

The Effects of Inundation on Pavements

Contractors Report

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1. Introduction

The Highways Agency's Network Services Technical Services Team commissioned Research and Development support for the delivery of a project looking into the effect on the highway pavement infrastructure of inundation by flood water and freeze/thaw conditions.

The commission was designated Task 383(1308) JCBS, under the Provision of Services Framework Contract: 2/1308 Technical Consultancies.

The scope of the study was to carry out the following and report the findings:

- 1.1 Review existing available published information and previous research relating to the in-service performance of road pavements subject to inundation on a short or long duration.
- 1.2 Use the HAPMS database to detect any change in parameters as a result of inundation and evaluate their significance on the performance of the pavement.
- 1.3 Seek information from Local Authority and HA staff on any regularly inundated roads in their area, what they use to surface them and how these materials have performed.
- 1.4 Review existing available published information and previous research relating to inundation of road pavements and the effect of saturation on road making materials separately, i.e. sub base and asphalt.
- 2.1 Construct a trial pavement in our laboratory and test the sub base and complete surface for the effects of inundation.
- 2.2 Take samples of asphalt and assess the effect of water saturation on stiffness.

The first element concerns seeking information on existing roads and road making performance under inundation conditions. This involves web and library searches, and by asking Local Authorities for their strategy and details of what maintenance they have done in situations where inundation has or might occur.

A site was identified by the HA project sponsor on the M50 for consideration using past records from HAPMS to evaluate the effects of an inundation event on an existing motorway pavement.

The second element simulates inundation in the laboratory on a small scale by manufacturing a box containing sub base and asphalt and testing with the Portable Dynamic Plate and Falling Weight Deflectometer, with dry and inundated cycles.

2. Background

Flooding has become a regular feature of the weather over the last few years. Major floods occurred in England as follows:

- 2004 Boscastle and N Cornwall
- 2005 (January) Carlisle and Cumbria
- 2006 (August) Aldershot
- 2007 (June) E. Yorkshire, Midlands, Gloucestershire, Hereford and Worcester
- 2007 (July) Gloucestershire, Herefordshire, Worcestershire, Oxfordshire, Berkshire, South Wales
- 2008 (January) Lincolnshire West and South Yorkshire, (December) Devon, Somerset, Dorset
- 2009 (February) Chelmsford, Essex, W Sussex, Wiltshire, Devon, Somerset, Hampshire, (June) Sheffield, Derbyshire, (November) Cockermouth and W Cumbria
- 2010 (July) Merseyside, (August) Kent, Hampshire, (November) Cornwall

The motorway network was particularly affected in 2007 (M5, M50 Hereford and Worcester, A40 Gloucestershire, and also in 2008 (M1 and M62, M5, A30, A303) and 2009, (M4 at Severn crossing).

The effect of climate change will be to not only increase the amount of rainfall by about 20% in the next 40 years, but also the intensity of storms. The latter is particularly important as it directly affects the amount of flooding.

In rural areas traversed by motorways, the effect of development by increasing the speed and amount of water run off is likely to be small. However, in small catchments, especially those closer to the urban centres, the catchment itself could become more prone to flash flooding, such as happened in Boscastle and Cockermouth. In these cases the water runs off roofs, hardstandings and access roads rather than soaks into the ground, as happened before the development was constructed. Industrial and retail park developments can have a very large impermeable area as can modern high density housing.

In an attempt to mitigate this effect the Government set up the Pitt Review in 2007 which concerned itself with the administrative responsibilities for managing flood events. This led to the Flood and Water Management Bill 2010, still to be implemented.

From a highway engineering point of view it set down a requirement that the run off from a development should be no more than would have happened on a normal 'greenfield' site. Developers therefore need to take measures such as permeable and porous pavements, swales, ditches and ponds and the use of soakaways, to ensure this happens. It will be able to be implemented when the necessary supporting National Standard documents are in place. Motorway designs since 1970's have implemented such practices by the presence of drainage lagoons.

It has been recognised however that catchments will continue to flood and the extent and frequency may well increase as a result of Climate Change. The Environment Agency (EA) now recommend that the current designs for a 1 in 100year return period for precipitation levels are increased by 30%. If this increase in capacity does not happen, either directly or indirectly by flood mitigation

measures, there is approximately a 50% chance of what we would consider to be a 1 in 100 year storm every 25 years. Current drains and culverts are not designed for this storm intensity.

HA (Mike Whitehead 2011) have identified that 10% of their network is susceptible to flooding now and that flooding causes 2 fatal, 10 serious and 54 minor incidents per year as well as the extensive disruption to the network. They have therefore prepared a flood ‘hotspots’ map using the RAG system - red, 50% chance, amber, 25% chance and green, 10% chance, of flooding in any year. This provides input into the Highways Agency Drainage Data Management System (HADDMS). A typical map is shown in Figure 2.1:

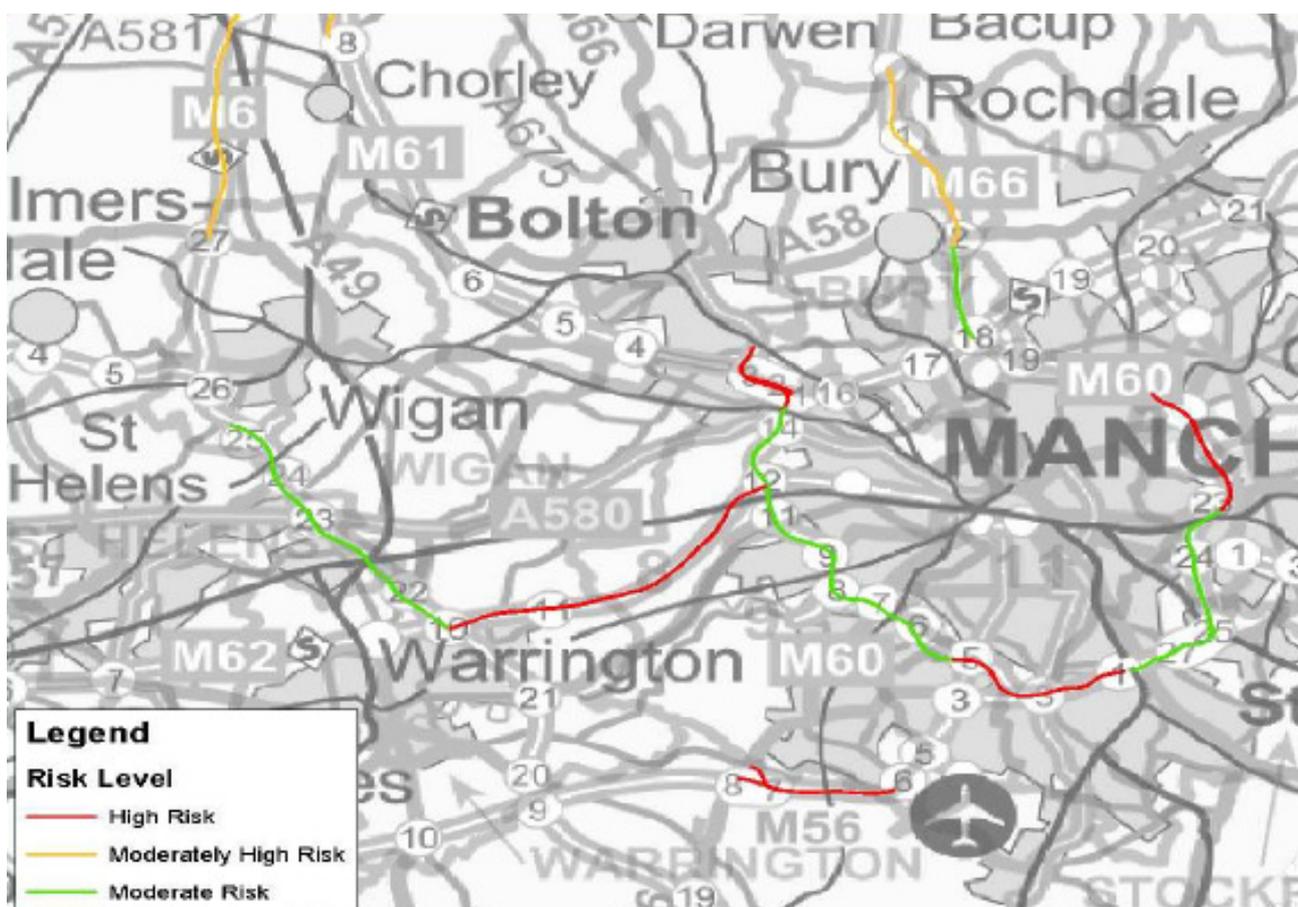


Figure 2.1: Map showing HA flood risk areas

From this map, culverts and bridges can be identified that need further investigation, the former for their ability to have capacity improved (potentially about 30 No.), and the latter for foundation scour protection. The flood map also identifies cuttings and stretches of highway that might be at risk from flooding from adjacent rivers with a view to possible protection by false cuttings.

An example of where this was carried out is on M26 as it crosses the Darenth Valley, Otford near Sevenoaks, Kent (designed in 1976, opened 1979). The false cutting on the right side of the picture (Figure 2.2), i.e. an embankment formed of unsuitable material between the two bridge embankments, prevents the river flooding the motorway. The drainage lagoon (now surrounded by a high fence) collects the water flowing down from the cutting behind the bridge and stores it until the river flood recedes. In the light of climate change it may be prudent to revisit these calculations wherever such a facility exists, to check they are adequate.



Figure 2.2: M26 Darenth Valley, Kent

The consequences of flooding on the asset can be illustrated as shown in the chart (Figure 2.3) by HA below:

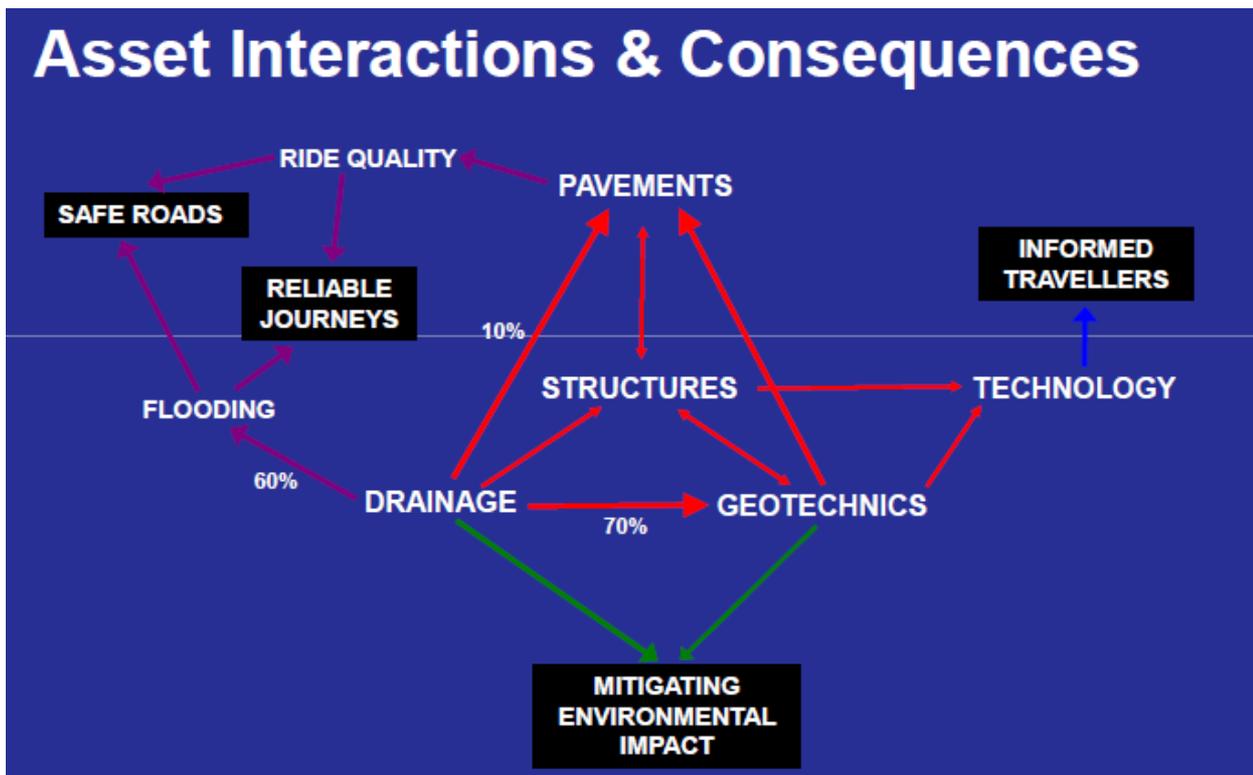


Figure 2.3: Consequences of flooding

A purpose of this report is to identify the potential effects of flooding on the pavement construction itself. This could be loss of strength of the pavement as a result of sub base or the asphalt becoming saturated. This is thought to be a temporary phenomenon of a few days duration after which the material should drain. Part of the work is to see if the previous properties are restored.

In addition water flowing at high speed over the surface and traffic running on the waterlogged surfaces can scour the asphalt reducing the durability. This is very difficult to simulate in the laboratory. This often occurs where road drainage is not maintained and water ponds in the carriageway after a rainstorm. It also can occur on coastal roads subject to waves and spray as discussed in more detail in Section 3.1.



Figure 2.4: Scour of HRA chippings as a result of almost constant wetting on the carriageway from water dripping from the rail overbridge.

3. Task Reports

3.1 Review existing available published information on in-service roads

The amount of published information relating to existing roads, rather than theoretical exercises or work done on laboratory specimens, is lamentably small and scarcely enough on which to base any major changes to current UK highway design practice. Indeed only a limited amount of literature exists on water movement in pavements and much of that is 20 - 30 years old, with much of it done in USA with different climate and soils.

The degree of saturation of a pavement depends upon entry and egress routes. Ingress can be from the sides through poor or inadequate verge/centreline drainage or from the top through the joints, cracks and porosity of the surfacing, especially as it deteriorates. Where the road is in cutting, water gains access from below as a result of a high water table or capillary action not controlled by side drainage. Which of these is the more important will depend upon the circumstances of the individual site, as they are not mutually exclusive.

Egress can be by soakage down into the subgrade below or by exit to the side drains (if any) or services trenches. It has also been observed that considerable variability in permeability can occur laterally, for example in wheel path locations as a consequence of possible densification within them, and edge effects along the shoulders and drainage trenches, and vertically within the sub base. These are caused largely by construction operations, e.g. smearing of fine grained soils and contamination of sub base.

Last but not least the amount of water available is seasonal and weather dependent so any short term investigations can only be snap-shots, and overall the volume of water present is unpredictable.

We do however know from numerous trial holes taken into pavements, that the amount of water present between bound layers and at the sub base/bound layer interface, which is often impermeable as a result of construction activity, can be very high, but the presence of saturated sub base is unusual. Any design criteria for sub base strength is therefore based upon this unknown degree of saturation.

Current design guidance is still all based upon TRL Report LR 1132 (1984). This was the result of a combination of laboratory work and the observed performance of roads built to the then current standards.

One of the key parameters was that the load spreading ability of the sub base and capping layers should be adequate to provide a satisfactory construction platform. Changes in the strength of the sub base as a result, for example, of saturation, was only considered by inference, in that the total strength of a pavement needs to be mobilised to maintain the stresses on the sub grade to acceptable levels. However whilst the sub base can be structurally significant, particularly on thin pavements, it contributes only one third of the strength of asphalt, thickness for thickness.

The pavement design thickness for sub base is based upon what is required for constructing a motorway on a green field site carrying about 1000 standard axles of construction traffic (LR1132 Fig C.3). In practice on many maintenance schemes, the construction traffic is far less than that, indeed on many widening or patching projects no construction traffic will use the sub base. In that case extrapolation of C.3 suggests that, for example, 200mm sub base (5% CBR subgrade) can be replaced with 150mm sub base. Alternatively the 150mm of sub base with CBR 15% [100MPa stiffness modulus] could be replaced with 200mm of a granular material of a lesser strength. Current design thickness therefore contains a fair element of conservatism.

What this means is that, once in place, the 200mm of material would still be strong enough even if it lost some strength as a result of wetting up. In laboratory work Hicks and Monismith (1971) found

that the resilient modulus was not greatly affected when measured on dry and partially saturated specimens, but this study will attempt to quantify this. Further references are given in Section 3

What is also not known is the permeability of typical in-service sub bases. Simple calculations, based for example on the Hazen formula, show that the permitted range of fines (0-10% passing the 63micron sieve) can lead to several orders of magnitude change in permeability. Jones and Jones (1989) and HA 41/90 provided a test method based upon a horizontal permeability device for measuring this property. The results of tests carried out compared to the Hazen formula are given in Figure 3.1.

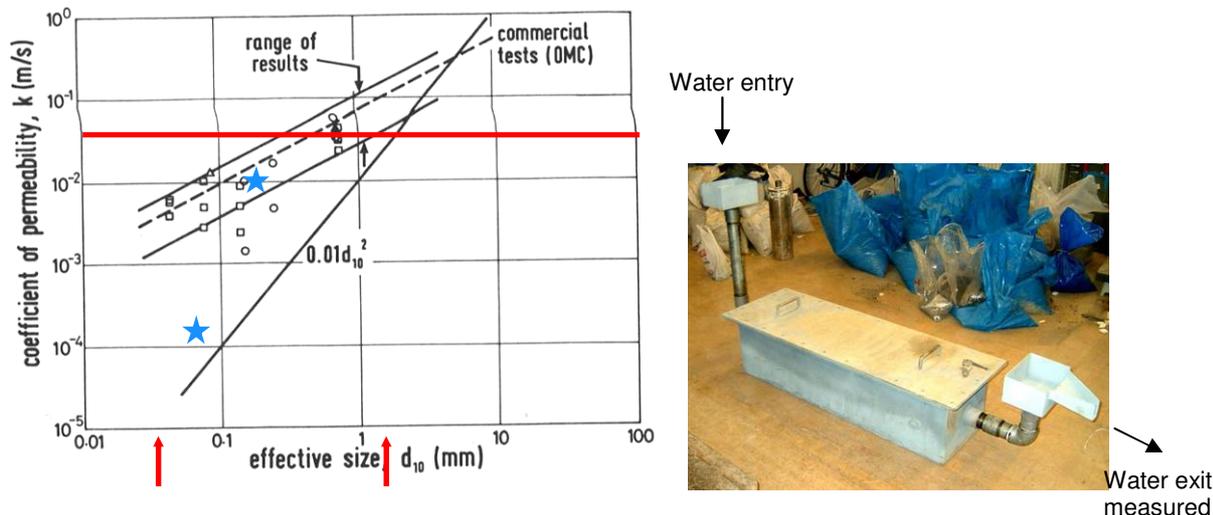


Figure 3.1: Horizontal permeability of sub base

This test gives the saturated permeability, however there is no specified acceptance value. Experience with this test in Kent CC using crushed limestone and granite Type 1 sub bases, has found that a value of 8×10^{-2} m/sec has been found suitable.

Work in USA has suggested a slightly higher value of $>3.4 \times 10^{-2}$ m/sec for hydraulic conductivity, to ensure adequate free draining, particularly beneath jointed concrete pavements where water ingress is known to occur as joint seals fail. This is not a surface type used on Kent CC roads but extensive lengths of M25 in Surrey have now unsealed longitudinal joints that are visibly letting in rain water. Concrete motorway surfacing in UK is laid on stabilised sub base.

It may be prudent to consider introducing a permeability requirement in the Specification for Highway Works for Type 1 sub base to complement the stiffness requirements in IAN73 for a Class 2 foundation or the existing recipe in Clause 803.

In practice the ingress and exit of water will be the amount able to flow through an unsaturated sub base. This is analogous to a pipe more or less clogged with coarse and fine particles. Despite some efforts in Universities, this has proved to difficult to model and predict, depending as it does on the number and size of interconnected voids, possible flow paths through the material, the microtexture on the aggregates' surfaces and the effective head of water available.

As a sub base becomes more permeable, it tends to have greater voids and hence less structural strength/load spreading capability. There is a difference of view amongst some observers as to whether a strong but relatively impermeable sub base that will also protect the sub grade from water softening, e.g. a Cement Bound Granular Material (CBGM), as used beneath concrete pavements, is to be preferred to a more free draining product that will keep the structural pavement dry.

Free draining sub base will ensure that the base asphalt is kept dry. This may well be water susceptible as a consequence of low bitumen binder content. However, especially in flooding situations or where the side drainage ceases to work effectively, the layer can act as conduit feeding water under the pavement from the edges.

As material complying with the Type 1 specification (SHW Clause 803), which has remained largely unchanged for over 40 years, can vary between free draining and low permeability. The spatial variations and the drain trench/sub base interface properties are unknown therefore the response of the sub base layer to inundation in any particular situation is not predictable.

Since flood events in the UK last rarely more than 36 hours, a free draining material will be immediately affected but will also drain and revert to the normal situation quite rapidly. The effect of this on the pavement is being simulated in the laboratory work reported in Section 3.6. Less permeable material will be affected far less but the effects will last longer.

In order to make some estimate of the effect of flooding on the sub base some assumptions are necessary. Entry of flood water into a sub base layer will depend upon the site conditions on and below the surface, for example the permeability of the central reservation at the time of the inundation. Infiltration into grassed surfaces is in the range (1.44 - 2.52) l/hr/sq.m. Unless the sub base layer extends beneath the central reservation only a percentage of this will enter the sub base layer and the quantity will be dependent upon the condition of the sub base locally.

