

Flushing in chipseals

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Abbreviations and acronyms

AADT	average annual daily traffic
AAV	aggregate abrasion value
ALD	average least dimension
ANOVA	analysis of variance
HCV	heavy commercial vehicle
IRI	International Roughness Index
MD	Micro-Deval test
NZTA P17	<i>NZTA P17: October 2012: Performance based specification for reseals</i>
OMD	Opus modified Micro-Deval test
PSV	polished stone value
PSV test	<i>BS EN 1097-8: 2009: Tests for mechanical and physical properties of aggregates. Determination of the polished stone value</i>
RAMM	NZ Transport Agency Road Assessment and Maintenance Management database
Transport Agency	New Zealand Transport Agency
vpd	vehicles per day

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Executive summary

Flushing is the process whereby chipseal texture depth is lost over time, resulting in a loss of skid resistance. It is the single most important reason for resealing on New Zealand state highways.

This report details research carried out from 2012 to 2015. In the first part of the work the aim was to identify and investigate the physical mechanisms causing flushing. The aim of the second part of the project, undertaken by researchers at the University of Auckland, was to use pavement condition data to develop a model to predict the rate of flushing progression in chipseals.

Physical mechanisms

Key physical mechanisms investigated were:

- 1 Chip reorientation, compaction of the seal layer and loss of 'trapped' air voids in the body of the seal (as distinct from the surface air voids)
- 2 Excess binder application
- 3 Abrasion of the seal aggregate to produce fines that increase the volume of the bitumen mastic and reduce the size of the sealing chip
- 4 Water-induced migration of bitumen to the surface through the formation of 'blisters' also known as 'volcanoes'
- 5 Embedment of the sealing into the basecourse or substrate (this could include underlying asphalt patches)
- 6 Low binder viscosity resulting from excess kerosene remaining in the bitumen
- 7 Thermal expansion of the bitumen volume.

A variety of laboratory studies and field site sampling and assessments were undertaken. Flushing is a complex process involving multiple physical mechanisms that may be operating simultaneously and contributing to the loss of surface texture in different proportions (which may also change over time). Overall conclusions from the work were:

Factors making a major contribution to flushing:

- Aggregate abrasion and breakdown leading to a reduction in the size of the sealing chip and the build-up of fines in the seal void volume.
- Compaction and reorientation of the seal layer under traffic reducing the available void volume in the seal layer.
- Water venting and sub-surface stripping in seal layers due to water trapped at the seal-basecourse interface and probably principally arising by ingress through the seal surface.

Factors having no or making only a minor contribution to flushing:

- Thermal expansion of the bitumen (the effect is minor).
- Excess bitumen application. Use of very high, non-standard bitumen application rates will obviously act to fill seal void volume, but this appears to be the exception rather than the rule.
- Binder viscosity. The binder viscosity (at least over the range of standard sealing grades), will have a major effect on bleeding and tracking of bitumen but does not affect the rate of seal texture loss.

Factors with insufficient information to draw conclusions:

- Embedment into the basecourse. It is recognised that embedment occurs but very little quantitative work on this mechanism has been carried out, and the extent or variability of the effect is unknown.

Modelling flushing

A two-part model using parameters in the NZ Transport Agency's Long-Term Pavement Performance (LTPP) database was developed. The first part uses a logistic model to predict the onset of flushing. The model was found to have an accuracy of 74% when used to predict the initiation of flushing on a separate data set.

The second part uses a linear regression model to predict the rate of flushing progression and was modelled with variations for first-coat seals, and second and higher generation seals.

The linear model was statistically strong (R^2 of 0.445 for first-coat seals and 0.628 for second and higher generation seals). The developed linear model was tested using a separate set of LTPP data and the model predictions revealed that the developed model was robust at predicting the progression of flushing.

Recommendations

Physical mechanisms

The work presented in this report illustrates the fact that there is no single simple method to prevent flushing in chipseals; a number of approaches are needed:

- 1 A test based on the Micro-Deval (MD) test should be included in the *NZTA M/6 2011: Specification for sealing chip* to control aggregate breakdown based on aggregate source.
- 2 The permeability/ drainage of the basecourse to water, to prevent build-up of water at the base of the seal needs to be improved. This may require a rethink of the M/4 grading envelope or simply a more rigorous enforcement of construction requirements.
- 3 The reasons why seals 'leak' need to be urgently addressed. Is it inherent in the technology or can construction practices be changed to minimise leakage?
- 4 Use of highly polymer or crumb rubber, modified binders or epoxy bitumen (or similar very high strength, thermosetting polymer modified bitumen) is required to minimise chip reorientation and seal layer compaction. Recent research has shown that compared with conventional binders, thermosetting epoxy binders may have sufficient strength to resist chip embedment and reorientation. Reactive epoxy binders may also react with the aggregate surface and resist sub-surface stripping.
- 5 More seal void volume should be built in through appropriate seal design, minimising bitumen application and maximising development of a stone-on-stone skeleton (mimicking open-graded porous asphalt or stone mastic asphalt mixes). There is a limit to what can be achieved here as the seal must also, as far as possible, attempt to waterproof the basecourse and retain chip under traffic stresses.

Modelling flushing

Data from accelerated pavement deterioration studies using different seal types should be used to investigate the accuracy of the model and improve its performance.

It is recommended that the items of pavement data currently collected as part of the LTPP programme be extended to include data relating to the soil moisture environment of a pavement, particularly the dry density, wet density and water content of soils. The soil moisture data items were identified as an

important predictor of flushing from previous flushing modelling attempts but were unable to be included in the modelling work presented here. Having soil moisture data available from future LTPP surveys will likely be very useful in the development of further distress prediction models.

Abstract

This report details research carried out from 2012 to 2015 into chipseal flushing. The physical mechanisms causing flushing were investigated and a model was developed to predict the growth of flushing over the New Zealand state highway network.

Factors making a major contribution to flushing are:

- aggregate abrasion and breakdown
- compaction and reorientation of the seal layer under traffic
- water venting and sub-surface stripping in seal layers.

Factors having no or making only a minor contribution to flushing are:

- thermal expansion of the bitumen
- excess bitumen application
- binder viscosity.

Further work is needed to quantify the significance of chip embedment into the basecourse.

A two-part model using parameters in the NZ Transport Agency Long-Term Pavement Performance database was developed. The first part uses a logistic model to predict the onset of flushing and an accuracy of 74% when used to predict the initiation of flushing on a separate data set.

The second part uses a linear model to predict the rate of flushing progression. First-coat seals, and second and higher generation seals were modelled separately.

The linear model was statistically strong (R^2 of 0.445 for first-coat seals and 0.628 for second and higher generation seals).

1 Introduction

Chipseal flushing is the process whereby seal texture depth is lost over time (figure 1.1), resulting in a loss of skid resistance. In the worst (and not uncommon) case, bitumen completely covers the surfacing aggregate. Flushing is an international problem (Jackson et al 1990; Bahia et al 2008; Gransberg and James 2005; Lawson and Senadheera 2009) and the single most important reason given for resealing on New Zealand state highways, accounting for almost 30% of lane kilometres in 2010/11 (Towler et al 2010 ARRB paper).

Flushing is distinct from 'bleeding' and 'bitumen tracking' although both of these phenomena can, and are more likely, to occur in flushed seals. Bleeding occurs typically in hot weather when the bitumen viscosity is very low. Bitumen can run off into gutters due to the road camber or easily adhere to tyres resulting in tracking along the surface (as can be seen in figure 1.1).

Previously the NZ Transport Agency (the Transport Agency) defined a seal as flushed when the texture depth was $<0.7\text{mm}$ (for posted limits of $<70\text{km/h-1}$) or $<0.9\text{mm}$ (for posted speed limits of $>70\text{km/h-1}$). Some local authorities specified a seal with texture depth of $<0.5\text{mm}$ as flushed. (Transit New Zealand et al 2005). Currently the Transport Agency recommends that a seal is defined as 'fully flushed' when both the skid resistance as measured by the equilibrium SCRIM coefficient (ESC) is ≤ 0.35 , and the mean profile depth is $\leq 0.7\text{ mm}$ or the sensor measured texture depth (SMTD) is $\leq 0.4\text{ mm}$ (Donbavand et al 2011; Whitehead et al 2011).

This research falls into two distinct parts. In the first part of the work the goal was to identify and investigate the physical mechanisms causing flushing. The aim of the second part of the project undertaken by researchers at the University of Auckland (chapter 3), was to use pavement condition data to develop a model to predict the rate of flushing progression in chipseals.

Figure 1.1 Seal with flushed patches and showing bitumen tracking in left wheel path

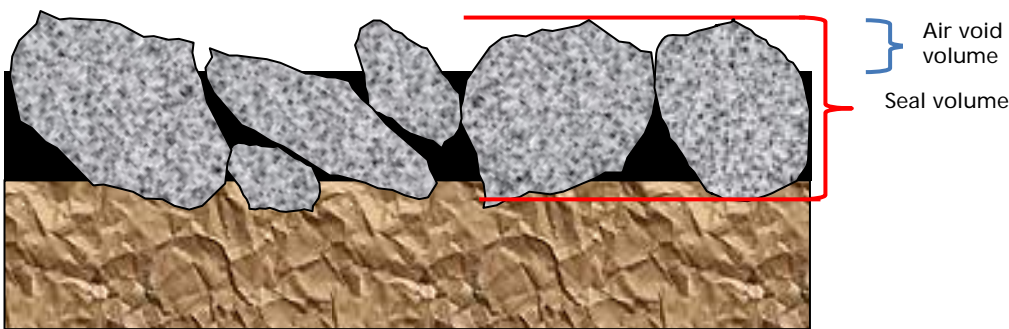


2 Physical mechanisms causing flushing

Chipseal flushing can be understood in terms of the composition of the seal layer volume (shown schematically in figure 2.1).

The seal layer volume is defined as the volume contained within a layer defined by the top and bottom points of the sealing chip. The height of this layer is generally somewhat greater than the chip average least dimension (ALD). This seal volume is composed of air voids, sealing chip, bitumen and basecourse material (due to chip embedment). Flushing results from any process that gives rise to a gradual loss of the surface air void volume (texture).

Figure 2.1 Chipseal layer volumes



It is important to note that, in theory, seals in New Zealand are designed to result in bitumen occupying no more than 60% to 70% of the seal void volume (Transit New Zealand et al 2005). Even in the hypothetical case that the chip becomes fully reoriented under trafficking so the seal layer is the ALD thickness, a properly designed seal should still not be in a flushed condition.

A range of potential causes for flushing in chipseals has been presented in the technical literature and suggested by practitioners (Alderson and Oliver 2008; Alderson 2008; Holtrop 2011; Patrick 2009b; Transit New Zealand et al 2005). In some cases flushing is described as due to 'binder rise' (or similar terms which describe the seal condition) without any explanation of why the binder should 'rise'. Suggested causes for flushing related to a physical mechanism or condition can be summarised as follows:

- chip reorientation, compaction of the seal layer and loss of trapped air voids in the body of the seal (as distinct from the surface air voids)
- excess binder application
- abrasion of the seal aggregate to produce fines that increase the volume of the bitumen mastic and reduce the size of the sealing chip
- water-induced migration of bitumen to the surface through the formation of 'blisters' also known as 'volcanoes'
- embedment of the sealing into the basecourse or substrate (this could include underlying asphalt patches)
- low binder viscosity resulting from excess kerosene remaining in the bitumen
- thermal expansion of the bitumen volume.

These mechanisms, earlier relevant literature and work undertaken in the current project are discussed in detail in the following sections.

2.1 Reorientation and compaction of the seal layer

Trafficking of a seal surface reorients the chips to lie with their ALD closer to the perpendicular with respect to the surface, with a resultant loss of surface air voids. Potter and Church (1976) show that the chip layer thickness reaches about 1.25 times the ALD. Similar results were obtained by Gaughan and Jordan (1994); Dickinson (1990); Alderson (2002).

The rate of air void volume loss for a range of seals as a function of traffic level was found to the form of equation 2.1 (Patrick and Donbavand 1996; Transit New Zealand et al 2005). Texture depth was expressed as V_v/ALD to allow for comparison seals with different chip grades:

$$V_v/ALD = A - B \log(e_{lv_{tot}}) \quad (\text{Equation 2.1})$$

Where: V_v = volume of surface air voids ($\text{mm}^3 \text{mm}^{-2}$)

$e_{lv_{tot}}$ = cumulative equivalent light vehicle number (assuming one heavy commercial vehicle = 10 cars)

ALD = chip average least dimension (mm)

A, B = constants

Field texture loss data may also be affected by other flushing processes so the rate of texture loss due to reorientation alone is not known.

2.1.1 Layer instability

Gradual reorientation of the chips will tend to produce a more compact seal layer, although this packing may not represent the closest possible packing of the chip. If other processes are ignored (eg basecourse embedment, chip breakdown) then the rate of reorientation should eventually slow and a 'stable state' with the chip fixed in position, achieved (unless the traffic stresses imposed are markedly changed). It may be hypothesised that if this stable state is not reached before the next seal is applied, then the bitumen application rate needed for that seal may be effectively higher than it should be as the substrate chip may continue to reorient. Ultimately compaction and shearing of multi-layer seals built up in this way will lead to the excess bitumen being forced to the surface resulting in loss of texture. This concept of 'seal instability', was introduced in the 1990s by researchers in Hawke's Bay (Harrow 2008; Gray and Hart 2003) to account for observations of multi-layer seals in which subsequent layers had ever shorter lives before flushing.

Layer instability is defined as: 'inadequacies in the structural performance of surface layers that are often associated with the build-up of multiple seal layers until the combined thickness is greater than 40mm with an excess of binder' (Transit New Zealand et al 2005). The concept is that flushing is primarily a result of excess application of bitumen. The resulting seal layer is 'soft', allowing increasingly rapid texture loss through shearing of the layer and embedment of the topmost sealing chip.

