoing routine maintenance.

Options for exposed concrete pavements are limited when compared with asphalt surfaced pavements. Partial depth inlays to restore the surface whilst maintaining pre-existing site levels have rarely been a preferred option for concrete pavements as milling can damage joints or leave insufficient pavement thickness and cover to any reinforcement, resulting in further, more rapid deterioration of the retained pavement.

Restoration options are presented in this chapter for each pavement type and are characterised in the CPMM dependent on their intended use:

- Surface treatments (including retexturing) and non-structural overlays. These treatments are intended to improve the pavement surface characteristics and maximise the life of existing exposed concrete pavements. Some treatments can improve ride quality.
- Strengthening, where a structural capacity increase is required to achieve a specified design life. Options include:
    - application of a thick overlay; and,
    - partial depth and full depth reconstruction.

7.1. Assessment of suitability

The selection and design of an appropriate restoration option requires assessment of site specific conditions and limitations, and is typically dependent on:

- pavement type (URC, JRC or CRCP) linked to the presence of joints;
- site constraints and consequential costs including:
    - headroom (see CD 127 [53]);
    - weight limits (for bridges or other structures);
    - lane closure time;
    - drainage;
    - vehicle restraint systems (VRS); and,
    - acceptable noise generation requirements;
- pavement condition, the type of defects present (if any) and residual pavement life;
- condition of the underlying pavement foundation; and,
- future trafficking demands versus acceptable periodic maintenance.

The first action when determining the appropriate option will be to review the existing pavement construction and condition following the process outlined in CD 227 [1]. Any option used should be appropriate for the pavement condition and future demands. Existing structural defects, or structural defects that arise post-treatment, will over time reflect through any overlay to the surface. If the treatment fails prematurely as a result of the condition of the underlying concrete, there will be increased life cycle costs, and disruption to the road user.

Most concrete pavements tend to maintain a good condition for the majority of their anticipated design life then deteriorate rapidly as they approach their end of life. Therefore, defect and maintenance history are often good indicators of whether a surface treatment or non-structural overlay is suitable. If the pavement has a history of reactive maintenance or is in a poor visual condition showing structural defects, then strengthening treatments are likely to offer better whole-life cost value than the application of a surface treatment or non-structural overlay.

In addition to the presence of defects, the condition of any joints, the type and condition of the underlying pavement layers, and the effectiveness of drainage assets will have a great impact on the rate of deterioration of the pavement.

It is possible to gain a wider understanding of the general condition of the pavement using the non-destructive testing, intrusive testing and asset monitoring techniques outlined in Chapter 2. Guidance on establishing the condition and integrity of pavement and foundation layers using back-analysed stiffness values obtained from FWD testing is given in Chapter 2 and CD 227 [1].

In any event, for surface treatments and non-structural overlays to be successful, the pavement should be, or be made up to be structurally sound i.e. free from any of the structural defects discussed within Chapter 3. Surface defects at joints will also require repair.

Any site constraints that may limit the choice of treatment options such as overlays should be identified concurrently with the pavement condition assessment. There needs to be a clear understanding of any headroom or weight limit restrictions; then consideration of any limitations and

/ or work required to the associated linear assets such as drainage and VRS. It is not uncommon for sections of a scheme with headroom or weight limit restrictions (for bridges or other structures) to be locally reconstructed whilst other sections, without these constraints, are overlaid.

Environmental factors, such as the noise sensitivity of the adjacent areas will also be a major factor in determining suitable treatment options. A combined understanding of the pavement deficiency, restrictions and future demands, will reduce the number of available restoration options.

Assessment of the suitability of each option should take into consideration user disruption during construction and the service life together with the LCCA - using assumptions on the likely level and frequency of periodic maintenance.

7.2. Considerations for URC and JRC

The primary consideration for URC and JRC, except for where reconstruction is undertaken, is limiting future maintenance liabilities that may arise at the location of joints in the existing concrete post-treatment.

Restoring surface characteristics through retexturing can:

- damage joint seals, necessitating replacement to minimise the ingress of water, de-icing salts and incompressible materials into the pavement; and,
- damage joint faces and increase the rate of deterioration at joints.

Section 7.4.1.1 contains more information on identifying the suitability of a concrete pavement for retexturing, including guidance on raw materials and the risks associated with any reduction in pavement thickness.

Overlays to URC and JRC can be subjected to higher strains (in a flexible material such as asphalt) and stress (in a rigid material such as concrete) at the location of any joints and wide cracks in the underlying pavement. These movements can result in the appearance of reflective cracking in the new surface (see Figure 7.1). This is attributable to a combination of:

- load induced movements;
- long-term temperature induced movements (seasonal);
- short-term temperature induced movements (diurnal); and,
- drying shrinkage movements.

Reflective cracking will allow the ingress of water and other external substances into the pavement, compromising the pavement structure. These cracks also can fret (a loss of aggregate), and water ingress through the crack can result in a loss of bond between the overlay and the concrete substrate causing delamination and potholing then untimely resurfacing works. Therefore, for overlay treatments, preventing or delaying reflective cracking is one of the primary design considerations.

Figure 7.1 Reflective cracking in an asphalt overlay caused by an underlying joint / crack

Reflective crack propagation is influenced by the asphalt overlay thickness, so fundamentally reflective cracking can be delayed by the application of a thick asphalt overlay (> 180 mm). However, the thickness required to delay reflective cracking may exceed any structural capacity requirements for the pavement, and site constraints and consequential costs may limit the practicable overlay thickness for a site.

Reflective crack mitigation systems can be used to delay reflective cracking within structurally sound pavements where a thick asphalt overlay is not viable. This topic is an area of ongoing research and development, current systems include:

- asphalt overlay with saw-cut and seal joints (see Section 7.4.2.1) at the same locations as joints (and wide cracks) in the underlying pavement.
- stress absorbing membrane interlayers (SAMIs) (see Section 7.4.2.2) to absorb and distribute strains, increasing the number of repeated movement cycles required for a crack to initiate. SAMI options are:
  - asphalt SAMI;
  - spray applied SAMI; and,
  - geosynthetic SAMI.
- polymer modified bitumen (PMB) asphalts for crack mitigation used in conjunction with one of the above:
  - highly modified PMB thin surface course system (TSCS) (see Section 7.4.4) or hot rolled asphalt (HRA).
  - PMB binder and base course (see Section 7.4.5).

Conversely, if only a short service life, i.e. three years, is required and / or routine maintenance to seal cracks and repair potholes is acceptable, then it may be appropriate to not utilise a reflective crack mitigation system within the overlay.

One possible detrimental effect of applying asphalt overlays onto a concrete pavement is the tendency of the asphalt overlay to induce greater thermal gain into the underlying slab because of the black surface. The most noticeable result may be increased horizontal movement and increased slab curling, potentially resulting in local heave of the asphalt at underlying joint locations. Thermal gain will reduce with increased asphalt overlay thickness.

Where thick overlays can be accommodated, thick overlays will offer a significant structural contribution to the pavement. Fractured slab techniques are an option to reduce the total movement (and strain) at joints, prior to asphalt overlay. These techniques can reduce the asphalt overlay thickness needed to delay reflective cracking, by fracturing the slabs to create fine cracks at regular intervals where small amounts of movement can occur. Fractured slab techniques such as crack and seat are discussed in Section 7.4.8.

Long term monitoring of overlaid URC and JRC pavements indicates that the occurrence of reflective cracking from underlying joints or cracks is accelerated at locations with low load transfer efficiency. Therefore, satisfactory load transfer efficiency (≥ 75 %) at joints should be confirmed through FWD surveys (see Section 2.2.1.1) before proceeding with a non-structural overlay treatment whose success is reliant on reflection cracking being delayed.

For non-structural overlays, particular attention also needs to be given to the existing concrete joints, ensuring that defective joint seals are replaced, and any locations of joint spalling are repaired prior to the works. Routing and sealing of joints and cracks following the inlaid crack repair procedure in Section 5.3.3 is an option to address minor spalls whilst resealing joints prior to overlay.

Whilst load transfer efficiency is an indicator of the performance of load transfer devices, it is also an indicator of slab support. A degree of reflective cracking associated with horizontal movement is expected in an asphalt overlay at the location of underlying joints in a structurally sound URC or JRC pavement. However, reflective cracking and the subsequent deterioration of the surfacing in proximity of the cracks, can occur as a result of excessive vertical movements at joints or cracks in the underlying pavement.

If unsatisfactory load transfer is identified, further investigation should be undertaken to characterise the slab support conditions. As outlined in Chapters 2 and 4, options to examine a reduction in slab support include visual (settlement, stepping, and pumping) or audible assessment, routine profile monitoring, or by voiding assessments undertaken with an FWD, GPR or ultrasound.

Where pumping, dynamic vertical movement and / or ongoing settlement is prevalent, signifying drainage or foundation issues, the only long-term solution is full depth reconstruction (see Section 7.4.9). Otherwise, stabilisation with under slab grouting and lifting may be used to restore the carriageway profile, at which point overlays can be undertaken with or without fracturing the slabs. Some regulation of profile can be achieved with an asphalt overlay.

7.2.1. Strengthening design for URC and JRC

The design for pavement strengthening will be dependent on multiple factors including the condition of the pavement and foundation, the materials proposed within the strengthening treatment and the anticipated future traffic, following the requirements outlined in CD 224 [41], CD 225 [54] and CD 226 [12].

For asphalt overlays, the overlay thickness for strengthening will be the greater value of the thickness required to:

- meet future trafficking requirements; and,
- mitigate the risk of reflective crack propagation.

For overlays, an analytical design will need to be undertaken by an experienced pavement engineer using multi-layer linear elastic modelling, the principles of which are documented in CD 226 [12]. This primarily considers the properties, stiffness and thickness of each layer of the pavement as well as that of the proposed overlay. The properties of the concrete include an assumed layer stiffness of the intact or fractured slabs, which can be determined by establishing the compressive strength and the coarse aggregate type [55]. Analytical design requires a departure from standard.

The minimum asphalt overlay for strengthening is a topic of ongoing research and development and refinement, the thickness depends largely on the existing pavement condition. Where horizontal or vertical movements are significant, or load transfer efficiency is poor, reflective cracking may occur more rapidly. It may therefore be appropriate to use a thicker asphalt overlay, or potentially a reflective crack mitigation system within the overlay construction.

Table 7.1 outlines the recommended strengthening options for URC and JRC. Service lives for the strengthening options are not provided on the basis that each option will be designed to deliver a pavement with a structural life in accordance with the requirements of CD 227 [1].

The technology readiness level for each option presented in Table 7.1 are correct at the time of publication of the CPMM, see Section 1.6 for more information.

Further guidance on the implementation of the options described in this chapter can be obtained through liaison with Highways England Safety, Engineering and Standards (SES). Further detail on the options outlined is contained in Section 7.3.2.

Table 7.1 Strengthening options for URC and JRC

Option	Thickness (mm)	Technology readiness level	Key considerations
Bonded concrete overlay	Min. 100	5	No history of use in UK but used in the USA and continental Europe. As neither a specification clause nor a design process currently exist a departure from standard is required. Joints in the underlying pavements need to be matched in the overlay. Bonded concrete overlays tend to deteriorate rapidly at end of life.
Asphalt overlay	CD 226 Min. 180	7	A departure from standard is required for analytical pavement design. Reflective cracking will be delayed but not prevented.
Fractured slab techniques with asphalt overlay	CD 226 Min. 150	7	A departure from standard is required for analytical pavement design. Suitable for pavements in fair to poor condition. Reflective cracking will be delayed but not prevented. Potential future maintenance liability.
Unbonded concrete (CRCP) overlay	CD 226 Min. 200	7	A departure from standard is required for analytical pavement design. An initial asphalt regulating layer may be required to inhibit bonding. An asphalt surface course may be required to give acceptable surface characteristics.
Reconstruction	CD 226	9	Significant cost and disruption. Foundation replacement may be necessary to maintain site levels and accommodate pavement design thickness.

7.2.2. Surface treatments and non-structural overlays for URC and JRC

Figure 7.2 and Table 7.2 outline the recommended options for surface treatments and non-structural overlays for URC and JRC. Table 7.2 includes expected service lives for each option. The understanding of the service life of each option is evolving based on findings from the ongoing monitoring of in service pavements. The expected service life is influenced by multiple factors, including those described below. Ultimately, the expected service life is an output from the design process based on a knowledge of the site where the option is being proposed, plus experience with the options and techniques used.

Designing for maintenance is equally important for surface treatments and non-structural overlays and strengthening as it is for new construction. Design considerations include the implications on the cover to reinforcement following retexturing works, the recyclability of reflective crack mitigation interlayers, and the potential for single layer overlays to delaminate as they approach the end of their service life.