A typical sub base layer on a motorway, 250 mm thick, if dry initially, will accommodate 350 litres of water per linear metre of road. With positive drainage, i.e. the verge/central reservation is grassed and all the water enters the layer, one can assume 83 hrs of continuous rain or flood would be required to saturate it, with no egress. No egress is a possible scenario if the area is completely flooded.

However if the verge/central reserve has 'over the edge' combined surface water and filter drains, with a highly porous granular material up to the surface, this will rapidly fill with water during flood conditions and water can flow out under the pavement through the sub base under the head of floodwater. In the M50 flood of 2007 this head was in excess of 1.4 m. The extent of saturation of the sub base will depend upon the edge conditions and its permeability, however in these conditions it is likely that the whole sub base layer could be saturated in less than 1 day flooding.

If the sub base does become saturated, the traffic stresses will be carried by the water and not by the aggregate, this is a very unstable situation that can lead to piping (fines migration) and fines loss, as effectively the road is floating as discussed by the researchers in Section 3.4.

Where a road is inherently weak, or a concrete carriageway has been laid on a sub base rather than a cement bound material, evidence of sub base piping and fines pumping up to the surface are commonly seen as shown in the photographs below:

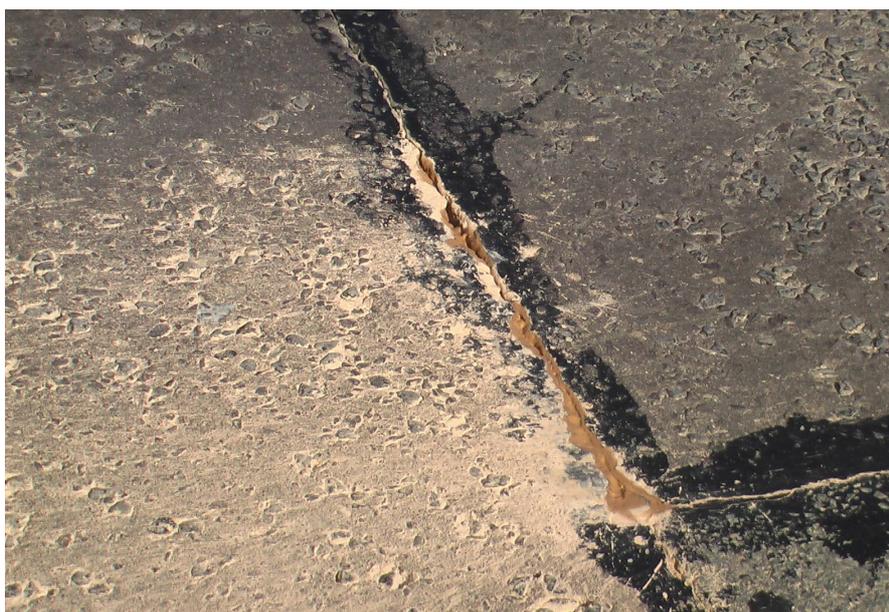


Figure 3.2: Rigid pavement



Figure 3.3: Flexible pavement

Where a flood has persisted for a long period of time, e.g. >3days on a road with positive drainage and a sealed verge/central reservation, or >1day with combined drainage, it may be prudent to close the road to traffic not just whilst it is under water but also until the sub base is no longer saturated. It may not be all that easy to tell when it is safe to reopen it.

Reid et al (2006) were particularly interested in the topic from the point of view of assessing the risk of leaching when potentially contaminated recycled materials are used as sub base, e.g. slags and Incinerator Bottom Ash (IBA). In particular the researchers looked at infiltration through a jointed concrete and then cracked concrete trial pavement at TRL, recognising the paucity of real data available in the literature. Despite the serious deficiencies they identified in the trial, they were confident that sub base run off was about 0.5% to 0.75% of surface run off and moisture levels were higher in winter than in summer. On this basis, unless a flood occurs, it is unlikely that a sub base will be fully saturated in practice. Of course this is only of limited relevance in flood situations but summer floods are less likely to saturate the sub base than those in winter as the reservoir space is larger, 30% of the 12 flood events above occurred in summer.

The major concerns in the design and construction of a road structure are the stability of embankment and cutting slopes, surface and sub soil drainage, frost heave, elasticity, rutting etc. Research work in Europe and USA has been carried out on these aspects in laboratories. However until relatively recently long-term inundation by floodwater was not a very common phenomenon and research in this regard was found to be almost non-existent. Even in other parts of the world, where flooding is commonplace, data from comprehensive research on the effect of inundation by floodwater on road pavement is sparse.

It often concentrates on the effect of embankments damming water flows, which then scour culverts and bridge foundations and can wash away part of the road surface. The damage to the pavement structural fabric is less obvious and longer term. This form of damage is the one most preoccupying Highways Agency in their Asset Management Plan discussed in Section 1.

When considering overseas research due regard must be given to the different sub grade soils, sub base materials, and surfacing types. For example in Africa and Australia self cementing lateritic materials are commonplace.

Alam and Zakaria (2001) reported on the effects of the devastating floods in Bangladesh in 1998 that affected 76% of the country when the pavement of 600km of national highway and 330km of

regional highway was damaged. They went on to carry out laboratory work on the effect of soaking over periods up to 45 days on sub base materials. They found there was a significant loss of CBR strength in the sub base and of stiffness in the Marshall Asphalt. This has also been reported by other research. The implication was that national specifications should choose test methods including soaking regimes rather than dry values for sections of road prone to flooding. This is discussed further in Section 3.4.

In Western Australia the climate is such that unsaturated design is the norm. However for areas subject to inundation the design moisture content for sub base is assessed in using 120% of the modified AASHTO optimum moisture content.

The Queensland Government gives the following guidance in a published leaflet 'Flooding on Roads in Queensland' (Figure 3.4).

The New Zealand supplement to Pavement Design - a Guide to structural design of road pavements (Austroads 2004) (2007) gives the following advice.

'Soaked' laboratory CBR values are appropriate where the pavement could be subject to inundation by flooding'.





Saturated roads

When a road is inundated, especially for a long time, the materials in the road pavement become saturated. This reduces the strength of the road pavement material.

The department undertakes inspections and testing to re-open roads as soon as possible after inundation. In its custodial role, the department balances protection of the road asset with community and industry access.

Roads that have been saturated must be carefully managed. To do this, road closures, load restrictions and traffic management are the only options available.

Traffic management

Managing traffic on saturated road pavements as the roads 'dry-back' is important for the safety of road users and for the life of the road asset. Traffic management measures used include:

- using signage to encourage loaded vehicles to travel safely in the middle of the road where possible
- restricting the use of heavy load permits.



Load restrictions

Once road pavements are saturated, it takes time for moisture to drain from the material and for full-strength properties to return. Restricting the loads on roads during this 'dry-back' period is essential to preserving this important community asset.

Based on technical data, reducing the load by 20% reduces the impact on the road pavement materials by about 50%. Experience shows that a single trip by a fully-loaded transport vehicle may cause permanent failure of the road pavement material, leading to expensive repairs and affecting accessibility to the road network.

The responsible departmental Regional/District Director will progressively lift load restrictions according to the local conditions. The stages include:

- 5 tonne limit to allow access for local residents and emergency services vehicles
- 80% of the legal axle loads on transport vehicles as shown in the table
- removal of load restrictions.

Axle Group	Legal Load	80% Load
Steer axle	varies	no reduction
Single 4 tyre	9.0 t	7.2 t
Tandem 8 tyre	16.5 t	13.2 t
Tri 12 tyre	20.0 t	16.0 t

Reference: Transport Operations (Road Use Management - Mass, Dimensions and Loading) Regulation 2005

In making these decisions, the Regional/District Director relies on specialist advice based on measurement and analysis of road surface movements under test loading of the pavement. This technique models the 'dry-back' process of the road pavement material, and allows the decision to be made about when it is safe to remove the load restrictions and open the road fully to all road users.

Figure 3.4: Queensland Government guidance 'Flooding on Roads in Queensland'

They also draw attention to the fact that some unbound sub-bases have shown from experience and repeated load axial testing that they perform poorly when saturated. They highlight the need for

water to escape quickly from such moisture sensitive layers and drainage systems must operate effectively during the life of the pavement.

As already identified, they state that the drainage system should not be a conduit for flood water entering the pavement. They point out that water will enter the pavement and will flow downhill until prevented to do so by an area of lower permeability such as can pertain within the wheel paths. The water pressure can lead to significant deterioration in the pavement structure. Their preference in situations where flooding is likely is for a stabilised base layer to keep water out and which will remain strong even when wet.

Useful references

Baldwin et al (1997) *The movement of water in roads* Rational Road Drainage Design - R6990 TRL.

Reid et al (2006) TRL PPR082 *A study of water movement in road.*

Powell WD, JF Potter, HC Mayhew M Nunn LR1132 (1984) *The structural design of bituminous roads.* HA 41/90 A permeameter for road drainage layers.

Biczysko, SJ. (1985) Permeability of sub-base materials. UNBAR 2. Nottingham University.

Dawson AR (1985) Water movement in road pavements. UNBAR2, pp 7-12. University of Nottingham.

Alam MJB and M Zakaria (2001) Engineering Concerns of Flood Proc 1st Intl Conf IEB Chittagong, Bangladesh, 2-3 November.

Vuong B (2003) Development of performance based specifications for unbound granular materials - Part B use of RLT test to predict performance. Austroads Engineering Road Note No. 5 (2003) Main Roads Western Australia.

3.2 Use the HAPMS database to detect any change in parameters

The Highways Agency identified a section of the M50 at Junction 2 (Figure 3.5) that had been the subject of inundation in July 2007 as shown in the photograph (Figure 3.6) below



Figure 3.5: M50 J2: Normal conditions



Figure 3.6: Inundation at July 2007

Using the HAPMS data the gradient of the motorway in this vicinity was able to be plotted as shown in Figure 3.8 below.

The photograph (Figure 3.6) shows that in the background trucks are parked up and therefore the extent of the flooded area can be identified behind the camera, situated on the motorway east overbridge.

To define the extent of flooding, the longitudinal profile data from the TRACS survey was utilized. A flood depth of 1m has been used to define the extent of the flooding measured from the 'dip' in the longitudinal profile for both carriageways. Using this methodology the approximate length of inundation is some 700m. The longitudinal profiles are represented in Figure 3.8 showing the approximate extent of inundation together with an overall plan of the site as indicated below (Figure 3.7).

TRACS survey condition data has been reviewed with respect to identifying parameters which may indicate specific surface deterioration. This is related to the area of carriageway which is subject to inundation, compared to surrounding areas of pavement unaffected by inundation.

The parameters reviewed were:

- Wheel track rutting (left and right).
- Ride quality (3m, 10m & 30m Longitudinal Profile Variance).
- Low texture.
- Fretting (for HRA surfacing only).

The surfacing on this stretch is unknown but based upon the measured ruts would appear to be Hot Rolled Asphalt.

The 100m sub section data from the 2010 TRACS Survey has been utilized to compare/contrast the current surface condition. Previous survey data from 2006, 2007, 2008 and 2009 has also been

reviewed to determine if there is any increased rate in surface deterioration for the above defined parameters within areas of inundation.

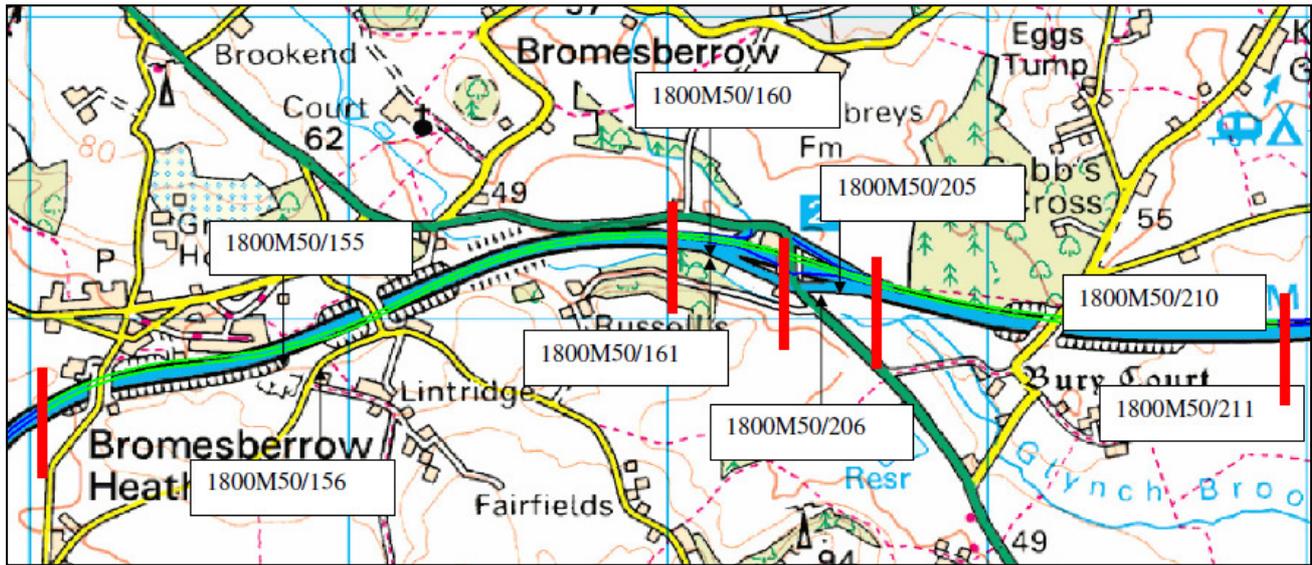


Figure 3.7: M50 Location Plan

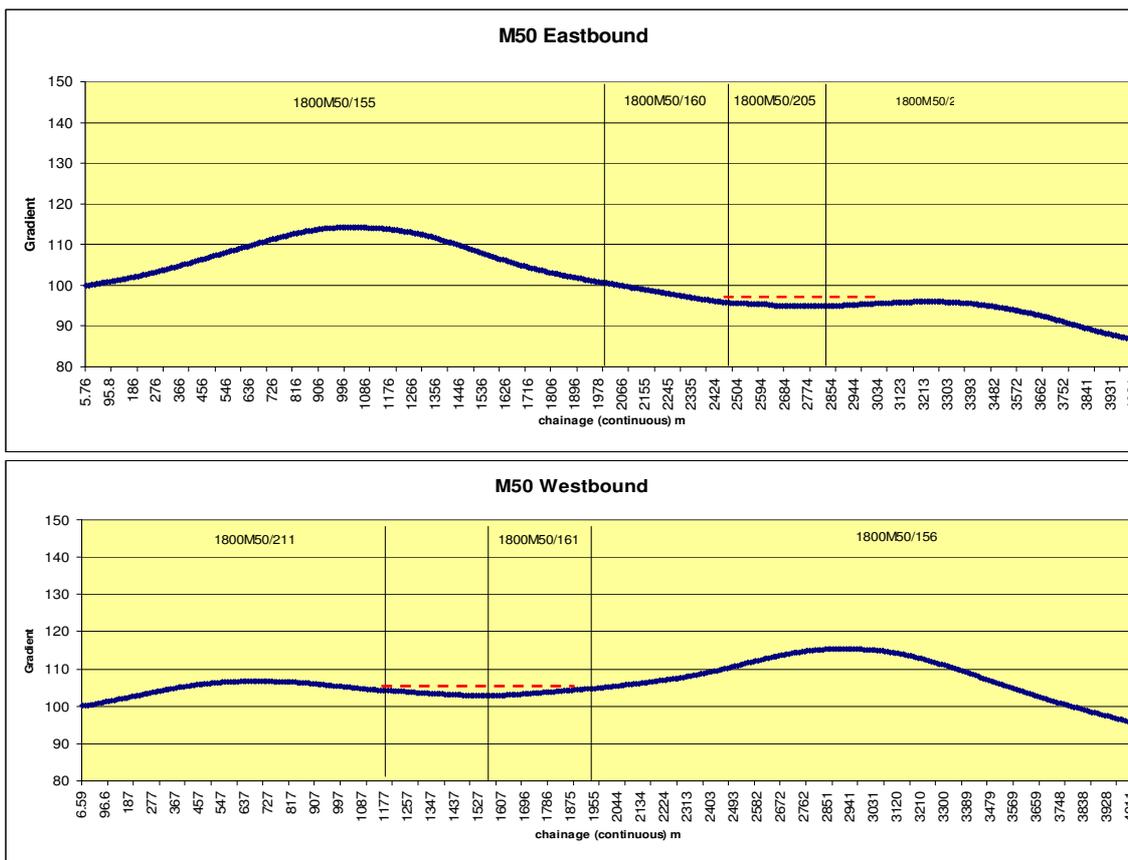


Figure 3.8: M50 Motorway Gradient Plots

Based on this, both carriageways have been sub divided into 700m lengths either side of the length of inundation and each parameter averaged within these lengths to compare with the length of inundation. The results are given in table Figure 3.9.

Eastbound	Section	Section Chainage(m)		Continuous (m)	Left rut	Right rut	LPV3	LPV 10	LPV 30	Texture Depth	Fretting
		start	end								
	1800M50/155	406	1106	506 to 1106	3.83	5.59	0.42	1.59	6.74	1.71	0.56
	1800M50/155	1106	1806	1106 to 1806	3.71	5.64	0.43	1.93	7.97	1.82	0.55
	1800M50/155//160	1806	496	1806 to 2474	4.77	7.04	0.35	1.53	11.6	1.71	0.43
Area subject to flooding	1800M50/205//210	0	310	2474 to 3151	5.74	6.19	0.47	1.94	8.52	1.74	0.47
	1800M50/210	310	1028	3151 to 3869	4.29	4.14	0.47	1.96	13.0	1.53	0.11
									3		
Westbound	1800M50/211	507	1185	507 to 1185	4.38	6.01	0.22	0.96	6.67	1.21	0.00
Area subject to flooding	1800M50/211//206	0	318	1185-1875	5.13	6.15	0.20	0.68	5.45	1.40	0.02
	1800M50/206//161//156	318	611	1875 to 2533	4.62	6.72	0.23	0.72	4.60	1.44	0.00
	1800M50/156	611	1338	2533 to 3260	2.65	2.84	0.20	0.58	3.02	1.38	0.03
	1800M50/156	1338	2065	3260 to 3987	0.87	1.03	0.21	0.92	5.09	1.48	0.00

Figure 3.9: TRACS Data (2010 summary averages)

A review of the 2010 summary averages shows no significant trend in data values between the length of inundation and the surrounding lengths of pavement. There are however, comparatively increased values with respect to rut depth. The highest value for left wheel track rut depth is within the area of inundation for both the west and east bound carriageway; however, it is highly unlikely that this is related to inundation of the pavement.

There is an increased value for fretting (HRA surfacing) within the inundated length of the eastbound carriageway which might possibly be due to water on the surface but values remain extremely low and inconsequential.

A review of the rate of deterioration between 2006 and 2010 parameters, similarly shows no significant increase in deterioration for the areas of inundation. All parameter values remain within the bounds of variability which could be expected for a typical pavement.

On the basis of the information from this site it would appear that the effects of this inundation incident in 2007 were benign.

However it also highlights that sag curves in the motorway alignment especially those, as here, that are apparently in conjunction with cuttings, are particularly vulnerable in times of flood as they are entirely dependent upon the surface water drainage for removal of the water.

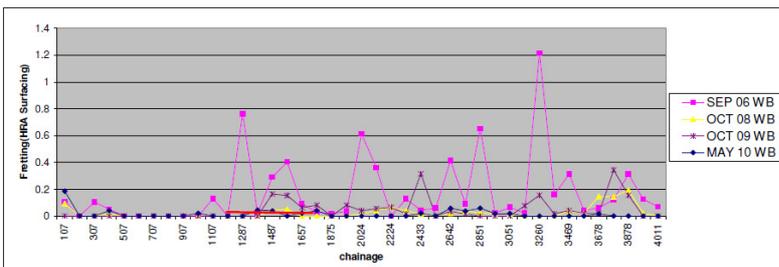
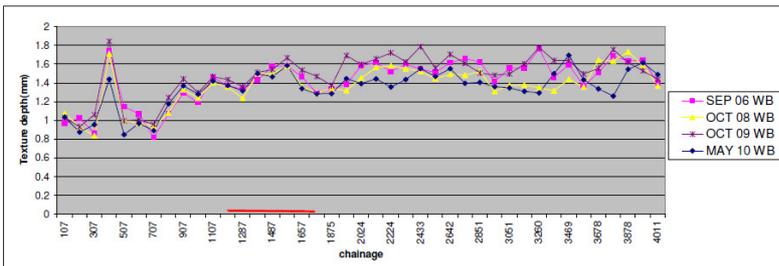
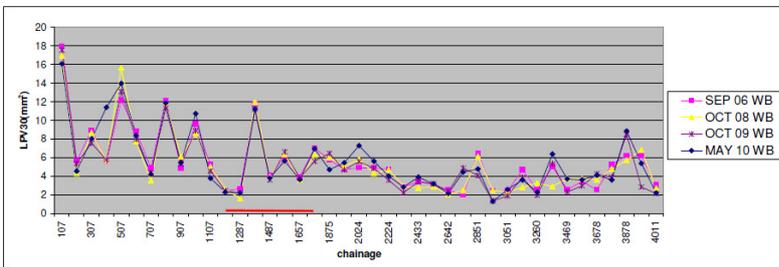
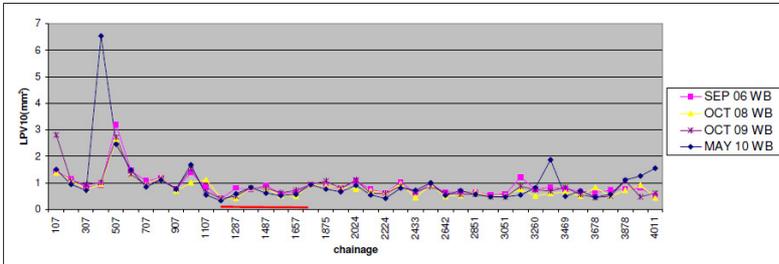
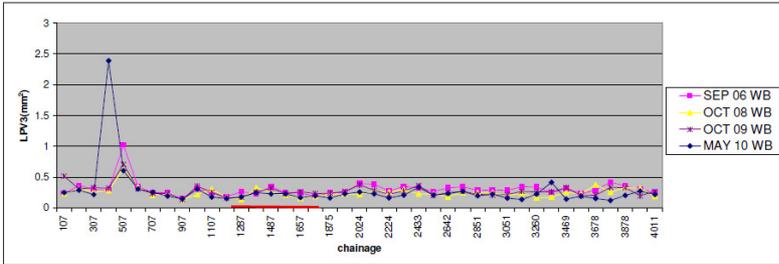
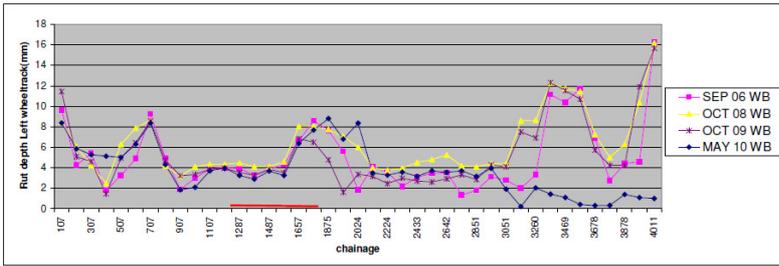
However it is reported that HA is carrying out work to re-construct and stabilise approximately 450m of the flood damaged westbound embankment and other associated works in the west embankment of the M50 motorway. The scheme is located in the westbound verge east of M50 J2 near Bromsberrow Heath in Gloucestershire.