Not all multi-layer seals, however, demonstrate layer instability and the reasons for this were investigated by Ball and Patrick (2005). The researchers performed a range of tests on samples taken from sites with multiple seal coats both with premature flushing (12 sites) and those with standard lifetimes (five sites). Simple ball penetrometer measurements of surface hardness were made but no relationship was found between these results and the tendency of seals to flush prematurely. The shear strength of a selection of surfaces was measured by a laboratory test that dynamically loaded (constant stress) core sample surfaces with a circular ram (50mm diameter). The embedment of the ram as the sample sheared was recorded as a

function of the number of impacts. A standard test temperature of 45°C was adopted. The change in embedment level of the ram (in mm) between 300 and 3,000 impact pulses was used to compare rates of compaction.

The wheel path specimens showed no obvious relationship between rate of embedment and the likelihood of premature flushing, but interpretation of the results is complicated by the fact that the sites had had different levels of trafficking and compaction before testing. More significant was the finding that the between wheel path results (untrafficked seal), showed the rate of embedment of the prematurely flushed sites was 1.6–3.4 times that for the normal life seals. Also for sites where premature flushing had occurred, the rate of embedment outside the wheel path was significantly greater than that occurring within the wheel path. For sites without evidence of premature flushing the wheel track and outside wheel tracks rates were similar.

The results indicate the multi-layer seals demonstrating premature flushing were more prone to compaction and shearing than others but this was not clearly correlated with the binder content. There was also no clear pattern in terms of seal design of the layers. Differences in compaction and shearing were possibly related to the packing of and freedom to reorient the aggregate in the layer which might have related to the timing of the seal layers and the traffic levels of the different sites.

2.1.2 Air voids in the body of the seal layer

Further work on rates of seal compaction and shearing was undertaken as described in Koddipily et al (2014), see also appendix C. Cores were examined by x-ray tomography to determine the percentage of air voids present. Measurements taken before and after wheel tracking show that air voids trapped in the body of multi-layer seals are lost as the seal compacts. The variability in air void percentages found in different seal core samples may help explain why some seals are more prone to layer instability than others.

2.1.3 The significance of seal compaction as a cause of flushing

The potential for seal compaction to result in flushing was further investigated by Ball and Patrick (2005). Experiments were conducted to determine the hypothetical minimum void volume available after complete reorientation and compaction of a multi-layer seal and this was compared to volume of bitumen likely to be present.

Minimum void volume layers were achieved by hand packing (by two different operators), of aggregate particles on their ALD. Various grades were layered by hand in a high walled container following various typical chip sequences. Each subsequent layer of chips was gently rolled with a rubber-covered hand roller to encourage movement of chips into the underlying voids.

The following seal combinations were studied:

- 1 Single-coat grade 2 (grade 2 surface)
- 2 Grade 4 over grade 2 (grades 2/4 surface)
- 3 Grade 3 over grade 4 over grade 2 (grades 2/4/3 surface)
- 4 Grade 5 over grade 3 over grade 4 over grade 2 (grades 2/4/3/5 surface)
- 5 Grade 2 over grade 2 (grades 2/2 surface)
- 6 Grade 3 over grade 3 over grade 2 over grade 3 (grades 3/2/3/3 surface).

The volume of void space in the seal was calculated by filling the packed chip layers with water.

Theoretical bitumen application rates for the separate seal layers were calculated using the equations given in *Chipsealing in New Zealand* (Transit New Zealand et al 2005, equations 9–11, p339), for a traffic volume of 1,000 vehicles/lane/day ($v/l/d$) and 11% heavy vehicles, which reduce to:

$$V_b = (ALD + 0.7T_d)(0.291 - 0.25\log_{10}(2.0 \times v/l/d \times 100)) = 0.166(ALD + 0.7T_d) \quad (\text{Equation 2.2})$$

Where V_b is the volume of binder (Lm^{-2}), T_d is the substrate texture depth (mm) and $v/l/d$ is the number of vehicles per lane per day. Spray rates were calculated for an existing texture depth $T_d = 0.5\text{mm}$ (ie assuming the substrate seal had flushed). Based on work by Potter and Church (1976), it was assumed that 30% of the total voids in the first layer were taken up with basecourse due to embedment. The results from Ball and Patrick (2005) are shown in table 2.1 with additional calculations taken from Herrington et al (2012).

Table 2.1 Summary of seal void data

Chip grades	2	2/4	2/4/3	2/4/3/5	2/2	3/2/3/3
Surface thickness mm	13.0	22.0	33.0	33.9	26.0	42.0
Total voids L/m ²	6.5	10.9	16.0	13.6	12.3	16.7
Percent voids L/m ²	46.0	47.8	46.9	39.5	48.0	39.5
Available voids L/m ²	4.5	8.9	14.0	11.6	10.3	15.0
Percent voids allowing for basecourse embedment L/m ²	31.8	39.0	41.0	33.7	40.2	35.5
Calculated total spray rate L/m ²	2.13	3.60	5.40	6.34	4.26	7.53
Percent voids allowing for basecourse embedment and applied bitumen L/m ²	16.8	23.2	25.2	15.3	23.5	17.7
Available voids/calculated total spray rate	2.1	2.5	2.6	1.8	2.4	2.0

Source: Ball and Patrick (2005); Herrington et al (2012)

The total voids range from 39.5% to 48% of the total layer volume, similar to the 39.2% to 45.6% range reported by Potter and Church (1976) and the 45% reported by Dickinson (1990) using similar experimental methods to Ball and Patrick (2005) but in both cases for single coat seals. Values in the last row of table 2.1 show available voids are typically about twice the volume of bitumen hypothetically required to construct the seals.

If a higher value of T_d of 1.5mm is used in the calculation with Ball and Patrick's aggregate ALD data, then the volume of bitumen required increases by only about 3% to 6% and the volume of voids present is still more than sufficient to accommodate the binder. In practice of course, the seal aggregate packing is likely to be far less efficient than in these experiments and can potentially range up to nearly 50%, the approximate void volume obtained for a loosely poured single size stone (Towler and Dawson 2008). Based on these findings in order to fill available void volume with bitumen (and give rise to a flushed seal) more than twice the normal application rates would be required, which seems an unlikely scenario.

2.1.4 Conclusion

Compaction and shearing occurs in some multi-layer seals. The extent and rate of compaction may relate to the extent of reorientation that takes place before the next seal is applied and the quantity of air voids 'built in', ie sealing over a highly textured surface is likely to result in the formation of more air voids and the underlying seal will also be more prone to continued chip reorientation, compaction and shearing.

In general terms, however, even a 'fully' compacted seal should still possess sufficient void volume to accommodate all the bitumen likely to have been used in its construction. Simple compaction of the seal layer is, in itself alone, not sufficient to cause flushing.

2.2 Excess bitumen application

Excess bitumen application in a chipseal could arise, for example, from resealing of a badly stripped seal, operator error or a deliberate strategy of using high application rates to minimise the risk of early chip loss. The work of Ball and Patrick (2005) discussed in section 2.1.3, however, indicates that bitumen application rates of more than twice standard rates would be required to fill the available voids, even in a fully compacted seal.

2.2.1 Measurement of bitumen/stone ratios in flushed seals

To investigate the possibility that flushing had arisen from excess application of bitumen, the bitumen/stone mass ratios of the cores analysed in section 2.2.1 were measured and compared to those expected from calculations using standard, bitumen and chip application rates.

The bitumen/ stone mass ratios for the 64 cores analysed in the present study are shown in figure 2.2. Most cores had ratios between 7% and 12% with mean of 10.3%. Additional data collected in Hawke's Bay for over 80 cores taken from flushed sites over the 2004 to 2008 period show that the mean bitumen/ stone ratio was 9.8 % (std dev 1.3) (Jones 2012). The latter figures agree well with the results in figure 2.2.

To understand the significance of the results in figure 2.2 two simulations were carried out in which calculations of bitumen/stone mass ratios were made for a hypothetical sequence of seals (table 2.3).

An estimation of the seal texture depth after a variable period of time was made using the seal design equations in *NZTA P17: 2012 Performance based specification for reseals* (P17).

Initially it was assumed that a grade 2 chip (12mm ALD) was applied to a surface with a sand circle test value of 300mm (ie 0.64mm texture depth), as a first-coat seal. This gave a bitumen application rate for 1000v/d of traffic of 1.97Lm^{-2} . The volume of aggregate in each subsequent seal was calculated using a ratio of 800/ALD and converted to mass by assuming bulk density of $1,300\text{kgm}^{-3}$. The simulation assumed that the grade 2 seal was resealed after one year. The texture depth from the P17 equations was estimated and converted to a sand circle of 137mm and the calculations repeated. The total binder and aggregate masses were then used to calculate the bitumen/stone ratio.

Figure 2.2 Bitumen/stone mass ratios

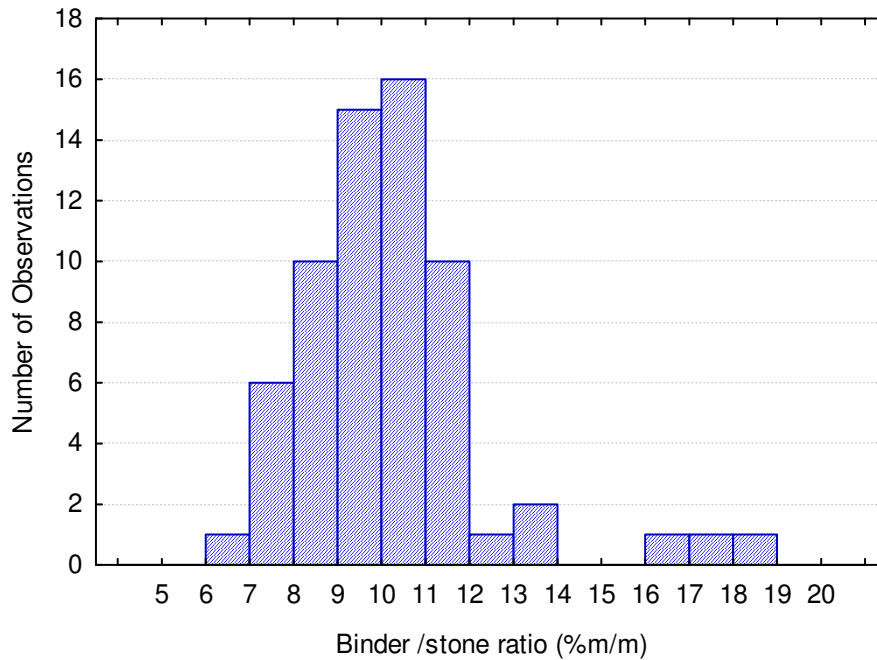


Table 2.2 Binder/stone mass ratio calculations (traffic = 1,000 v/l/d, elv = 2,000 v/l/d)

ALD (mm)	Sand circle (mm)	Texture depth (mm)	Bitumen application rate (Lm ⁻²)	Age (year)	Volume of aggregate (m ³)	Weight of aggregate (kg)	Total bitumen (kg)	Cumulative weight of aggregate (kg)	Total bitumen/stone mass ratio
Repeated grade 2 seals									
12	300	0.64	1.97	1	0.015	19.5	1.972318	19.5	10.1
12	137	3.05	2.24	1	0.015	19.5	4.213737	39.0	10.8
12	143	2.80	2.21	1	0.015	19.5	6.425304	58.5	11.0
Grade 2/4/3/5 sequence of seals									
12	300	0.64	1.97	10	0.015	19.5	1.972318	19.5	10.1
7	161	2.21	1.36	5	0.007	9.1	3.328183	28.6	11.6
10	215	1.24	1.72	10	0.013	16.25	5.050327	44.9	11.3
5	180	1.76	0.99	1	0.006	6.5	6.039439	53.0	11.4

Table 2.3 Binder/stone mass ratio calculations using bitumen application rates 25% above design (traffic = 1,000 v/l/d, elv = 2000 v/l/d)

ALD (mm)	Sand circle (mm)	Texture depth (mm)	Bitumen application rate (Lm ⁻²)	Age (year)	Volume of aggregate (m ³)	Weight of aggregate (kg)	Total bitumen (kg)	Cumulative weight of aggregate (kg)	Total bitumen/stone mass ratio
Repeated grade 2 seals									
12	300	0.64	2.47	1	0.015	19.5	2.47	19.5	12.6
12	149	2.58	2.73	1	0.015	19.5	5.20	39	13.3
12	158	2.30	2.70	1	0.015	19.5	7.90	58.5	13.5

ALD (mm)	Sand circle (mm)	Texture depth (mm)	Bitumen application rate (Lm ⁻²)	Age (year)	Volume of aggregate (m ³)	Weight of aggregate (kg)	Total bitumen (kg)	Cumulative weight of aggregate (kg)	Total bitumen/stone mass ratio
Grade 2/4/3/5 sequence of seals									
12	300	0.64	2.47	10	0.015	19.5	2.47	19.5	12.6
7	182	1.73	1.63	5	0.007	9.1	4.09	28.6	14.3
10	243	0.97	2.12	10	0.0125	16.25	6.21	44.85	13.8
5	204	1.38	1.18	1	0.005	6.5	7.39	51.35	14.4

It can be seen in the simulation that the bitumen/stone ratio for this extreme case is approximately 11%. When the calculation was repeated for a grade 2/4/3/5 sequence, the binder/stone ratio was approximately 11.5%. These values are consistent with the data in figure 2.2 (note that the calculation of aggregate mass is conservative (lower than may be occurring in the field) which results in higher calculated binder/stone ratios). If the binder application rates assumed in this study were increased by 25% then the ratios increase to around 13% to 14% (table 2.3). Clearly to obtain the very high ratios seen in a few cases in figure 2.2 then application rates well in excess of the design values would be required.

Except in a few extreme cases the quantity of bitumen present in the cores analysed is consistent with expected (standard) application rates and the conclusion drawn from the analysis in table 2.1, that the void volume available in the seal layer should easily accommodate the bitumen (though not necessarily the volume of bitumen *and* fines) likely to be present appears valid.