The expected service lives quoted in Table 7.2 are for pavements with design traffic in excess of 50 msa over a 40 year period. Where the traffic is less than 50 msa, the service life is likely to be higher than the quoted upper limit. The service life for retexturing techniques refers to the period over which the skid resistance is elevated above the site threshold values outlined in CS 228 [2]. The service life will be influenced by multiple factors, including the:

- type and geometry of road;
- quantity and behaviour of the traffic; and,
- materials that make up the surface.

Section 7.4.1 contains more information on the use and applicability of retexturing techniques.

For non-structural overlays, the expected service lives in years quoted in Table 7.2 are aligned to the levels of commercial vehicle traffic typical on the SRN and defined as the likely point in time at which network surveys and subsequent evaluation triggers an intervention, or where unplanned localised maintenance is required that has a negative consequence on the network, e.g. a negative impact on ride quality.

For non-structural overlays, the service life will be severely impacted by:

- the state and condition of the concrete pavement (if unsound or otherwise deteriorated);
- dynamic vertical movement under trafficking; and,
- poor drainage and the potential for water to be trapped between layers.

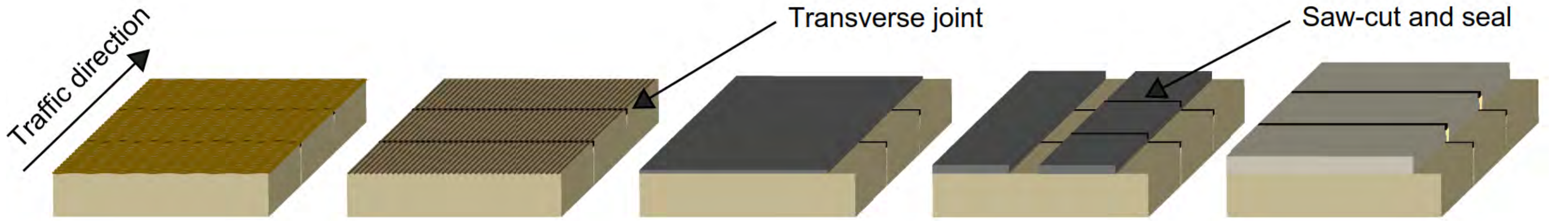
The service life of non-structural overlays will also be influenced by factors including the:

- load transfer efficiency across joints / cracks;
- amount of horizontal movement at joints / cracks;
- thickness of the overlay applied;
- surfacing material used; and,
- type and effectiveness of any reflective crack mitigation system used.

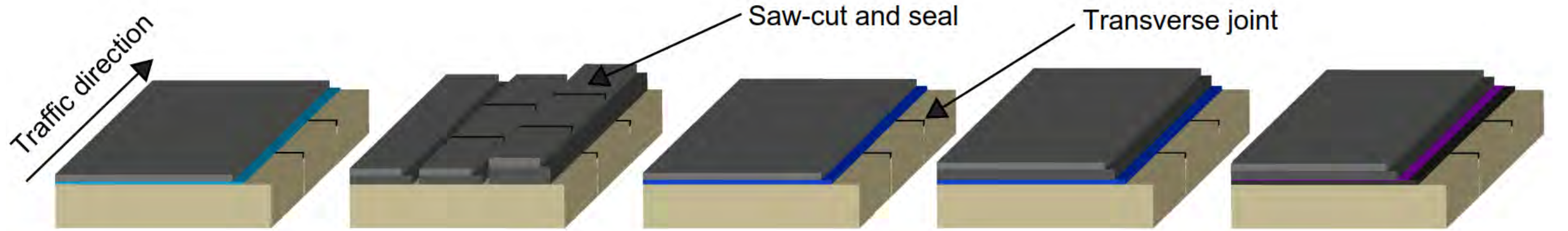
The service life of a subsequent resurfacing treatment is likely to be less than the original overlay treatment. Where the noise constraints permit, a Clause 943 HRA 35/14 F PMB with pre-coated chippings may offer improved service life versus a Clause 942 thin surface course system as the material is less prone to fretting and potholing once reflective cracks occur.

The technology readiness level for each option presented in Table 7.2 are correct at the time of publication of the CPMM, see Section 1.6 for more information.

Further guidance on the implementation of the options described in this chapter can be obtained through liaison with Highways England Safety, Engineering and Standards (SES). Further detail on the options outlined is contained in Section 7.3.2.



1	2	3	4A	4B	5
Fine milling	Longitudinal diamond grinding	20 mm cold applied ultra-thin surfacing (CAUTS)	20 - 50 mm PMB TSCS (4B includes saw-cut and seal)		50 - 100 mm fully bonded concrete overlay



<table>
<tr><th colspan="2">6</th><th colspan="2">7A | 7B | 7C</th><th colspan="2">8</th><th colspan="2">9</th><th colspan="2">10</th></tr>
<tr><td rowspan="2"></td><td rowspan="2">25 - 50 mm PMB TSCS</td><td></td><td>25 - 50 mm PMB TSCS</td><td rowspan="2"></td><td rowspan="2">25 - 50 mm PMB TSCS</td><td></td><td>25 - 50 mm PMB TSCS</td><td></td><td>25 - 50 mm PMB TSCS</td></tr>
<tr><td></td><td>50 - 100 mm PMB binder course</td><td></td><td>50 - 100 mm PMB binder course</td><td></td><td>50 - 100 mm binder course</td></tr>
<tr><td rowspan="2"></td><td rowspan="2">~15 mm spray applied SAMI</td><td colspan="2" rowspan="2">(7B and 7C includes saw-cut and seal)</td><td rowspan="2"></td><td rowspan="2">20 - 30 mm asphalt SAMI</td><td rowspan="2"></td><td rowspan="2">20 - 30 mm asphalt SAMI</td><td></td><td>Geosynthetic SAMI</td></tr>
<tr><td></td><td>15 - 20 mm regulating layer</td></tr>
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Figure 7.2 Surface treatments and non-structural overlay options for URC and JRC

Table 7.2 Surface treatments and non-structural overlay options for URC and JRC

Option (see Figure 7.2)	**Thickness (mm)**	**Expected service life**	**Technology readiness level**	**Key considerations**
1 Fine milling	(reduction) 2 - 5	4 - 5 years	8	Surface may be noisier or have a different tone. Life expectancy is dependent on the coarse aggregate type in concrete. A departure from standard may be required dependent on treatment length (see CD 236 [56]).
2 Longitudinal diamond grinding	(reduction) 2 - 5	4 - 6 years	8	Low noise surface. Life expectancy is dependent on the coarse aggregate type in concrete. A departure from standard may be required dependent on treatment length (see CD 236 [56]).
3 CAUTS	< 20	3 - 5 years	9	Potential high output option. May have unacceptable noise generation characteristics for some sites (see CD 236 [56]). Reflective cracking will occur. This seasonal process needs daytime installation between April and August. Significant pre-planning is required.
4A TSCS overlay	20 - 50	3 - 5 years	7	Omitting saw-cut and seal requires a departure from standard. Reflective cracking will occur, potentially within 1 year post-surfacing. Therefore, potential future maintenance liability. Low initial cost and low disruption option. Very thin layers may require specialist plant. Unsuitable for regulation.

Option (see Figure 7.2)	Thickness (mm)	Expected service life	Technology readiness level	Key considerations
4B TSCS overlay with saw-cut and seal	20 - 50	5 - 8 years	9	The occurrence of reflective cracking is linked to the condition of joints, and the workmanship associated with the saw-cut and seal technique. Very thin layers may require specialist plant. Unsuitable for regulation. Saw-cut and seal is a time intensive process and only effective for bays < 12 m long.
5 Bonded concrete overlay	50 - 100	N/A	5	No history of use in UK, but used in the USA and continental Europe. As neither a specification clause nor a design process currently exist, a departure from standard is required. Joints in the underlying pavement need to be matched in the overlay. Bonded concrete overlays tend to deteriorate rapidly at end of life.
6 TSCS and spray applied SAMI	40 - 65	N/A	6	Spray applied SAMIs have a limited history of use in UK, although they are used in continental Europe. A specification clause for spray applied SAMIs does not currently exist so a departure from standard is required. This process requires daytime installation between April and August. There is a risk of bitumen bleeding plus loss of surface texture in certain scenarios.
7A Multi-layer PMB overlay	75 - 100	5 - 8 years	7	Reflective cracking will occur. The rate of reflective crack propagation linked to the, overlay material and thickness and the condition of joints. Omitting saw-cut and seal requires a departure from standard for overlays less than 100 mm thick.

Option (see Figure 7.2)	Thickness (mm)	Expected service life	Technology readiness level	Key considerations
7B Multi-layer PMB overlay with saw-cut and seal	75 - 100	8 - 12 years	9	The occurrence of reflective cracking is linked to the condition of joints, and the workmanship associated with the saw-cut and seal technique. Saw-cut and seal is a time intensive process and only effective for bays < 12 m long.
7C Multi-layer PMB overlay with saw-cut and seal	100 - 150	> 12 years	9	
8 TSCS and asphalt SAMI	45 - 80	8 - 12 years	7	Asphalt SAMIs have a track record in this application. Track record is product specific and information should be obtained from the supply chain. A specification clause for asphalt SAMIs does not currently exist, so a departure from standard is required. Paver laid reducing construction interfaces for asphalt works (versus techniques requiring specialist plant or processes). Recyclable.
9 Multi-layer overlay with asphalt SAMI	95 - 150	> 12 years	7	
10 Multi-layer overlay with geosynthetic SAMI	90 - 150	> 12 years	9	Geosynthetic SAMIs have a track record of performance in this application. Track record is product specific and information should be obtained from the supply chain. Potentially limited recyclability of geosynthetic SAMI and surrounding asphalt layers at end of life. The geosynthetic SAMI should be placed under the binder course to reduce the risk of debonding failures and damaging the geosynthetic SAMI during resurfacing works.

7.3. Considerations for CRCP

CRCP does not have expansion or contraction joints (except at the terminations). Instead, thermal stresses are relieved by naturally occurring closely spaced transverse cracking within the concrete slab. The closely spaced transverse cracks are held closely together by continuous longitudinal reinforcement, ensuring aggregate interlock and load transfer are maintained. Relatively small thermal movements occur at the location of inherent cracks, which in a well-designed and well-constructed pavement are present as hairline cracks (< 0.5 mm wide).

During the life of CRCP, improvements to surface characteristics such as noise generation and wet-skid resistance may be necessary. Furthermore, the defects outlined in Section 3.5 (see also 'significant defects' in CD 227 [1]) can develop, which in themselves may not cause a serviceability issue, but will accelerate the deterioration of the pavement through the ingress of water, de-icing salts and incompressible materials into the pavement structure, which may lead to corrosion of the reinforcement and spalling of cracks.

Non-structural overlays may be undertaken to improve surface characteristics, and to slow the rate of deterioration of the pavement, particularly where 'significant defects' have occurred, by effectively providing a barrier (i.e. an overlay) to any ingress into the pavement structure.

7.3.1. Strengthening design for CRCP

Where strengthening of the pavement is required, such as instances where the pavement has been calculated to have insufficient residual life for the future predicted traffic, but the CRCP is otherwise intact or has been satisfactorily repaired, the overlay thickness should be designed using the process outlined in CD 226 [12].

For asphalt overlays, the assumptions used for the design of new continuously reinforced concrete base (CRCB) pavements in CD 226 can be followed, where 15 mm of concrete is considered to be equivalent to 100 mm of asphalt [12]. This standard equivalency factor can be used as a guide to determine the overlay thickness requirement over intact CRCP in conjunction with the in situ concrete pavement properties, and the past and future traffic. This makes allowance for the different thermal stresses generated in an asphalt overlaid concrete pavement as a result of the darker surface colour.

Bonded concrete overlays have a history of effective use internationally and can enhance structural capacity with a lesser overlay thickness versus asphalt; however, at the time of publication of the CPMM they have not been used in the UK.

7.3.2. Surface treatments and non-structural overlays for CRCP

The surface characteristics of CRCP in otherwise good condition can be improved by use of retexturing techniques. However, retexturing is unlikely to be appropriate for CRCP that has been subject to inlaid crack repairs, as the flexible nature of the inlaid crack repair material can result in shear damage to the repair or plucking of the repair entirely when impacted by milling / grinding equipment.

As thermal movements are relatively small at inherent cracks, there is less need to mitigate risk of reflective cracking, so overlays can generally be thinner than for URC and JRC pavements. Therefore, options for CRCP are linked to pavement condition and the type and severity of the significant defects available, following the general principle that an increased overlay thickness will increase the amount of time for defects such as wide cracks to reflect through to the surfacing.

Most intact CRCP will be suitable for a single layer overlay such as CAUTS (see Section 7.4.3) or TSCS incorporating a PMB (see Section 7.4.4) to give the surface material flexibility and mitigate risks of any reflective cracking from thermal movements at inherent crack locations.