Whilst undertaking the embankment repair they will also be upgrading the existing barrier, re-surfacing the hardshoulder, upgrading the existing drainage system, installing boundary fencing, installing new asphalt kerb and renewing road markings and traffic signs.

The works started on 14th February 2010 and will last for approximately 10 weeks.

From the picture it appears that this is over-the-edge drainage rather than positive drainage now preferred. The introduction of positive drainage with an asphalt kerb can only be beneficial.

M50 WESTBOUND
 2006 to 2010 TRACS Survey Data



3.3 Seek information from Local Authority and other staff

Gaining information from Local Authority staff on the effects of inundation was difficult, as they may know suitable locations but have no maintenance history. No HA staff contributed to the study. The main places where roads are subject to inundation are those coastal roads that are subject to occasional or regular tidal or wave inundation and river fords. In both cases the traffic levels on these roads are very small certainly compared to motorways. In the former case the lack of guaranteed access acts as a constraint on development, in the latter case, if the road was sufficiently important, a bridge has been built. A number of coast roads in N. Ireland, Kent, Devon, Lancashire and North Yorkshire were identified. Additionally there are tidal causeways for example to Lindisfarne (Figure 3.10) and St Michael's Mount (Figure 3.11).



Figure 3.10: Lindisfarne Pavement Surface



Figure 3.11: St Michaels Mount Causeway



Figure 3.12: Coast Road, Co. Antrim

Verbal evidence from staff was universal that these roads were constructed with dense macadam surfaces as evolved roads, i.e. layers of macadam and surface dressing had built up over the years from an unbound granular base laid in the 19th century.

An alternative was the use of stone sett paving installed many years ago; cobbles and stone paving/pitching are also frequently found in river fords. These may have subsequently been overlaid with asphalt as these materials become slippery; their microtexture is the primary friction parameter and this can become clogged with slime, algae and detritus.

The NI Roads Service for Coast Road (Figure 3.12) in Ulster has, in the last 10 years, chosen a proprietary SMA with what we would now call ‘low’ texture depth (0.8mm Patch new) as the preferred choice. They tried a BBA HAPAS type Thin Surface Course System (TSCS) about 15 years ago and found that the material was rapidly destroyed by being so frequently saturated that it fretted out. There was also concern about water penetrating down to the lower layers that may have been cracked/oxidised. They have also found that in this environment, they had chip loss from Hot Rolled Asphalt, a phenomenon also found in other places where a road is frequently wet. The denser SMA did not suffer this defect. However the probable reason for using SMA here and in many local authority situations is that the material can be installed within a paver width, unlike HRA, so that work can proceed on relatively narrow roads under one way working. This is a serious matter where diversion routes may be lengthy.

An interesting observation from Roads Service NI in their evidence is that the denser SMA mixtures are better than HRA surface course in coping with road movements such as pertain on water susceptible soils such as peats and alluvium. Work by Jacobs for the Londonwide Asphalt Specification on the cracking resistance of Thin Surface Courses also found that some TSCS could have equivalent or better crack resistance than HRA.

As with most evolved roads, the budget for maintenance is small. However, with low traffic levels and rocky, or at least granular foundations, the universal feedback was that relatively frequent maintenance with a sealing treatment to the surface was quite adequate, in conjunction with reactive pothole filling on receipt of feedback from residents.

It was observed that surface scour occurs from traffic running over a saturated surface, for example at Lindisfarne and Coast Road Antrim, as shown in the photographs. This condition is also

observable on road surfaces subject to occasional localised flooding in times of heavy rainfall. Feedback also suggested that where an asphalt road surface was frequently wetted moisture could be retained within it although it could look dry. This water could then try to evaporate and delaminate, debond or loosen the surface dressing chippings which then scour off under traffic to give the effect shown above. Research quoted in Section 3.5 suggests asphalt can remain wet for 20hrs after rainfall ceases, though the wheel path may dry faster as a result of tyre suction and air pumping. Surface dressing in damp areas under trees is known to be very difficult so that in many cases the dressing is lost even though the rest of the road is performing well. This however may be down to the shadier area having a harder surface and therefore less embedment takes place.

Surface scour can also occur where sand is blown onto the carriageway which can act as an abrasive under tyre wheels. Where this type of road also floods at certain stages of the tide, the road surface is then clogged with sand, producing a very smooth potentially slippery surface with a high percentage of rounded sand on the surface. The water is also trapped in the pores of any open textured surfacing exacerbating the effect of water on the bitumen bound material. For these sorts of sites a dense surface of bitumen macadam (asphalt concrete) or hot rolled asphalt could be preferable.



Figure 3.13: Coast road through dunes

In the UK there is a preference for quite rough surfaces so that tyres can get some sort of purchase when it is very wet. This is not the case necessarily in Europe where they are much more willing to accept a much smoother surface which should in theory be more durable.

For the surface to the Passage du Gois, Ile de Noirmoutiers, France, the Highway Authority has tried concrete slabs, small pavers and dense surfacing. However, the dense asphalt suffers from the ingress of water and needs regular patching as shown on the photograph below (Figure 3.14). The mix design is not known.



Figure 3.14: Passage de Gois Vendée, France

For areas identified by the Environment Agency as having a high risk of flooding (Figure 3.15 - the dark blue areas on their flood mapping), Kent County Council has changed the routine pavement design on their major road network from the standard Thin Surface Course Systems to Hot Rolled Asphalt. This is even more important where the blue areas and the shaded areas indicating a clay subgrade coincide, as these are particularly moisture susceptible. This can be done automatically by the Jacobs Carriageway Asset Manager (JCAM) software used by Kent CC that allocates surface treatments. This choice is in recognition of the need to provide a dense surface to prevent water ingress and damage to the asphalt and possibly the fragile layers below. This acknowledges that the BBA HAPAS bond coat to Thin Surface Course Systems is not a seal coat.

An alternative could be Asphalt Concrete; however the poor feedback on durability, probably as a result of low binder content, and the inadequate surface texture for roads with speeds in excess of 30mph makes these only suitable if surface dressed which nullifies the cost advantage.

An alternative could be the use of a dense Stone Mastic Asphalt, as used in Scotland and Northern Ireland with success, together with a lower texture depth requirement and gritting for early life skid resistance. No such products are BBA HAPAS approved and so are technically out-with the Specification for Highway Works Clause 942. Notwithstanding they are proprietary products complying with EN 13108-5 and a 5 year guarantee and should be acceptable as a departure.

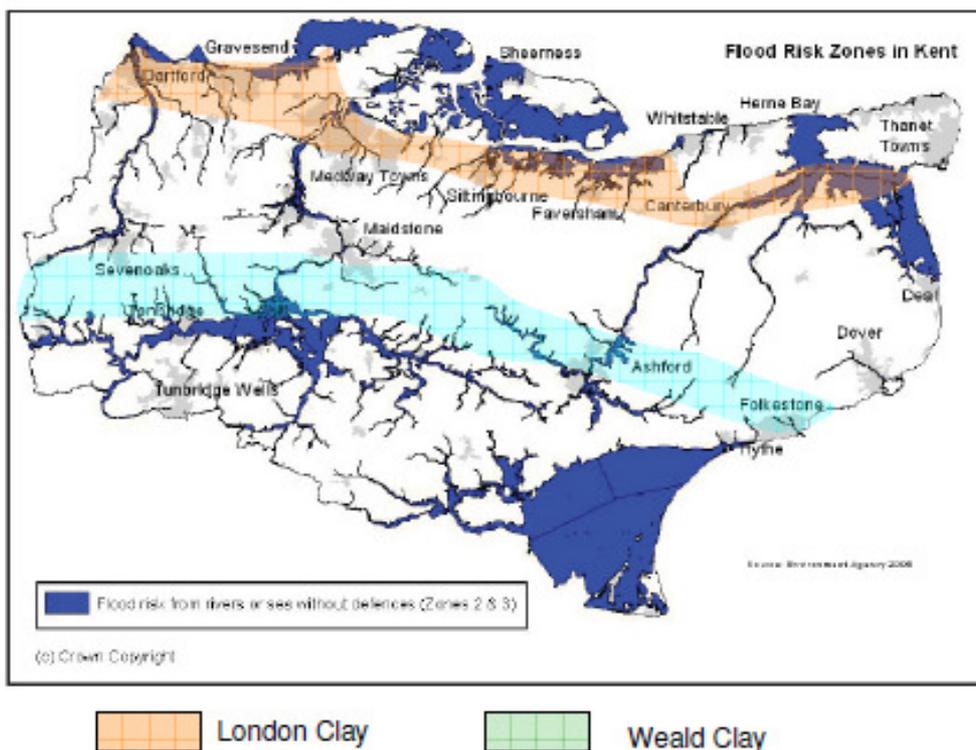


Figure 3.15: Kent Flood Mapping

3.4 Review of existing available published information - Sub base

A review has been conducted by Dr David Woodward of University of Ulster of existing available published information and previous research relating the effect of saturation on road making materials separately, i.e. sub base and asphalt. Whilst the role of the sub-grade and its moisture sensitivity is of critical importance to the strength of the pavement this is out-with the scope of this project.

This review took the form of a desk top study using the internet and the University’s own extensive technical library.

3.4.1 Sub base - Introduction

The modern pavement is designed to meet both the requirements of both present and future trafficking conditions. The unbound layers are used to provide support during construction and load bearing especially on lightly trafficked pavements, preferably using local materials for economic reasons. Much research has considered the role of grading and moisture contents to achieve optimum compaction and stability.

The sub-base provides a stress dissipation layer that protects the sub-grade from over stressing, acts as a working platform during construction and sometimes may act as a drainage layer. Sub-base thickness is pre-calculated and depending on the under-lying sub-grade a capping, geotextile or additional drainage may be used.

Current practice with sub-base aggregate selection in the UK is covered in the Specification for Highway Works. This has recently been revised in line with EU policy regarding standardisation of testing methods. For example, the wet Ten Percent Fines Value Test has been replaced by the dry Los Angeles Fragmentation Test to assess the 10/14mm sized fraction.

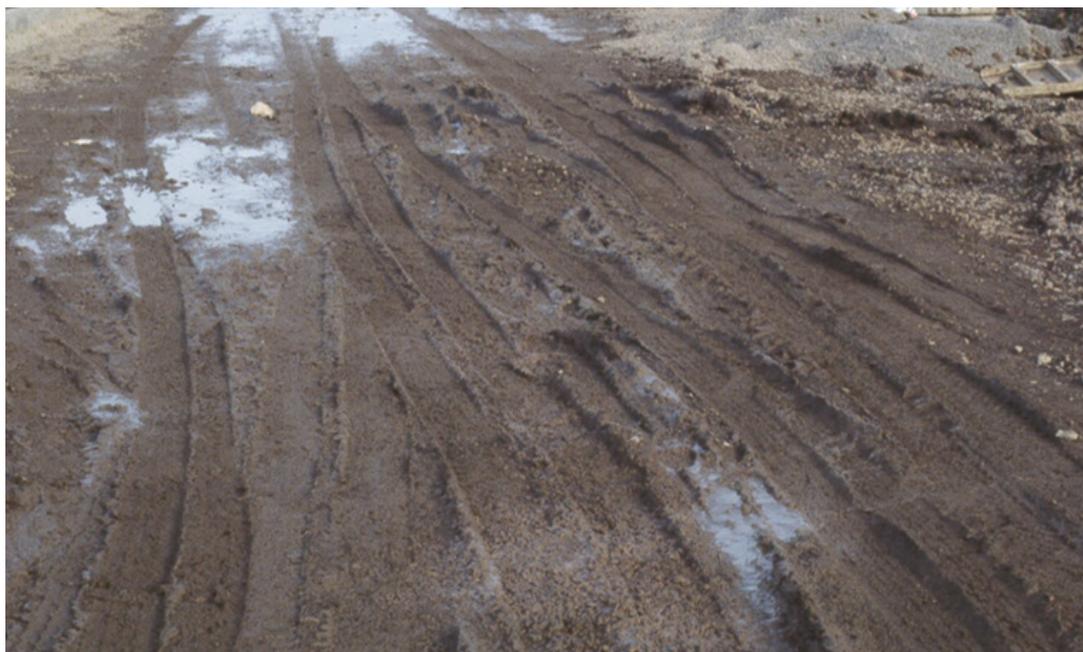


Figure 3.16: A gritstone aggregate sub-base badly contaminated by site traffic

The Plasticity Index remains to assess potential plastic problems of the <0.425mm fraction. As the only method used to identify the finer component within the sub-base it could be important that it adequately highlights the potential for weakening problems. However, there are reported instances where material certified as non-plastic has caused problems either during construction or later in-service, i.e. loss of stability due to inability of the interlocking particles to withstand external stresses.

Certain conditions can result in excess fines or their concentration and lead to premature and typically rapid failure of the road pavement. This has been attributed to factors such as sub-grade intrusion and segregation. However, there are other instances of failure where these factors were not the problem and some other mechanism or conditions associated with the sub-base, its compaction or subsequent trafficking were contributing factors. Compaction and trafficking in conjunction with a weak material can concentrate a layer of fine material of low stability, at the top of the sub-base or at a level within it reducing the ability of the layer to perform. This is illustrated in the picture Figure 3.16.

Although the top layer was scraped off and replaced, the first asphalt layer applied to this failed immediately because an impermeable layer still remained at half depth in the sub-base.

3.4.2 Review of SHW specification requirements

The table below (Figure 3.17) is a review of the main unbound aggregate related requirements given in the SHW. This gives the main Series Clauses and what is required. It is assumed that these criteria, if properly implemented, will result in a long lasting pavement structure.

However, if the unbound aggregate has been assessed, laid, compacted to these criteria and then fails in-service it may be argued that these standard requirements are missing some aspect in relation to predicting performance with respect to inundated conditions.

An issue with CE Marking and the SHW is the presumption that an aggregate type approved after testing a stockpile will, after storage, installation and compaction, and possibly including saturation at some stage in the process, still satisfy the necessary stability and permeability requirements after installation. This has been addressed to an extent by IAN73 Rev 1, however this does not take ask for any tests on a layer after saturation.

Series 600	Grading, uniformity coefficient, moisture content, Moisture Condition Value, Los Angeles, un-drained shear strength of remoulded material, bulk density and plastic limit.
Series 701.2	Choice of permitted materials for sub-bases and bases, have regard to the nature of those materials and of the subgrade or any capping and the need to protect them from deterioration due to the ingress of water, the adverse effects of weather and the use of constructional plant
Series 801.1 asks for:	Grading, los Angeles, micro-Deval, Magnesium sulphate, water absorption, volume stability of BFS, BOF and EAF slags, <0.425mm non-plastic, minimum CBR
Clause 801.6 for unbound mixtures shall:	Satisfy the minimum CBR requirement of Appendix 7/1 tested in a soaked condition, tested at the density and moisture content likely to develop in equilibrium field conditions which shall be taken as being the density relating to the uniform air voids content of 5% and the value of optimum water content declared when tested as required by BS EN 13285
Clause 801.8 material shall:	Be classified as non-frost susceptible if the mean heave is 15 mm or less, when tested in accordance with BS 812-124. Comparator specimens in accordance with Annex B of BS 812-124 shall be used
Clause 802.1	be protected from drying out and segregation both during transit to the point where it is to be laid and whilst awaiting tipping
Clause 802.11	Any permanent thickening shall be across the whole width of the pavement. Temporary thickening shall not impede drainage of any layer or the subgrade
Clause 803.6	Transported, laid and compacted without drying out or segregation
Clause 804.7	Transported, laid and compacted without drying out or segregation, at a moisture content within the range 1% above to 2% below the declared value of optimum water content when tested as required by BS EN 13285
Clause 805.1 Type 3 (open graded)	Permitted for use in circumstances where a free draining layer is preferred. There are additional requirements in to ensure a well graded mixture. Aggregate requirements relate to those given in BS EN 13242.

Figure 3.17: The main unbound aggregate related requirements given in the SHW

3.4.3 Aggregate degradation and premature failures

The aggregate used in a sub-base is assumed not to alter either proportionally or physically, i.e. the grading remains constant and is mechanically sound. However, some materials are prone to greater degradation than other when subjected to stress when wet. The nature of the degraded material and its subsequent effect vary depending on the aggregate source. A particular example is the weathered basalts of Northern Ireland. Problems have also been noted with recycled aggregates containing poor quality concrete and brick. It is not however a problem with the majority of UK sub base aggregate sources.

Another example is a heavily weathered basic igneous rock with very high expansive clay minerals being used as a capping / sub-base layer. As the picture below (Figure 3.18) shows, the material is rutting under construction traffic. This material will breakdown in the presence of water. In an inundated situation this layer will have very little stability or supporting structure.



Figure 3.18: Sub base rutting under construction traffic

An even more unusual case is a Sonnenbrand type failure of a basic igneous rock being used in the UK as sub-base, illustrated in the picture below (Figure 3.19). The crushed aggregate reacts with the air and water and starts to decompose. This aggregate was quarried less than 2 weeks before this image was taken. Sonnenbrand type failure is claimed not to occur in the UK.



Figure 3.19: Example of Sonnenbrand type failure

The issue of aggregate degradation has been investigated in the laboratory for over 100 years. Lord (1916) and Woolf and Runner (1935) recognised the relationship between aggregate degradation and performance.

Minor (1969) considered water to be a major factor associated with aggregate breakdown accelerated by rock to rock abrasion as the layer flexes. He attributed some types of degradation problem to the presence of altered minerals such as montmorillonite. This was found in rocks that had passed conventional laboratory testing.

He also reported that it was necessary to assess poorer quality aggregate or that which degrades quickly to problematic fines and they mention cases where plastic slurry of up to ½ inch thick was found to underlie the succeeding layer. This clearly affects the ability of a sub base to drain. In order to assess this he combined a rolling degradation test with evaluation of the produced fines using a modified Sand Equivalent procedure. Aggregates with values <40 were prone to generate plastic-like fines. This combination methodology is much better than the PI and crushing type tests traditionally used to assess sub-base aggregates.

In a review of test methods Venter (1980) concluded that there is no one test for all purposes. When evaluating the Los Angeles test, Eske and Morris (1985) found that soaking the aggregate sample prior to testing caused a further 10% degradation compared to dry testing.

Sampson and Netterburg (1989) stated that due to the complex nature of aggregate breakdown many aggregate tests are only suitable for certain applications. They reported that although most problems were associated with aggregates containing plastic fines, non-plastic fines also caused failures. Although materials with PI values of 12 performed adequately, they considered that this was due to moisture control.

As shown in the Table above a relatively simple number of tests are used to assess the suitability of a sub-base. Moisture is critically important for a sub-base material. However, current use of the dry Los Angeles (LA) fragmentation test is limited in its ability to consider the effect of moisture where totally dry conditions rarely exist in any road in the UK.

Megaw (1993) found that most ingress of water and so its detrimental impact when combined with loading occurred within 20 minutes. The importance of fines, i.e. material <0.075mm and smaller would appear to be of importance. Whilst the LA test generates fines, it does not include the effect of moisture.

Traditional types of test such as Ten Percent Fines Value (TFV) and Los Angeles (LA) fragmentation represent a small fraction of the entire sub-base particle range. They do not adequately assess the effect of degradation and formation of fines. Wet wear types of test such as the Rolling Mill Degradation (RMD) test used by Megaw (1993) to assess fines quality and the wet micro-Deval offer more appropriate generation forces.

Plasticity does not adequately assess the impact of the smallest fraction, i.e. as little as 0.002mm sized material forms the fines for slurry migration. It does not take material wear into consideration or predict fines generation. Woodward (1995) looked at methods of assess aggregate durability and recommended a modified version of the wet micro-Deval test to assess the wear of a graded fine aggregate.