2.2.2 Bitumen/stone mass ratios and chipseal layer instability

High binder/stone mass ratios in multiple chipseal layers have been found to relate to 'layer instability', a condition characterised by shortened seal lives from flushing (see section 2.2.1). *Chipsealing in New Zealand* (Transit New Zealand et al 2005), notes the following ranges and expected performance based on extensive testing in the Hawke's Bay region (table 2.4).

Table 2.4 Bitumen/stone ratio effects

Bitumen/stone ratio (m/m)	Expected performance
<12%	Low risk of instability
12-15%	Possible instability. Treatment by water cutting can be effective
15-20%	High risk. Sandwich seal can be considered
>20%	Whole layer needs treatment

Source: Transit New Zealand et al (2005, pp178-179).

Binder/stone mass ratios for the flushed cores analysed in section 2.2.1 had ratios between 7% and 12% with a mean of 10.3%; only relatively few had the high ratios quoted in table 2.4. This was unexpected as the cores analysed were all clearly flushed. The methodology used to obtain the bitumen/stone ratios in table 2.4 was not provided but it is now understood that because of the extraction method used in the original derivation of the figures, the sub 2.36mm aggregate fraction was not separated from the bitumen fraction and the mass was counted as bitumen (Jones 2012). In the present study, the fraction passing the 2.36mm sieve comprised on average about 10% by mass of the aggregate fraction, counting this as part of the bitumen phase increases the mean binder stone ratio to about 22%, which is more consistent with the ranges in the table.

Additionally data collected in Hawke's Bay for flushed seal cores taken during the 1999–2008 period, had a mean binder/stone ratio ranging from 19.7% to 22.8% m/m when the 2.36mm aggregate fraction was included as binder, but 9.1% to 11.1% m/m when it was counted as aggregate (Jones 2011). The latter figures are also consistent with the data in table 2.4.

2.2.3 Comparison of actual bitumen application rates to those stated in RAMM

To further investigate potential over application of bitumen it would be useful to undertake case studies to compare the actual bitumen application rates in flushed seals to those that should have been applied following the standard seal design equations. This would be a large exercise and was beyond the scope of the present work.

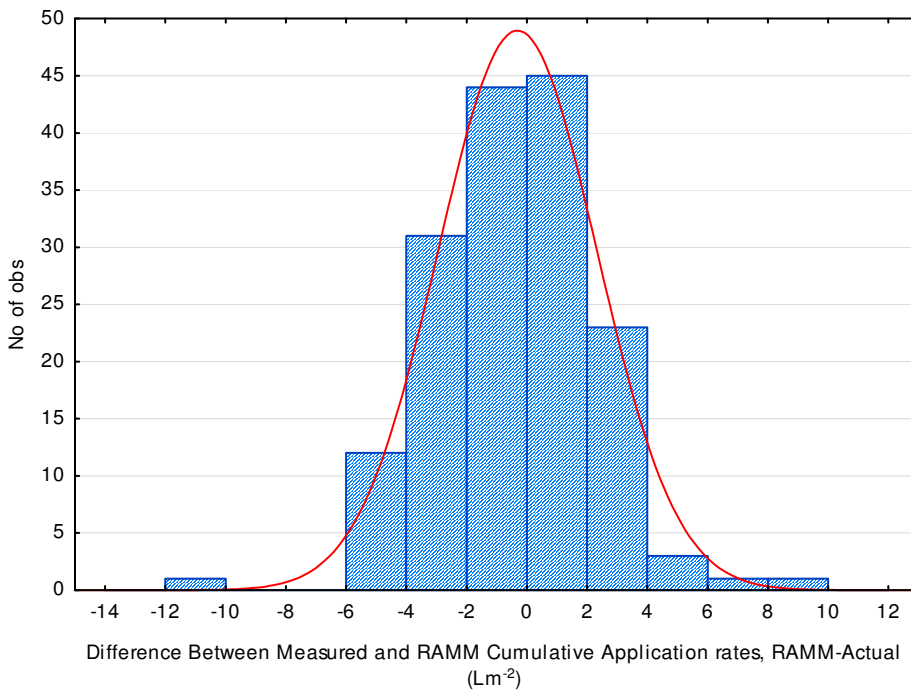
Instead a comparison was made of actual application rates and those for the same sites given in the NZ Transport Agency Road Assessment and Maintenance Management database (RAMM). A number of sites were selected and the assumptions made that *the seal designs for these sites were largely in accordance with the standard equations* given in *Chipsealing in New Zealand* (Transit New Zealand et al 2005), and the data had been correctly entered into RAMM (unfortunately not a trivial assumption). The measured application rates were then compared to the RAMM data to highlight the presence of excess bitumen.

A total of 161, 150mm diameter cores taken from seals on SH3, SH4, SH41 and SH43 (Wanganui–Taranaki) were extracted by the continuous centrifuge method described in section 2.3.2 and the bitumen contents measured. Essentially all aggregate particles down to sub-micron sizes are removed from the bitumen by this method. Seals ranged from one to seven layers deep with an average of three layers and with measured bitumen contents averaging 5.7Lm^{-2} . The cores were collected by the Transport Agency for other purposes and the opportunity was taken to make use of them in this project. None of the cores were heavily flushed, most showed some spot flushing but with generally good texture.

The measured cumulative bitumen application rates (calculated from the area of the core and assuming a bitumen density of 1) were compared with those calculated from the cumulative application rates given in RAMM for the core locations. For each site the difference between the RAMM data and actual application rate was calculated (see figure 2.3).

The differences show an approximately normal distribution around zero which suggests the actual and design application rates given in RAMM are overall the same. A *t*-test comparing the mean difference for all cores to zero showed the difference to be not statistically significant at the confidence 95% level ($p = 0.07$).

Figure 2.3 Comparison of actual and RAMM cumulative application rates



2.2.4 Conclusion

Bitumen application rates are not generally in excess of that expected from standard seal design and are in most cases probably not responsible for flushing. The bitumen/stone mass ratios listed in NZ Transport Agency (2005) are calculated by including the sub-2.36mm aggregate mass in the bitumen fraction, so are much higher than the true values, which even in flushed seals are as expected from standard design. The relationship found between increasing, bitumen (plus sub-2.36mm fines)/stone ratio, and layer instability and likelihood of flushing, as given in NZ Transport Agency (2005), appears to be due to increasing levels of fine aggregate in the bitumen mastic rather than excess bitumen (see section 2.3).

2.3 Aggregate abrasion and breakdown

2.3.1 Reduction in chip size

Abrasion, polishing and breakdown of the sealing chip under traffic will reduce the size of the sealing chip and hence the available surface texture. An example of breakdown material from a chipseal core constructed from grades 2, 3 and 5 chip only is presented in figure 2.4. As shown in the following sections, an approximately continuous grading of material is observed.

Figure 2.4 Example of aggregate breakdown material from a multi-layer chipseal core constructed from grade 2, 3 and 5 chip only



Aggregate breakdown has been identified as a possible cause of flushing in Australian seals (Alderson 2008). Paige-Green (2001; 2004) in South Africa studied the crushing and breakdown of low crushing strength aggregates in artificial seals under a laboratory wheel tracker and heavy vehicle simulator. Damage to the chip occurred largely within the first 16,000 passes of the vehicle simulator and was related to tyre inflation pressure rather than total load. Aggregate size reduction through traffic wear has long been included as a factor in seal design in South Africa (Marais 1969). The *NZTA M/6 2011: Specification for sealing chip* (NZTA M/6) has a general aggregate strength requirements but no specific test measurement to properly control weathering and traffic abrasion (see section 2.3.5 for further discussion).

2.3.2 Aggregate fines build-up

In addition to a reduction in size of the sealing chip, large amounts of wear material from aggregate abrasion has been found to be retained in the seal, forming a mastic with the bitumen.

Sixty-four seal cores from eight different flushed state highway sites around the country were analysed to determine the volume of aggregate and bitumen present (the volume of surface air voids was negligible). Only seals with grade 5 or larger chip were examined to avoid possible confusion between generated fine particles and material that may have been added as part of the sealing aggregate itself. The bottom of the (150–200mm diameter) cores were cut with a concrete saw to remove any basecourse aggregate (this did not introduce any significant error to the fines content measurement – see appendix A). The resulting cores were typically 30–110mm thick and usually composed of three to four seal layers. The cores were extracted with trichloroethane using an automatic extraction system so that even very fine aggregate particles (passing the 75 micron sieve) are separated from the bitumen and collected by centrifugation. Further details of the experimental procedure have been published separately (Herrington et al 2012).

All specimens showed the presence of significant volumes of fine aggregate passing the 4.75mm sieve (mean value 15%), which in theory should not have been present at all (figure 2.5).

Figure 2.5 Seal core volume of aggregate passing the 4.75mm sieve

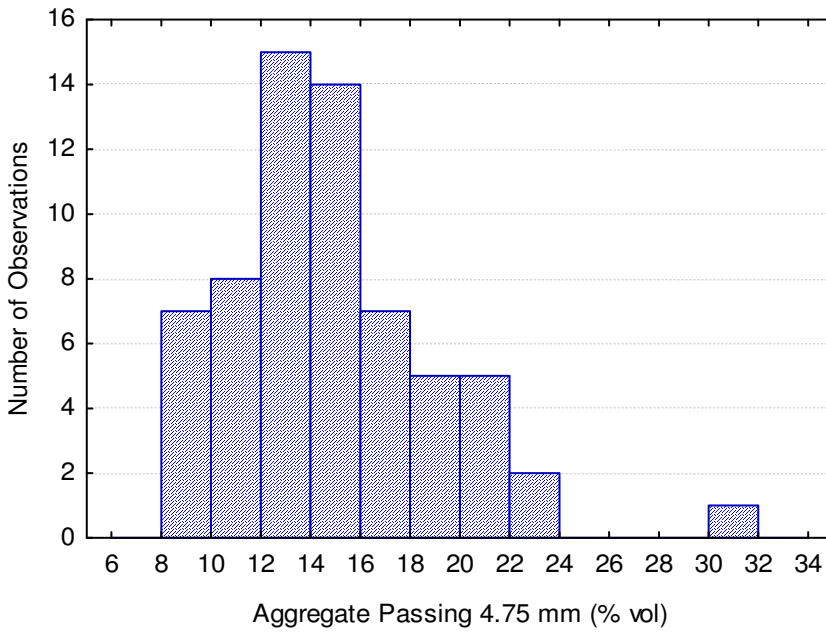
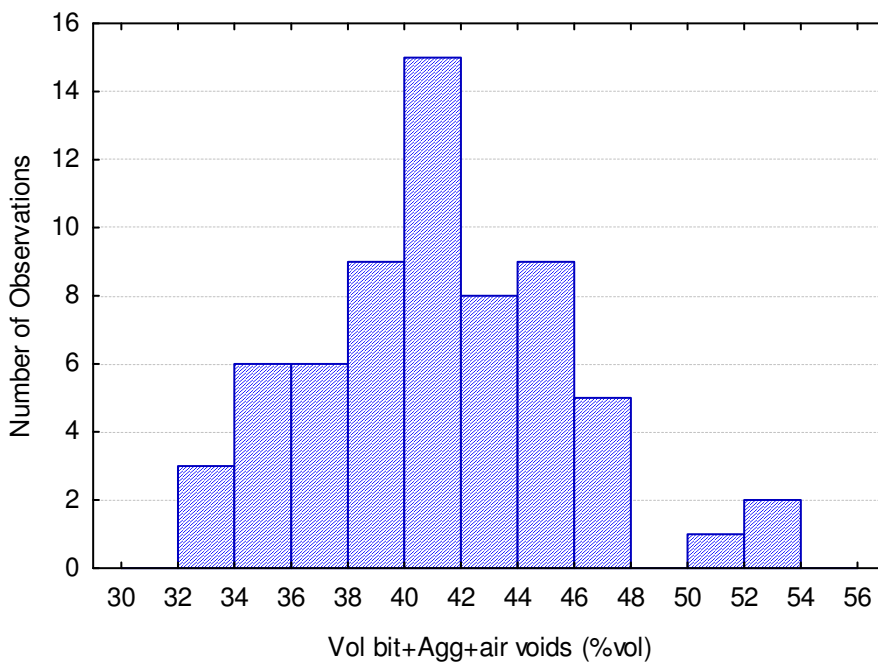


Figure 2.6 Seal core volume of aggregate fines (passing the 4.75mm sieve) +bitumen +air voids



Two samples were examined petrographically to confirm the fine materials were indeed from the sealing aggregate and not from windblown detritus or tyre rubber etc. Depending on the local environment, however, the contribution of windblown material in some sites may be significant. The proportion of non-aggregate derived fine material was very low in the two specimens examined (2.4% and 0.5%). The volume of fine aggregate plus bitumen and body air voids, ranged from 33% to 53%, with a mean value of 41.3% (figure 2.6).

The values obtained agree with the void calculations in table 2.1 which predict that for a seal to be flushed (ie all available void volume occupied) then the volume of volume of fine aggregate plus bitumen and air voids must exceed 30% to 40% of the total void volume. It is important to emphasise that the absolute mass or volume of fine aggregate material is not a simple indicator of the likelihood for flushing as this will depend on the total void volume in a given seal (which may vary widely).

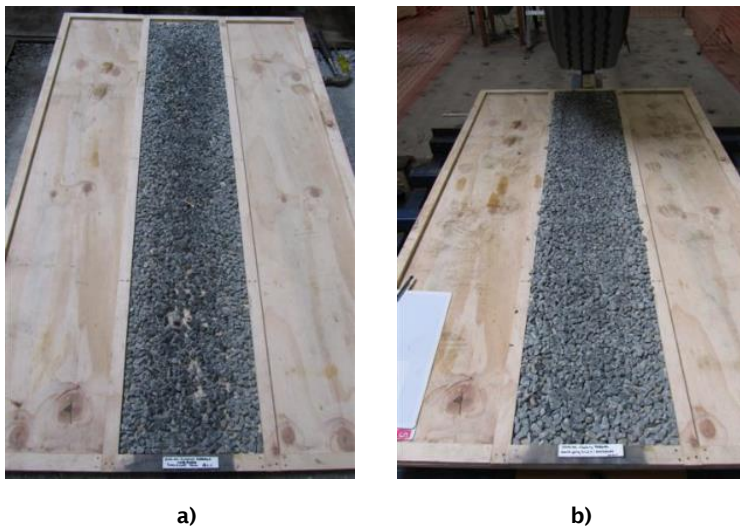
The results strongly suggest retention of aggregate in the seal is a major contributor to flushing and aspects of the process are discussed further in the following sections.