However, where deterioration has started to occur, such as wide cracking and crack spalling, or where ride quality improvements are required, two layers of PMB asphalt (total thickness ≥ 70 mm) is recommended.

Reconstruction will be necessary where there is severe deterioration or site constraints prevent overlay (see Section 7.4.9).

7.4. Materials and options

This section describes different options and materials available for undertaking restoration works. The suitability for existing pavement types is described in the previous sections.

7.4.1. Retexturing

The road surface condition influences the skidding resistance, ride quality and noise levels experienced by the road user and the noise levels experienced by the wider environment. For URC and JRC pavements, the condition of the transverse joints, when trafficked, can further reduce the overall ride quality and increase the noise levels.

The skidding resistance of a road surface is dependent on the interaction of vehicle tyres with the surface:

- surface microtexture: the roughness of the aggregates and mortar at the surface; and,
- surface macrotexture: the introduced texture of the road surface itself to provide a path for water and air to be dissipated from under tyres.

At higher speeds the macrotexture becomes more significant as it facilitates water drainage to ensure that the tyre / surface contact is maintained. However, excessive positive macrotexture increases the rolling resistance and thus the fuel consumption of the vehicle.

For concrete surfaces, microtexture comes primarily from the angularity of the fine aggregate used. Some coarse aggregates used in concrete are either smooth to begin with or tend to readily polish smooth under trafficking. Macrotexture is applied to the concrete surface by various methods including brushing and grooving.

In service, the microtexture and macrotexture of concrete pavement surfaces are gradually reduced by trafficking. The exposed fine and coarse aggregates are gradually abraded and polished, reducing microtexture until an equilibrium level of skidding resistance is reached.

The macrotexture is reduced as the cement mortar matrix and any grooves or ridges applied to the surface break down. This results in a surface with reduced wet skidding resistance and reduced water shedding ability that can result in increased spray and reduced visibility. Noise generating characteristics can also change with changing macrotexture.

Brushed finished surfaces are particularly prone to losing skidding resistance and generally, concrete pavements constructed with softer, less wear-resistant aggregates (e.g. limestones) are more susceptible to skid resistance deterioration than those constructed with harder aggregates.

In service, skidding resistance can be restored by roughening the worn surface. There are several different retexturing techniques that involve the mechanical reworking or reshaping of a sound road surface to restore skid resistance and / or texture depth. Some of the retexturing techniques can improve the surface profile and reduce noise generation at the tyre interface with the surface. However, others can result in an increase in noise generation and / or change the tone of the noise generated.

Retexturing techniques offer the benefit that carriageway levels are not increased, so they are suitable for sites with carriageway level sensitives due to headroom or weight restrictions and do not generally require work to be undertaken to associated linear assets such as drainage and VRS. Most techniques can be repeated multiple times depending on the cover to any steel reinforcement and impact on linear assets and carriageway profile.

7.4.1.1. Retexturing techniques

The preferred options for retexturing are fine milling (see Section 7.4.1.2) and longitudinal diamond grinding (see Section 7.4.1.3). Both techniques have specific provision in the MCHW and are commonly used over significant lengths of pavement on the SRN to restore surface characteristics. These techniques offer long service lives, can improve carriageway profile and in the case of longitudinal diamond grinding, significantly reduce the noise generation characteristics of the concrete surface.

Other retexturing techniques are available that can be used over short sections to treat high-risk sites whilst another treatment is planned; however, the techniques have either a short effective service life or produce a very noisy road surface. This manual does not go into detail on these techniques, which include:

- transverse grooving;
- water jetting;
- flailing;
- bush hammering; and,
- shot blasting.

A departure from standard may be required, dependent on the treatment length, due to the low expected service life and noise generation characteristics of some retextured surfaces. Requirements for departures from standard for retexturing techniques are outlined in CD 236 [56].

This section covers retexturing techniques for existing concrete surfaces. These retexturing techniques do not need to be used for new concrete being placed to repair or widen the existing pavement. The finished surface requirements for new concrete are outlined in Clause 1026 [7].

Selection of the most appropriate retexturing technique is dependent on:

- the asset management policy;
- the deficiency being treated;
- performance requirements;
- practical considerations:
    - closure time;
    - level and profile constraints; and,
    - weather;
- environmental considerations:
    - contaminants produced by retexturing;
    - noise generated during treatment; and,
    - noise generation characteristics of the finished surface.

The durability of the resultant increase in skid resistance and texture depth can depend on the:

- retexturing techniques used;
- type and geometry of road;
- quantity and behaviour of the traffic; and,
- raw materials that make up the surface.

Retexturing is most effective on a concrete road surface that is generally sound. Most processes can be carried out at any time of year in all but the most severe weather conditions. Caution is needed with some treatments over JRC and URC pavements as the joint grooves or joint seals may be damaged, meaning joint reinstatement may be required post-treatment (see Figure 7.3).

A thin layer of the existing concrete surface is removed during fine milling and longitudinal diamond grinding, and if needed further concrete can be removed, these techniques can be used to reprofile any surface irregularity (Section 3.1.1) to improve ride quality.

If the carriageway is being reprofiled using these techniques, at the same time as retexturing, both cover to any reinforcement and structural capacity of the pavement should be assessed. Insufficient cover to reinforcement will accelerate the occurrence of structural defects, whilst a significant reduction in the concrete thickness from deeper or multiple treatments will reduce the structural life of the pavement. Where reprofiling is proposed, concrete bays should be intact and joint performance good. Joint resealing and local repair may be required after reprofiling.

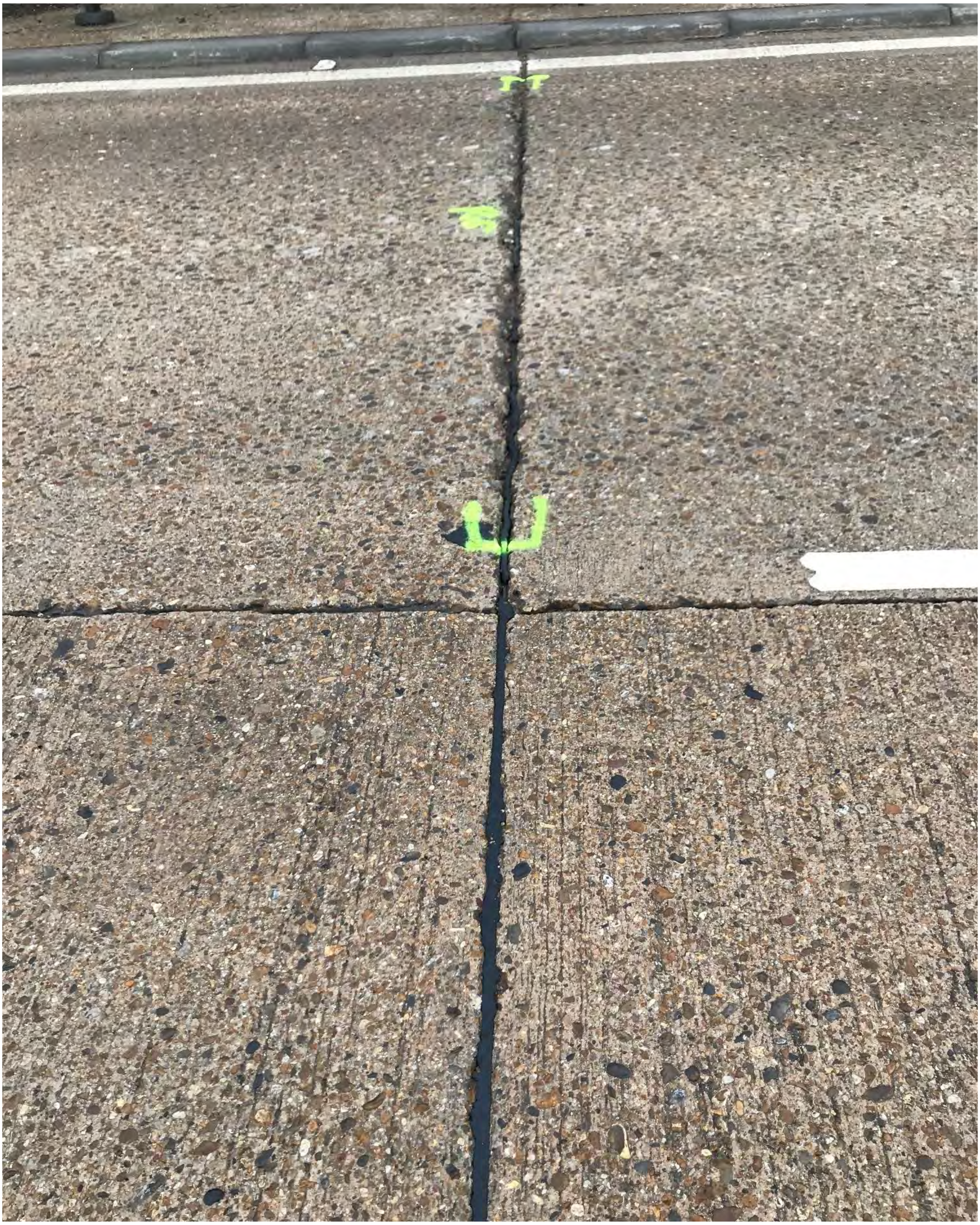

Figure 7.3 Joint groove in furthest lane damaged by retexturing requiring a thin bonded repair. Closest lane joint groove not subject to retexturing in good condition

Reprofiling of the pavement when retexturing is only suitable where:

- the site is in a good structural condition;
- there is adequate cover to reinforcement;
- there is sufficient pavement thickness versus trafficking requirements; and,
- constraints can accommodate a slight reduction in carriageway height.

The type of coarse aggregate within the concrete should be confirmed before works commence so the specialist contractor can prepare their works programme and equipment. The mechanical hardness of the aggregate in the concrete can affect the equipment set up and speed at which the reprofiling process can progress. For example, flint has a greater resistance to diamond cutting than limestone and may require slower movement with a greater load applied to the cutting drum. The equipment may need to be adjusted to mitigate the risk of pop-outs when treating concrete with softer coarse aggregates such as limestones.

Retexturing techniques expose the coarse aggregate in the concrete. Therefore, the suitability of these retexturing techniques will be dependent on the type of coarse aggregate in the concrete, particularly the polishing and water absorption characteristics of the aggregate. These characteristics will affect the:

- expected service life of the retexturing treatment; and,
- durability of the retextured surface.

Concrete comprising flint or siliceous gravels is likely to be acceptable for these retexturing techniques without further investigation. However, it is recommended that concrete comprising limestone and other crushed rock aggregates are subject to petrographic examination in accordance with BS EN 12407 [57] to give an understanding of the technical properties and the suitability of the aggregate to exposure to both traffic and environmental loading including freeze-thaw cycles.

Furthermore, limestone aggregates tend to polish readily. Therefore, even if the technique is deemed suitable in terms of durability of the retextured surface, a lower expected surface life of the retexturing should be expected on concrete with a limestone coarse aggregate fraction.

The techniques are further explained in the following sections.

7.4.1.2. Fine milling

Fine milling uses a specialist milling machine, with closer pick spacings than conventional milling machines, to introduce shallow fine grooves in the pavement. Fine milling can be used to provide an improved ride quality by milling at a variable depth to improve the surface profile. Fine milling produces a significantly roughened surface that improves both the microtexture and the macrotexture. However, it has been found to increase the tyre / road noise.

Requirements for fine milling are outlined in Clause 1093 [7].

7.4.1.3. Longitudinal diamond grinding

Longitudinal diamond grinding involves the removal of a thin layer of concrete using diamond tipped saw blades to restore texture and enhance the skidding resistance characteristics. The process uses closely spaced blades to cut longitudinal grooves at predetermined widths and depths and expose a clean unpolished aggregate face (Figure 7.4).

Figure 7.4 Longitudinal diamond grinding equipment (left) and typical finished surface (right)

The blade spacing is often project specific, dependent on the aggregate in the concrete. Harder aggregates (such as granites) may require closer blade spacings than softer aggregates (limestones).

The process can follow the profile of the surface, meaning a negligible nominal depth of concrete is removed thereby reducing any risk of damage at the joints. There can be a notable reduction in the tyre / road noise following this treatment.

Alternatively, the grinding can be undertaken at a variable but greater depth to improve the pavement surface profile by removing any steps or irregularities. The improved profile can result in a significant improvement in the ride quality but there may be some damage to joints, and resealing may be required after this deeper treatment. Requirements for longitudinal diamond grinding are outlined in Clause 1092 [7].

7.4.2. Reflective crack mitigation systems

Reflective crack mitigation systems are in various stages of development and observation. Available options include:

- asphalt overlay with saw-cut and seal joints;
- stress absorbing membrane interlayers (SAMIs), which includes:
  - asphalt SAMI;
  - spray applied SAMI; and,
  - geosynthetic SAMI;
- polymer modified bitumen (PMB) asphalts used in conjunction with one of the above:
  - PMB thin surface course system (TSCS) or hot rolled asphalt (HRA); and,
  - PMB base and binder course.