Woodward (1995) extended the number of soaking cycles in the magnesium sulfate soundness test from 5 to 10 and 15. This found that the majority of the 75 aggregates assessed would have been classified as sound using a value of >75% retained after the standard 5 cycles. However, with an increase in the number of test cycles, a significant number of aggregates continued to break-up.

The FRAS project investigation determined the effect of adding just 1% salt to water during the freeze/thaw test (Petersen and Schouenborg, 2004). The addition of just 1% NaCl to the water during testing had the effect of considerably increasing the amount of aggregate failure compared to the standard Freeze / Thaw without salt.

Given the very wet summers that flood unbound layers, the amounts of repeated salt application in winter and its probable soaking into the sub-base layers these two examples are important to achieve better understanding of the premature failure.

3.4.4 Examples of premature failure due to fines

Under severe frost conditions in Alaska, layers with <6% fines did not show surface problems. Esch and McHattie (1984) recommended large pore spaces to be left in the particle mix to give adequate space for water to swell any moisture susceptible particles thereby reducing damaging impact on the pavement.

Conversely, Brooks (1968) reported that open textured sub-bases may experience problems if compacted on a weak subgrade due to intrusion destroying the benefits of drainage and stability (Farrar, 1968). Many design guides now recommend the use of a Geotextile separation layer to prevent this happening, for example on subgrade values of $\leq 3\%$ CBR.

Black and Lester (1978) considered that when a cohesive soil penetrates the voids of a sub-base layer its strength or stability is reduced by the height of the penetration.

Evidence has been seen where the soft subgrade has completely penetrated the sub base beneath joints in concrete roads vertically moving under heavy traffic.

3.4.5 Influence of drainage

Permeability is the property of permitting a fluid to pass through the body which possesses this property. Where there are no voids, i.e. they are filled with fines, this cannot happen.

Effective drainage aids longevity of the pavement system. This was recognised by the likes of Metcalf, Telford and Macadam in their early pavement designs that considered cross slopes, side ditches and sub-surface drainage for removing excess water before the sub-grade can be affected.

Most damage is done when water alters the engineering properties of the sub-base layer. Water in combination with induced stressing such as traffic may result in excess pore pressure or super-saturation. This displaces finer material that should be acting as a stability provider, causing weakness and structural problems. The displaced water in supersaturated conditions can transport fines causing concentrations of a slurry layer within and at the top of the sub base layer and reducing the effective permeability. This has also been discussed in Section 3.6.

3.4.6 The effect of grading

This property is controlled at production and influences a wide range of issues such as mechanical strength, permeability, and plasticity of the finer components. By control of quarrying, crushing and blending processes grading can be altered to suit particular needs. The grading envelope of a sub-base material is based on the need for an optimum high density and/or permeability.

All types of unbound aggregate materials have an optimum moisture content for compaction purposes, i.e. sufficient lubricant to allow re-orientation of particles into a denser state under an applied effort. The Proctor and Modified tests can be used to determine the optimum moisture content to predict the maximum dry density.

Megaw (1993) used a Modified Marshall uniaxial test to assess the cohesion/stability of different gradings using water as the binding agent. It was found that reducing fines in the material reduced the cohesion of compacted aggregate.

McCullough and Bell (1985) found an optimum stability grading. Using shear box testing and grading analysis they found that maximum stability occurs at the coarser side of the Type 1 grading. Shear strength of the sub-base is related to stability.

A well graded mix is more compactable than the uniform grading of a single sized aggregate associated with drainage layers. It is also less susceptible to secondary compaction and penetration by subgrade material.

3.4.7 Influence of moisture content

Moisture can significantly influence the ultimate dry density of a graded material. Acting as a lubricant it allows the particles to slide more easily into a denser structure. This can lead to consolidation of a saturated granular material by traffic. The replacement of air by water encased particles adds to an overall denser mass. When the optimum moisture content is exceeded the moisture starts to replace the particles and density decreases.

Fine clay particles can become coated with thin films of water. Combined with Van der Waals forces and hydrogen bonding they can aid bonding of the mix. These films of moisture can be <10 molecules thick (Thompson, 1984) and in comparison to the void infill are minute. A very small amount of water can bind the clays together.

With a coarser graded material there are less of these fine-sized particles and the mix will require a larger volume of water to fill the voids for maximum surface tension. Although the presence of hydroscopic material such as clay initially appears to bind the material together it is ultimately detrimental to the pavement. Less than 8% of these particles can significantly affect internal friction resulting in a material unable to withstand external stresses without deformation, i.e. excess moisture lubricates the system leading to premature deformation.

Campen (1954) considered the effect of clay increasing lubrication on reducing stability. Haynes and Yoder (1963) and Thompson (1984) found moisture content to strongly influence permanent deformation of dense crushed rock layers. They state that most pavement layers can perform adequately but during a wet period when the ground is saturated problems can occur and show themselves as surface failures.

Raad (1982) considered the Coefficient of Volume Compressibility (M_v) and the Coefficient of Permeability (k) to be significant properties. Raad found the increased permeability of coarser gradings and their reduced compressibility to reduce dynamic pore pressure and pumping potential.

Thompson (1984) stated that moisture sensitivity is reduced as the amount of <0.075mm and plasticity decreases..

Using a modified Marshall methodology Megaw (1993) found maximum stability to vary depending on moisture content and grading for limestone and greywacke aggregate. A decrease in fines content, i.e. a coarser grading, gave rise to maximum stability at a lower moisture content, but with a lower maximum value.

The amount of variation in maximum stability values for the limestone was less than for the greywacke aggregate assessed. Optimum moisture content increased with increasing coarseness for both rock types. However, the gradient for the greywacke is much greater than the comparatively flat limestone. However, this may have been influenced by the weaker limestone breaking down in the test mould during compaction.

Using the Proctor test Megaw (1993) found that as maximum stability increases with increase in C_u and C_z , the optimum moisture content required to obtain this peak stability also increases. This apparent contradiction is due to the nature of the two test methods and what each tried to measure.

Proctor measures the optimum moisture required to achieve maximum dry density. The finer the grading, or the larger % of 0.075mm material, means a larger surface area available for the water film. The modified Marshall method produced a curve of peak unconfined stability that did not coincide with the maximum dry density for the same moisture and grading. This shows that the optimum moisture content needed for maximum stability is different from the optimum moisture content required for maximum dry density.

McRae and Rutledge (1952) stated that this optimum condition producing a maximum density for a given compaction method is generally the strongest and most permanently stable condition for a soil resulting from the particular compaction procedure.

Seed and Monismith (1954) found that too much water and over-compaction was deleterious. For saturated subgrade conditions they concluded that higher density indicates higher stability.

McGee (1974) considered how the amount of fines affected performance of the graded aggregate and reported that sub-bases with a fines percentage in the range 10-15% were associated with poor pavement performance.

Megaw (1993) replaced the 0.075mm content with an inert lime. Using a modified Marshall test this was found to increase stability, i.e. the nature of the fines in a mix has a significant bearing on performance. For the finest graded material the addition of inert lime did not significantly raise maximum stability, but rather moved optimum moisture content from 5.5 to 7.5% .

The optimum moisture content needed for maximum dry density was 9%. Greatest improvement in stability by addition of lime occurred for the coarser grading. These findings confirm those associated with use of lime additives to graded materials with high fines content to increase their shear strength.

Megaw (1993) felt that internal friction is more important than density. Normally friction increases with density but with the presence of detrimental fines the lubrication of this system allows the characteristics of the fines to become more important.

Use of high moisture contents based on laboratory testing combined with over-compaction could lead to over-densification and movement of fines leading to their concentration. This is greater when using vibratory rollers compared to static rollers or with longer terms of trafficking.

3.4.8 Investigating the effect of induced dynamic stress using a wheel tracking test

Megaw (1993) modified an immersion wheel tracking apparatus to accommodate a 100mm deep crushed rock test sample unconfined. Type 1 grading variations were assessed at a range of moisture and density conditions. These were subjected to 750 passes of a 10kg loaded wheel and rut depths measured.

For each grading there was a moisture content threshold where the material gained stability as moisture content was increased, i.e. a moisture content of least deformation. Thereafter there was a dramatic increase in the rut depth coinciding to a loss of particle interlock. This investigation showed that there is a relationship between the void content derived from grading and compaction, and the moisture content required to induce a load bearing reduction within the material.

Once the voids were saturated excess water replaces the space normally taken by aggregate and reduction in dry density occurs. The development of positive pore pressure within the system reduces the interlocking ability of the material making it less likely to withstand the externally induced force, i.e. forces the particles apart allowing rutting.

This development of positive pore pressure will only readily occur when the moisture cannot escape from the system faster than there is a decrease in the voids / moisture ratio. The object of compaction is to reduce void content thereby effecting permeability, i.e. improved compaction reduces permeability. Therefore materials with low permeability as a result of their grading that suffer a decrease in the voids/moisture ratio will be more susceptible to the development of positive pore pressure within the system than one that is more free draining.

As soon as a sub-base becomes saturated from ingress during flooding or inundation due to extreme rainfall events, a sudden increase in rutting can occur. Predicting when a specific material with a certain water to void ratio becomes unexpectedly de-stabilised as loading is applied is also a problem. Water is relatively incompressible and when there is more water present than void space the water exerts pressure on the surrounding particles forcing them apart. This reduces interlock resulting in deformation.

3.4.9 Laboratory investigation of fines generation and migration within sub-base

Particle distribution affects the ability of a material to maintain stability under applied stresses. Megaw (1993) considered the breakdown and subsequent migration of aggregate fines within the sub-base. An apparatus called the Primary Cell Conditions Test (PCCT) was designed to allow measurement of fines production and its migration in relation to moisture content, grading and stress application.

This consisted of a series of loading cell composed of interlocking, water tight rings with a plunger attached to a hydraulic ram capable to applying variable loads and frequency. The loading cell had a 198mm internal diameter with each ring having a depth of 40mm. The maximum particle size in the aggregate was 20mm with the total layer thickness being 200mm. Each test sample was compacted according to its maximum dry density using Proctor. Testing was carried out at 5Hz and 5kN load for 1800 seconds. After testing the cell was dismantling and the material at each level assessed for grading.

Analysis of dry coarse graded material showed little movement of 0.075mm material. The larger 0.075/0.6mm and 0.6/5mm particles increased towards the bottom of the cell, i.e. it was influenced by gravity.

Further testing found that test samples with lower moisture contents resulted in a lower amount of fines generation with a vertical distribution affected by moisture change after testing.

The uppermost level of cell saturation was found to coincide with the level where the highest fines concentration was found, i.e. the fines migrated to the top of the saturated material under the cyclic pumping action.

In practice this would accumulate at the base of the asphalt over-layer or continue to migrate laterally. Testing weaker material resulted in a greater amount of fines. The amount of migration was greatest in the super saturated examples.

Analysis of the migrated fines found that it was the finest material <0.002mm that was most susceptible to movement. The formation of fine particles under wet conditions does not produce equal volumes, i.e. as each fines particle becomes encased in water envelope a slightly greater volume of slurry is present.

The nature of the individual mineral will dictate this volume increase with more absorbent mineral such as montmorillonite and other swelling clays, holding a higher percentage of moisture in this way. The greater this swelling capacity, the easier it is to induce saturation of the whole structure as more moisture is retained reducing air voids.

The generation of fines was found to relate to the number of load applications but not to its frequency. Migration of fines can occur if the material has the potential for fines generation and is subject to the correct conditions.

For example, use of vibratory rollers apply a greater number of cyclic stress applications compared to a static roller. This causes particle re-orientation to form a denser mass and possible migration effects. Their greater effective loading may also cause greater particle degradation.

Combination of both provide a greater opportunity for both degradation and migration than a simple static method. Megaw (1993) observed that saturated conditions were requisite to create the necessary environment for fines migration. Using large vibratory rollers to quickly reduce void content may actually make the conditions such that the change from saturated conditions will occur with relatively little increase in moisture.

3.4.10 Predictive equations relating to fines creation and migration

Figure 3.20 summarises a series of equations relating to fines creation and migration for sub-base materials subjected to simulated trafficking (Megaw, 1993).

Value predicted	Equation	R value
Percentage of fines created	$AFP\% = 0.0032t + 1.42L_{kN} + 0.095\lambda_{Hz} - 0.24RMD - 6.07$	0.7
Percentage of fines created (voids 11.5 to 7.7%)	$AFP\% = 1.84V\% - 12.48$	0.95
Percentage of fines created	$AFP\% = L - 0.08RMD + 0.0025t + 0.083\lambda_{Hz} - 0.59CU - 4.89$	0.9
Total fines content	$TFC = 0.889O_f - 0.007RMD + 0.0031t + 0.1\lambda_{Hz}$	0.97
Total fines content	$TFC = 0.91O_f - 0.009RMD + 0.0067LA$	0.96
Total fines content	$TFC = 0.86O_f + 0.0017TFV + 0.00063LA$	0.94
Percentage of fines in the top layer	$TLC = 1.17TFC + 0.0002LA - 0.006TFV$	0.7

Where:

AFP%	Average Fines Produced	LA	Number of load applications
L _{kN}	Load magnitude limited to a range of 2.5 to 7.5kN over an area of 0.0307m ²	V%	Volume of voids
t _s	Test duration 0 to 2880 seconds	C _U	Uniformity of original grading
λ _{Hz}	Frequency 3 to 50Hz	O _f	Original percentage of fines
RMD	Rolling Mill Degradation value 8.8 to 65		
TFV	Ten percent fines value		
TLC	Top later content of fines		
TFC	Total fines content		

Figure 3.20: Predictive equations relating to fines creation and migration

3.4.11 Factors related to the formation of a fines slurry

The formation of an enriched slurry surface or fines accumulation in a sub-base is dependent on the following scenario. These will become more dominant should the sub-base be inundated.

- Ideally, but not always, the sub-base material must be susceptible to produce fines of a certain quality.
- Where a crushed rock aggregate grading of lower quality is to be used the initial fines should be reduced to reduce the likelihood of future problems during compaction and in-service.
- Although rocks likely to degrade to produce expansive clays should be avoided, any type of particle can be suspended in a slurry depending on its surface area to mass ratio.
- The finer the grading the less water is required for it to become super-saturated and be susceptible to fines creation and fines migration.
- Increased fines content increases the magnitude of migration even though density of the sub-base is increased.
- The created fines are of single micron size and capable of movement within the inter-connected void structure.
- Although coarser graded material is more susceptible to degradation due to inter-particle actions the effect of fines generation is limited.
- Induced stress leads to creation of positive pore pressure within a saturated or near saturated material providing the mechanism for fines to migrate to form a lens within the layer or at its surface.

3.4.12 The conflict between grading and inundated sub-base failure

Sub-base material has to provide a range of conflicting criteria. Density is important as low void content reduces secondary compaction. The following summarises the main issues relating grading to failure of inundated sub-base materials.

- Water is essential for optimum compaction during construction but detrimental to the completed pavement.
- Attempts to reach zero voids during construction increases the water/voids ratio, increasing the risk of a positive pore pressure.
- Excessive pore pressure will reduce the materials ability to withstand external stresses leading to premature failure.
- For a given grading, moisture content is critical especially in finely graded materials where the percentage of voids is small.
- Over-compaction occurs when a material designed to be compacted at an optimum moisture content is further densified, resulting in fewer voids leading to positive pore pressure.
- It would appear that each grading has an optimum moisture content for resistance against rutting. Immediately beyond this moisture content the material is susceptible to rapid rutting.

3.4.13 Solutions to the problem of migrating fines

With respect to inundated sub-base materials and what would appear to be the inevitable creation and subsequent failure due to migrating fines, the following are possible solutions that need to be considered, i.e. these relate to choice of aggregate, grading / moisture content and induced stressing.

- Aggregate supplied with a high percentage of fines or susceptible to degrade to form fines during construction need to be recognised.
- Varying ground water conditions that allow saturated conditions to develop require additional drainage.
- Saturated sub-base conditions allow positive pore pressure and cause slurry migration.
- A specification for high density materials that necessitate the use of vibratory rollers may require a drainage or filter layer.
- Compaction of marginal materials using vibratory rollers should be avoided.
- There is no substantial movement of fines unless the voids are filled with the fines slurry.
- Fines movement will occur when the total amount of fines reaches a point of void saturation.

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3.5 Review of existing available published information - Asphalt Concrete

3.5.1 Introduction

Pavement damage is caused mainly by traffic loads, unsuitable material and the environment. The first two factors can be estimated for design purposes by empirical and mechanistic methods. However, the effect of environmental factors such as rainfall on the performance of pavements is not as well understood. Water on the road surface must be allowed to drain off the surface as soon as possible because it may contribute to accidents and reduce the service life of the surface material.

This review considers research relating to the effect of moisture on the properties of asphalt materials in general and Asphalt Concrete (AC) in particular. As the majority of this type of work has been done in America where AC is the predominant asphalt type, the American experiences are valid and appropriate to the UK. Moisture sensitivity is directly related to asphalt and all of the failure mechanisms listed above. Such understanding requires a holistic approach in evaluating and appraising elements of hydraulics, materials response and material performance that cannot be fully developed in this short review paper.

The effect of moisture, i.e. rain falling on an asphalt surface, is particularly interesting as surfacing asphalt layers can be designed to be porous, i.e. porous asphalt or some Thin Surface Course Systems, or be impermeable, i.e. Hot Rolled Asphalt. Even within the sub-group of AC mixes there is a very wide range of design based on air voids and void interconnectivity possible.

Water may enter the pavement structure through cracks in pavement surface, permeable surfaces, pavement edges, lateral movement from the road shoulder, percolating water, high water table, and liquid and vapour movement from the water table. Water can directly affect the performance of the surface layer by contributing to a wide range of failure phenomena. These include:

- Aggregate durability caused by issues such as repeated wetting and drying, thermal stressing in cold weather accelerated by application of de-icing materials.
- These types of aggregate failure may occur on-mass, i.e. every stone particle in the mix, as a percentage of unsound particles in the coarse aggregate, or due to the finer aggregate particle size.
- Loss of the aggregate bitumen bond, either within the outer layer of the aggregate, at the aggregate bitumen interface or due to the presence of dust coatings on the aggregate.
- Ravelling and fretting of the surface.
- Bitumen ageing.
- Separation from underlying asphalt layers.
- Failure of bond and tack coats.
- Potholes and delamination.

Climate change is considered as a threat to our common future and it predicts more frequent extreme weather events such as greater temperature fluctuation and more frequent extreme rainfall events. This change in climate, whereby the frequency and amount of rainfall is increasing due to global climate changes, has important implications on the life of all layers in the pavement structure.

3.5.2 The hydraulic characteristics of rainfall on asphalt surfacing materials

During a rainfall event a sequence of events occurs. Initially surface ponding occurs as the road surface texture gets filled with water. This varies depending on the degree of hydraulic conductivity, i.e. void content and interconnectivity. If the infiltration rate is smaller than the rainfall rate the difference of the two, i.e. the rainfall-excess rate, becomes available for overland flow. Rainwater then forms a layer of increasing thickness as it flows to the edge of sloped pavement surface. Slope, rainfall intensity, spacing of roughness elements are some of the factors considered in the overland flow of thin water films (Singh, 1996; Akan, 2003).

For an AC mix with very low voids, i.e. a mix that is almost impermeable, water film thickness or water depth is the total thickness of the water film on the pavement minus the water trapped in the macro-texture of the pavement surface. Water film thickness is reduced in direct proportion to the increase in macro-texture. Very porous material such as some Thin Surface Course Systems, or poorly compacted AC mixes, have an additional element of internal drainage that can significantly reduce surface flow during a rainfall event.

The mean texture depth (MTD) is a measure of the macro-texture of the pavement surface. Flow occurs in the total flow layer. This is the water film thickness (WTF) plus the mean texture depth. Increasing the macro-texture or depth is important because it allows a reservoir for storage water (depth below the MTD) and enhances drainage (depth above the MTD).

However, this storage property may be detrimental to the surfacing material as it isolates pockets of water that may then be subjected to hydraulic pumping due to trafficking or concentrate ice formation or de-icing materials acting as loci for premature failure (Millar et al. 2011).

Flow conditions affect resistance to water flow and hence speed of run-off. Flow resistance increases for unsteady, non-uniform flow (Kolosseus and Davidian, 1966; cited by Singh, 1996). Reed and Kibler (1983) state that flow resistance changes with the depth of flow, relative to the size of surface roughness elements such as the aggregate particles used in the AC mix. The Manning roughness coefficient ' n ' is the most popular resistance parameter in channel flow calculation and can be directly related to Darcy friction factor and coefficient of Chezy. Ross and Russam (1968) and Gallaway et al. (1971) looked at ' n ' as a basis measure of resistance in favour of developing equations from experimental data taken from other variables. Reed and Kibler (1983) investigated the relation between pavement texture and hydraulic resistance and used Manning's ' n ' as a basic measure of resistance, as opposed to an empirical study where the variables are measured and correlated statistically. They found that Manning's ' n ' is inversely proportional to the sheet flow depth. Huebner et al. (1998) reported the hydraulic resistance for 3 different types of pavement surface, i.e. Portland cement concrete, porous asphalt and dense-graded asphalt concrete.

Much of the work done on flow resistance has been based on impermeable boundaries. Much less work has investigated resistance to flow over porous surfaces. For laminar flow, reduced frictional resistance confirms interfacial velocity slip (Beavers and Joseph, 1967). For turbulent flow, significant momentum transfer occurs at the bed interface, which increases frictional resistance counteracting the reduction in flow resistance by velocity slip.