2.3.3 Effect of over-chipping on fines generation

Over-chipping during construction may potentially accelerate aggregate breakdown through increased stone-on-stone abrasion caused by trafficking.

To explore the potential for over-chipping to contribute to fines generation, a simple experiment was conducted using a wheel tracking device based on a truck tyre (Firestone type 11R22.5), loaded to 20kN (2 tonnes). Details of the machine are given in Herrington et al (2012). The device tracks the wheel on a fixed path (in one direction only) at about 1.6km/h⁻¹. A grade 3/2 mixture of greywacke chip was placed on a plywood base, either as a single layer with gaps or as a two to three chip deep layer, to represent an over-chipped seal as shown in figures 2.7a and b respectively.

Figure 2.7 Single layer a) and multiple layer b) chip beds after rolling



In the case of the over-chipped 'seal' the aggregate was redistributed three to four times over the course of the experiment to simulate chip movement likely under actual trafficking. After 12,600 passes the aggregate was swept from the base and sieved. The results are compared with those of the original untrafficked aggregate in table 2.5.

Table 2.5 Aggregate grading after 12,600 wheel passes

Sieve size (mm)	Percent passing (by weight)		
	Original aggregate	Single layer trafficked	Over- chipped layer trafficked
13.2	84.5	88.4	82.9
9.5	3.5	5.4	4.4
6.7	0.1	0.5	0.8
4.75	0.1	0.4	0.7

The percentage of material passing the 4.75mm sieve has increased markedly in the trafficked seals compared with the original aggregate. The material passing from the untrafficked aggregate is essentially a fine dust whereas that from the trafficked layers contained a significant quantity of millimetre scale particles.

The results suggested that over-chipping may have an effect on fines generation and to investigate this effect further, two existing chipseal sites (Clintons Road and Telegraph Road), on local roads near Christchurch where extremes of high and low chipping levels had been used during construction were sampled. The gradings of heavily chipped sections were compared with those of sections with very low chip application rates. The sites were constructed as part of an earlier trial conducted on early seal failures through chip loss (Waters 2011).

The seals at both sites were a single coat grade 3, constructed in each case using the same chip and binder in February 2007 (table 2.6); full details of the sites are given by Waters (2011). Photographs of the surfaces from Clintons Road are shown in figure 2.8. Neither surface (nor the Telegraph Road sites) was flushed; the patches of bitumen visible in the low application rate site surface in figure 2.8a are exposed patches of bitumen resulting from chip loss.

Figure 2.8 Clintons Road seal surfaces resulting from different chip application rates



a) Very low application rate

b) Very high application rate

Table 2.6 Seal sites for examining the effect of over-chipping on aggregate breakdown (based on data provided in Waters 2011)

Site	AADT ^(a)	Seal	Binder application rate (Lm ⁻²)	Chip source	Relative number of chips per unit area	
					Low chipped area	High chipped area
Clintons Road	126 ^(b)	Grade 3 single coat	2.8	Pound Road quarry	1	1.6
Telegraph Road	646 ^(b)	Grade 3 single coat	2.7	Pound Road quarry	1	1.2

^(a) Annual average daily traffic; ^(b) Negligible numbers of heavy vehicles.

Four cores from each area of each site were taken in December 2012 approximately five years after construction. The total seal layer at each site was about 20–25mm thick (two seal layers) including some adhered basecourse. The specimens were cut and the top 15mm of seal analysed (table 2.7).

Table 2.7 Effect of over chipping on fines generation

Sieve size (mm)	Clintons Road (% mass passing)					Telegraph Road (% mass passing)				
	High application		Low application		Difference (high-low)	High application		Low application		Difference (high-low)
	Mean ^(a)	95% CI	Mean ^(a)	95% CI		Mean ^(a)	95% CI	Mean ^(a)	95% CI	
13.2	95.6	0.6	95.5	2.6	0.1	96.5	1.8	97.5	1.5	-1.0
9.5	33.8	1.4	34.9	3.0	-1.1	41.3	3.3	50.0	5.3	-8.7
6.7	12.8	2.8	11.1	1.1	1.7	26.2	1.9	25.4	3.1	0.8
4.75	8.9	2.6	7.4	0.8	1.5	14.0	0.8	13.2	0.8	0.8
2.36	6.3	2.2	4.9	0.5	1.4	6.7	0.3	6.9	0.3	-0.3
1.18	5.1	1.6	3.8	0.4	1.3	4.7	0.3	5.0	0.3	-0.3
0.6	4.5	1.3	3.3	0.4	1.2	3.8	0.2	4.1	0.3	-0.3
0.3	3.6	0.8	2.9	0.3	0.7	3.1	0.1	3.4	0.2	-0.3
0.15	2.8	0.4	2.4	0.2	0.4	2.5	0.1	2.8	0.2	-0.2
0.075	2.2	0.2	2.0	0.2	0.1	2.0	0.1	2.2	0.2	-0.2

^(a) Mean of 4 replicates in each case

The gradings of seals at both sites do not show any significant change in grading between the low and high chipped areas. There is a small but consistent increase in fines for the over-chipped site at Clintons Road compared with that of the low application rate, but the differences are comparable to the variation in the core replicates. For each site the mean % mass passing the 2.36mm sieve of the four replicate cores for high and low chip application areas were compared using a *t*-test and did not show any significant difference at the 95% confidence level. The gradings are compared in figures 2.9 and 2.10.

The fines present in the seals are a combination of those present in the underlying seal (although this effect was minimised by only extracting the top 15mm of seal) and those generated by traffic over the five years from construction, so it is possible that small differences due to chip application levels may have been masked. The level of over-chipping was also substantially less than that in the laboratory experiment discussed above but at the upper end of that which would occur in practice (Waters 2011). In practice also traffic will rapidly distribute loose chip outside of the wheel paths in contrast with the laboratory experiment where the chip was constrained.

Figure 2.9 Aggregate gradings Clintons Road

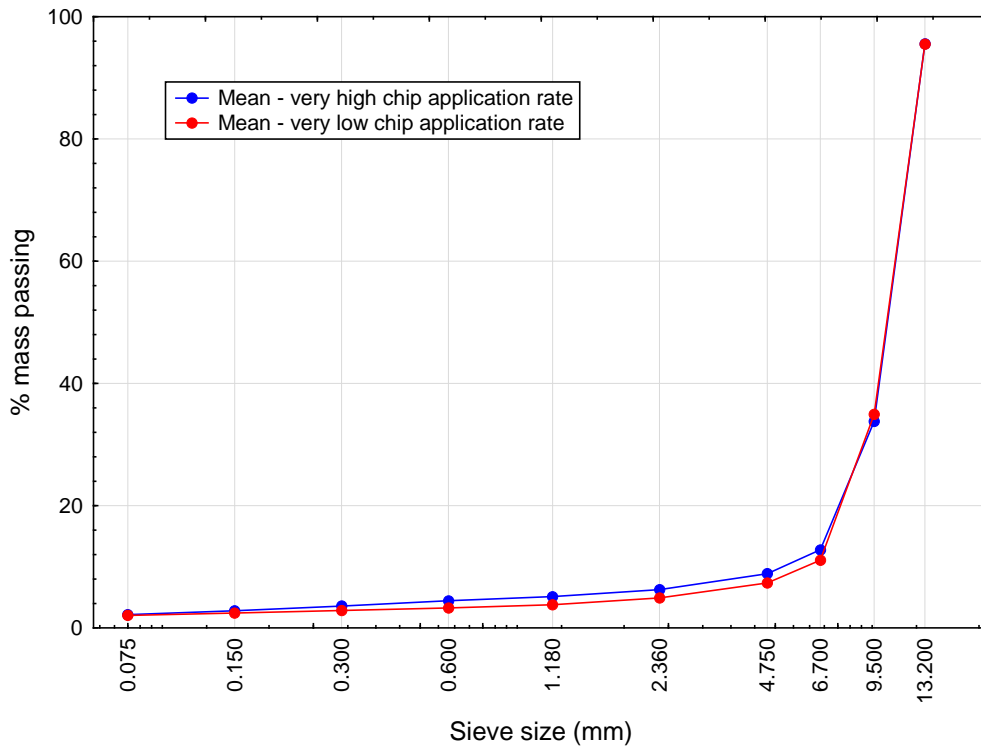
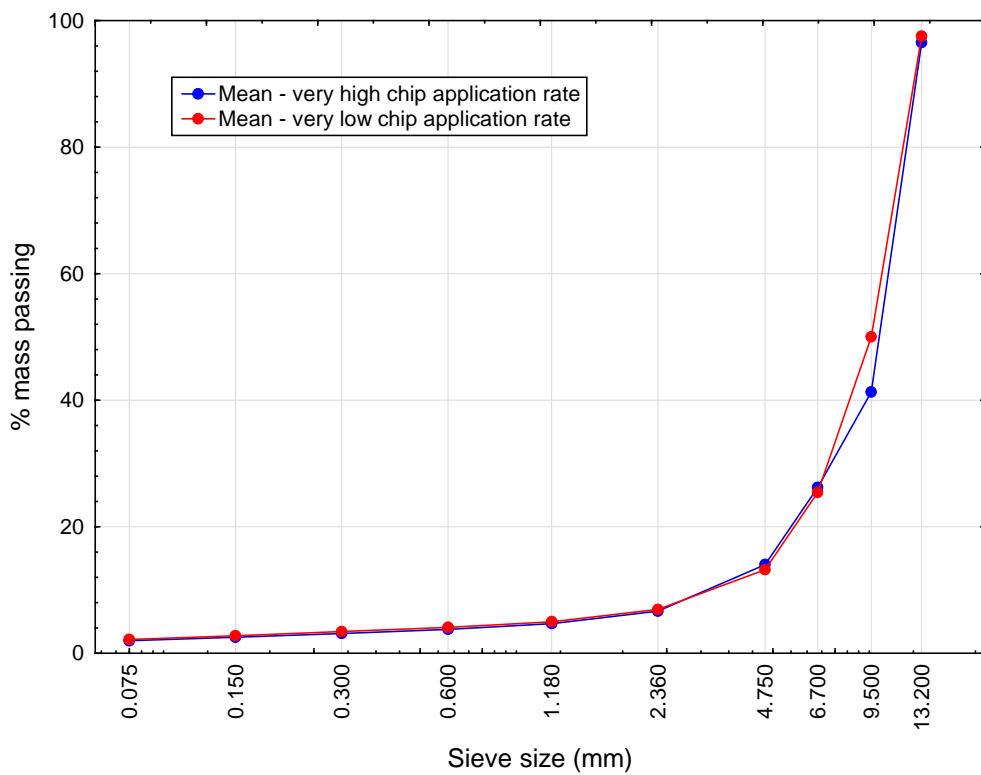


Figure 2.10 Aggregate gradings Telegraph Road



2.3.4 Effect of seal type on fines generation

Seal design may also potentially contribute to differences in the rate of chip breakdown arising from the extent of chip to chip contact. It was not possible within the current project to fully explore this possibility but two adjacent seals constructed within a few days of each other in March 2009 on SH2 at Featherston (AADT 9148, 8% HCV) were examined.

The seals were a racked-in grade 3/5 and a two-coat grade 3/5 (2 years 8 months old at the time of sampling), constructed by the same contractor over the same existing seal using the same aggregate and carrying the same traffic. A racked-in seal consists of a single application of bitumen followed by a grade 3 chip spaced to allow the second application of smaller chip to fit between. The two-coat seal consists of a first bitumen application followed by a grade 3 chip, a second bitumen application and a grade 5 chip. The surfaces are shown in figure 2.11.

Four cores were taken from the outer wheel-path of the two-coat seal and six from the same wheel path of the racked-in seal. The cores were taken from sections of the seals with visually uniform texture (not flushed) and at a spacing of 300–500mm. Only the top 15mm of each core was extracted to minimise any effects due to fines present in the underlying seals. The grading results are shown in figure 2.12 and table 2.8.