7.4.2.1. Asphalt overlay with saw-cut and seal

Saw-cut and seal is a standard technique to manage reflective crack propagation in asphalt overlaying URC and JRC pavements on the SRN. Clause 713 covers requirements for saw-cut and seal of bituminous overlays over existing jointed concrete pavements [6].

Saw cuts are made in the asphalt overlay using a specialist blade directly above the location of transverse joints (and wide cracks) in the underlying pavement. The saw cut comprises a sealant slot and a crack initiation slot. The slot is cleaned and dried and a bond breaker tape installed to prevent the sealant entering the crack initiation slot, followed by the application of a hot applied sealant in the sealant slot (see Figure 7.5). The sealant should be able to accommodate stresses from the movement at joints in the underlying concrete pavement, preventing reflective cracks from occurring.

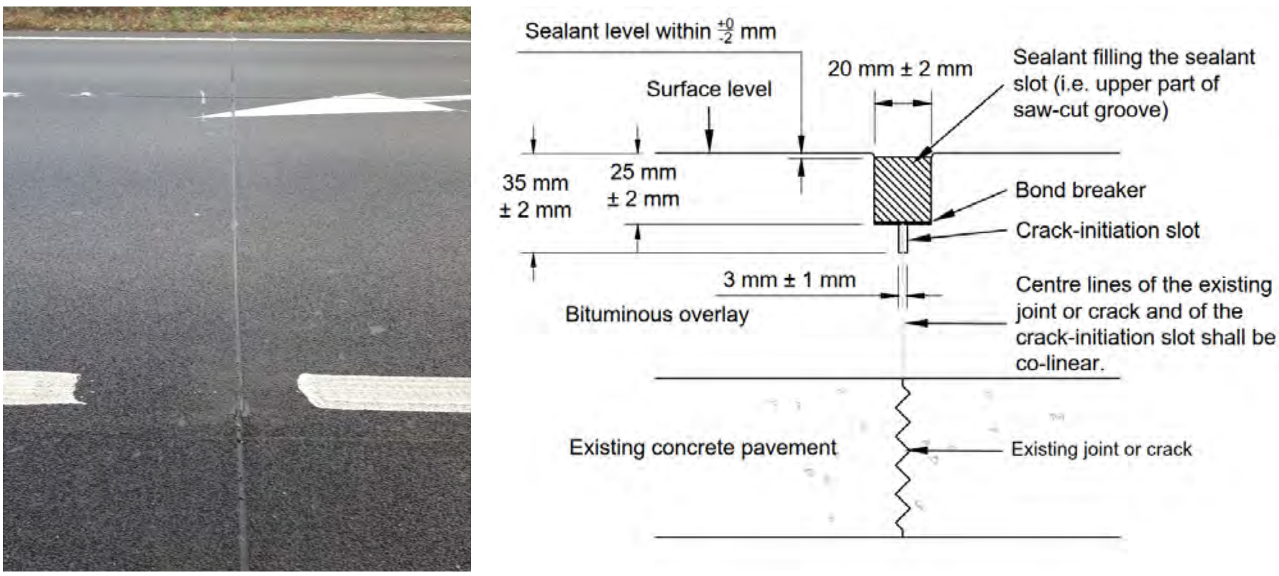

Figure 7.5 Saw-cut and seal

The performance of saw-cut and seal with thin overlays has been variable in the UK. Where the overlay thickness is insufficient, linked to the width and condition of the sealed joint, then cracks can form parallel to the sealed slot in the overlay. The risk of parallel cracks proximal to the transverse saw cut can result in localised ravelling of the joint; creating an ongoing maintenance issue [58]. As a result, joint condition and width should be considered in conjunction with overlay thickness and overlay materials.

As a general guide, an asphalt overlay of 70 mm or greater has been shown to effectively mitigate risk of parallel cracking away from the saw cut. Clause 713 requires a minimum asphalt overlay thickness of 40 mm to use the saw-cut and seal treatment; however, saw-cut and seal with an alternative detail to that outlined in Clause 713 has been effectively implemented on asphalt overlays 25 mm thick. Use of a PMB TSCS is recommended for saw-cut and seal to reduce the risk of parallel crack formation.

Site control is critical throughout the process to ensure performance of the technique. The crack initiation slot needs to be accurately aligned with the underlying concrete joint and the slot needs to be thoroughly cleaned and dried.

The saw-cut and seal process is labour and time intensive and only suitable for URC and JRC pavements with a bay length of up to 12 m. Beyond this length, the movement at joints is too great to be accommodated and failure of the joint sealant can rapidly occur.

Depending on the joint condition and the degree of movement, it may be appropriate to undertake saw-cut and seal at longitudinal joints. Where transverse joints do not traverse the full carriageway width, it is recommended that the saw cut is extended across the full carriageway width to reduce the risk of sympathetic cracking of the surface.

7.4.2.2. Stress absorbing membrane interlayers (SAMIs)

SAMIs are flexible pavement interlayers placed over URC or JRC pavements. They are alternatives to saw-cut and seal when an asphalt overlay is being undertaken. Their purpose is to ‘absorb’ stresses caused by thermal contraction and expansion movements in underlying cracks and joints. They are designed to deform horizontally or vertically under stress and movement, dissipating the energy produced and as a result, prevent the initiation, or otherwise delay the onset of cracking in the overlaying asphalt layers generated by horizontal thermal movements at the joints in an underlying URC or JRC pavement [59].

SAMI is an all-encompassing term for various types of proprietary products. SAMIs generally fall into the following product types:

- paver laid asphalt mixtures (asphalt SAMIs);
- spray applied SAMIs [59]; or,
- geosynthetic SAMIs.

Asphalt SAMI

Asphalt SAMIs are proprietary paver-laid mixtures that generally employ fine graded aggregates and a high content of heavily modified PMB (i.e. incorporating a high percentage of polymers) to enable deformation and subsequent recovery when stress is applied (see Figure 7.6).

They are typically installed between 20 - 35 mm thick and can be applied directly onto the concrete. They can then be overlaid with a surface course or a binder course followed by a surface course. At the date of publication of this manual, there is not a specification clause in the MCHW for asphalt SAMIs, so a departure from standard will be required for their use.

Products are available with a good track record of performance in delaying reflective cracking compared with control sections on the SRN for over 10 years; however, not all products will offer equivalent performance. In the absence of a specification clause, any products used should have an established history of successful use in delaying reflective crack propagation over jointed concrete pavements or have evidenced performance in a trial scenario.

Asphalt SAMIs offer construction benefits:

- they can be removed by the conventional milling equipment with no increased cost, programme or recycling implications; and,
- there is reduced risk of unintentionally damaging the interlayer during resurfacing.

Figure 7.6 Asphalt SAMI over URC

Spray applied SAMIs

Spray applied SAMIs are bitumen based proprietary products that are applied by a spray tanker. Spray applied SAMIs are, generally, heavily applied (2 - 3 kg/m^2) bond coat that can be used in conjunction with various fibres. A crushed aggregate or microsurfacing is then applied over the SAMI before they are overlaid by asphalt. This prevents binder being picked up by construction traffic and any bitumen bleeding occurring.

Additional labour and plant will be required on site for installation compared with asphalt SAMI. Furthermore, some techniques are seasonal if a bitumen emulsion is used as the emulsion breaks more slowly in cool or damp conditions.

Spray applied SAMIs have limited historic use in the UK [58].

At the date of publication of this manual, there is not a specification clause in the MCHW for spray applied SAMIs, so a departure from standard is required for their use. Where a product is being considered for use, evidence of successful use over URC or JRC pavements should be provided and submitted with any departure from standard application.

Geosynthetic SAMIs

Geosynthetics are synthetic polymer materials (fabrics or composites incorporating grids) installed at pavement layer interfaces. When applied atop of a heavy rate of bond coat (> 1.0 kg/m^2 residual bitumen) over a jointed concrete pavement they can act as a SAMI. In this application, some products have a track record of performance in delaying the onset of reflective cracking when placed over a jointed concrete pavement on the SRN versus control sections.

Whilst geosynthetic SAMIs can delay the onset of reflective cracking, they are not considered to be structural elements of the pavement and do not provide a structural contribution; therefore, they cannot be used to reduce any structural design thickness of the asphalt overlay.

The application of a heavy rate of bond coat means they can inhibit water ingress into lower layers of the pavement, thereby reducing any adverse impacts of freeze / thaw cycles on the pavement structure and foundation.

Clause 936 outlines requirements for the installation and end product performance of geosynthetics [60]. To be used on the SRN they need to have a product acceptance scheme certificate that demonstrates their performance and suitability for use. Particular attention should be paid to the product acceptance scheme certification that it includes evidence of satisfactory performance when used over a URC or JRC pavement as opposed to any other type of pavement construction.

Furthermore, the product acceptance scheme certification may include limitations on the maximum crack or joint width over which the geosynthetic product can be used. Therefore, some products may be unsuitable for certain pavement types and additional assessment of pavement condition may be required prior to use.

The system is installed close to the interface between the concrete and asphalt overlay. The geosynthetic needs to be placed sufficiently deep within the bound layers (under the binder course) to reduce the risk of the geosynthetic:

- delaminating during service, as deeper placement reduces shear forces on the geosynthetic under trafficking, which if too high can lead to shoving, delamination then potholing of the above asphalt layers; and,
- being damaged during future resurfacing activities.

Geosynthetics may need to be installed on an asphalt regulating layer as opposed to directly on top of the jointed concrete, dependent on the product used. This additional layer may mean a thicker overlay is required, which needs to be considered with any site constraints.

Installation requires specific technical expertise and experience. Quality control is critical as their performance is highly dependent on the quality of installation. In hot weather, asphalt delivery trucks and the paving machine can 'pick up' the placed geosynthetic, causing debonding and folding if the bonding material softens.

Rolls of material should be installed in straight lines to avoid folds forming in the interlayer. This means that geosynthetics are not generally suitable for roundabouts and bends, as significant lapping is required at these locations. The risk of debonding due to a combination of increased shear forces at these locations could outweigh the potential benefits from their use.

If geosynthetics are being used to replace an existing asphalt overlay over a URC or JRC pavement, then a 'crack map' of those cracks present in the current overlay, often produced as part of the pavement investigation, can be retained to allow the performance of the geosynthetic to be assessed for compliance against requirements in the specification.

At end of life, it may be more difficult and time consuming to remove geosynthetics compared to other reflective crack mitigation systems. They may impact on the recyclability of the overlying asphalt if the geosynthetic is removed in the same action as the asphalt. Any additional programme and cost implications for future works need to be considered as part of designing for maintenance when selecting the most appropriate non-structural overlay option for a particular site.

Further guidance on the use of geosynthetics can be found in the RSTA/ADEPT 'Code of practice for geosynthetics and steel meshes' [61].

7.4.3. Cold applied ultra-thin surfacing (CAUTS)

Cold applied ultra-thin surfacing (CAUTS) systems are surface treatments which, due to the speed of application, can result in significant time and cost savings to the construction programme and reduced user disruption when compared with other surfacing methods.

CAUTS are applied up to 20 mm thick so they can potentially be applied over the existing concrete pavement without having to adjust drainage or VRS assets, although height limits should be carefully checked. They are a good solution for skid resistance deficient sites with site constraints, such as headroom or underbridges with weight limits.

There is a range of CAUTS products available, consisting of microsurfacings and high quality surface dressings. CAUTS products should be either CE marked or have product acceptance scheme certification. CAUTS are typically installed in one or two very thin layers (10 to 20 mm) as an overlay to an existing concrete or asphalt surfacing. Clause 923 outlines the design, installation and end performance requirements for CAUTS [60].

CAUTS applied in isolation will not inhibit reflective crack propagation; however, they are generally resistant to fretting at cracks.

CAUTS generally have higher noise generation characteristics than TSCS. This can restrict usage of CAUTS on 'noise sensitive' sites which require a 'low noise surfacing'. More information on what constitutes a 'noise sensitive' site and requirements for low noise surfacings is given in CD 236 [56].

In addition, the service life expectancy of CAUTS can be shorter than asphalt such as a TSCS; however, this may be balanced by the reduction of upfront cost and construction time. An assessment including life cycle cost savings may be required to ascertain if this technique is appropriate.

CAUTS treatments require careful programme planning as they are seasonal and sensitive to cold weather, wet weather and high humidity during installation. CAUTS should be applied between April and August during daylight. Early engagement with supply chain is recommended when considering the use of CAUTS.

For some CAUTS treatments, trafficking is essential to the process to embed chippings. Application of micro grit can be an integral part of the process to achieve early life skidding resistance.