As permeability increases, lateral momentum exchange begins to dominate the overall resistance because small-scale turbulences penetrate large pore spaces (Richardson and Parr, 1991; cited in Singh, 1996). The physical interaction between permeable boundaries and fluid motion is of considerable significance and needs to be accounted for properly (Singh, 1996). Both laminar and turbulent flows can exist.

Overland flow conditions can be significantly affected by infiltration and percolation through the AC layer in terms of time and space. Infiltration rates are governed by material flow characteristics such as void content and its inter-connectivity. Since overland flow and infiltration are affected by different properties and processes, they need to be analyzed separately.

When two components of a single problem are analyzed separately the results of the separated components must be coupled (Motha and Wigman, 1995). Ridgeway (1976) reported on permeability measurements on un-cracked specimens of Portland cement concrete and dense-graded AC. They found that the amount of infiltration through un-cracked areas was insignificant compared to the amount of infiltration that occurs through joints and cracks in the pavement surface. Ridgeway (1976) stated that the amount of water that enters the pavement structure through cracks or joints depends on the water carrying capacity of the crack or joint; the area that drains to each crack or joint; and the intensity and the duration of the rainfall.

Hydraulic conductivity may be defined as the rate of flow of a fluid through a material based on Darcy's Law. Laboratory and in-situ testing can be used to measure the coefficient of hydraulic

conductivity or permeability of an asphalt surfacing material. The theory of asphalt mix permeability, including Darcy’s Law, was explained by Huang et al., 1999 (cited in Mohammad et al., 2003).

Studies have shown that the permeability of asphalt mixes is a function of percent air voids, aggregate gradation, aggregate shape, specimen thickness and compaction procedure. The interaction of these factors in addition to differences among the methods used to measure permeability, make it very difficult to develop an analytical equation to accurately relate permeability to these factors (Masad et al., 2002).

Gogula et al. (2003) reviewed the problems in measuring permeability for both field and laboratory testing. They stated that the majority of recent work in permeability testing had been conducted on either core specimens cut from the compacted pavement or on specimens compacted in the laboratory. This is important as Darcy’s Law is applicable for one-dimensional flow as would be encountered in a laboratory permeability test. Measuring the in-situ permeability of a compacted pavement is theoretically more difficult because water can flow in two dimensions. Other potential problems include the degree of saturation, boundary conditions of the flow and the type of flow.

Another potential boundary problem is the flow of water across or through the pavement layers. Without some type of destructive test such as coring, there is no way of knowing whether water has flown across the pavement layers. While conducting field permeability tests, Gogula et al. (2003) observed that the drop in water level during the first test usually took less time compared to subsequent replicate tests. One explanation is that during the first test water fills up the voids including some that were not interconnected. During the second and third test the water cannot go through these non-interconnected voids and only flows through the interconnected void network (Mallick et al., 2001; cited in Gogula et al., 2003).

Gogula et al. (2003) concluded that there was a significant difference between field and laboratory permeability values. The field permeability values were always much higher than laboratory permeability values, i.e. the higher values for the in-situ pavement are not confined to one-dimensional flow but also to water entering the pavement in vertical and/or horizontal directions.

The relationship between permeability and air void content was found to be exponential. As the asphalt content (bitumen content) in a mix is increased the effective air void content was reduced thus reducing permeability (Chen et al., 2004). Figure 3.21 shows typical void content and approximate hydraulic conductivity values for a range of asphalt surfacing materials (Daines, 1995). Figure 3.22 summarises permeability category values reported by Westerman (1998).

Material	Typical void content (%)	Approximate hydraulic conductivity (cm/sec)
Mastic asphalt	<1	$< 10^{-9}$
Rolled asphalt surfacing (30% stone)	2-8	$10^{-9} - 10^{-8}$
Rolled asphalt surfacing (50% stone)	4-9	10^{-4}
Asphalt concrete	3-5	$10^{-8} - 10^{-6}$
Close-graded bitumen macadam surfacing	4-7	$10^{-6} - 10^{-3}$
Open-graded bitumen macadam surfacing	12-20	$10^{-6} - 10^{-5}$
Porous asphalt surfacing	15-25	$10^{-6} - 10^{-3}$

Figure 3.21: Typical void content and approximate hydraulic conductivity values for a range of UK bituminous materials (Daines, 1995)

Permeability category	Permeability rate (cm/sec)
High Permeability	$10^1 - 10^{-4}$
Low Permeability	$10^{-4} - 10^{-6}$
Practically Impervious	$10^{-6} - 10^{-9}$

Figure 3.22: Permeability categories (Westerman, 1998)

3.5.3 Moisture sensitivity and asphalt mix durability

Although there is a significant amount of literature relating to moisture sensitivity, the specific effect of rainfall on surfacing mix durability has not been the subject of significant investigation. This is important given the predictions of global climate change and rising costs of materials. The UK’s climate is likely to suffer greater extremes of wetness whilst world demand for oil based products is causing major increases in the prices of bitumen. It is proposed that there may be an increasing number of asphalt surfacing materials failing prematurely due to rainfall or moisture sensitivity related issues.

The main piece of work that has impacted the UK was the SATS test developed by Collop et al. (2004). The SATS test was developed as a means to evaluate the performance of coated macadam (AC) binder course and base mixtures with relatively low binder contents and high air void contents. This methodology subsequently became adopted as Series 953 in the SHW Durability of Bituminous Materials – Saturation Ageing Tensile Test (SATS). Assuming that it is constructed properly they identified age-hardening and moisture damage as the primary factors relating to durability test methods for bituminous paving mixtures. This subsequently directed development of the SATS methodology.

In contrast, there has been much research into moisture sensitivity done in America where it is considered a national concern. It formed one of the areas investigated in the SHRP programme. A National Seminar on Moisture Sensitivity of Asphalt Pavements (TRB, 2003) stated that identifying the problem and isolating the contributing factors i.e. materials and construction are equally challenging. The topics addressed in this seminar considered the following:

- Identification of the problem – distinguishing between materials induced and construction related factors.
- Fundamental concepts – binder and aggregate considerations and failure mechanisms.
- Test methods – field and laboratory.
- Remediation – additives and construction practices.
- Field performance and case studies.
- Specifications – shortcomings and needs for improvements.
- Environmental and health issues.

In general terms Collop et. al. (2004) stated that moisture damage significantly influences mix durability due to loss of cohesion caused by failure of the adhesion between the bitumen and the aggregate (Kennedy, 1985 and Terrel and Al-Sailwai 1994). Coree and Kim (2005) describe moisture damage as separation of the bitumen coating from the aggregate surface in a compacted mixture in the presence of water under the action of repeated traffic loading. Shah (2003) concluded that moisture damage can be generally classified in two mechanisms that may either occur separately or together, i.e. loss of adhesion and loss of cohesion. The loss of adhesion is due to water getting between the asphalt and the aggregate and stripping away the asphalt film. The loss of cohesion is due to failure within the mastic.

Moisture related failure of a surfacing layer typically occurs as loss of fine and coarse aggregate known as raveling or fretting. Moisture infiltrates into the asphalt and weakens the bond and/or

mastic even without mechanical loading. However, under the combined action of moisture and traffic loading accelerated damage will occur. A wide range of variables have been identified. These include the type and use of the mix, bitumen and aggregate characteristics, environment effects during and after construction (Coree and Kim, 2005).

Solaimanian et al. (2003) stated in their review of literature relating to laboratory experiments and field studies that moisture sensitivity is correlated to:

- Aggregate source.
- Drainage and condition of drainage.
- Pavement structure e.g. lack of bonding between layers that can trap water, fabrics and interlayers that can trap water.
- Mix design such as binder content, grading, dust to bitumen content, film thickness, mix permeability, additives.
- Construction variability – segregation that can create areas of high air voids content, variance from job mix formula.
- Climate – determines the presence and amount of water, freeze/thaw action, asphalt temperature.
- Traffic which applies stress to the mix whilst in a weakened condition.

The interactions of these variables and the different levels of interaction at which laboratory testing can measure relevant properties or simulated performance are complex. However, Solaimanian et al. (2003) state that there is no clear relationship between laboratory results and field performance.

Laboratory tests used to estimate moisture sensitivity in Hot Mix Asphalt can be described into two basic types, i.e. on loose mixtures and on compacted mixtures prepared in the laboratory/cores taken from compacted pavements. These laboratory tests are summarized in Figure 3.23 and Figure 3.24 (taken from Solaimanian et. al. 2003) and Figure 3.25 (taken from Collop et. al. 2004). The actual test methods listed in Figure 3.23 and Figure 3.24 are summarized by Solaimanian et. al. (2003).

Test	ASTM	AASHTO	Other
Methylene blue			Technical Bulletin 145, International Slurry Seal Association
Film stripping			(California Test 302)
Static immersion	D1664*	T182	
Dynamic immersion			
Chemical immersion			Standard Method TMH1 (Road Research Laboratory 1986, England)
Surface reaction			Ford et al. (1974)
Quick bottle			Virginia Highway and Transportation Research Council (Maupin 1980)
Boiling	D3625		Tex 530-C Kennedy et al. 1984
Rolling bottle			Isacsson and Jorgensen, Sweden, 1987
Net adsorption			SHRP A-341 (Curtis et al. 1993)
Surface energy			Thelen 1958, HRB Bulletin 192 Cheng et al., AAPT 2002
Pneumatic pull-off			Youtcheff and Aurilio (1997)

* No longer available as ASTM standard.

Figure 3.23: Moisture sensitivity tests on loose samples (Solaimanian et al. 2003)

Test	ASTM	AASHTO	Other
Moisture vapor susceptibility			California Test 307 Developed in late 1940s
Immersion-compression	D1075	T165	ASTM STP 252 (Goode 1959)
Marshal immersion			Stuart 1986
Freeze-thaw pedestal test			Kennedy et al. 1982
Original Lottman indirect tension			NCHRP Report 246 (Lottman 1982); Transportation Research Record 515 (1974)
Modified Lottman indirect tension		T 283	NCHRP Report 274 (Tunnicliff and Root 1984), Tex 531-C
Tunnicliff-Root	D 4867		NCHRP Report 274 (Tunnicliff and Root 1984)
ECS with resilient modulus			SHRP-A-403 (Al-Swailmi and Terrel 1994)
Hamburg wheel tracking			1993 Tex-242-F
Asphalt pavement analyzer			
ECS/SPT			NCHRP 9-34 2002-03
Multiple freeze-thaw			

Figure 3.24: Moisture sensitivity tests on compacted specimens (Solaimanian et al. 2003)

Test method	Thermal cycling	Performance tests
Freeze-thaw pedestal test (FTPT) ²⁰	23°C for 3 days followed by -12°C for 15 h, 23°C for 45 min and 49°C for 9 h	Cracking of specimen over a fulcrum
Immersion compression test ²¹ Marshall stability test ²²	49°C for 4 days or 60°C for 24 h, 23°C for 4 h Vacuum treatment under water at 0°C to 1°C, 60°C for 48 h	Compressive strength Marshall stability
Duriez test ²³	18°C for 7 days	Unconfined compression at 18°C and 1 mm/s
Lottman procedure ²⁴	Distilled water at partial vacuum of 600 mmHg for 30 min, atmospheric pressure for 30 min, -18°C to -12°C for 15 h, 60°C for 24 h	Indirect tensile strength and stiffness
Tunnicliff and Root procedure ²⁵	Distilled water at partial vacuum of 508 mmHg until 55-80% saturation, 60°C for 24 h	Indirect stiffness
Modified Lottman procedure ²⁶	Distilled water at partial vacuum of 508 mmHg until 60-80% saturation, -18°C to -12°C for 15 h, 60°C for 24 h	Indirect tensile strength and stiffness
Bitutest protocol ⁶	Partial vacuum of 510 mmHg at 20°C for 30 min, saturation at 60°C for 6 h, 5°C for 16 h	NAT ITSM testing at 20°C
Immersion wheel tracking test ²⁷	Submerged in water at 40°C	Wheel tracking at 25 cycles/ min
Hamburg wheel tracking device ²⁸	Submerged in water at 50°C (25-70°C)	Wheel tracking at 50 passes/ min
Environmental conditioning system (ECS) ¹⁹	Water at partial vacuum of 254 mmHg or 508 mmHg for 30 min, 3 hot cycles at 60°C for 6 h, one freeze at -18°C for 6 h	Resilient modulus (stiffness) and permeability at 25°C

Table 2. Summary of moisture damage tests for compacted asphalt mixtures

Figure 3.25: Summary of moisture damage tests for compacted asphalt mixtures (Collop et al. 2004)

Review of these tables show duplication of the listed methods indicating that most of the methods identified by Collop have been carried out in America. There are a number of additional methods such as Duriez, the Bitutest protocol and Immersion wheel tracking test used in UK.

To further complicate any attempt to relate laboratory testing with field experience, Figure 3.26 shows the classic example by Schmidt and Graf (1972) that shows the effect of moisture measured using Resilient modulus as being reversible.

The TRB report (2003) is recommended reading for anyone interested in moisture sensitivity.

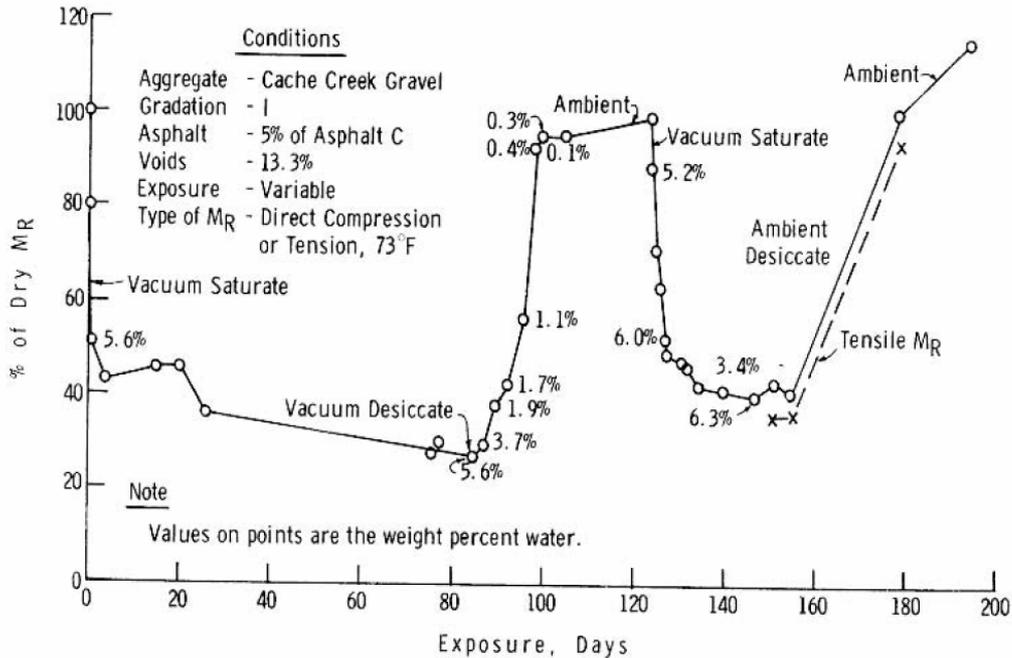


Figure 3.26: Effect of moisture on resilient modulus may be reversible (after Schmidt and Graf, 1972)

3.5.4 Aggregate type

Aggregates can greatly influence whether a mixture will be moisture sensitive or not. Their surface chemistry and the presence of clay fines are important factors affecting the adhesion between the aggregate and the asphalt binder. Common methods of combating these factors are through the use of anti-strip agents such as liquids or lime and by the elimination of detrimental clay fines through proper processing or specification (Little and Jones, 2003). These issues are discussed in more detail in the paper dealing with chemical and mechanical processes.

The detrimental effect of aggregate quality on durability was investigated by Woodward (1995). This PhD considered the laboratory prediction of surfacing aggregate and contrasted the BS 812 with the then proposed EN alternatives. Each aggregate property is considered in detail with the data broken down to rock type, whether sampled from the stock-pile or crushed in the lab, comparison of BS and 812 methods, inter-relationships between variables, modified versions of the BS 812 and proposed EN methods. It is considered that this still remains the most extensive investigation of surfacing aggregate laboratory testing carried out in the UK.

The investigation found many issues relating to the inadequacies of predicting aggregate performance in the UK. For example, Figure 3.27 shows the difference between wet and dry micro-Deval testing, i.e. water during testing causes considerably greater amounts of wet wear.

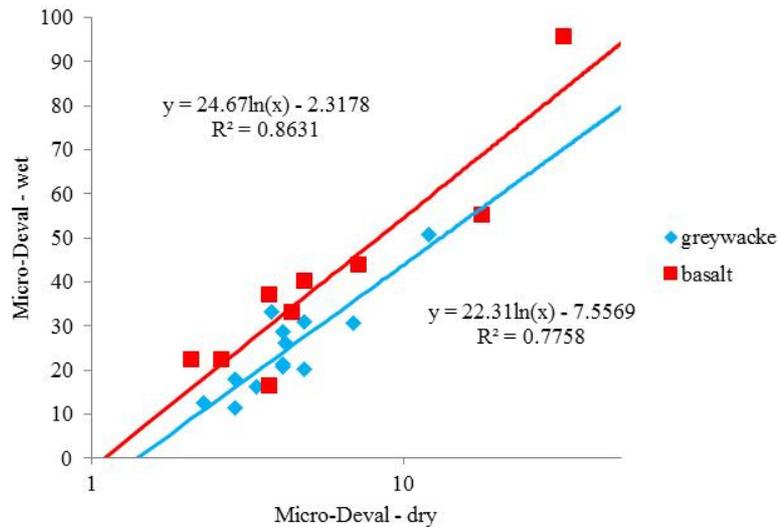


Figure 3.27: Plot of dry and wet micro-Deval data for a range of aggregates (after Woodward, 1995)

Woodward (1995) extended the number of soaking cycles in the magnesium sulfate soundness test from 5 to 10 and 15 as shown in Figure 3.28. This found that the majority of the 75 aggregates assessed would have been classified as sound using a value of >75% retained after the standard 5 cycles. However, with an increase in the number of test cycles, a significant number of aggregates continued to break-up. Given the excessive frost and salt used in the last two winters this is worrying.

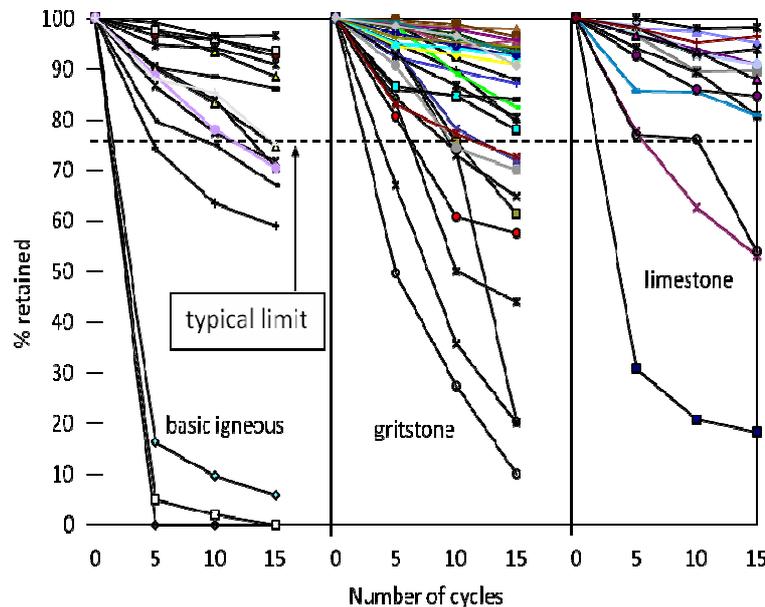


Figure 3.28: The effect of extended test cycles on aggregate durability using modified magnesium sulphate soundness testing (Woodward, 1995)

The FRAS project investigated the effect of adding just 1% salt to water during the freeze/thaw test (Petersen and Schouenborg, 2004). A summary of the results is plotted in Figure 3.29 for a range of European aggregates including some sourced in the UK.

The addition of just 1% NaCl to the water during testing had the effect of considerably increasing the amount of aggregate failure compared to the standard freeze/thaw without salt.

These simple examples show how aggregates that may be considered as durable based on standard test methods used in the UK may be susceptible to premature failure when the asphalt surface gets wet, has salt applied as a de-icer or subjected to freeze/thaw cycles.

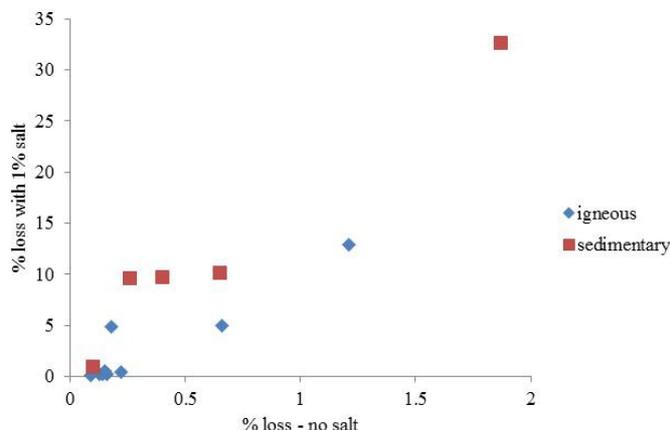


Figure 3.29: Plot of FRAS data showing effect of 1% salt addition to water during freeze/thaw testing (after Petersen and Schouenborg, 2004)

3.5.5 Binder coatings and water

Jamieson et al. (1995) stated that the SHRP adhesion model concludes that aggregate properties have a greater impact on adhesion than do various binder properties. This model found that stripping is controlled by cohesive failure within the aggregate rather than at the bitumen–aggregate interface (Jamieson et al. 1995).