Figure 2.11 SH2 Featherston two-coat and racked-in seals at the time of sampling



a) Two-coat seal



b) Racked-in seal

Figure 2.12 Aggregate gradings SH2 Featherston site

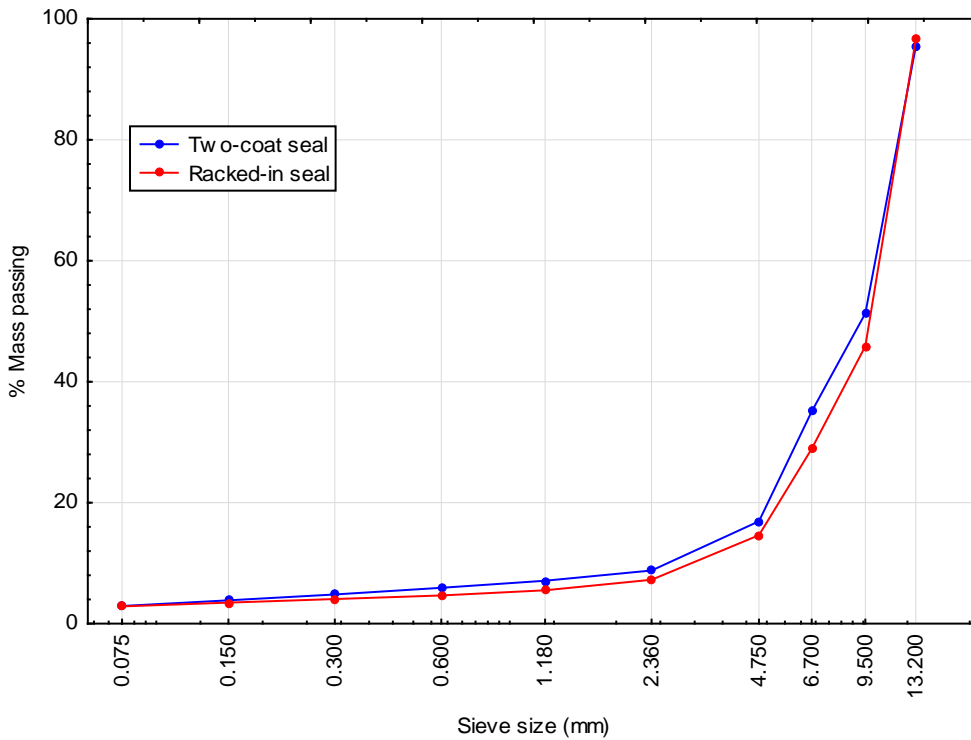


Table 2.8 Effect of seal type on fines generation

Sieve size (mm)	Racked- in seal (RIS) (% mass passing)		Two- coat seal (TCS) (% mass passing)		Difference: RIS-TCS
	Mean	95% CI	Mean	95% CI	
13.2	95.4	3.7	96.6	2.8	-1.2
9.5	51.3	6.4	45.9	2.2	5.4
6.7	35.1	4.4	28.9	3.9	6.2
4.75	16.9	2.6	14.6	1.1	2.3
2.36	8.8	1.2	7.3	0.6	1.5
1.18	7.1	1.2	5.5	0.7	1.6
0.6	5.9	1.1	4.7	0.7	1.3
0.3	4.8	0.9	4.1	0.6	0.8
0.15	3.9	0.7	3.5	0.6	0.4
0.075	2.9	0.6	2.9	0.5	0.1

Substantial differences were found between the two sites for the larger sieve sizes though this may just reflect the differing and unknown proportions of grade 3 to grade 5 chip used in the two seals.

The NZTA M/6 specification for sealing chip allows grade 5 chip to contain up to 2% of material passing the 2.36mm sieve but zero passing the 0.3mm sieve. To eliminate potential differences between the sites arising from differing quantities of grade 5 chip being applied during construction, the fines levels were compared in terms of material passing the 0.3mm sieve. The mean percentages passing the 0.3mm sieve were compared by *t*-test and found not to be significantly different at the 95% level indicating that

aggregate breakdown is not affected by seal design, at least for two-coat and raked-in seals. The results suggest that differences between other seal types are also likely to be insignificant.

2.3.5 Assessment of aggregates for fines generation

Probably the major variable affecting fines generation is the aggregate mineralogy. Aggregates from different quarries or even different faces within a single quarry may have different abrasion characteristics.

The physical properties of aggregates used in chipseal surfacings are controlled by the NZTA M/6 specification. Specified test properties broadly related to abrasion and resistance to attrition are:

- *NZS 4407 1991 Test 3.10: The crushing resistance test*
- *AS 1141.32-1995: Methods for sampling and testing aggregates – weak particles (including clay lumps, soft and friable particles) in coarse aggregates*
- *NZS 4407 1991 Test 3.11: Weathering quality index test*
- *BS EN 1097-8 2009: Tests for mechanical and physical properties of aggregates. Determination of the polished stone value (PSV test)*

However none of these procedures are primarily designed to control or test for the resistance of sealing aggregate to the abrasion and breakdown that occurs during handling (chip to chip abrasion) or under trafficking (tyre-chip abrasion):

- The crushing resistance test is known as a ‘constrained comminution’ test. The particles are largely constrained and cannot move freely. The number of contacts each particle has determines the resulting tensile stress in the particle. The results of such tests depend on the particle size (McDowell et al 2003). This test approach simulates the stresses imposed on aggregates used in basecourse (more so than sealing) during compaction and afterwards from traffic stresses where the particles are confined and there is no direct contact with traffic.
- The weak particle test is essentially qualitative and is limited to detection of only very weak particles (the test involves testing the strength of particles by hand).
- The weathering test accounts for the action of water on the aggregate properties and measures the resistance to breakdown under a steel roller but without chip to chip abrasion and the loading process is not very repeatable.
- The PSV test measures abrasion under trafficking to some degree but the procedure is designed (in particular in the way the test aggregates are carefully positioned to avoid macro-texture effects) to specifically measure polishing of the aggregate micro-texture, not the more severe damage and changes to the chip size and shape that can occur through abrasion in the field.

2.3.5.1 Opus modified Micro- Deval (OMD) test

The MD test is a procedure that is finding increasing application overseas in assessing the abrasion resistance of aggregates used in asphalt mix and basecourse. This test was investigated as a means of comparing fines generation from aggregates produced at different quarries. The background and application of the MD test is discussed in appendix B.

The general principal is that the test aggregate is soaked in water for a period then rotated in a drum with water and 5kg of 9.5mm steel balls for two hours at 100rpm. The fine material produced is weighed and the loss from the test sample expressed as percentage of the initial sample weight. There are two principle methods in use (US and European methods) and numerous variants of these (see appendix B).

Preliminary experiments with the ASTM method (D6928), showed that qualitatively the procedure resulted in largely fine (<1.0mm) material in contrast to the high proportion of larger 2–4mm fragments observed in material extracted from seal cores. For this reason the procedure was modified as follows:

Grade 3 aggregate (300g, 100% passing the 13.2mm and retained on the 9.5mm sieve), was soaked for one hour at $20 \pm 5^{\circ}\text{C}$. The sample was drained and placed in a steel drum with 25 9.5mm steel balls, one 53.9mm steel ball and 0.39L of water. The drum was sealed and rotated at 100rpm for 27 minutes and the material recovered, dried and sieved. The drum used was 198mm id x 173.5mm internal height (meeting the ASTM D6928 specification), but to increase the severity of the test the drum was modified by adding four, 20mm high protrusions to the inside of the drum (as shown in figure 2.13), to catch the aggregate and balls so that they fell to the bottom of the drum on each rotation.

Figure 2.13 OMD test drum. The four metal protrusions on the inside of the drum catch the steel balls and aggregate and allow them to drop to the bottom of the drum increasing the amount of breakdown



2.3.5.2 OMD trial test results for various aggregates

The OMD test was carried out in duplicate or triplicate on four randomly selected, different grade 3 sealing chip aggregates from quarries in both the North and South Islands and the mean grading curves in each case calculated. The purpose of the testing was to establish if any significant differences existed in aggregate breakdown between the sources and if further investigation was warranted. Three of the aggregates (A, B & C), were described by the suppliers as 'greywacke' and the other (D), as a greywacke with quartz and schist present and all met the NZTA M/6 specification requirements.

The results, shown in figure 2.14, show a substantial variation in the formation of breakdown material amongst the four aggregates. Figure 2.15 shows the results overlaid with gradings from three of the flushed field cores used in the analysis in section 2.3.2. The field cores were constructed from different grade 3 (and 3/5) aggregates so a direct comparison with the OMD tests results is not possible, but the gradings show a similarly wide range giving some confidence that the test conditions are a reasonable simulation of field conditions.

Figure 2.14 OMD test results for four different aggregate sources

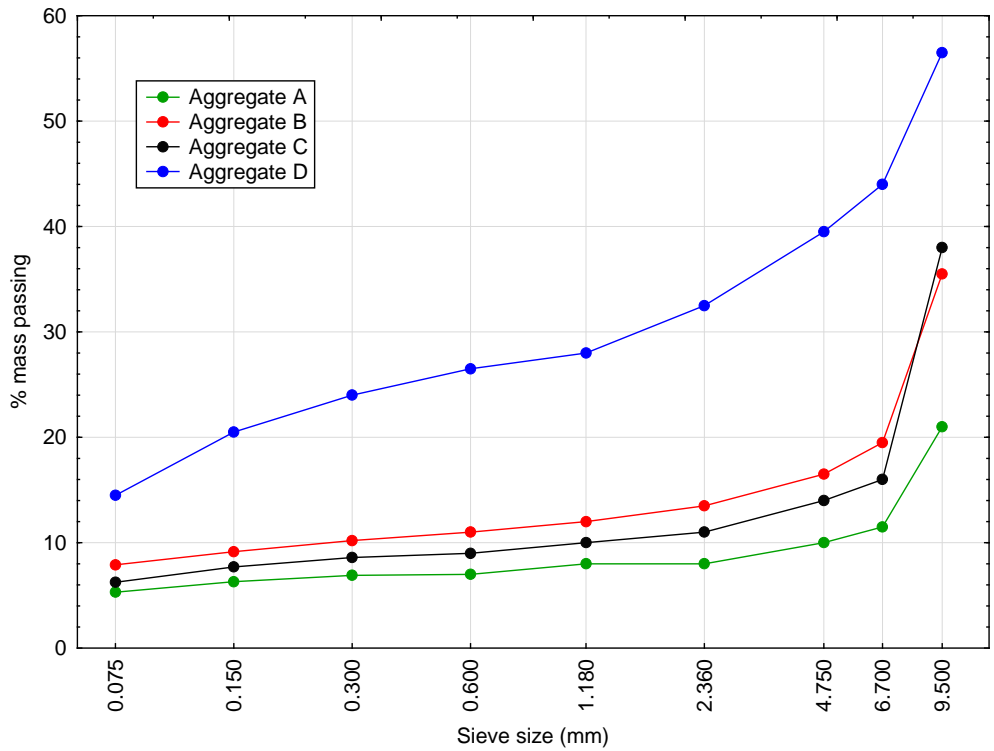
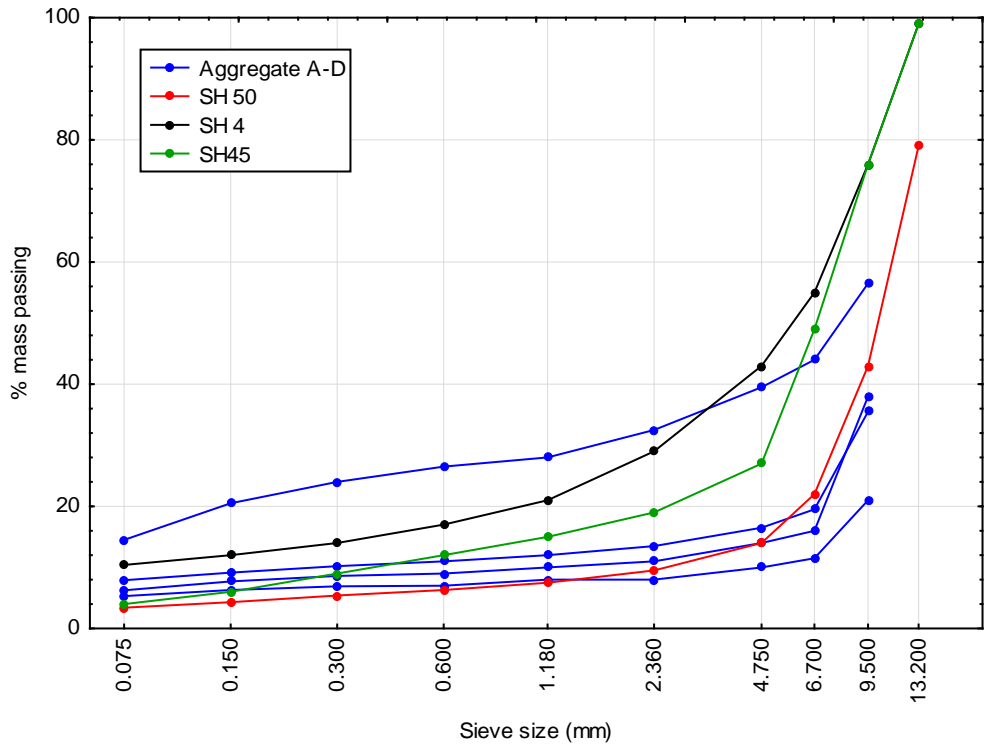


Figure 2.15 OMD test results for four different aggregate sources compared with field core gradings



2.3.5.3 OMD test verification with field data

To be practically useful the OMD (or any other abrasion test), must be verified against field performance. Rankings of aggregates in terms of the extent of breakdown in the test procedure should also apply to breakdown in the field (other factors such as traffic being equal). A proper verification of the OMD test was beyond the scope of the current project but an initial comparison to field performance was undertaken. Existing, approximately comparable sites were identified where aggregates A and D had been used in construction of seals, one on SH8 and the other on SH5, see table 2.9 for details.

Table 2.9 Site details for seals used to verify predictions of the OMD test

Site	AADT	Seal	Constructed	First sample	Second sample
SH5 (near Rotorua) RP 5/77/13.50	4,470 (10.1% heavy vehicles)	Grade 2/4 two-coat, 130/150 binder	October 2013	April 2014	April 2015
SH8 (near Cromwell) RP 8/297/13.17	3,961 (10.2% heavy vehicles)	Grade 3 single coat, 130/150 emulsion with 4% styrene-butadiene rubber polymer	November 2012	April 2014	April 2015

The seals had very similar traffic levels but differed in design (probably not significant in light of the results presented above), the age at first sampling and that a polymer modified binder had been used at the SH8 site. Possibly because of the polymer modified binder a significant amount of larger chip had been lost from this site at the time of first sampling.

OMD test results for the two aggregates, showing the repeatability of the test (two different operators) and the significant difference in breakdown behaviour, are presented in table 2.10.