The longevity and performance of CAUTS is dependent on the underlying pavement layer which should be in a reasonable condition. Therefore, before specifying the use of CAUTS, both the structural and surface soundness of the existing concrete pavement should be assessed.

7.4.4. Thin surface course systems (TSCS)

Thin surface course systems (TSCS) are proprietary systems in which a hot bituminous bound mixture is machine-laid onto a bond coat to form a textured surface course 20 mm to 50 mm thick (see Figure 7.7). Requirements for the design, manufacture and installation of TSCS are contained within Clause 942 [60].

The application of TSCS over existing concrete pavements can improve wet skidding resistance and noise generation characteristics. However, the application of a PMB TSCS directly over a jointed concrete pavement without the use of a reflective crack mitigation system is not recommended, unless ongoing maintenance is acceptable to address regular reflective cracks from underlying joints and cracks, which may occur rapidly after construction.

Acceptable serviceability is likely to be maintained with JRC pavements owing to increased spacing of joints versus URC; however, even with a pavement in good condition, reflective cracking should be expected to have occurred, and some will have bifurcated and spalled, within 5 years of application [58].

Figure 7.7 TSCS installation

When used over concrete pavements, TSCS should incorporate a PMB. PMBs offer multiple benefits over penetration grade bitumen; most pertinent is the additional flexibility and as a result the increased resistance to cracking. More information on PMBs is contained in Section 7.4.5.

TSCS overlay is an option for CRCP that is not suffering from defects described in Chapter 4, to address high noise or wet skidding resistance deficiencies. It can seal the surface if there is a concern that deterioration may occur rapidly through ingress of water and de-icing salts into any less frequent and therefore wider transverse cracks in the CRCP.

When a TSCS is applied directly over a concrete pavement, it is recommended to use a thick layer of bond coat to improve waterproofing as well as provide a satisfactory bond between the concrete and TSCS. Application of bond coat through by integral spray bar paver may also reduce bond coat pick up under the tyres of construction traffic, improving bonding between the concrete and TSCS.

7.4.5. Asphalt materials incorporating PMB

The ability of asphalt to resist cracking is largely dependent on the volumetrics and binder content of the mixture and the viscoelastic characteristics of the bitumen. Use of polymer modified bitumen (PMB) within asphalt can improve the ability of the asphalt to accommodate strain thereby improving the resistance of the mixture to cracking. Therefore, with the exception of where thick overlays (≥ 150 mm) are being applied, asphalt materials incorporating PMB are preferred for overlaying URC, JRC and CRCP.

Asphalt binder course and base mixtures incorporating PMB are available and can be specified through the MCHW [60], including:

- Clause 943 hot rolled asphalt (HRA);
- Clause 937 stone mastic asphalt (SMA); and,
- Clause 930 EME2 (Note: specialist binder required with extensive design protocol).

In addition to 'standard' MCHW mixtures, some asphalt manufacturers can supply proprietary base and binder course mixtures that may be used under a departure from standard application. These proprietary mixtures may use alternative aggregate gradation and contain heavily modified PMBs (i.e. incorporating a high percentage of polymers) to provide enhanced resistance to cracking.

Whilst the use of PMB asphalts alone as a treatment for jointed concrete pavements has been shown to delay reflective cracking versus asphalts incorporating standard penetration grade bitumen, the most important factor in delaying reflective cracking remains the asphalt overlay thickness and the use of a reflective crack mitigation system.

A framework of the different ‘classes’ of PMB is outlined in BS EN 14023 [62]. This framework is useful for comparing the performance of the PMBs used within asphalt mixtures between various asphalt manufacturers. Information from asphalt manufacturers on the penetration and softening point of the PMB used is readily available.

Examples of a product specification in terms of penetration and softening point for standard types of PMB are contained in Table 7.3. PMB 75/130 75 can be expected to have better resistance to cracking than PMB 45/80 60, the higher penetration and softening point is indicative of greater polymer modification. However, there is a balance to be struck between cracking resistance and workability of the mixture. Other properties outlined in BS EN 14023, in particular Fraas breaking point and elastic recovery, will also give an indication of a PMB’s resistance to cracking.

Early engagement with the supply chain is recommended prior to product specification to understand the availability of different types of PMB. Asphalt manufacturers may only use one PMB across all PMB asphalt mixture designs across all manufacturing plants, so acquiring alternative PMBs may be costly and require additional mixture design works.

The additional requirements outlined in BS EN 14023 [62] may be specified as necessary.

Table 7.3 Typical PMB product specification

Product specification	**Property**	
	Penetration at 25 °C	**Softening Point**
PMB 45/80 60	45 - 80 mm	Not less than 60 °C
PMB 75/130 75	75 - 130 mm	Not less than 75 °C

7.4.6. Bonded concrete overlays

Bonded concrete overlays have had little use in the UK, but have been widely used in the USA and Europe, where they have been shown to have a lower life cycle cost than asphalt overlays [19]. In principle, bonded concrete overlays can provide significantly better structural improvement per unit depth than asphalt overlays. However, there are practical considerations related to design, construction and use, notwithstanding the initial cost and disruption, which is higher than an asphalt overlay of equivalent thickness.

Notably, failure characteristics of bonded concrete overlays involve debonding at the interface with the underlying slab followed by rapid break-up of the overlay, resulting in the formation of large potholes / spalls requiring immediate attention and causing user disruption [19]. This unpredictable and disruptive failure mode may be unacceptable to some organisations.

At the time of publication of the CPMM, the performance of bonded concrete overlays has not been demonstrated on the SRN; there is not a specification clause in the MCHW for bonded concrete overlays and there is not a design process in the DMRB for bonded concrete overlays. Therefore, a departure from standard is required for their use.

There are several construction practicalities to consider. A bonded concrete overlay relies on a good bond with the existing concrete. Factors such as early contraction, warping or differential thermal movement can result in either an initial lack of bond or a gradual reduction in bond, both of which will significantly compromise the structural capability of the combined section. Bond failure could lead to major defects, with the concrete cracking and spalling across the carriageway width. The risk of failure of bonded concrete overlays may outweigh the potential benefits.

Furthermore, an extended curing period is often necessary before trafficking. As bonded overlays tend to have a high ratio of surface area to volume, correct curing is important to prevent excessive moisture loss by bleeding and evaporation. The curing period just after placing is particularly important to minimise drying shrinkage.

Contraction of the overlay during curing can be limited by selecting appropriate constituent materials and mixture design. Limestone aggregate has a lower coefficient of thermal expansion than flint and siliceous gravel and granite, which will reduce contraction during curing. To minimise the risk of early cracking, a low water / cement ratio concrete should be considered. Fabric reinforcing mesh in the overlay may be required to help control shrinkage-induced cracking and consideration may be given to the use of concrete containing steel fibres to help control cracking.

Any joints and wide cracks in the underlying pavement should be reproduced in the bonded concrete overlay, ensuring that the appropriate joint type (e.g. contraction or expansion) is used. However, in practice, this is difficult and labour-intensive and requires close supervision.

Site specific noise requirements need to be understood to ensure that the finished concrete surface has acceptable noise generation characteristics.

Further guidance on bonded concrete overlays is available from ‘concrete overlays for pavement rehabilitation’ [43].

7.4.6.1. Substrate preparation prior to bonded concrete overlay

The preparation of the concrete before overlay plays an important role on the long-term performance of the pavement. The overlay and existing pavement need to perform as a monolithic structure; therefore, good design, preparation and construction practices are essential for ensuring bonding is achieved.

Bonding of a concrete overlay is typically achieved by roughening the existing surface through shot blasting. Milling is not recommended as this can cause plucking of aggregates and microcracking that weakens the surface. Some proprietary bonded concretes can employ a bonding grout or epoxy. Once roughened, the surface needs to be thoroughly cleaned and dried to ensure all dust and standing water, which could inhibit bonding, is removed.

If a grout is used to assist the bond, it needs to be applied immediately before the overlay concrete is placed, otherwise premature setting of the grout may create a slip layer. The existing joints should be cleaned and resealed and any spalling at joints repaired before overlaying.

Placement temperature and careful control of the curling and warping stresses that may occur during the initial curing period post placement are also crucial to assuring that a good bond forms and is maintained. To this end, humidity and temperature differences between the layers needs to be minimised. Traffic loading should not be permitted until the concrete has reached trafficking strength to avoid disruption to this bond.

The optimum condition to place a bonded concrete overlay is when the temperature variations between the overlay surface and the existing concrete are minimal during the placement and curing period to reduce risk of debonding. Factors that affect this will be day-night temperature variations, exposure to sunlight, wind and rain.

7.4.7. Unbonded concrete overlays

When structural requirements dictate that a thick overlay is required and level constraints permit, then an unbonded overlay may be used. Unbonded concrete overlays do not need as much preparation as for bonded concrete overlays. Typical treatments prior to overlay include slab stabilisation (under slab grouting) in areas of excessive vertical movement at joints under trafficking, and full depth repairs of punchouts.

CD 226 can be used to determine the pavement thickness required, dependent on existing pavement and foundation conditions and future traffic requirements [12].

The pavement design should include a mechanism to ensure that debonding occurs by some positive means. A partially unbonded overlay should not be attempted since this cannot be specified nor produced in a uniform manner in practice. Normally debonding is achieved by use of an initial asphalt regulating layer, which adds to the total overlay level increase. An asphalt surface course may also be needed to provide noise and skidding resistance characteristics. This additional thickness should be considered when reviewing options versus site constraints.

Use of an unbonded concrete overlay does not constrain the designer to the same slab size as the underlying pavement. Therefore, CRCP may be used over a jointed concrete pavement, joints do not need to coincide with those in the existing pavement retained below.

Whilst overlays can be either URC, JRC or CRCP, CRCP is the preferred option for the SRN. CRCP overlays are more suitable for strengthening existing concrete pavements that have deteriorated to a considerable degree as well as those that are still in good structural condition. URC and JRC overlays will require a departure from standard.

7.4.8. Fractured slab techniques

If the pavement has deteriorated beyond the condition to which it may be cost-effectively maintained, or if delaying reflection cracking is the dominant design thickness criteria, then fractured slab techniques may be appropriate before overlaying with an asphalt overlay or bonded concrete overlay. Fractured slab techniques minimise the movement of the concrete pavement by reducing the effective slab length and seating the broken concrete.

The fractured slab technique used is dependent on the type of concrete pavement:

- For URC, crack, seat and overlay (CSO) as described in Clause 716 [10] is used.
- For JRC, saw cut then CSO as described in Clause 715 [10] is used.

Fractured slab techniques involve inducing fine vertical transverse cracks to create closely spaced locations where thermal contraction can take place while retaining satisfactory load carrying and load transfer characteristics (see Figure 7.8). The purpose of this is to divide the large horizontal thermal movements concentrated at the widely spaced joints amongst a larger number of cracks as a way of delaying or controlling reflective cracking in an asphalt overlay.

Figure 7.8 Crack and seat of URC pavement after slab cracking at 1 m intervals prior to asphalt overlay

Crack spacings are typically 1 m, but the recommended range is 0.75 to 2 m. This will depend on the performance required of the overlaid pavement and dividing the existing bays equally. For example, a 2 m crack spacing is not appropriate for 5 m long bays.

When a concrete pavement is cracked and seated, its load-spreading ability, or stiffness, is reduced. The reduction in stiffness will depend on factors including the remaining degree of aggregate interlock at the cracks and the crack spacing. A balance needs to be made between inducing infrequent (therefore wide) cracks that result in localised very low stiffness and increased certainty of reflection cracks developing through the asphalt overlay (albeit at less frequent intervals), and inducing frequent (therefore narrow) cracks with good aggregate interlock that result in a general lower stiffness and less risk of reflection cracks developing through the asphalt overlay.

The former situation would demand an increased asphalt overlay thickness (otherwise a mitigating saw-cut and seal treatment) to manage the anticipated reflection cracking. The latter situation needs to avoid rubblising the existing concrete pavement, i.e. providing an acceptable balance between a general reduction in stiffness that minimises the risk of reflection cracking but without compromising the structural integrity of the overlaid concrete to carry the future traffic.

Before main works commence, a site trial must be undertaken to establish the optimum crack spacing that includes measurement of the cracked concrete stiffness. The crack pattern on the surface of the concrete needs to be predominantly transverse; any longitudinal cracks longer than the crack spacing should be avoided. To help produce such crack patterns, the plant for the cracking process should have a guillotine action capable of delivering variable pre-set impact loads to the concrete surface.

For JRC, before the CSO process, longitudinal steel reinforcement is severed by transverse saw cutting between each guillotine position. To maintain aggregate interlock after saw cutting, the saw cuts should be sufficiently deep to sever the steel reinforcement only.

Once slabs have been fractured, an asphalt or unbonded concrete overlay is applied. Asphalt is common, but CRCP in particular offers good load-spreading properties that enable it to accommodate some localised variation in support from the underlying materials.