Surfaces rich in alkali metals are more susceptible to de-bonding than surfaces rich in alkaline earth metals because the latter form water-insoluble salts with acid and other groups with the bitumen. The superior stripping resistance of some limestones is due to the formation of water insoluble (covalent) bonds between calcium sites on the aggregate and bitumen constituents, but stripping of calcareous aggregate can occur where their water solubility is high.

The net adsorption test (NAT) was developed under SHRP in the early 1990s and is documented in SHRP Report A-341 (Curtis et al. 1993). The test is used to determine the affinity and compatibility of an asphalt–aggregate pair and the sensitivity of the system to water.

Cheng et al. (2002) found that the diffusion of water vapour through a binder coating can be considerable and that asphalt mastics can hold a rather surprisingly large amount of water.

3.5.6 Void content

Terrel and Shute (1989) and Terrel and Al-Swailmi (1994) proposed the concept of pessimum air voids, i.e. the range of air void contents within which most asphalt mixtures such as AC are typically compacted (between about 8% and 10%). Above this level the air voids become interconnected and moisture can flow out under a stress gradient developed by traffic loading. Below this value the air voids are disconnected and are relatively impermeable and thus do not become saturated with water. This concept is shown in Figure 3.30.

In the pessimum range, water can enter the voids but cannot escape freely and is, thus subjected to pore pressure build-up upon repeated loading. This theory suggests that as most mixes are constructed within this pessimum range, they are at higher risk especially in early life. Later in life the material densifies with trafficking to the impermeable range. This secondary densification may not occur with the use of polymer modified binders or too low a bitumen content mix is used.

Water trapped in the pessimum range is slow to evaporate and may cause freeze/thaw issues or issues due to concentration of de-icing salts.

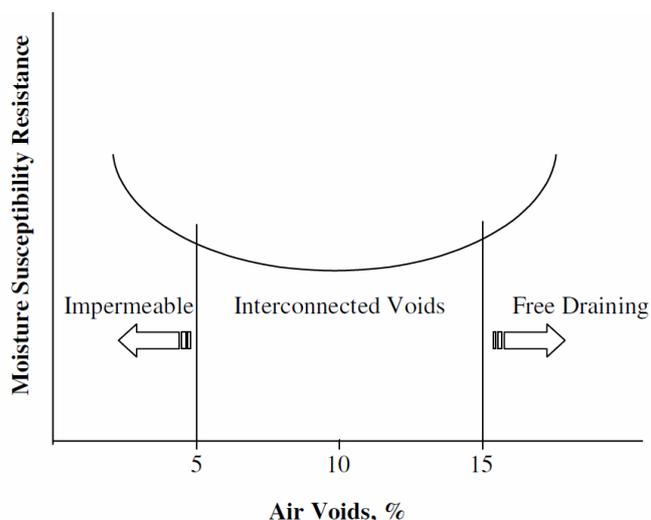


Figure 3.30: Concept of pessimum air voids (Terrel and Al-Swailmi (1994)).

3.5.7 Dynamic loading

Little and Jones (2003) describe pore pressure distress caused by entrapped water. Stresses imparted to the entrapped water from repeated traffic load applications will worsen damage as the continued build-up in pore pressure disrupts the asphalt film from the aggregate surface or can cause the growth of micro-cracks in the asphalt mastic.

They refer to work by Bhairampally et al. (2000) who used a tertiary damage model developed by Tseng and Lytton (1987) to demonstrate that well-designed asphalt mixtures tend to strain harden on repeated loading. This “strain hardening” is of course not classical strain hardening that occurs when metals are cold-worked to develop interactive dislocations to prevent slip but is the locking of the aggregate matrix caused by densification during repeated loading.

They report that some mixtures exhibit micro-cracking in the mastic under heavy repeated loading. This results in progressive cohesive or adhesive failure, or both, and is evident in a plot of accumulated permanent strain versus number of load cycles as the rate of damage dramatically increases as the micro-cracking progresses. The rate of this accelerated or tertiary damage is exacerbated in the presence of water as the pore pressure developed in the micro-crack voids increases the rate of crack growth and damage through the development of higher pressures at the crack tip and through a weakening of the mastic and of the adhesive bond between the mastic and the aggregate.

Little and Jones (2003) refer to hydraulic scour occurring at the pavement surface. This is stripping due to the actions of tyres on a saturated surface. Water is sucked under the tyre into the pavement by the tyre action.

3.5.8 Effect of void content on absorbed water and evaporation

Nursetiawan (2009) considered whether water contained on or within the void structure of an asphalt surfacing mix would affect its durability. The laboratory testing concentrated on two aspects, i.e. water retention as an asphalt mix dries out and change in asphalt mix properties due to water conditioning.

For four types of asphalt surfacing i.e. proprietary 6mm Open Texture (6mm OT), 10mm Dense AC (10mm AC) wearing course, proprietary 10mm Marshall Asphalt (10mm MA) and proprietary 14mm Marshall Asphalt (14mm MA). Test specimens 100mm in diameter were compacted using gyratory

compaction at 10, 50, 100, 200, and 700 gyrations to provide a range of void/interconnected void contents.

The hydraulic conductivity of the 100mm diameter test specimens was assessed using the University of Ulster in-house falling head method. The hydraulic conductivity data is plotted in Figure 3.31. This shows a ranking in the permeability of the materials used in relation to void content i.e. the 14mm MA was impermeable with the 10mm AC being most permeable.

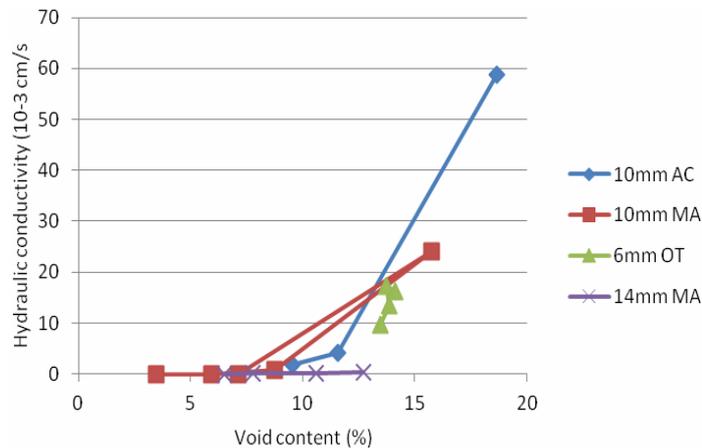


Figure 3.31: Void content v. hydraulic conductivity using falling head apparatus for 100mm diameter specimens (Nursetiawan, 2009)

The base and sides of each test specimen was sealed using wax and then submerged in water for 48 hours. Each test specimen was then removed, dried with a towel and weighed. The wax ensured that any moisture loss during drying occurred from the top. The weight of each specimen was determined. The effect of air void content on mass of initial absorbed water for the four types of mix is shown in Figure 3.32.

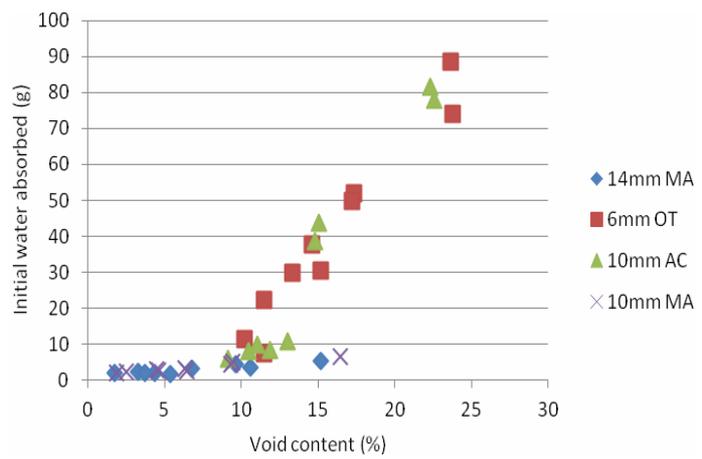


Figure 3.32: Initial water absorbed v. air void content (Nursetiawan, 2009)

Moisture loss due to drying at 20°C was measured periodically over a 170 hour period. This was determined by weighing each specimen in the temperature controlled room. As the base and sides of each test specimen was sealed with wax any change in specimen mass was due to evaporation. Figure 3.33 and Figure 3.34 show the data during the drying process for the dense 10mm MA and open 6mm OT at 20°C. It was found that the greatest amount of water loss occurred within the first 20 hours. Comparison of drying out rate at 10 and 20°C found no significant difference

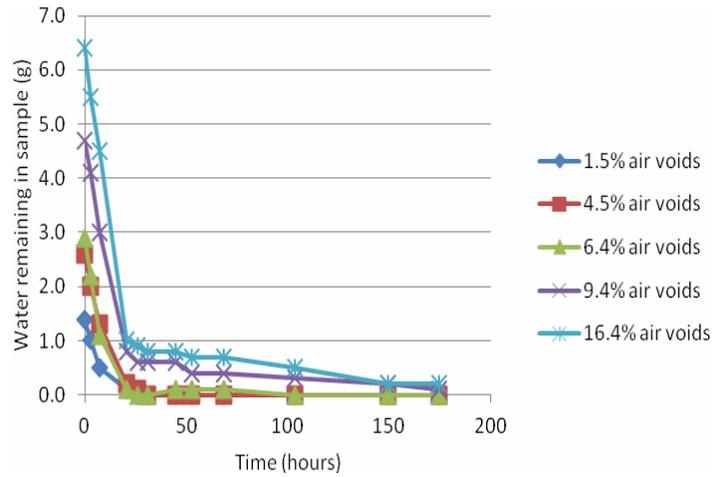


Figure 3.33: Water remaining in 10mm MA samples at 20°C (Nursetiawan, 2009)

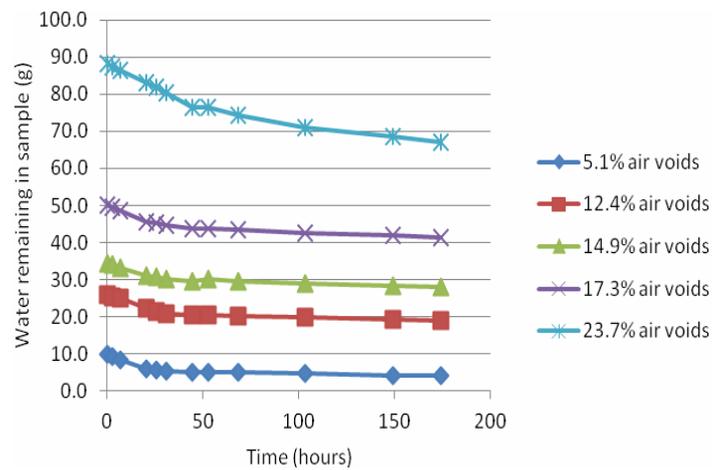


Figure 3.34: Water remaining in 6mm OT specimens at 20°C (Nursetiawan, 2009)

The drying out test was repeated using 305mm x 305mm slabs at 20°C. The method was similar to the 100mm diameter test specimen test. Figure 3.35 shows the amount of water retained for the 305mm square slabs tested at a room temperature of 20°C.

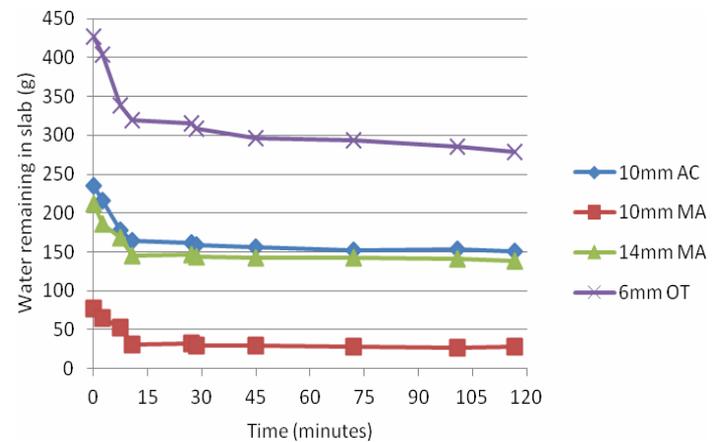


Figure 3.35: Water mass loss with time for 305mm slabs at 20°C (Nursetiawan, 2009)

3.5.9 Effect of asphalt aging

Nursetiawan (2009) investigated the effect of simulated rainfall on aged asphalt durability measured using the ITSM test method. 150mm diameter test specimens were used as these had a greater surface area compared to the 100mm diameter specimens. The 14mm Marshall Asphalt mix was used. The test specimens were compacted at a range of void contents that would be greater than the value typically used for this material. Three asphalt mix conditioning protocols were assessed:

- Condition 1- this used artificially aged asphalt to prepare the test specimens that were subjected to simulated rainfall.
- Condition 2 – this used un-aged asphalt to prepare the test specimens that were subjected to simulated rainfall.
- Condition 3 – this used un-aged asphalt to prepare the test specimens that were conditioned by being immersed in a water bath.

The 150mm diameter test specimens were prepared using a gyratory compactor with increasing number of gyrations used to give a range of void contents. The material had been reheated twice for a period of approximately 15 hours. Other test specimens were produced with fresh material reheated for 3 hours at 170°C.

The maximum density of the fresh re-heated 14mm Marshall Asphalt was found to be 2478kg/m³ using the Rice Method. Using this value, the air void content of the aged asphalt mixes ranged from 13 to 23%. It should be noted that properly compacted test specimens for the asphalt used should have had a void content of 6 to 8%.

The Ulster Rainfall Simulator (URS) was used to simulate 4 rainfall events over a 24 hour period (see Figure 3.36). A rainfall event consisted of 30 minutes rainfall and 5.5 hours drying at room temperature.



Figure 3.36: Test specimens in the URS being subject to simulate rainfall (Nursetiawan, 2009)

The Indirect Tensile Stiffness Modulus (ITSM) was used to assess the effect of repeated for a total of 235 rainfall and drying events. The data plotted in Figure 3.37 for the Test Condition 1 aged asphalt shows that Stiffness Modulus decreases with increasing number of rainfall cycles. In contrast, there is little change in the ITSM data for the test specimens made with the Test Condition 2 un-aged asphalt as can be seen in Figure 3.37.

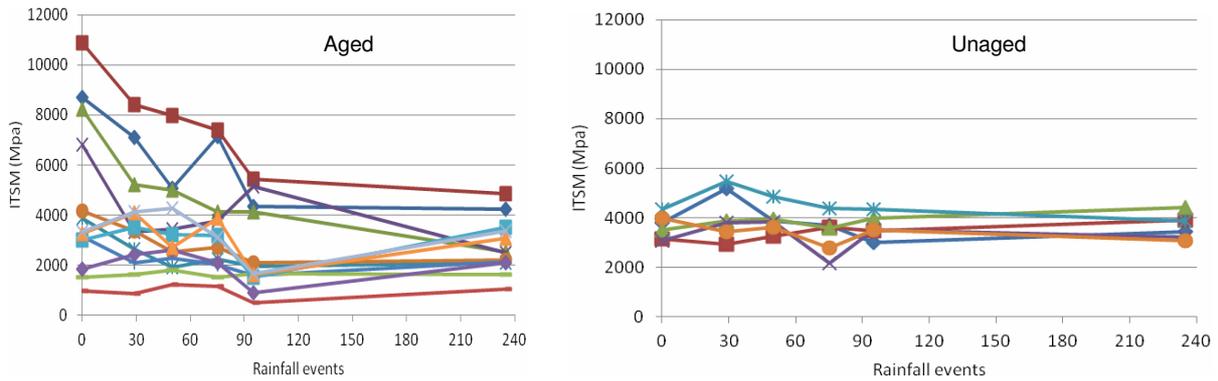


Figure 3.37: Change in ITSM for aged and unaged asphalt test specimens due to increasing number of rainfall events (Nursetiawan, 2009)

Figure 3.38 plots void content for the aged specimens against Stiffness Modulus for increasing number of rainfall cycles. The graph shows that Stiffness Modulus decreases with increasing void content. The plot shows that the greatest decrease in ITSM occurred for the lowest air void content test specimens. Although the higher void content mixes did not have as high ITSM, their relative loss due to simulated rainfall was considerably less.

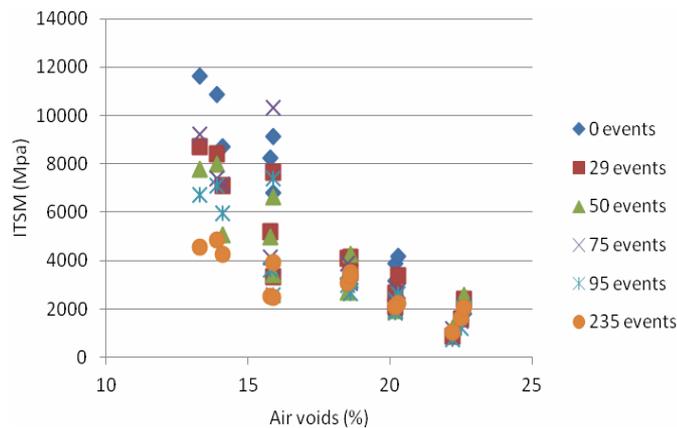


Figure 3.38: Relationship between Stiffness Modulus and void content for increasing number of rainfall cycles (Nursetiawan, 2009)

AC durability is a fundamental property and relates to global issues ranging from sustainability and climate change. For example, if the surface mix can be designed to have a longer life then there will be less spent of its maintenance. Climate change predictions imply that roads will be wetter for longer periods of time therefore increasing the risk of moisture related premature failures. Therefore, this issue of mix durability is a factor that needs to be understood. Although there have been attempts to look at moisture sensitivity few have fully considered the simple effect of rainfall on the surface layers of a road.

The issue of how road surface materials dry out, i.e. how long do they remain wet, and how repeated rainfall events effect mix stiffness, i.e. does a road lose stiffness during prolonged periods of wet weather, illustrates the impact that changing weather could have on AC mixes. In real-life both conditions will affect the life of the AC surfacing layer and possibly the structural failures underneath.

The drying out process is a dynamic process and influenced by many environmental factors such as air temperature, relative humidity, solar radiation and wind speed. It can be affected by the

temperature of the asphalt material and its surface area where a larger surface area will result in a faster drying time. In simple terms even poorly compacted 14mm MA and 10mm MA absorbed small amounts of water and so should have good resistance to moisture sensitivity issues.

In contrast, poorly compacted 10mm AC and 6mm OT will absorb a much larger amount of water and potentially be at greater risk from moisture sensitivity issues.

Considering the concept of pessimum air voids however, this simplistic argument may need reconsidered. The 10mm MA had absorbed very little water irrespective of how well they were compacted and lost this water quickly within the first 20 hours. In contrast the much greater amounts of water within the 6mm OT was not significantly reduced even after 170 hours or 7 days.

This was unexpected as it was thought that the interconnected voids would have allowed the drying out process to have quickly occurred. This may be attributed to the wax coating around the edge and base of the test specimens acting as an impermeable barrier. Any drying would be simply due to evaporation and not lateral drainage as would occur in a road.

It is believed that this experiment represents a worse case condition where the rain that has already percolated through the porous surface material is trapped and cannot flow freely to the drainage system. This can occur for example where construction joints are painted. In this case it will take a long time for the road surface layer to dry out by natural processes exacerbating the problems as a result of compaction issues.

This work was limited in its extent but clearly suggests that there are important issues that warrant further research to relate void content and its degree of interconnection in relation to the effect of rainfall on mix durability.

Two temperatures were assessed during the drying out test process. The results show that there is no significant difference in the drying out process between the two air temperatures used, i.e. 10°C and 20°C respectively. The graphs show that it was very little difference in drying rate for these two different temperatures.

Based on this drying out test, where only temperature effect was observed, it was concluded that for dense well compacted materials, the drying process related only to the water trapped on the road surface, i.e. trapped within the depth of its surface texture.

In the case of porous materials, poorly compacted AC mixes or road surfaces that may be cracked or suffering from other types of deterioration water will typically penetrate into the material and take a long time to slowly dry out. If the frequency of rainfall events is high this can lead to moisture induced damage on mix durability. The effect of moisture can be a significant problem that can damage and shorten a pavement's life.

Aged asphalt with the lowest void contents (14%) gave the biggest drop in stiffness. In contrast the aged asphalt with 22% voids had the lowest stiffness and this remained the same during testing. The results show that aging is an important issue to consider. The aged material was difficult to compact as shown in the very high void contents and when subjected to water the binding properties of the bitumen were poor resulting in the significant drop in stiffness as water affects bonding within the material.

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3.6 Construct a trial pavement in the laboratory and test

A small trial pavement facility was constructed in Jacobs laboratory, Wrexham comprising a strong box 0.6m x 0.6m x 600mm deep with permeable sides and base contained within a tank and sitting on a permeable foundation. This was filled with 350mm Type1 sub base and after testing, the sub base was surfaced with 200 mm asphalt compacted with a vibrating plate trench compactor.

The box design is as shown in Figure 3.39 and Figure 3.40 below.