Table 2.10 Gradings for aggregates A and D after the OMD test, seven replicate tests were carried out in each case

Sieve size (mm)	Aggregate A (% mass passing)		Aggregate D (% mass passing)		Difference: D-A
	Mean	95% CI	Mean	95% CI	
9.5	53.4	6	32.3	5	21.1
6.7	42.1	7	18.4	3	23.7
4.75	37.7	6	15.7	3	22.0
2.36	31.6	5	13.1	2	18.4
1.18	27.6	4	11.9	2	15.7
0.6	25.7	4	11.0	2	14.7
0.3	23.9	3	10.4	1	13.5
0.15	20.6	2	9.4	1	11.2
0.075	15.0	2	7.9	1	7.1

Cores were taken from the wheel tracks of the field sites and the gradings for the SH5 samples were determined using the extraction method given in section 2.3.2. The SH8 samples contained styrene-butadiene rubber polymer that was not properly extracted using trichloroethane in our continuous extraction system. The large aggregate (>2.36mm) was recovered using that system, but the fine aggregate and undissolved polymer with some residual bitumen was further extracted manually by stirring in hot (30–40°C), dichloromethane for two to three hours. After settling, the solution was carefully decanted off and passed through a grade 4 glass sinter funnel. The process was repeated at least three

times (until the dry fine aggregate showed no evidence of cohesion). The clean aggregate was dried and added back to the large aggregate fraction for grading.

Photographs of cores from the sites are shown in figure 2.16. The samples were first taken in April 2014 and again in April 2015 and the gradings compared to try and determine the relative extent of chip breakdown over that period. This relatively short time frame was dictated by the project timetable. Aggregate gradings for both sites are shown in table 2.11, initially and after one year of trafficking.

Differences in the gradings are small but do indicate an increase in the overall level of fines although only some increases are statistically significant, more so for aggregate D, which is consistent with the predictions of the OMD test. The negative change in grading at the larger sieve sizes for aggregate D probably reflect continued chip loss as evident in figure 2.16. Overall the changes after only one year are too small to draw conclusions about the predictive ability of the test.

Figure 2.16 Wheel track cores from SH8 (left) and SH5 (right), sites used to verify the OMD test, note the chip loss from the SH8 core



Table 2.11 Aggregate gradings from SH5 and SH8 aggregate breakdown sites. Statistically significant differences are highlighted in red, eight cores were analysed in each case

Sieve size (mm)	SH5 (Rotorua) (% mass passing)					SH8 (Cromwell) (% mass passing)				
	Initial		One year		Difference (one year - initial)	Initial		One year		Difference (one year - initial)
	Mean	95% CI	Mean	95% CI		Mean	95% CI	Mean	95% CI	
9.5	43.4	3.1	45.6	4.7	2.2	50.2	3.6	50.0	2.5	-0.2
6.7	24.9	0.8	27.4	1.6	2.4	28.2	1.9	26.1	1.4	-2.1
4.75	14.9	0.7	16.2	1.1	1.3	18.4	1.2	17.0	4.1	-1.4
2.36	8.2	0.9	12.0	1.7	3.8	9.5	0.7	11.1	0.6	1.6
1.18	5.8	0.8	7.8	4.5	2.0	6.2	0.6	8.2	0.8	2.0
0.6	4.4	0.7	6.1	1.1	1.8	4.6	0.5	6.7	0.9	2.0
0.3	3.3	0.7	4.8	1.1	1.4	3.6	0.5	5.6	1.0	2.0
0.15	2.6	0.7	3.6	1.1	1.0	2.6	0.5	4.5	1.1	1.9
0.075	1.9	0.6	2.6	1.1	0.7	1.8	0.4	3.4	1.2	1.6

2.3.6 Conclusion

Aggregate breakdown contributes to flushing by both reducing the size of the sealing chip but also through the build-up of aggregate fines in the bitumen mastic. Aggregate fines rather than excess bitumen application are responsible for the filling of available void volume in the seal leading to flushing.

Although only a few sites were able to be examined in the current work, there does not appear to be any major effect on aggregate breakdown from over-chipping during construction nor from differences in seal design.

Significant differences in rates of aggregate breakdown in the laboratory using a variant of the MD test, however, were observed due to the aggregate mineralogy (quarry source). An attempt to validate the test method used against aggregate breakdown rates in the field was inconclusive due to problems with chip loss from one site and the short time frame available for the study. A more extensive study to validate the test procedures is required.

2.4 Water-induced flushing

Water has long been implicated as a potential cause of flushing (Major 1972), most obviously through the formation of blisters or 'volcanoes' caused by venting of water vapour (figures 2.17 to 2.23). In hot weather bitumen blisters form, which under the correct conditions can later be found to contain liquid water.

Venting of water in this way is extremely common and is in fact almost always observed where flushing has occurred. The blisters may burst or remain intact on cooling to form a hollow shell (figure 2.19) or collapse to form a 'blob' of bitumen sometimes referred to as 'spot flushing'. In trafficked areas the blisters are rapidly flattened, but their position is apparent as vent holes, sometimes several millimetres in diameter (figure 2.20).

As well as appearing in the wheel tracks, blisters can be commonly observed on the road shoulder and on areas of seal that have little or no traffic (eg figure 2.21). Venting has also been observed on artificial seals constructed on steel plates and exposed outdoors 200mm above the ground (figures 2.22 and 2.23).

Figure 2.17 Bitumen blisters forming in hot weather (Melbourne, Australia)



Figure 2.18 Bitumen blisters (SH77 Darfield)



Figure 2.19 Blisters after cooling. The central blister has had the top removed to show the hollow interior (SH75 Banks Peninsular)



Figure 2.20 Water vapour vent holes in a flushed wheel track (SH2 Featherston)



Figure 2.21 Blisters forming on the road shoulder as well as in the wheel path (SH75 Banks Peninsula)



Figure 2.22 Bitumen blisters formed on an artificial seal constructed on a steel plate and exposed outdoors



Figure 2.23 Bitumen blisters formed on an artificial seal constructed on a steel plate. The base plate is visible on the bottom left where earlier a sample had been removed



2.4.1 Blister formation

Blisters are not formed due to volatiles from the bitumen or from kerosene added as cutter (typically 1–3pph but up to 6–7pph). As kerosene and bitumen are completely miscible the vapour pressure of the mixture depends on the individual vapour pressures and proportion of each component (Raoult's Law). Bubbles will only form if the mixture vapour pressure is equal to atmospheric pressure. At the low concentrations of kerosene used, the mixture vapour pressure is almost entirely due to the bitumen which has a negligible vapour pressure at road surface temperatures (theoretical boiling point of well over 500°C at atmospheric

pressure). This is confirmed from experience in heating cutback bitumens in the laboratory and in bitumen sprayers where bubbles do not form. Kerosene is lost from the bitumen through diffusion to the surface and evaporation.

An early hypothesis to explain the formation of bitumen blisters was that they resulted from vaporization of moisture in the basecourse in hot weather (when the bitumen viscosity is also low). This mechanism was investigated by Ball et al (1999). Calculations showed that mean water vapour pressure at 54–60°C (near the upper seal temperature in summer) was about 17kPa. If a 0.05mm average diameter capillary channel was assumed to exist between the seal chips then the flow rate of bitumen through this channel caused by the water vapour pressure, would be more than sufficient to fill a typical surface void volume over the course of a year.

2.4.1.1 Bitumen film rupture pressure

To confirm the above calculations a simple experiment was conducted using the apparatus shown in figure 2.24. An approximately 3.5mm thick, 14.3mm diameter film of 40/50 penetration grade bitumen was formed in a metal annulus and supported by a fine wire mesh above a closed volume connected to a sensitive barometer (Vaisala model PTB330). The vessel and film were brought to 45°C in an oven over half an hour with the system open to the atmosphere. The system was then closed, air introduced from a cylinder and the pressure increase monitored until the film ruptured as shown in figure 2.25. The rupture pressure at 45°C and two different rates of pressure increase are given in table 2.12.

Figure 2.24 Apparatus for estimating rupture pressure for a 40/50 bitumen

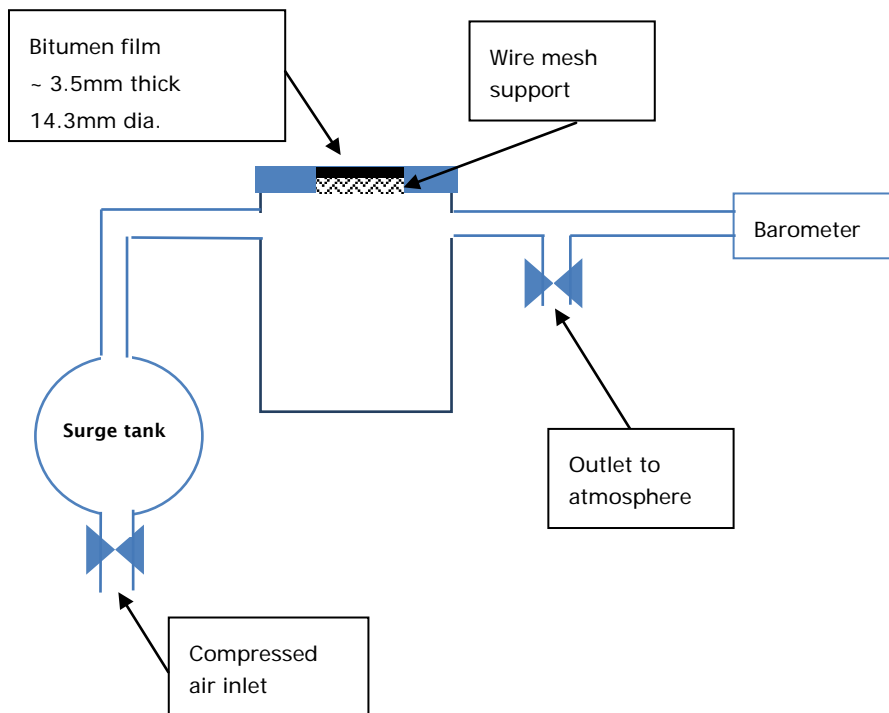


Figure 2.25 Rupture of a 3.5mm film of 40/50 bitumen pressurised at 0.19mbar s⁻¹

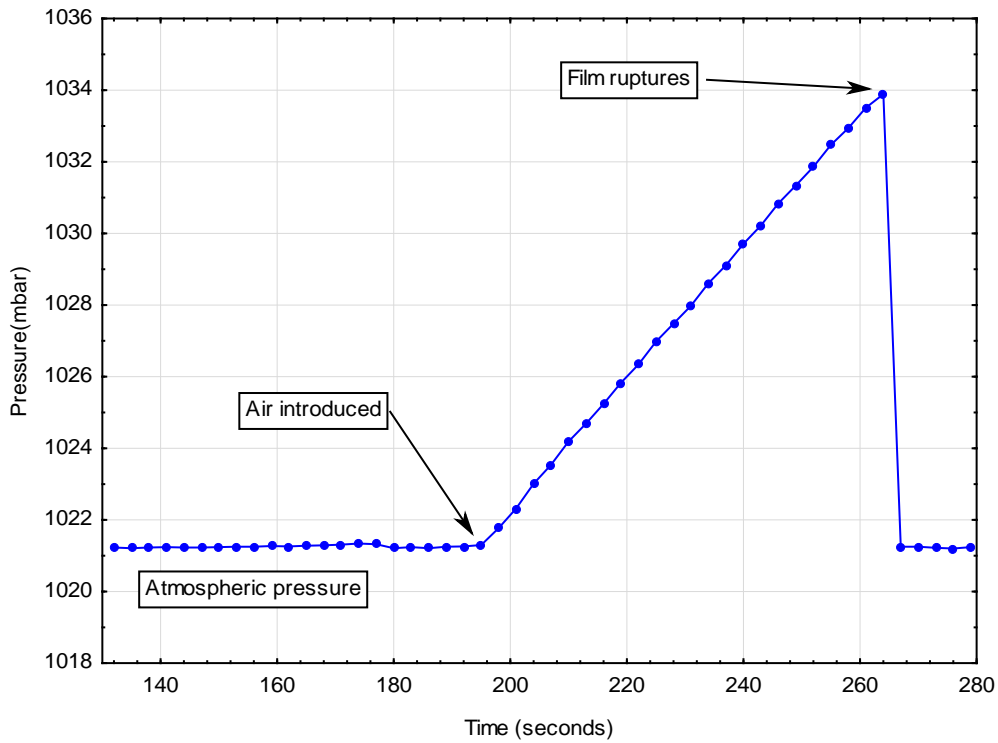


Table 2.12 Rupture pressure and time for 40/50 bitumen in a 3.5mm film at 45°C*

Pressure increase rate (kPa)	Rupture pressure (kPa)	Time to rupture (seconds)
0.019 ± 0.01	1.44 ± 0.14	69 ± 6
0.002 ± 0.01	1.00 ± 0.01	540 ± 50

*Mean ± range of three replicate measurements in each case

A rupture pressure of 1.4kPa is comparable to water vapour pressure at only 11°C and suggests that water vapour could result in very rapid formation of blisters.

Approximately 0.56cm³ of bitumen was displaced from the metal annulus by the action of the air pressure giving a flow rate of 8.1 x 10⁻³ cm³s⁻¹. For a 2mm diameter annulus of the same height (representing a more realistic diameter channel through the seal aggregate), under the same pressure and temperature conditions and assuming Poiseuille’s law holds (ie flow rate is proportional to r⁴ where r is the radius of the channel), then the flow rate would be approximately 3.1 x 10⁻⁶cm³s⁻¹. It would thus require only approximately one hour for the bitumen to be forced from the 2mm diameter channel. This appears to confirm the conclusions of Ball et al (1999) and suggests that water vapour pressure beneath the seal could potentially give rise to bitumen blisters and is consistent with the field observation that blisters can form within hours under the correct weather conditions.

However, the lowest rate of pressure increase used in the experiments above is still approximately 15 times that of the theoretical value if the basecourse was a closed system, which, assuming a temperature increase from 10°C to 45°C over 12 hours, would be about 0.0003 kPa s⁻¹. The basecourse though is not a

closed system, and at such a slow rate of increase there is a possibility that the pressure would dissipate through air movement out of the basecourse. This is discussed further below.

2.4.1.2 Air permeability of the basecourse

The formation of blisters through vaporisation of basecourse moisture relies on sufficient back pressure being generated to allow water vapour pressure to develop under the seal over the course of the day, as the road temperature increases. If that is the case then the question is also raised as to why the expansion of *air* in the basecourse does not also result in blister formation, which should be uniform across all seals (but which is not observed).