The structural overlay thickness will be determined based on future trafficking requirements and the assumed condition of the pavement. Slab fracturing reduces the effective stiffness modulus of the concrete and this needs to be considered when designing the overlay. The overlay should be designed using the analytical design approach in CD 226 [12]. This initial assessment may be based on FWD testing before slab fracturing.

The overlay thickness will need to be sufficient to delay reflection cracking based on the reduced horizontal movement at joints resulting from the cracking operation. Where the existing concrete pavement is already highly distressed such that the cracking operation reduces the integrity further and / or the joints exhibit poor load transfer characteristics, CSO with CRCP overlay may be more advantageous than CSO with asphalt overlay.

The stiffness of the fractured slabs is confirmed using an FWD prior to the overlay, to satisfy that the minimum design stiffness performance requirements are met, i.e. that each slab has not been overly cracked or improperly seated. The stiffness value required to be achieved by the FWD on site may be higher than the assumed design value to account for deterioration over time. If this is not satisfied, additional overlay thickness may be required, or localised reconstruction undertaken.

7.4.9. Partial depth and full depth reconstruction

Partial depth or full depth reconstruction may be the only option available if:

- overlays are unsuitable due to headroom constraints, underbridge capacity restrictions;
- level changes cause significant effects on adjacent infrastructure; and / or,
- the foundation requires replacement.

Partial depth reconstruction involves replacement of all the of the bound pavement layers down to the top of the foundation, whereas full depth reconstruction involves replacement of all the bound pavement layers and the foundation.

With full depth reconstruction, the subgrade will be exposed at various times. This may be disadvantageous where the foundation has stiffened with time and surface water ingress could reduce the equivalent subgrade surface modulus. Therefore, full depth reconstruction introduces an element of risk not found with partial depth reconstruction.

The design for reconstruction is covered in CD 226 [12].

Full depth reconstruction may be necessary only for just one of the lanes within a carriageway. In this case, the potential for differential thermal movements across the carriageway width would need to be evaluated and designs prepared to accommodate them, considering the thermal properties and material constituents in the adjacent concrete to be retained.

If full depth reconstruction is necessary because of constraints on raising levels, reconstruction with concrete may be preferable, as concrete is likely to require a thinner pavement than the asphalt alternative of equivalent life and so impinge less on the level constraints or less deep excavation into the foundation. Furthermore, recycled materials from the original concrete slab can be used more readily in a new concrete slab (including roller compacted concrete (RCC)) and / or hydraulically bound foundation than in a new asphalt pavement. There may also be the opportunity to undertake recycling of the foundation layers.

Appendix A - Innovations in concrete pavement maintenance

There are emerging innovations and technologies related to concrete pavement maintenance that could offer future benefits including durability, sustainability and whole-life cost. This appendix provides an outline of some of the emerging innovations in concrete pavement maintenance, their potential benefits and their current state of technology readiness.

Self-healing concrete

Concrete has self-healing ability, known as ‘autogenous healing’, it allows micro-cracks to be closed in the presence of moisture without external help. However, the process does not occur if tensile stresses are imposed on the concrete, which is unlikely with concrete pavements, and can only close cracks up to 0.3 mm wide [63].

An alternative technique is to embed dormant healing agents into concrete, which can be activated manually or by cracks appearing in the concrete. The ‘Materials for Life’ (M4L) project trialled self-healing concrete by constructing five panels using different self-healing techniques including microcapsules, bacteria, shape memory polymers and flow networks. It is understood that the project will further explore the application in concrete structures [64]. So far, self-healing concrete technologies are in the early development stages with no large scale trials having been undertaken.

Engineered cementitious composite (ECC)

ECC is a type of ductile fibre reinforced cementitious composite incorporating only a small proportion of steel fibre, typically 2 %. Developed in the 1990s and with materials design theory based on micromechanics and fracture mechanics, *“the material reveals a metal-like strain hardening behaviour under uniaxial tensile loading, normally with a tensile strain capacity of over 3 %, which is about 300 times compared with normal concrete”* [65] [66]. The use of ECC as an alternative concrete pavement material could enhance the long term durability performance by “*developing extensive micro-cracks under flexural fatigue loading, in addition to a much enhanced modulus of rupture*” [67].

Another area of potential application of ECC is to use its self-healing and self-sensing attributes. ECC’s autogenous healing feature is promoted by the control of micro-cracks. The self-sensing feature is achieved by introducing carbon black powders or milled carbon fibre as conductive filler in ECC to enhance the piezo-resistive behaviour [68].

Internal curing concrete

Internal curing provides curing water from a constituent within the concrete to replace water lost during the hydration process, resulting in a reduction in drying shrinkage and curling of the concrete. Internal curing has been used mostly on concrete bridge desks, typically through use of expanded lightweight aggregate; however, superabsorbent polymers, cellulose fibres, or recycled concrete have been used.

Applications have been extended to CRCP, URC new construction and full depth repairs. In CRCP, the crack spacing is reported to be 20 - 30 % greater than that in conventional concrete and the cracks that developed later had reduced widths. In URC, the internal curing is reported to improve pavement durability and performance through small reductions in unit weight, elastic modulus, coefficient of expansion and curling stresses and a small increase in strength.

A trial in Indiana, USA in 2014 using internal curing with expanded slag aggregate in high early strength full depth concrete pavement repair suggested the benefit of reduced shrinkage cracking and increased water curing even after traffic opening. LCCA of the project in the US suggested lower whole-life cost for internally cured concrete compared to conventional concrete [69].

Geopolymer concrete, or Earth friendly concrete (EFC)

The geopolymer concrete or Earth friendly concrete (EFC) does not contain ordinary Portland cement, instead a combination of by-products from industrial processes is used, such as water purification, waste incineration and production of steel, with quarry dusts, agricultural waste and recycled aggregates. The formation of geopolymers is based on aluminosilicate and it hardens with the addition of an alkaline activator, usually a combination of sodium silicate solution (water glass) and sodium hydroxide (NaOH) [70].

One of the key drivers for the use of geopolymer concrete is the environmental benefit of reducing the CO_2 emissions by 80 % compared to Portland cement. Other benefits include high flexural strength, low shrinkage and good workability which could qualify for a suitable pavement concrete.

The application of geopolymer concrete in the UK construction industry is still in the development stage, although it has been rapidly advancing in Europe and Australia. A 40,000 m^3 geopolymer concrete pavement was built on Brisbane West Wellcamp airport using a slip form paving machine in 2014 [71]. Heath et al. [72] discussed the potential for using geopolymer concrete in the UK and suggested the barriers to implementation includes:

- the anticipated decrease in PFA and GGBS produced in the UK;
- a less developed market in the UK; and,
- a lack of standards and long-term performance data.

Appendix B - Treatment options for defects

Table B.1 Surface defects treatment options

<table>
<tr><th>Defect
(CPMM Section)</th><th colspan="4">Treatment options (See Note 1)</th></tr>
<tr><td>Surface irregularities (3.1.1)</td><td>Bump cutting
(see Section 5.4.7)
(HR)</td><td>Thin bonded repair
(see Section 5.3.2)
(HR)</td><td>Full depth repair
(see Sections 5.4.2 to 5.4.4)
(LTR)</td><td>Bay replacement
(see Section 5.4.1) (LTR)</td></tr>
<tr><td>Surface scaling
(3.1.2)</td><td rowspan="2">No treatment</td><td rowspan="2">Retexturing
(see Section 7.4.1)
(HR)</td><td rowspan="2">Thin bonded repair
(see Section 5.3.2) (LTR)</td><td rowspan="2">Shallow repair (see Section 5.3.2)
(LTR)</td></tr>
<tr><td>Crazing
(3.1.3)</td></tr>
<tr><td>Pop-outs
(3.1.4)</td><td>No treatment</td><td>Single particle pop-out repair (see Section 5.3.1) (LTR)</td><td>Thin bonded repair
(see Section 5.3.2) (LTR)</td><td>Shallow repair (see Section 5.3.2)
(LTR)</td></tr>
<tr><td>Defective joint seal
(3.1.5)</td><td>Replace joint seal
(see Section 5.2.1)
(LTR)</td><td colspan="3">Where excessive movement is identified, additional treatments are recommended to address movement.</td></tr>
<tr><td>Shallow joint spalls
(3.1.6)</td><td>Widen joint sealant groove (see Section 5.2.2) (LTR)</td><td>Thin bonded repair
(see Section 5.3.2)
(LTR)</td><td>Shallow repair with cementitious or resin repair material (see Section 5.3.2)
(LTR)</td><td>Shallow repair with polymeric material (see Section 5.3.2) (HR)</td></tr>
<tr><td colspan="5">HR = Holding repair (expected service life 3 - 7 years)
LTR = Long term repair (expected service life > 10 years)</td></tr>
</table>

Note 1: Treatment options are dependent on defect severity and characteristics of the defect. See Chapter 3 for more details on the selection of the appropriate treatment options.

Table B.2 Structural defects treatment options

Defect (CPMM Section)	**Pavement type**	**Characteristic**	**Treatment options**	
			Recommended (See Note 1)	**Other**
Depression (settlement) (3.2.1)	URC / JRC / CRCP	Intact pavement, no drainage issue	Under slab grouting and slab lifting (see Sections 5.4.5 and 5.4.6) (HR)	Bay replacements (see Section 5.4.1) (LTR)
		Localised area, drainage issue	Drainage renewal and bay replacement or full depth repair including foundation as necessary (see Sections 5.4.1 to 5.4.4) (LTR)	
		Extensive areas	Full depth reconstruction including drainage renewal (see Section 7.4.9) (LTR)	
Heave (3.2.2)	URC / JRC / CRCP	Localised high areas	Full depth reconstruction (see Section 7.4.9) (LTR)	Bump cutting (see Section 5.4.7) (HR)
		Severe cases	Full depth reconstruction (see Section 7.4.9) (LTR)	
Punchouts (3.2.3)	CRCP / JRC	-	Full depth repair with foundation replacement as required (see Sections 5.4.3 and 5.4.4) (LTR)	
Deep joint spalls (3.3.1)	URC	-	Bay replacement (see Section 5.4.1) (LTR)	Full depth repair (see Section 5.4.3) (LTR)
	JRC	-	Full depth repair (see Section 5.4.4) (LTR)	Bay replacement (see Section 5.4.1) (LTR)

Defect (CPMM Section)	Pavement type	Characteristic	Treatment options		
			Recommended (See Note 1)	**Other**	
Corner cracks (3.3.2)	URC / JRC	≤ 0.5 mm width	No treatment advised		
	URC	> 0.5 mm width	Bay replacement (see Section 5.4.1) (LTR)	Inlaid crack repair (see Section 5.3.3) (HR)*	Full depth repair (see Section 5.4.3) (LTR)
	JRC	0.5 - 1.5 mm width	Inlaid crack repair (see Section 5.3.3) (HR)	Full depth repair (see Section 5.4.4) (LTR)	
		> 1.5 mm width	Full depth repair (see Section 5.4.4) (LTR)	Inlaid crack repair (see Section 5.3.3) (HR)*	
Cracks around ironwork (3.3.3)	URC / JRC / CRCP	< 0.5 mm width	No treatment advised		
		≥ 0.5 mm width	Inlaid crack repair (see Section 5.3.3) (HR)*	Full depth repair or bay replacement and reinstate ironwork (see Sections 5.4.1 to 5.4.4) (LTR)	Relocate ironwork in verge and full depth repair or bay replacement (see Sections 5.4.1 to 5.4.4) (LTR)
Transverse or Diagonal cracks (3.3.5 and 3.3.4)	URC	≤ 0.5 mm width	No treatment advised		
		> 0.5 mm width	Bay replacement (see Section 5.4.1) (LTR)	Inlaid crack repair (see Section 5.3.3) (HR)*	Full depth repair (see Section 5.4.3) (LTR)
	JRC	≤ 0.5 mm width	No treatment advised		
		0.5 - 1.5 mm width	Inlaid crack repair (see Section 5.3.3) (HR)	Bay replacement (see Section 5.4.1) (LTR)	Full depth repair (see Section 5.4.4) (LTR)
		> 1.5 mm width	Bay replacement (see Section 5.4.1) (LTR)	Full depth repair (see Section 5.4.4) (LTR)	Inlaid crack repair (see Section 5.3.3) (HR)*