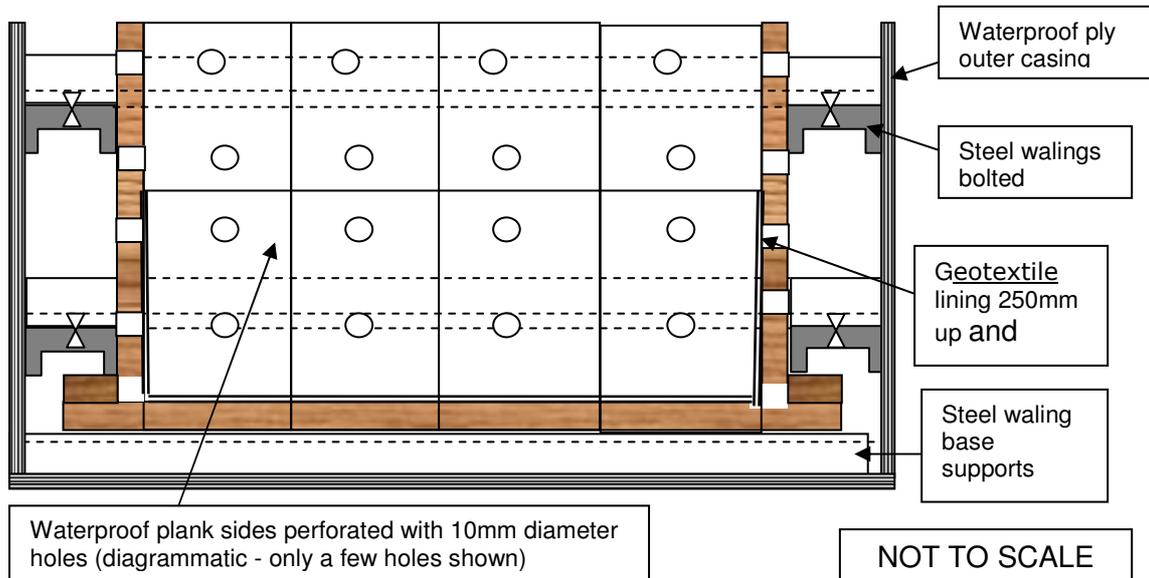


Figure 3.39: Permeable test box



Figure 3.40: Permeable test box (outer casing removed for transport)

3.6.1 Sub base testing

The SHW Type 1 limestone sub base was compacted in the box close at close to optimum moisture content (4.6%) in layers with a commercial plate vibratory compactor then tested without saturation, after saturating and after draining, using a lightweight Falling Weight Deflectometer, [the German Dynamic Plate] operated in accordance with the procedure in *Appendix A*. The device is shown in the photograph (Figure 3.41) testing sub base on a similar box.



Figure 3.41: Lightweight Falling Weight Deflectometer (LWFWD)

The sub base had a grading curve as shown in Figure 3.42 with 4.1% passing the 63micron sieve. The dry density moisture content relationship is given in Figure 3.43.

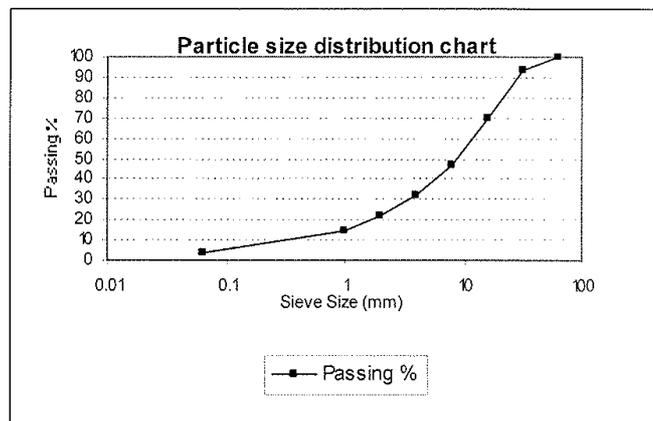


Figure 3.42: Sub base grading

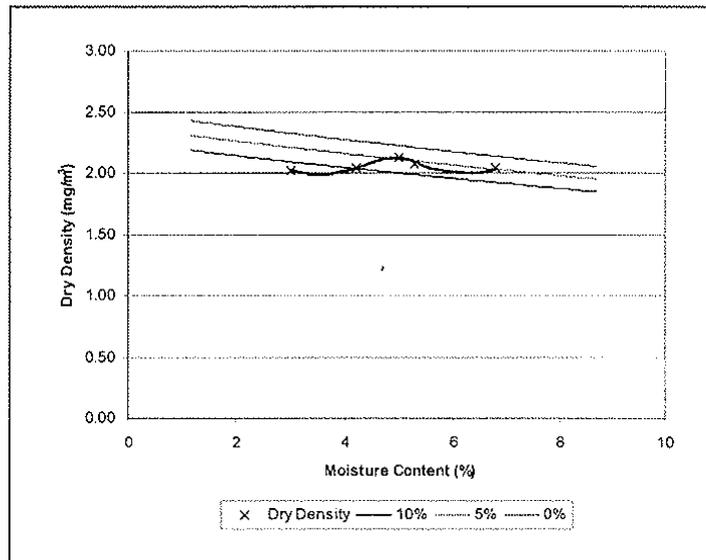


Figure 3.43: Dry density moisture content relationship

In Figure 3.44 the two soaking cycles are shown in blue. Between soaking the box was allowed to drain naturally. The relevant figures are those as blue diamonds and the right hand scale. The soaking periods are in pale blue. Base axis data is days.

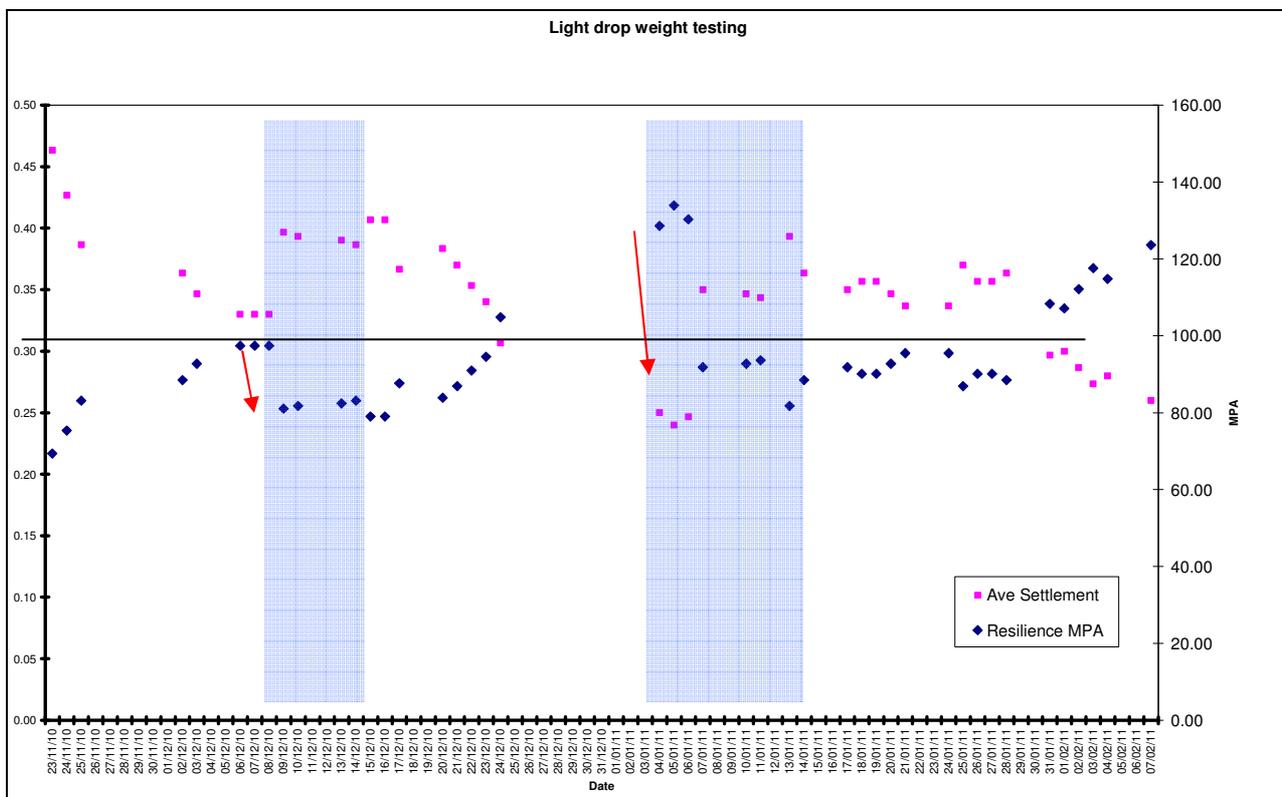


Figure 3.44: Layer stiffness modulus for sub base

The initial gain in stiffness is quite normal and is recognized in the Kent CC specification requiring the value to be declared after 24 hours, in the knowledge that further gains are possible. The

amount of stiffness gain is related to the aggregate petrography, for example in limestone it is greater than in granite.

Immediately on saturation there was a fall in stiffness of 17% from 97MPa to 87MPa on the first cycle and of 30% (from 130MPa to 92MPa) on the second cycle. The lower value lingered for a few days and it took just 6 days (1st cycle) but a longer time of 20 days after the second cycle, to revert to the pre-soaking levels.

It is frequently found that if a layer of fill is inundated settlement can occur. As discussed in the Section 3.4 dynamic compaction on a saturated layer, in the same location as pertains in the box, or for that matter in the wheel path, could increase the density of the sub base and hence explain the increase in stiffness modulus between the first and second cycles. This should not however continue to the extent that the sub base is destabilized.

It is known that no fine material escaped the box during the draining cycles as the box is lined with a Geotextile. However it is quite possible that there was fines migration as a result of the secondary compaction from the LWFWD. This could modify density and the permeability of the sub base itself, or conceivably the fines clog the Geotextile; this may be checked when the box is emptied. Both of these reasons would explain the slower speed of drainage of the box after the second soaking cycle. Sadly time did not permit a third cycle to determine whether this was a single event.

The sub base modulus measured prior to saturation was below the minimum long term surface stiffness modulus value of 100MPa required for a Class 2 foundation in IAN 73 Rev 1 (draft HD25/06). However it was above this when allowed to drain after saturation, and it returned to values above this when allowed to drain subsequently.

Normally crushed rock sub base, with levels of compaction in accordance with SHW Clause 802 Table 8/4, do readily achieve 100MPa. Therefore it is possible that the sub base in the box did not receive this compaction, or it was from a source difficult to compact.

The consequences of this evidence would appear to be that an occasional inundation event may not be deleterious and if the sub base is allowed to drain it may not suffer long term damage. However the laboratory evidence from others quoted in Section 3.4 does suggest this will be deleterious when sustained excess dynamic loading is applied when confined and saturated. It should be noted that the LWFWD testing was carried out on the sub base layer without surcharge so significant excess pore pressure could escape from the surface.

3.6.2 Asphalt testing

Two cores were taken of the asphalt from the box near the corners and they were separated into the base (two layers) and binder course. The core holes were backfilled with dense asphalt concrete and the following tests were carried out. Operations included in this task include:-

- 1 Measurement of the air voids. Good compaction was achieved in the box on both layers of asphalt but surprisingly the binder course had slightly higher voids (5.8% vs. 4.2%). This fell into the 'pessimum' range as discussed in 3.4.19.
- 2 The base (x2) and binder course were tested for ITSM in the Nottingham Asphalt Tester then saturated and retested (BBA HAPAS Thin Surface Course Systems [TSCS] Guideline Protocol Appendix A.2). This involves the following processes:

The specimen is covered with distilled water at 20±1 °C, and subjected to a partial vacuum of 510 mm Hg for 30 minutes. It is removed and placed in a hot water bath at a temperature of 601 °C for 61 hours. It is removed from the hot water bath and immediately placed in a cold water bath at a temperature of 5°C for 16 hours, after which it is immediately placed in a water bath at a temperature of 20±0.5°C for 2 hours. The specimen is removed from the water bath, surface dried and the conditioned stiffness determine at a test temperature of 20C for the first conditioning cycle. The procedure was repeated twice.

- 3 The specimens of two cores [1 and 2] after testing as above were heat ‘aged’ by the BBA HAPAS protocol for Thin Surface Course Systems [TSCS] Guideline Protocol Appendix A.12). This involves the following processes:

Place the prepared specimen(s), supported in the wire mesh basket(s), in a forced-draft oven for 120 hours at a temperature of (85±2), after which the oven was turned off and the specimen allowed to cool to room temperature for at least 24 hours prior to retesting for ITSM as above dry and saturated.

The penetration of recovered binder of the initial 40/60 grade bitumen was measured to assess the ageing effect. It was found to be 19pen, softening point 66.4°C. These are typical values for long term in service asphalt.

- 4 The specimens of another two cores [3 and 4] after testing as above were saturated and placed in the freezer for 48hrs and then thawed for 24 hours at room temperature. Two freeze thaw cycles were carried out and ITSM retested after each.

The results from the tests are presented graphically in Figure 3.45 and tabulated in Figure 3.46.

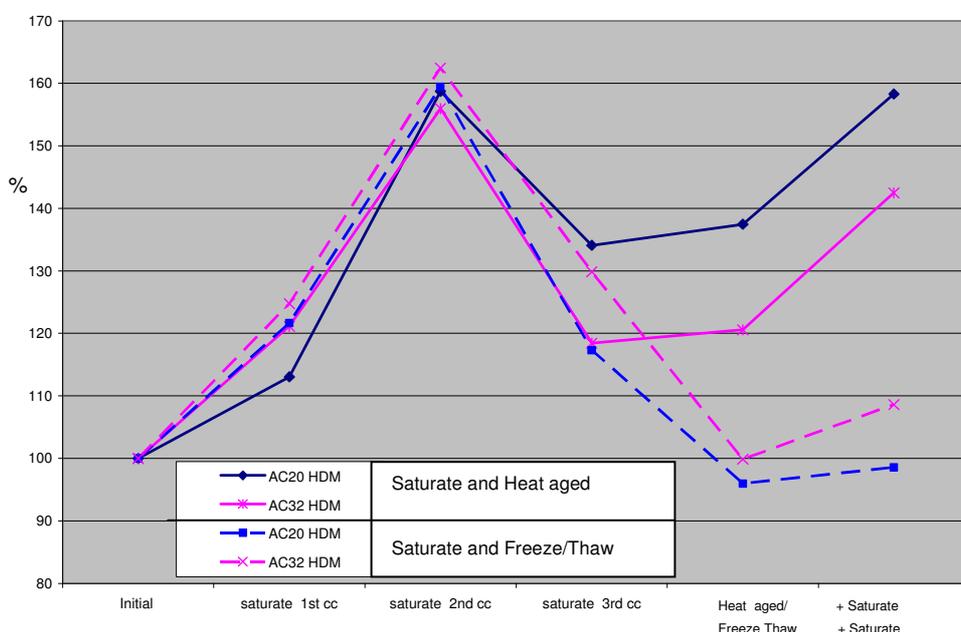


Figure 3.45: Effect of saturation, heat aging and freeze thaw cycling on binder course and base

The increase in ITSM stiffness after saturation (by an average of 159% over the original) is very common as a result of the pore water pressure not being able to dissipate fast enough beneath the platen so that the water in effect cements the core together. However after two cycles there may have been some disruption in the specimens that caused a small reduction in stiffness.

As expected, as the binder hardens stiffness increases. However freeze cycling a saturated specimen further disrupts the asphalt so that it falls by 30% compared to the immediately previous value, and falls just below the original.

This may be a significant finding in circumstances where binder courses can freeze, e.g. on 70% of the Local Authority network that is not salted in winter.

Core	Description		Initial	saturate 1	saturate 2nd cc	saturate 3rd cc	Heat aged	HA + satn	Air Voids
1A	binder	AC20 HDM	6918	7343	10360	7860	9418	10406	5.8
1B	upper base	AC32 HDM	7851	9270	12410	9087	11036	11494	3.5
1C	lower base	AC32 HDM	8886	10433	13480	11742	9247	11895	4.8
2A	binder	AC20	5504	6699	9360	8798	7655	9258	
2B	upper base	AC32 HDM	4398	5497	7073	4514	5336	6027	
2C	lower base	AC32 HDM	7356	9315	11478	8397	8735	11174	
Mean binder 1&2		AC20 HDM	6211	7021	9860	8329	8537	9832	5.8
Mean base 1&2		AC32 HDM	7123	8629	11110	8435	8589	10148	4.2
change vs initial %		AC20 HDM	100	113	159	134	137	158	
		AC32 HDM	100	121	156	118	121	142	
Core	Description		Initial	saturate 1st cc	saturate 2nd cc	saturate 3rd cc	FrzeTh 1st cc	FrzeTh 2nd cc	
3A	binder	AC20 HDM	8112	9688	11826	8815	7764	8040	
3B	base	AC32 HDM	6162	8243	10514	8055	6254	6704	
3C	base	AC32 HDM	6958	9188	12440	10041	7179	7385	
4A	binder	AC20 HDM	6151	7665	10913	7922	5926	6022	
4B	base	AC32 HDM	7234	8724	11254	8834	7124	7050	
4C	base	AC32 HDM	8952	10409	13399	11127	8714	10689	
Mean binder 3&4		AC20 HDM	7132	8677	11370	8369	6845	7031	
Mean base 3&4		AC32 HDM	7327	9141	11902	9514	7318	7957	
change vs initial %		AC20 HDM	100	122	159	117	96	99	
		AC32 HDM	100	125	162	130	100	109	
Mean binder all		AC20 HDM	6671	7849	10615	8349			
Mean base all		AC32 HDM	7225	8885	11506	8975			
change vs initial %		AC20 HDM	100	118	159	125			
		AC32 HDM	100	123	159	124			

Figure 3.46: Effect of saturation, heat aging and freeze thaw cycling on binder course and base

3.6.3 Sub base and asphalt testing with Falling Weight Deflectometer

The box was transferred to Jacobs Derby facility for testing both dry saturated and after draining using Jacobs Falling Weight Deflectometer [FWD] recently returned from calibration.

The FWD was lifted up onto a platform of the same height as the box so that the central deflection took place in the centre of the box.

Test details

Three geophone deflectors were used; the first at the centre of the loading plate, the second 200mm away from the centre and the third 300mm away from the centre.

Each test cycle included 50 drop sequences. Each drop sequence included 3 drops at load levels of 18kN, 30kN and 40kN. So each test cycle accounted for 150 test drops in total.

Test cycles were taken at the following stages:

1. Initial testing before first inundation.
2. During the first inundation (soaked for 72 hours).
3. 2 hours after the water retention valve was opened (i.e. during draining).
4. 3, 4, 5, 6, 7, 9, 20 hours after the water retention valve was opened.
5. During the second inundation (soaked for 24 hours).

In total 2,100 test drops were taken on the sample.
Summarised results of FWD testing are tabulated and shown graphically below (Figure 3.47).