The permeability of compacted basecourse aggregate to air is unknown. To further investigate back pressure development, experiments were conducted using an M/4 conforming basecourse (NZ Transport Agency 2006, *Specification for basecourse aggregate*) compacted into a 155mm internal diameter, 300mm high column in a cylindrical steel mould. The basecourse was compacted to 95% of the vibratory hammer maximum dry density (NZS 4402: Test 4.1.3), and had a moisture content of 5.1%. The absolute density was 2.318 tonnes m⁻³. An 8–10mm thick layer of 40/50 penetration grade bitumen was applied to the upper surface forming an airtight seal to the steel cylinder and with an attachment to allow connection of a sensitive barometer (Vaisala model PTB330) as shown in figure 2.26. The underside of the barometer attachment was free of bitumen (confirmed by visual inspection down the connection), so recorded the pressure at the base of the bitumen layer. To check that free air movement was not occurring at the interface between the wall of the steel cylinder and the basecourse a specimen was prepared as above but using a two-part split steel mould. This was carefully removed from the specimen and replaced with a heavy PVC pipe (about 200mm internal diameter). The annulus volume between the specimen column and the PVC pipe was filled with a fluid dental plaster to seal the sides of the basecourse. Tests conducted with this arrangement gave the same results as with the original design indicating no air leakage was occurring around the specimen.

Two experiments were conducted. In the first, the base of the column was covered with a perforated steel baseplate, placed in a 45°C oven and the barometer immediately connected. The pressure beneath the seal was monitored for over 2.5 hours with no change observed; by this time the bitumen layer had reached 45°C (figure 2.27). As the specimen temperature increased, the air and water vapour beneath the bitumen layer remained at atmospheric pressure, 1006 ± 0.5mbar (monitored separately). Gas movement through the basecourse column was sufficiently rapid to prevent any pressure build up even though the heating rate was well in excess of that likely in the field.

A second experiment was conducted in which the perforated base was sealed with silicone to make the whole system air tight. To increase the rate of heating the specimen was placed in the oven in a 10cm deep container which was then filled with 60°C water. The resulting pressure changes are shown in figure 2.28. No pressure change was observed over the first few minutes reflecting the slow heating rate of the large specimen mass but an almost instantaneous increase occurred when the hot water was added to the base and a bitumen blister formed.

The results of these two experiments indicate that at standard moisture contents the gas permeability of the basecourse is sufficient to prevent any pressure gradient developing due to thermal expansion of water vapour or air.

If the basecourse or a layer of basecourse near the seal base is saturated with water the situation may be different. In this case physical entrainment of liquid water through traffic pressure pulses is also very likely to occur and is discussed further below.

Figure 2.26 Apparatus used to assess development of back pressure in a basecourse column. Shown with air tight base seal and after test completion

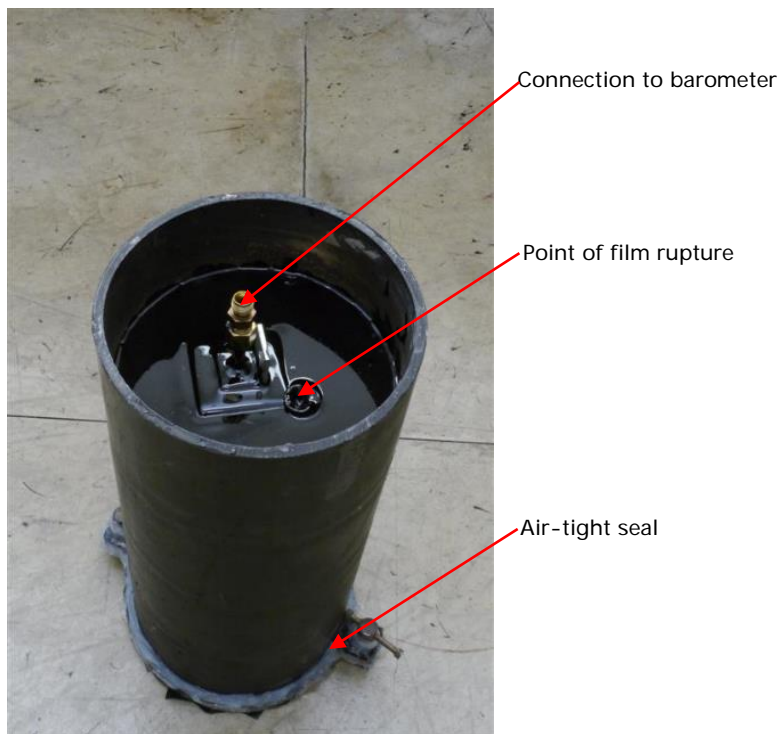


Figure 2.27 Pressure changes on heating the basecourse column with the base unsealed

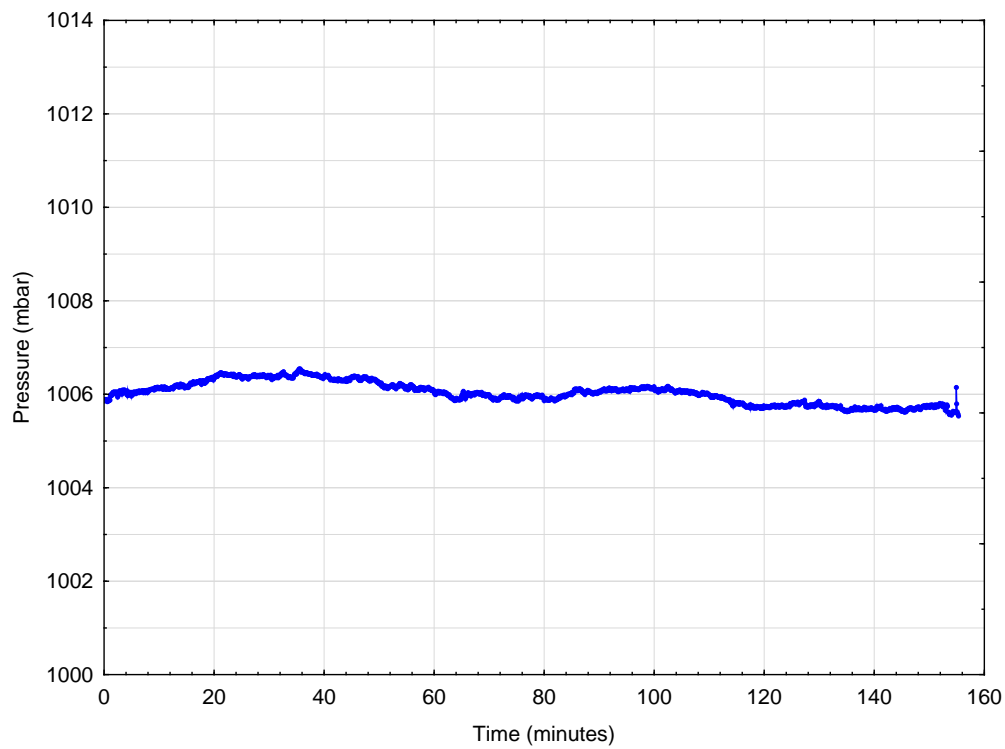
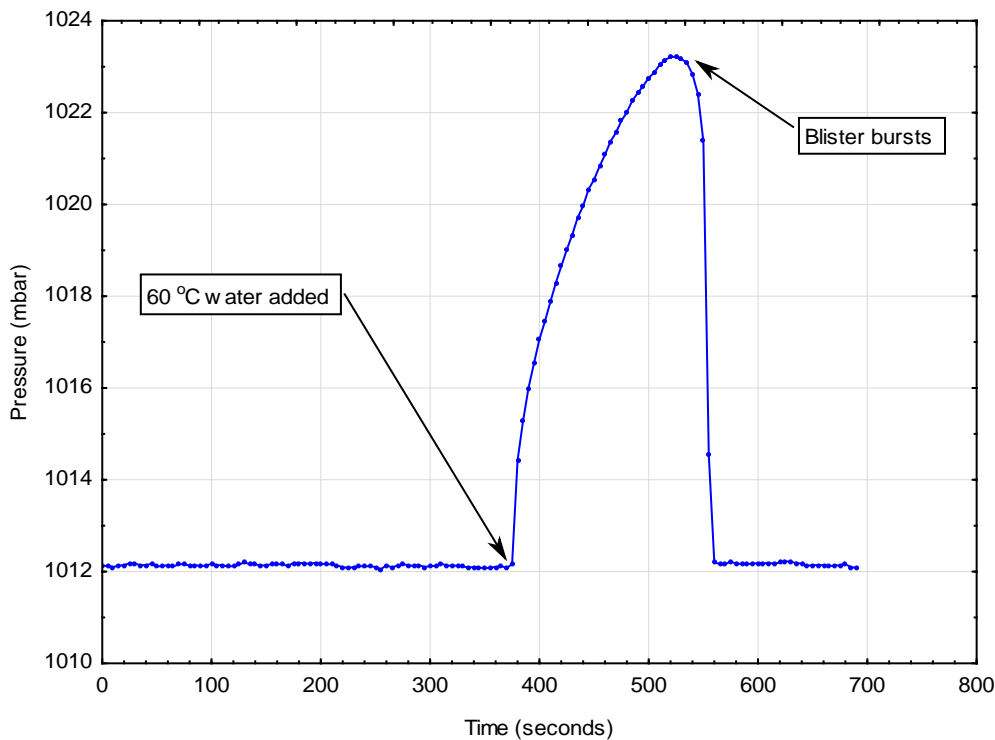


Figure 2.28 Pressure changes on rapid heating of the basecourse column with the base airtight

2.4.2 Sub-surface seal stripping- flushing variability

In addition to the possibility of moisture entering the seal from the basecourse, seals are also known to be permeable to rain water (Alabaster et al 2015; Patrick 2009a). If the basecourse is not sufficiently permeable or has a dense crust that prevents rapid drainage then this water is effectively trapped in the seal layer near the basecourse interface. This possibility was raised as long ago as 1972 (Major 1972). In the present study this process has been found to lead to what we have termed, 'sub-surface stripping' of multi-layer seals.

Flushing is often observed to be highly variable over the seal surface. Often it is apparent in only one wheel track or one lane. Short lengths or patches of severely flushed seal can alternate over only a few metres with well textured seal. While investigating a number of sites for the reasons for this phenomenon it was observed that cores taken from heavily flushed patches were found to have stripped (almost bitumen free) aggregate at the base. These findings are presented below.

2.4.2.1 Site 1: SH57 Shannon

Site 1 and the positions at which core samples were taken are shown in figure 2.29. The samples were taken close to one another to minimise potential differences arising from the presence of patches or other differences in the underlying seals. Patching was initially considered one of the probable reasons for flushing variability.

The texture of the flushed patch sampled on the site and adjacent well-textured seal are compared in figure 2.30. The flushed areas had obvious venting holes and were cracked with evidence of basecourse fines pumping to the surface. A range of cores are shown side by side in figure 2.31. A progression in core height is obvious. The cores from the flushed patch are shorter because it was not possible to extract the base of the seal which was present as loose aggregate from which the bitumen had been clearly

stripped (figure 2.32). This effect was found to varying degrees in all of the sites investigated. Figure 2.33 shows the highly flushed top surface of core one and stripped aggregate on the bottom surface. Figure 2.34 shows a crack on the surface of core 2 opened to show stripped aggregate on the sides.

Figure 2.29 Site 1 showing flushed patch and core positions



Figure 2.30 Site 1, comparison of flushed patch (showing vent holes and cracks) and well- textured surface (with some spot flushing evident) at about the position of core 4

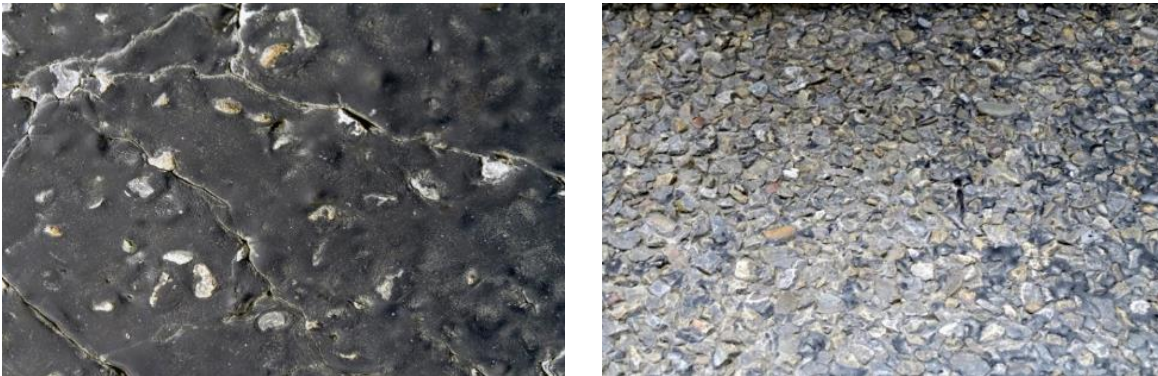


Figure 2.31 Site 1 cores showing a progression of seal loss through stripping. Core 1 (flushed) to core 7 (well textured)