Defect (CPMM Section)	Pavement type	Characteristic	Treatment options		
			Recommended (See Note 1)	Other	
Transverse / diagonal cracks (3.5.1)	CRCP	≤ 1.0 mm width	No treatment advised		
		1.0 - 1.5 mm width	Inlaid crack repair (see Section 5.3.3) (HR)	Full depth repair (see Section 5.4.2) (LTR)	
		> 1.5 mm	Full depth repair (see Section 5.4.2) (LTR)	Inlaid crack repair (see Section 5.3.3) (HR)*	
		>1.5 mm width and bifurcated, polygonal or closely spaced	Full depth repair (see Section 5.4.2) (LTR)		
Longitudinal cracks (3.3.6 and 3.5.2)	URC	≤ 0.5 mm width	Crack stitching (see Section 5.4.8) (LTR)	Inlaid crack repair (see Section 5.3.3) (HR)	
		> 0.5 mm width	Bay replacement (see Section 5.4.1) (LTR)	Inlaid crack repair (see Section 5.3.3) (HR)*	Full depth repair (see Section 5.4.3) (LTR)
	JRC	≤ 0.5 mm width	No treatment advised		
		0.5 - 1.5 mm width	Crack stitching (see Section 5.4.8) (LTR)	Inlaid crack repair (see Section 5.3.3) (HR)	
		> 1.5mm width	Bay replacement (see Section 5.4.1) (LTR)	Full depth repair (see Section 5.4.4) (LTR)	Inlaid crack repair (see Section 5.3.3) (HR)*
	CRCP	1.0 - 1.5 mm width	Inlaid crack repair (see Section 5.3.3) (HR)	Full depth repair (see Section 5.4.2) (LTR)	
		> 1.5 mm width	Full depth repair (see Section 5.4.2) (LTR)	Inlaid crack repair (see Section 5.3.3) (HR)*	

Defect (CPMM Section)	Pavement type	Characteristic	Treatment options	
			Recommended (See Note 1)	**Other**
Stepping (3.3.7) and / or Slab rocking (3.3.8)	URC / JRC	No drainage issue	Under slab grouting and slab lifting (see Sections 5.4.5 and 5.4.6) (LTR)	Bay replacement or full depth repair including foundation as necessary (see Sections 5.4.1, 5.4.3 and 5.4.4) (LTR)
		Drainage issue	Drainage renewal and bay replacement or full depth repair including foundation as necessary (see Sections 5.4.1, 5.4.3 and 5.4.4) (LTR)	Under slab grouting and slab lifting (see Sections 5.4.5 and 5.4.6) (HR)
Compression failures (blow-ups) (3.3.9)	URC / JRC	-	Full depth repair or bay replacement including introducing new expansion joints across full carriageway width (see Sections 5.4.1 to 5.4.4) (LTR)	
HR = Holding repair (expected service life 3 - 7 years) LTR = Long term repair (expected service life > 10 years) * Cracks of this size are expected to have little or no load transfer. Inlaid crack repairs over these cracks, depending on crack width, are likely to have a significantly reduced service life.				

Note 1: Recommended treatments are based on the characteristic of the defect. Other treatment options may be appropriate depending on pavement maintenance strategy and serviceability requirements.

Appendix C - Highways England 3D Process

The Highways England 3D process (Develop, Design, Deliver) is currently used for renewals and will also be used to manage maintenance works being delivered within the Concrete Roads Programme within RP2 and future road periods. The process consists of seven stages, as illustrated in Figure C.1, with associated go / no-go stage gates providing end-to-end scheme governance.

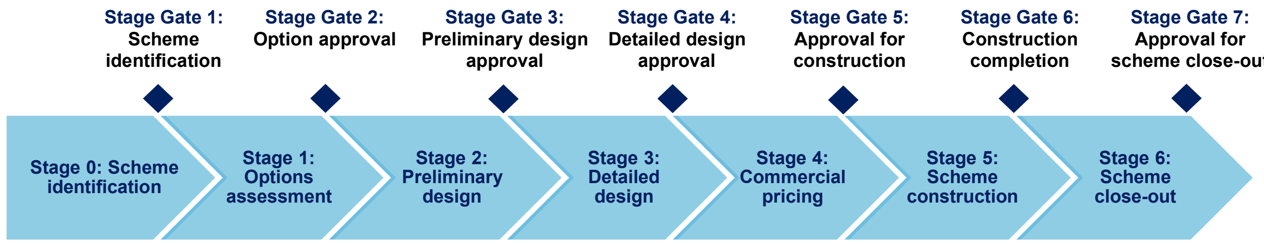

Figure C.1 3D Process showing Stages and Stage Gates

The Stage Gate reviews provide a check and challenge of a scheme's readiness to progress to the next stage, allow management of risks and opportunities, and capture agreed actions and decisions. Each scheme also has its own Scheme Passport which helps to standardise scheme management and acts as a central repository of scheme information through its lifecycle.

Key activities within each stage can be summarised as:

Stage 0: Scheme identification

- Need identified from national concrete survey data and prioritised
- Scheme type provisionally assigned as either "Lifecycle Extension Works" (LEW) or "Reconstruction" (Recon) (Note: the terms LEW and Recon have specific meaning in the context of the Concrete Roads Programme within RP2)

Stage 1: Options assessment

- Survey pack prepared and surveys are completed
- Information gathering
- Options are proposed and a final option is selected

Stage 2: Preliminary design

- Preliminary design is completed including all relevant documentation
- Cost estimate developed

Stage 3: Detailed design

- Detailed drawings, specification and schedules created
- Detail design completed

Stage 4: Commercial pricing

- Programme finalised
- Technical assurance and updating design documents
- Request for pricing sent to suppliers and responded to
- Pricing submission reviewed, and any clarifications sought and resolved

Stage 5: Scheme construction

- Task Order awarded
- Mobilisation
- Construction
- Testing and completion certificates finalised

Stage 6: Scheme close-out

- Asset data updated and handed over to Highways England
- Financial accounts closed
- Post project review
- Scheme formally closed

Concrete maintenance schemes within Highways England's Concrete Roads Programme will be delivered as either LEW or Recon, with the roles and responsibilities within the 3D process differing slightly for the two types of scheme. The main roles within the 3D process for concrete schemes will be delivered by:

- Highways England Concrete Centre of Excellence (CoE)
- Highways England Safety, Engineering and Standards (SES)
- Highways England Regional Operations Teams
- Concrete Framework Designers
- Concrete Framework Contractors

In recent years, maintenance of legacy concrete pavements has been limited to surface treatments and minor repairs which has resulted in a lack of experience in relation to more extensive maintenance interventions and reconstruction. Therefore, the CoE and SES play a key role in providing technical expertise and assurance within the 3D process to ensure that DMRB documents are complied with. The CoE and SES involvement ensures that:

- Lessons learnt and best practice can be held centrally and shared with Regions on a scheme by scheme basis
- Innovative techniques are appropriately adopted, managed and benefits realisation is achieved
- Efficiencies are realised
- As the Concrete Roads Programme develops, examples of best working practice can be used to develop improved standards

Appendix D - Concrete visual defect guide

Visual guide to identification of defects in concrete pavements

Introduction

This visual guide aims to support Highways England's staff and supply chain to identify and describe defects in concrete pavements.

Visual references and descriptions are provided for each defect. Descriptions are based on those given in *DMRB CD 227, Concrete Pavement Maintenance Manual 2021 and DIO Technical Standard - Airfield 05 -2019.*

The current scope of this guide is limited to recognition and characterisation of defects. Highways England's concrete pavement maintenance manual (CPMM) provides guidance on treatment techniques and materials to repair defects in concrete pavements.

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Asphalt or high friction surfacing (HFS) delamination

Description

Delamination is the loss of bond between concrete and surfacing material. It can result in the loss of a discrete area of the surface course or high friction surfacing, exposing the underlying layer.

Severe cases often cause ‘pothole’ like defects, where the full depth surface layer is lost.

Loss of the surfacing may be a result of the underlying layer being in poor condition.

What to record

Parameter	Unit	Methodology
Location	-	HAPMS section + chainage + lane + direction + GPS coordinates
Position in the lane	-	Nearside Wheel Track (NSWT) / Offside Wheel Track (OSWT) / Lane centre
Length of the delaminated area	[mm]	
Width of the delaminated area	[mm]	
Depth of the delaminated area	[mm]	
Additional information	-	Record associated features including ridges or bumps resulting from the defect

Note: Measurements may be approximated due to access restrictions

Reference: Asphalt delamination - Highways England

Reference: Asphalt delamination - Highways England highways england

Compression failures (blow-ups)

Description

Compression failures (also known as blow-ups) can occur at a transverse or longitudinal crack or joint that is not wide enough to permit thermal expansion of the concrete slabs.

The insufficient width can be caused by infiltration of incompressible materials into the joint space, locking of joints in adjacent slabs and / or insufficient space to expand due to high temperature.

When there is insufficient space for the concrete to expand horizontally, a localised upward movement of the slab (buckling) or shattering can occur in the vicinity of the joint.

A compression failure can also occur at utility cuts and drainage inlets.

What to record

Parameter	Unit	Methodology
Location	-	HAPMS section + chainage + lane(s) + direction + GPS coordinates
Joint or crack	-	Record where defect affected
Number of slabs affected	-	

Note: Measurements may be approximated due to access restrictions

Reference: Compression failure - Highways England

Reference: Compression failures and patching - Highways England

Corner crack(s)

Description

A corner crack is a crack across a corner of a concrete slab with a crack length between 0.3 m and 2 m.

Cracks that are 2 m or more in length should be considered as a diagonal crack. A corner crack less than 0.3 m long can be considered as a shallow or deep joint spall.

A corner crack differs from a corner spall (pg. 9) in that a crack extends vertically through the entire slab thickness, whilst a corner spall intersects the joint at an angle. Corner cracking can be caused by the restraint of thermal movements or a combination of traffic and environmental loading.

What to record

Parameter	Unit	Methodology
Location	-	HAPMS section + chainage + lane + direction + GPS coordinates
Position in the lane	-	
Length	[mm]	
Width	[mm]	
Additional information	-	Record any associated settlement or pumping

Note: Measurements may be approximated due to access restrictions

Reference: Corner cracking - Highways England

Reference: Corner cracking – Highways England

Cracks (transverse, longitudinal and diagonal)

Description

Cracks that divide the slab into two or three pieces are usually caused by factors including load repetition, curling stresses, poor support or shrinkage stresses.

Transverse cracks divide the slab into multiple pieces in the transverse direction. Longitudinal cracks are oriented parallel to the pavement centreline. Diagonal cracks traverse between perpendicular joints in slabs but exclude corner cracks (pg. 4).

For slabs divided into four or more pieces, see ‘shattered slab’ (pg. 16).

What to record

Parameter	Unit	Methodology
Location	-	HAPMS section + chainage + lane + direction + GPS coordinates
Position in the lane	-	As appropriate
Length	[mm]	
Width	[mm]	
Crack direction	-	Transverse / longitudinal / diagonal
Additional information	-	Record any associated spalling, settlement or pumping

Note: Measurements may be approximated due to access restrictions

Reference: Transverse crack - Highways England

Reference: Longitudinal crack - Highways England

Cracks around ironwork

Description

Cracks around ironwork are those emanating from a discontinuity caused by a gully, utility cover or core studs within a slab. Cracking around ironwork is likely to extend out from the corners.

Cracking can occur where ironwork penetrates through the slab or where the slab rests on the ironwork construction.

This cracking can occur under environmental loading alone and / or traffic loading.

What to record

Parameter	Unit	Methodology
Location	-	HAPMS section + chainage + lane + direction + GPS coordinates
Position in the lane	-	NSWT / OSWT / lane centre
Length	[mm]	
Width	[mm]	
Additional information	-	Record any associated spalling, settlement or pumping

Note: Measurements may be approximated due to access restrictions

Reference: Cracks Around Manhole - Highways England

Reference: Cracks Around Manhole - Highways England

Crazing

Description

Crazing (or ‘map cracking’) is a network of shallow, fine or hairline cracks which extend only through the upper surface of the concrete. The cracks tend to intersect at angles of 120 degrees and generally represent a spider's web, it usually develops progressively from a corner or edge of a bay.

Crazing is usually caused by excessive laitance or improper curing during construction, it may lead to surface scaling (pg. 21).

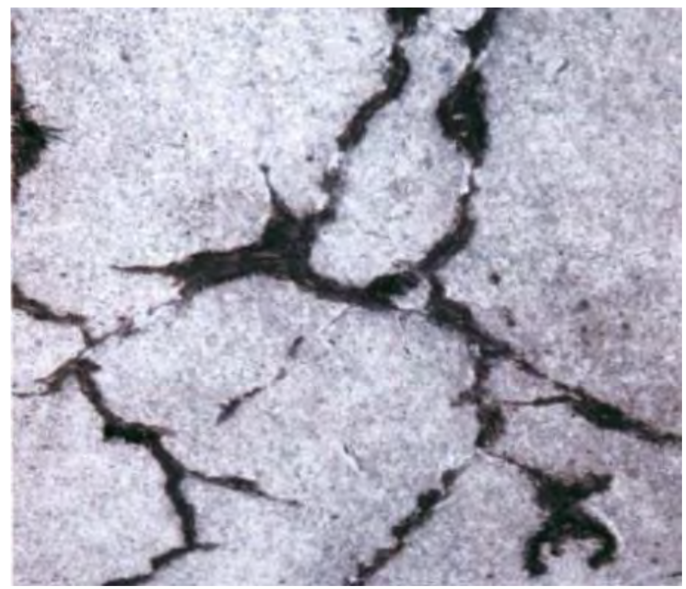
Visual signs of expansion and / or gel or staining at the surface is an indication of alkali silica reaction.