Condition	Minimum deflection reading			Average deflection reading			Maximum deflection reading			Standard deviation of data set		
	Central deflection reading	200mm deflection reading	300mm deflection reading	Central deflection reading	200mm deflection reading	300mm deflection reading	Central deflection reading	200mm deflection reading	300mm deflection reading	Central deflection reading	200mm deflection reading	300mm deflection reading
Original state	1315	1149	1080	1534	1360	1273	1751	1568	1483	137	128	122
Inundated	1063	937	883	1180	1045	991	1316	1178	1125	77	70	68
2 hrs after inundation	1148	997	912	1315	1137	1049	1457	1258	1151	94	74	62
3 hrs after inundation	1234	1076	983	1369	1180	1078	1511	1295	1186	101	78	73
4 hrs after inundation	1249	1087	986	1385	1198	1103	1533	1322	1214	109	86	80
5 hrs after inundation	1252	1101	1004	1403	1213	1111	1563	1337	1226	115	89	83
7 hrs after inundation	1250	1076	994	1401	1205	1109	1560	1322	1218	107	87	79
9 hrs after inundation	1244	1071	987	1393	1190	1095	1550	1308	1206	112	85	77
17 hrs after inundation	1224	1064	971	1372	1173	1081	1534	1292	1194	117	84	76
Inundated	1036	887	819	1147	985	914	1263	1087	1011	77	66	63
2 hrs after inundation	1163	1005	925	1295	1107	1023	1442	1223	1127	103	77	71
3 hrs after inundation	1171	1009	921	1311	1119	1032	1462	1243	1149	108	84	77
4 hrs after inundation	1179	1021	932	1317	1128	1035	1479	1260	1160	112	87	80
5 hrs after inundation	1183	1024	937	1323	1136	1041	1478	1265	1158	114	90	82

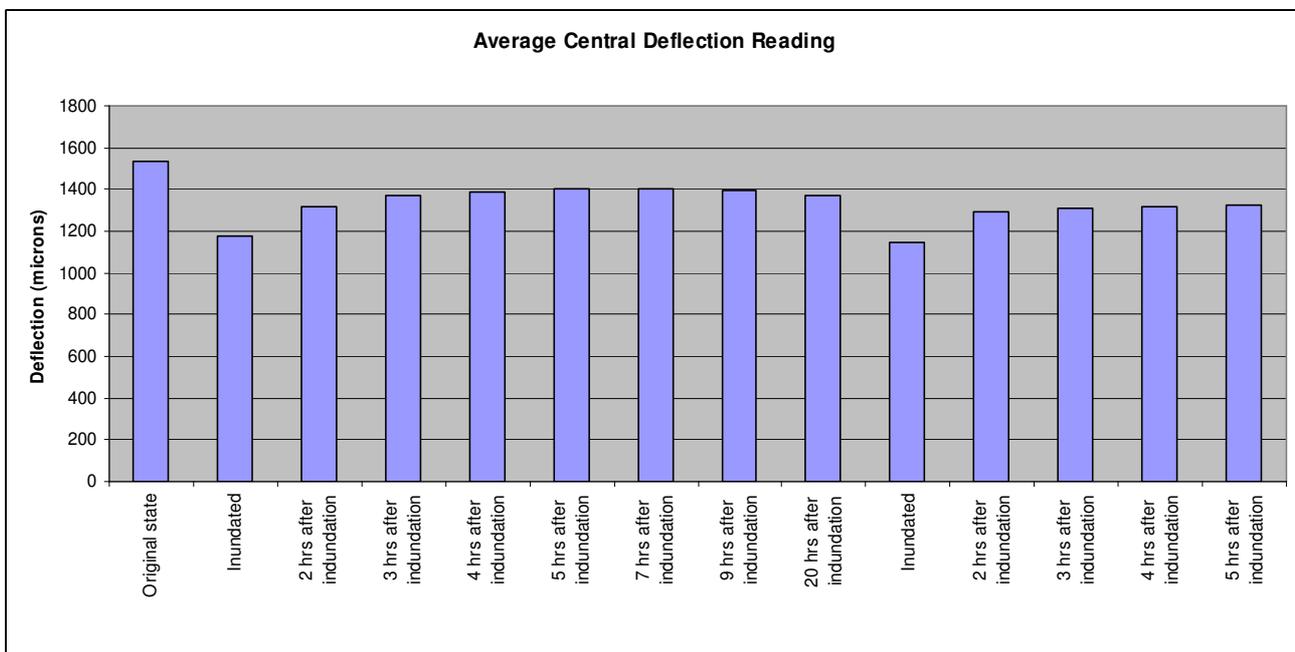


Figure 3.47: Deflection data and graph of average central deflections

Condition	Deflection Parameters		
	Central - 200mm	Central - 300mm	200mm - 300mm
Original state	173	261	87
Inundated	135	189	54
2 hrs after inundation	178	267	88
3 hrs after inundation	188	291	102
4 hrs after inundation	187	282	95
5 hrs after inundation	190	293	103
7 hrs after inundation	196	292	96
9 hrs after inundation	203	298	95
17 hrs after inundation	200	292	92
Inundated	162	233	71
2 hrs after inundation	188	272	84
3 hrs after inundation	191	278	87
4 hrs after inundation	189	283	94
5 hrs after inundation	187	282	94

Condition	Void
	Central - 300mm
Original state	35
Inundated	21
2 hrs after inundation	67
3 hrs after inundation	60
4 hrs after inundation	63
5 hrs after inundation	67
7 hrs after inundation	71
9 hrs after inundation	73
17 hrs after inundation	73
Inundated	30
2 hrs after inundation	65
3 hrs after inundation	65
4 hrs after inundation	67
5 hrs after inundation	66

Figure 3.48: Deflection Parameters and Voids Intercept of central deflection minus reading 300mm from centre

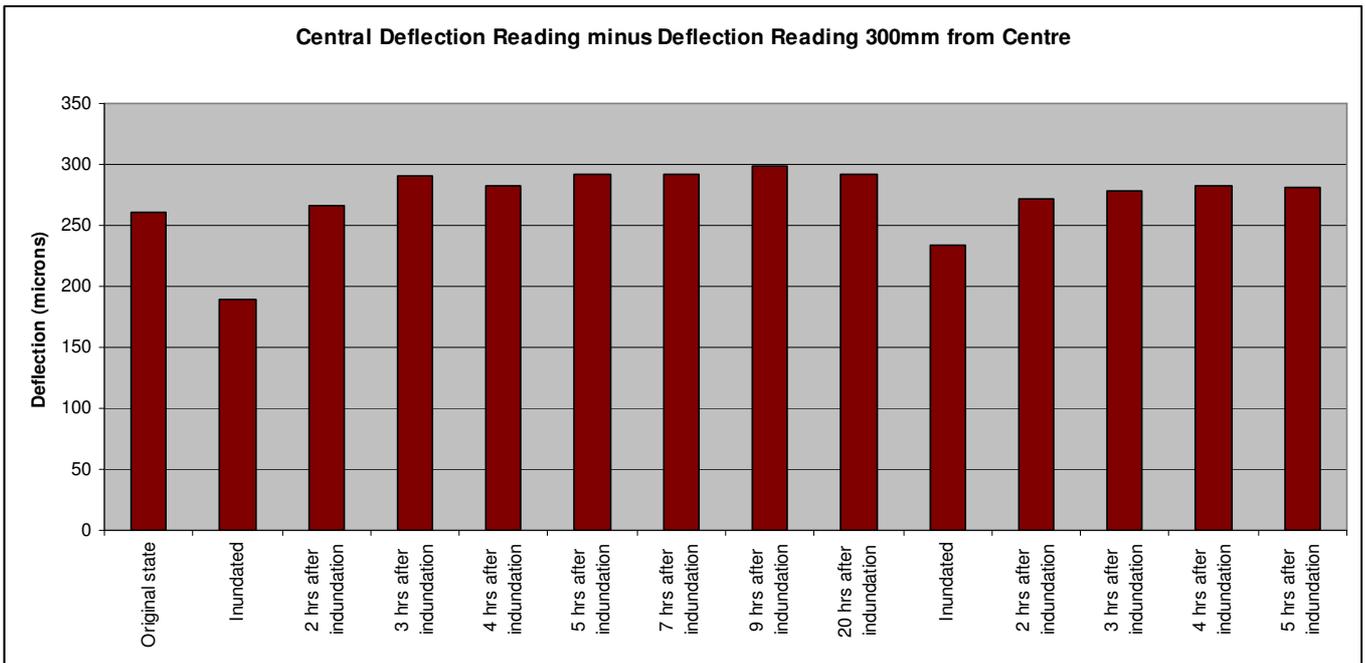


Figure 3.49: Central Deflection Reading minus Deflection Reading 300mm from Centre

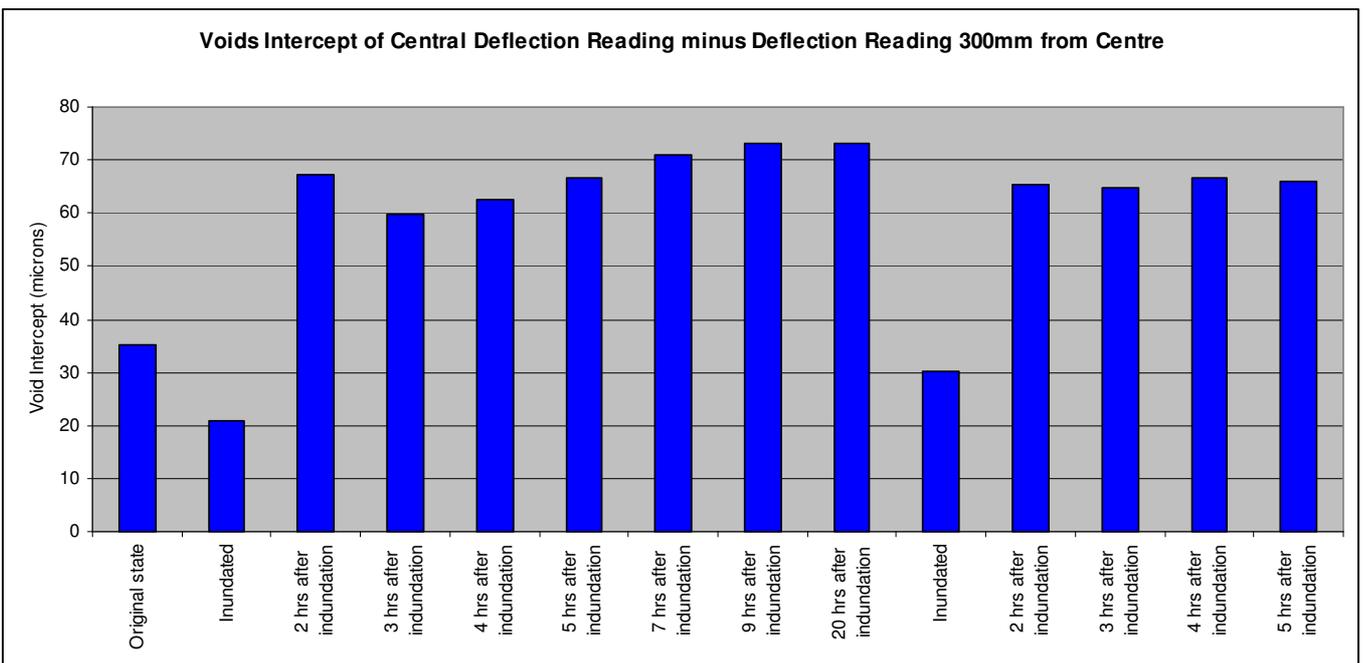


Figure 3.50: Voids Intercept of Central Deflection Reading minus Deflection Reading 300mm from Centre

Discussion of the results for the FWD testing

The average central deflection readings indicate that the initial central deflection reading was the highest; the higher the deflection the lower the stiffness of the pavement. However the average deflection reading measured 300mm away from the centre of the loading plate was also at its highest, which may suggest that the whole box was undergoing a seating action or

expansion under load despite the strength of the walings and timbers. The deflection parameter (central deflection minus 300mm deflection) can be used to remove the affect of the support outside of the sample, this gave a value of 261 microns. The void intercept of the three load levels for this parameter was 35 microns.

The void intercept is the value found when the deflections from drops of different loads are plotted on to an x-y graph and the best fit line of the points is taken. Where the line cuts the y axis generates the void intercept value. If the pavement is solid then the void intercept is zero; as the intercept increases the strength of the pavement reduces.

It may be assumed that the central value is the overall deflection and the central minus the 300mm value is an indication of the asphalt layer and possibly some of the granular layer as well.

Using the values from the central deflection minus the 300mm deflection for the void intercept calculation is an attempt to discount possible movement of the sidewall or base.

Following the first inundation, both the average central deflection and the average 300mm deflection dramatically decreased. The deflection parameter (central deflection minus 300mm deflection) also reduced (to 189 microns). The void intercept of the central minus 300mm deflection parameter also reduced to 21 microns. All this data suggests that the whole test sample had become stiffer on inundation and the saturated material was stronger than the dry material. This phenomenon is also discussed in Section 3.4.8.

Following the first inundation and after the water retention valve was opened to drain the box the central deflection reading increased by a significant amount to 1,306 microns [1.3mm] as the water drained away, although this was not as large as the initial reading of 1,315. Following this test sequence, the average central deflection steadily increased up to around 1,400 microns where it remained steady. This suggests the box drained over this time-scale to an equilibrium moisture content. The central minus 300mm deflection parameter also increased following the inundation and the 'steady-state' value was higher than the initial readings prior to the first inundation and suggestive that the sample had become slightly less stiff. A dramatic increase in the void intercept was observed in the test sequence performed 2 hours after the water retention valve was opened although this then decreased and then steadily increased up to around 70 microns which is over double the values from the pre-inundated testing.

The second inundation showed a similar change in deflection to the first, although the central minus 300mm deflection parameter was slightly higher, suggesting the sample was less stiff. The void intercept was 10 microns higher, suggesting that the water had not saturated the sample as previously, possibly as a result of air trapped in the sub base.

The results following the second inundation were similar to that of the first inundation with a marked increase in the central deflection and an increase in the void intercept value.

The results from the box are similar to values experienced in real road testing. When saturated roads have been tested in the past, the deflections are very low because it was not possible for the water to escape beneath the loaded patch fast enough and the water therefore acts as a solid. If the loading time was to be reduced, then the weakened strength of the sub-base might have a bearing on the strain level at the base of the asphalt layer. However when the loading time is quick (as in a moving vehicle) then the water acts as a solid mass. Under a moving vehicle there is in addition a horizontal component to the load in one direction. Over time this could lead to movement of fines from beneath the wheel path and loss of strength. This is a similar result to that found on the sub base alone when tested with the Lightweight Falling Weight Deflectometer as shown in Figure 3.46. However the values under the complete pavement recovered faster than with sub base alone.

The interesting result is that void intercept increases when the box starts to dry out. This may be explained in that the sub-base is losing the solidity of the fully saturated state and is gradually returning to equilibrium with a decreasing high moisture content.

In essence the results suggest that when the road is fully saturated it can still have adequate stiffness for moving loads. However as the foundation is returning to equilibrium then the damage might be at its highest and the more efficient the drainage system then the less amount of damage is likely to be caused.

3.6.4 Overall conclusions from the pavement testing

1. In the laboratory, subjecting an unconfined saturated sub base layer to a quick transient load with a lightweight FWD reduces its stiffness by about 17% from the initial value and more (up to 30%) if this is repeated. However all sub base increases in stiffness with time from the initial value, as the aggregate particles self cement together. Granite is poorer than limestone in this respect. When confined by an asphalt overlay and tested with a full scale Falling Weight Deflectometer, the sub base strength was increased as the water acts as a solid and cannot escape through the pore structure fast enough.
2. This 'strengthening' effect of water was also seen on asphalt, both in the in the NAT testing and with the full scale FWD. There was little evidence of deterioration of the pavement on the second cycle compared with the first. This suggests significant deterioration of the pavement subjected to repeat heavy loads when saturated, may not be an issue. However time precluded more cycles to verify this.
3. The effect of freezing saturated asphalt was significant and should be avoided in practice by ensuring adequate drainage and a good seal coat if de-icing is not carried out.

4. Summary conclusions

- 4.1 The amount of research on the effect of inundation on in-service roads is very small. Most work has been carried out in the laboratory on relatively small scale specimens.
- 4.2 The short term inundation of a pavement would appear to be relatively benign. However only one site was investigated and only a limited number of cycles of testing were carried out in the laboratory.
- 4.3 The consequence of this finding is that action to prevent road pavement damage by flooding can be delayed until routine maintenance is carried out when the measures can be implemented at low cost. Modest but valuable changes to the specification have been identified that can improve durability of asphalt and sub base in the wet.
- 4.4 The work of this study using a small trial pavement in a box and field tests corresponded to work done by others previously and could therefore be deemed to be valid.
- 4.5 Sub base was found to lose a significant amount of strength when wet and evidence from laboratory and in-service roads shows it can be prone to degradation and fines migration if loaded when saturated. Some sub base materials are more prone to degradation than others. Where appropriate stabilized sub base materials should be considered.
- 4.6 Prevention of saturation is important. This may be achieved by reducing the permeability of the surface and increasing the permeability of the sub base. Local Authorities generally use surface dressing to keep the pavement surface sealed.
- 4.7 Where a flood has persisted for a long period of time, e.g. >3days on a road with positive drainage and a sealed verge/central reservation, or >1day with combined drainage, it may be prudent to close the road to traffic not just whilst it is under water but also until the sub base is no longer saturated. It may not be all that easy to tell when it is safe to reopen it.
- 4.8 Open graded hence permeable materials have less strength than dense materials but there exists a window that can provide an acceptable material satisfying both criteria. Saturated dense materials still have higher strength than open graded mixtures.
- 4.9 The nature of the fines in the mix is of particular importance, crushed hard rock is helpful for stability and stiffness, limestone appears to have some self-cementing capability. Over compaction and heavy site traffic can lead to lower permeability and greater risk of saturation.
- 4.10 Asphalt loses strength when wet but regains it when it dries out provided that there is adequate resistance to water of the bitumen/aggregate interface. There are a number of tests for this property. However studies have found no clear correlation exists between these and in-service performance.
- 4.11 There is a 'pessimum' where water can enter voids but not escape easily; most mixtures fall within this range, of between 5% and 10% voids. This means that a material can stay wet for a considerable period, possibly in excess of 20 hours after rainfall ceases and be at risk of strength loss and stripping of binder from the aggregate. Testing the complete asphalt mix has proved to be difficult worldwide The UK SATS test in Clause 953 is probably as good as any. ADEPT guidance states that this test should be used if the aggregate source is unknown.
- 4.12 The increase in strength as a binder ages can be offset by the water present in pavement.

- 4.13 The strength loss of asphalt if it freezes whilst saturated was found to be significant and not likely to be reversed as it is an indication of mix disruption. This may also affect the aggregate within the mix, especially if the temperature falls low enough so that salted surfaces freeze.
- 4.14 Surface scour is widely evident where vehicles drive over a wet surface for a period of time. All surfacing materials are prone to this. Longitudinal and transverse gradients and location of gullies should be checked to ensure adequate surface drainage.

5. Recommendations to improve the asset at low cost

- 5.1 The specification could be amended to specify a) testing the strength parameters of sub base when saturated and possibly with the addition of salt, to ensure strength is adequate and, b) the measurement of the horizontal permeability of sub base to help ensure saturation does not occur or ensure it will drain quickly as water subsides. Parameters should be selected to maximize both strength and permeability.
- 5.2 Recommendations to use a Geotextile separation layer, to prevent penetration of soft subgrade into sub base layers during construction or subsequently, should be considered. For example on subgrade values of $\leq 3\%$ CBR.
- 5.3 Running traffic on a saturated sub base could be avoided by closing the carriageway until draining after saturation has occurred. The time this takes will be dependent upon the sub base permeability. It may be possibly assessed using a Falling Weight Deflectometer or traffic speed deflectometer comparatively between the high and low sides of a carriageway
- 5.4 In areas already identified as flood risk in HADDMS, low lying carriageways should be highlighted and a review carried out of the drainage design. Detailing should include sealing the verge and central reservation (topsoil and grass should be adequate) to take account of and prevent saturation occurring. False cuttings may be required to prevent the carriageway flooding as it crosses a river flood plain.
- 5.5 Surfacing pavement materials could take account of flooding risk and materials that are durable and not subject to surface scour selected. Testing of water sensitivity of new aggregate sources is essential.
- 5.6 Keeping the road surface sealed is an important focus for engineers on roads known to be frequently wet from river or sea water and in the case of evolved roads to maintain the granular fabric of the road structure dry. Treatments include patching, if necessary, and surface dressing on existing roads, an increased bond coat beneath surface courses and an impermeable Stone Mastic Asphalt binder course on new surfacing.
- 5.7 Dense Stone Mastic Asphalt surface course, with measures taken to prevent rutting and cracking if necessary, and gritted to provide adequate early life skid resistance would appear to give the best durability. These are not currently BBA HAPAS approved so cannot meet SHW Clause 942 but could be specified using EN 13108-5 with any additional requirements in Appendix 7/1 as a departure. These might be given as the preferred option for flood risk areas in national guidance.

6. Further research

- 6.1 This study was limited in scope in that only one Highways Agency site could be evaluated for the effects of a flood on in-service performance and only one combination of asphalt and sub base tested in the laboratory, although by confirming the work of others demonstrated its validity.
- 6.2 Research into the moisture sensitivity of the common UK sub bases materials should be carried out and especially the effect of water on the strength parameters.
- 6.3 Further work is necessary to evaluate and specify appropriate factors for sub base permeability in conjunction with strength testing.
- 6.4 There is some evidence that there may be an increasing number of asphalt surfacing materials failing prematurely due to rainfall or moisture sensitivity related issues. There are also important issues that warrant further research to relate void content and its degree of interconnection in relation to the effect of rainfall on mix durability as a consequence of water being retained within the voids for significant periods of time. Further research is necessary on this topic on UK materials and linked to in service performance.
- 6.5 TRACS data on the other Highways Agency sites that have been subject to inundation should be checked in conjunction with knowledge of the pavement surface type and the nature of the surface water drainage at the location.
- 6.6 Low lying carriageways in high flood risk areas should be identified and reviewed for the drainage and pavement design.
- 6.7 A check should be carried out if the sub base beneath a carriageway becomes saturated when the next motorway flood event occurs and the speed with which it drains after saturation. This may be carried out for example by identifying a flood prone section and installing some sub base drainage flow monitoring in advance.

Appendix A Test method used for Lightweight Falling Weight Deflectometer

Clause 887AK Materials approval using Portable Dynamic Plate Test

- 1 The material shall be compacted into a box of minimum dimensions: 610mm x 610mm x 420mm deep, with a Vibrotamper minimum mass exceeding 60kg in at least 3 layers with 8 passes on every point in each layer.
- 2 The moisture content shall satisfy the requirements for the particular material being approved.
- 3 The density shall be checked with a calibrated nuclear density gauge in direct transmission mode to ensure a minimum compaction of 95% of wet density achieved in BS 1377 Part 4 Method 3.7.
- 4 The materials shall be tested using the Dynamic Plate Test in accordance with Clause 889AK.
- 5 The mean stiffness modulus shall satisfy the requirements of the relevant Clause for the material.

889AK Dynamic Plate Test

1. Dynamic Plate Tests shall be carried out using equipment which has been properly calibrated to manufacturer’s specification and subject to a validation check prior to use.
- 2 The equipment shall be capable of delivering a total load pulse of peak magnitude 6-8kN, of total duration 15-40 milliseconds, to a rigid circular plate of 300mm diameter. Both the applied load and the transient deflection shall be measured.
- 3 The dynamic modulus shall be determined at each point tested using the following formula:-

$$\text{Dynamic Modulus, } E_{vd} \text{ (MPa)} = \frac{P(1-v^2)}{0.3y}$$

Where: P is the peak applied load (kN)
y is the peak deflection (mm)

v is the Poisson’s Ratio; a value of 0.35 shall be used in the absence of any other data

- 4 The stiffness modulus shall be obtained using the following formula
Stiffness Modulus, G (MPa) = $E_{vd} / 0.6$.

Note This formula relates to the German Dynamic Plate; other manufacturers’ plates may use different factors to obtain stiffness modulus from Dynamic Modulus. This factor has been obtained empirically.

- 5 The full technical specification and method of use of the Dynamic Plate Test apparatus is published by the German Federal Ministry of Transport, Road Construction Department in TP BF-StB Part B 8.3, 1992 (In German).