What to record

Parameter	Unit	Methodology
Location	-	HAPMS section + chainage + lane + direction + GPS coordinates
Position in the lane	-	NSWT / OSWT / lane centre
Length of the crazed area	[mm]	
Width of the crazed area	[mm]	
Additional information	-	Record any associated colouration, visible gel, spalling or cracking

Note: Measurements may be approximated due to access restrictions

Reference: Alkali silica reaction - DIO Technical Standard - Airfield 05 -2019 [2]



Reference: Cobweb crazing - Concrete Pavement Maintenance Manual [4]

Defective joint seals

Description

A defective joint seal is where the seal is not performing as intended and no longer serves its purpose.

Defective joint seals enable detritus to accumulate in the joint and / or allow significant infiltration of water. Accumulation of incompressible materials prevents the slabs from expanding and may result in the spalling, cracking or compression failures.

Typical types of joint seal damage include (a) stripping of joint sealant, (b) extrusion of joint sealant, (c) weed growth, (d) hardening of the joint sealant (oxidation), (e) loss of bond to the slab edges, (f) cohesive failure of the joint seal and (g) lack of sealant in the joint.

What to record

Parameter	Unit	Methodology
Location	-	HAPMS section + chainage + lane + direction + GPS coordinates
Joint type	-	Transverse or longitudinal joint
Number of joints affected	-	Count number of joints affected
Type of failure	-	Record failure type

Note: Measurements may be approximated due to access restrictions

Reference: Joint seal damage and spalling - Highways England

Reference: Joint seal damage - Highways England highways england

Joint / Corner Spall

Description

A joint / corner spall is the breakdown of the surface material of the slab edges within 600 mm of the joint / corner and typically intersects the joint at an angle.

The main causes of shallow spalls are infiltration of incompressible detritus into the joint groove, and damage due to traffic and environmental loading. They are typically wedged shaped and taper towards the joint. The possible causes of deep spalls are traffic loading, dowel / tie bar issues, ingress of solid material or lack of subbase support.

A shallow joint spall extends less than one third of slab depth. A deep joint spall extends more than one third of slab depth[7].

What to record

Parameter	Unit	Methodology
Location	-	HAPMS section + chainage + lane + direction + GPS coordinates
Position in the lane	-	NSWT / OSWT / lane centre
Length	[mm]	
Number of joints affected	-	Count number of joints affected
Number of slabs affected	-	Count number of slabs affected

Note: Measurements may be approximated due to access restrictions

Reference: Joint spall - Highways England

Reference: Joint spall – Highways England

Patching

Description

A patch is an area where the original pavement has been removed and replaced by a fill material that does not have equivalent properties to the parent concrete.

For condition evaluation, patching is divided into two types: small (less than 0.5 m^2) and large (over 0.5 m^2).

Patching is typically used as a short term repair.

What to record

Parameter	Unit	Methodology
Location	-	HAPMS section + chainage + lane + direction + GPS coordinates
Position in the lane	-	NSWT / OSWT / lane centre
Length	[mm]	
Width	[mm]	
Material		Asphalt / Resin / Polymeric
Condition		Describe condition of patch
Additional information	-	Record any associated spalling or cracking

Note: Measurements may be approximated due to access restrictions

Reference: Patching failure - Highways England

Reference: Patching – Highways England

0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20

Polished aggregate / Texture loss

Description

Texture loss is a reduction in surface macrotexture. Macrotexture is the visible roughness of a surfacing material, enabling drainage of water and dissipation of noise. Macrotexture is reduced as the cement mortar matrix and any grooves or ridges applied to the surface break down.

Polishing of aggregate is when exposed fine or coarse aggregates are gradually abraded and polished by traffic loading. Polishing is the reduction in the aggregate's microtexture. Microtexture is the microscopic properties of the surface that are associated with friction.

Texture depth is often used with skidding resistance surveys to identify at-risk areas for further investigation.

What to record

Parameter	Unit	Methodology
Location	-	HAPMS section + chainage + lane + direction + GPS coordinates
Position in the lane	-	NSWT / OSWT / lane centre
Length	[mm]	
Width	[mm]	
Additional information	-	Record any associated visible defects

Note: Measurements may be approximated due to access restrictions

Reference: Texture loss – Highways England

Reference: Good texture – Highways England

Pop-outs

Description

Pop-out is an isolated loss of a small area of surface material. It can be caused by a small piece of pavement that breaks loose from the surface due to freeze-thaw in combination with expansive aggregates as a result of a physical action or a chemical reaction.

Isolated pop-outs can be a result of the concrete containing clay, organic or friable materials. These pop-outs, caused by contamination, tend to occur in early life.

Retexturing may cause pop-outs depending on the strength of the aggregate in the concrete and strength of the concrete itself.

A pop-out typically ranges from 25 mm to 100 mm in diameter and from 10 mm to 50 mm deep.

What to record

Parameter	Unit	Methodology
Location	-	HAPMS section + chainage + lane + direction + GPS coordinates
Position in the lane	-	NSWT / OSWT / lane centre
Number of pop-outs	-	Only record if more than three pop-outs per square metre

Note: Measurements may be approximated due to access restrictions

Reference: Pop-out – Highways England

Reference: Pop-out – Highways England

Pumping

Description

Pumping is where fine-grained water-borne particles of subbase or subgrade material are pumped out of joints or cracks. It typically occurs where there are vertical deflections associated with heavy vehicular loading and inadequate slab support.

Pumping can be seen as staining of water or detritus on an otherwise dried concrete pavement surface. As a consequence of pumping of subbase or subgrade material, a void is formed under the slab and the volume of this void grows where these conditions continue.

Pumping and the associated loss of support may lead to cracking of the slab under repeated loading.

What to record

Parameter	Unit	Methodology
Location	-	HAPMS section + chainage + lane + direction + GPS coordinates
Joint or crack	-	Record defect type and direction e.g. longitudinal or transverse
Number of slabs affected	-	One pumping joint is counted as two bays affected

Note: Measurements may be approximated due to access restrictions

Reference: Pumping - Concrete Surfaced Airfields, Paver Distress Identification Manual [1]

Reference: Pumping - Highways England

Punchouts

Description

Punchouts are localised defects in reinforced concrete, in which fragments of broken concrete may be 'punched out' by the action of traffic downwards into the underlying subbase layer.

Mainly arise in CRCP but can occasionally occur in JRC, the concrete normally ruptures at the depth of the reinforcement.

A punchout typically occurs where localised closely spaced transverse cracks have developed along with longitudinal cracks, resulting in progressive slab disintegration under traffic.

What to record

Parameter	Unit	Methodology
Location	-	HAPMS section + chainage + lane + direction + GPS coordinates
Position in the lane	-	NSWT / OSWT / lane centre
Length	[mm]	
Width	[mm]	

Note: Measurements may be approximated due to access restrictions

Reference: CRCP punchout and patching failure - Highways England

Reference: CRCP punchout – Highways England

Rust staining

Description

Rust staining is an indication of corroded steel reinforcement, dowel bars or tie bars that may lead to accelerated deterioration of the concrete.

When steel corrodes, the resulting rust occupies a greater volume than the steel. This expansion creates tensile stresses in the concrete, which can eventually cause cracking and spalling.

Corrosion of dowel bars at joints is likely to reduce load transfer efficiency between bays and the dowel bars could shear at the joint.

What to record

Parameter	Unit	Methodology
Location	-	HAPMS section + chainage + lane + direction + GPS coordinates
Position in the lane	-	NSWT / OSWT / lane centre
Additional information	-	Record any cracking or spalling

Note: Measurements may be approximated due to access restrictions

Reference: Rust staining and patching

Shattered slab

Description

Shattered slabs are slabs that have been broken into four or more pieces by intersecting cracks. This is likely caused by overloading or inadequate slab support. Shattered slabs may deteriorate quickly into a safety hazard.

If all pieces or cracks are contained within a corner crack, the distress is categorised as a corner crack.

Where both longitudinal cracks and transverse cracks intersect, there is a risk of spalling and potentially the formation of punchouts in JRC and CRCP.

What to record

Parameter	Unit	Methodology
Location	-	HAPMS section + chainage + lane + direction + GPS coordinates
Number of slabs affected	-	Count number of slabs affected
Additional information	-	Record any associated spalling, settlement or pumping

Note: Measurements may be approximated due to access restrictions

Reference: Shattered slab - Highways England

Reference: Shattered slab - Highways England

Shrinkage cracks

Description

Shrinkage cracks are hairline cracks that are usually only up to a metre long and do not extend across the entire slab. They form during the setting and curing of the concrete and usually do not extend through the depth of the slab. [1]

They generally form in a pattern of short cracks approximately parallel to each other, oriented diagonally to the bay sides and not extending to the edges of the slab.

Pavement performance is generally not affected by these cracks because aggregate interlock is maintained.

What to record

Parameter	Unit	Methodology
Location	-	HAPMS section + chainage + lane + direction + GPS coordinates
Position in the lane	-	NSWT / OSWT / lane centre
Length	[mm]	
Width	[mm]	

Note: Measurements may be approximated due to access restrictions

Reference: Shrinkage crack - Concrete Surfaced Airfields, Paver Distress Identification Manual [1]

Slab rocking

Description

Slab rocking is a visible or audible vertical movement at a joint (or crack) under vehicle loading. It may only be detectable under HGV loading. It can cause pumping and is often identified from pumping stains.

It is a symptom of poor slab support and normally requires any load transfer devices at both ends of the slab (or reinforcement across cracks), to have yielded before it occurs.

Slab rocking can lead to more safety critical defects including stepping, spalling and cracking.

What to record

Parameter	Unit	Methodology
Location	-	HAPMS section + chainage + lane + direction + GPS coordinates
Elevation	[mm]	
Additional information		Record any associated spalling, settlement or pumping

Note: Measurements may be approximated due to access restrictions

Reference: Slab rocking - CORD [6]

Reference: Slab rocking - Highways England

Slab settlement / Joint stepping

Description

Slab settlement or joint stepping is a difference in elevation of a pavement either across a length of pavement (settlement) or across a joint or crack (stepping).

After initiation, stepping typically increases with time under repeated dynamic loading and continues to degrade the ride quality. This can result in spalling at the concrete joint or cracking in the slab.

Settlement or joint stepping can be associated with pumping (pg. 13) and can be linked to slab rocking (pg. 18). Major lane settlement may be associated with geotechnical issues.

What to record

Parameter	Unit	Methodology
Location	-	HAPMS section + chainage + lane + direction + GPS coordinates
Direction of step / settlement	-	Transverse / Longitudinal / Both
Elevation	[mm]	
Additional Information	-	Record any associated pumping, cracking or spalling

Note: Measurements may be approximated due to access restrictions

Reference: Joint stepping - Highways England

Reference: Slab settlement - Highways England

Surface irregularities (poor surface profile)

Description

Surface irregularities are bumps or depressions in the pavement which lead to a poor surface profile and often affect ride quality. They can be associated with defects such as:

- asphalt delamination;
- pop-outs;
- stepping;
- punchouts;
- settlement; or,
- spalling.

Poor surface profile is typically evaluated with machine surveys such as TRACS assessment and can be categorised in accordance with CS 230.

At ‘severe deterioration’ level it may be a safety hazard.

What to record

Parameter	Unit	Methodology
Location	-	HAPMS section + chainage + lane + direction + GPS coordinates
Defect type	-	Record visible defects
Additional information		Record any associated pumping, cracking or spalling

Note: Measurements may be approximated due to access restrictions

Reference: Surface irregularity - Highways England

Reference: Surface irregularity - Highways England

Surface scaling

Description

Surface scaling is the delamination or disintegration of the slab surface to the depth of the defect occurring over a portion of a slab.

Typically, it is caused by shallow cracks developing as a result of the expansion of water as it freezes in saturated concrete. Repeated freeze-thaw cycles allow further water ingress and crack propagation.

Surface scaling can also be caused by carbonation of the concrete.

What to record

Parameter	Unit	Methodology
Location	-	HAPMS section + chainage + lane + direction + GPS coordinates
Position in the lane	-	NSWT / OSWT / lane centre
Length	[mm]	
Width	[mm]	

Note: Measurements may be approximated due to access restrictions

Reference: Surface scaling – Highways England

Reference: Surface scaling – Highways England

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