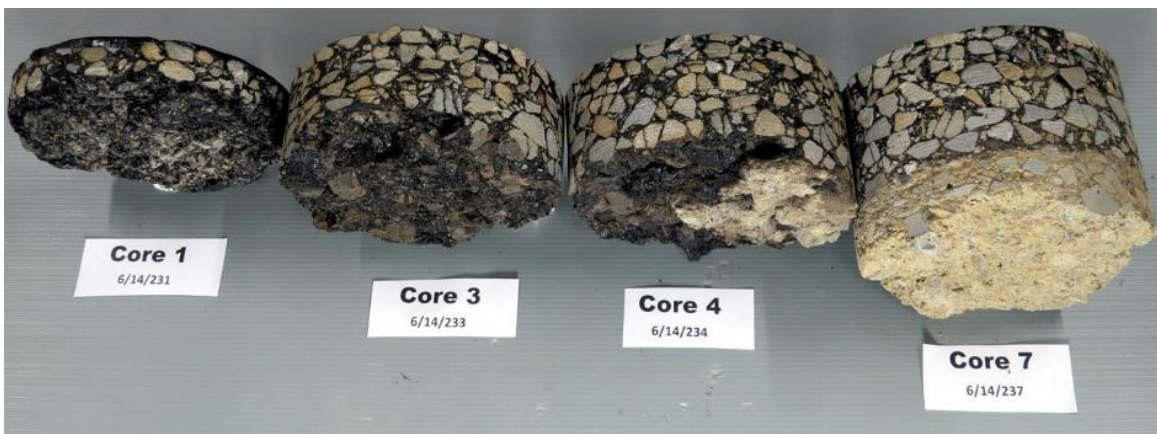


Figure 2.32 Site1, stripped, loose aggregate recovered from the base of core hole 2



Figure 2.33 Site 1, top and bottom of core 1 showing stripped aggregate

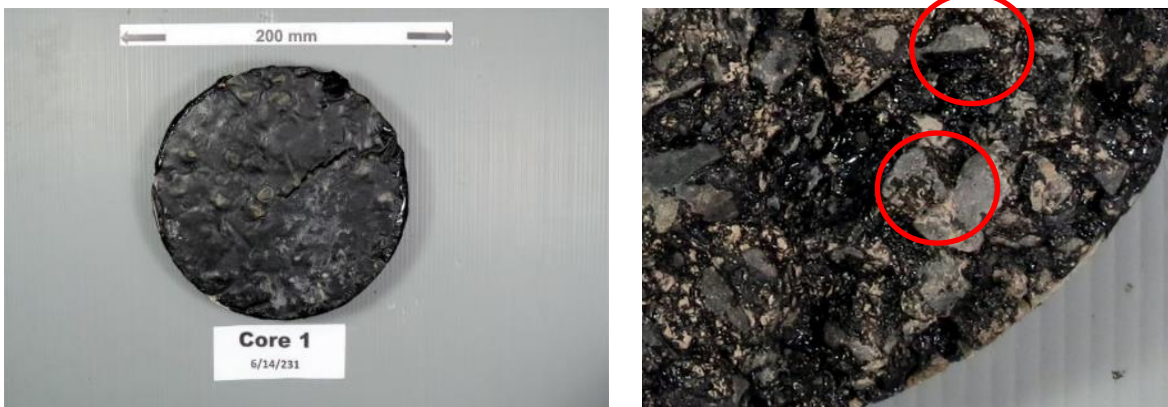


Figure 2.34 Site 1, crack in core 2 opened to reveal stripped clean aggregate on the sides of the crack



2.4.2.2 Site 2: SH2 Featherston

Site 2 and cores taken from the site are shown in figures 2.35 to 2.40. As at the previous sites, specimens taken from the flushed areas were shorter than those from the textured areas because bitumen had been clearly stripped from the aggregate below to such an extent that the seal had lost cohesion. Core 9 was an exception and was recovered intact. Figure 2.39 shows a specimen from a flushed patch that was manually broken in half to reveal the internal structure. Light brown basecourse fines from the coring operation are clearly visible throughout the depth of the layer. This indicates that a passage for water flow existed in the seal at least down to the stripped layer and in some cases the vents, clearly visible in the flushed areas (see figure 2.36), reached deep into the seal layer.

Figure 2.35 Site 2, flushed patches

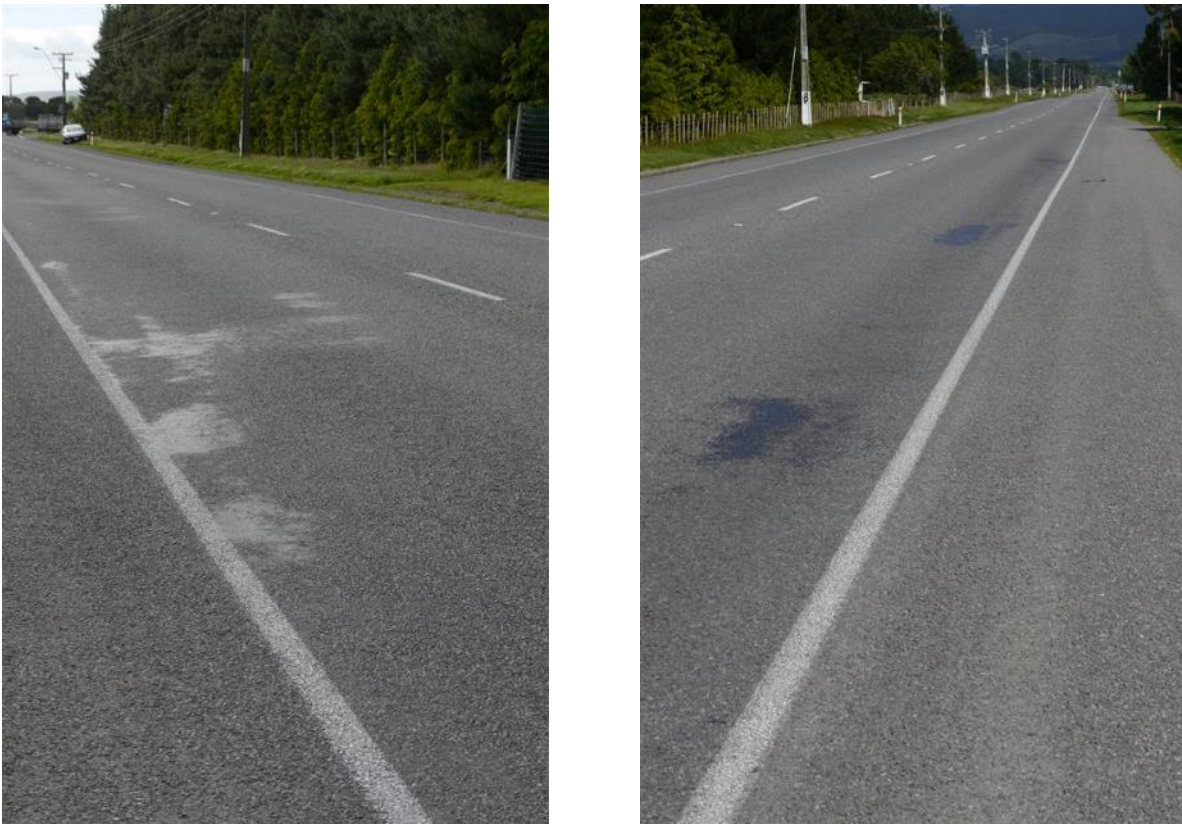


Figure 2.36 Site 2, flushed surface showing vent holes and texture adjacent to the flushed patch

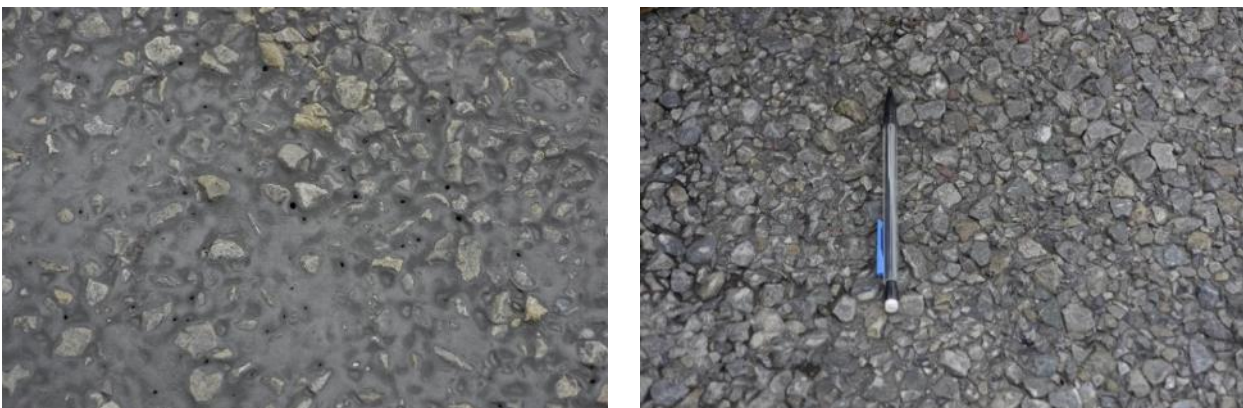


Figure 2.37 Site 2, specimen taken adjacent to the flushed patch showing the full depth of the seal layer (and the presence of an asphalt layer)

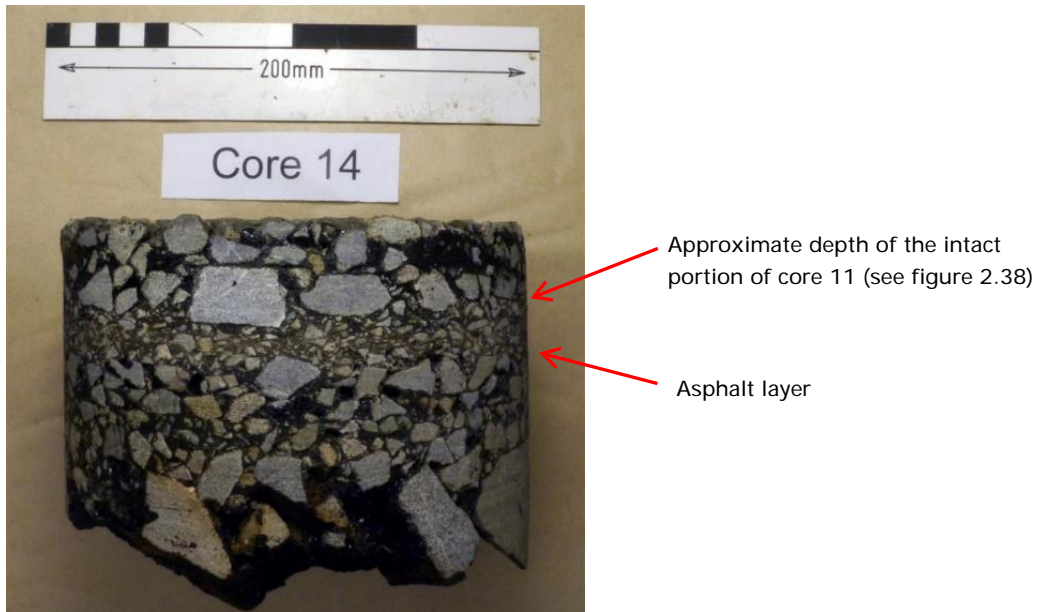


Figure 2.38 Site 2, intact top section of core 11 from a flushed patch



Figure 2.39 Site 2, close up of core 11 cross section from a flushed patch, manually broken to reveal the internal structure and evidence (deposited basecourse fines) for water migration through the layer

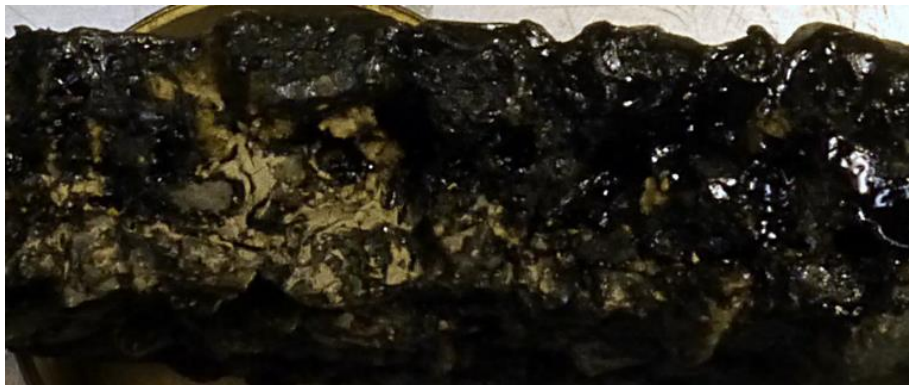
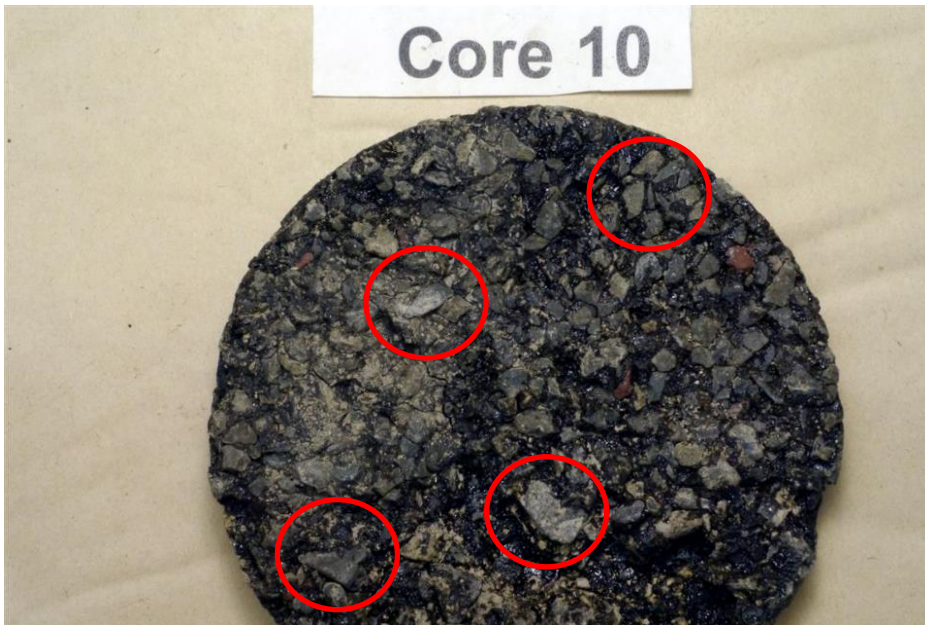


Figure 2.40 Site 2, base of specimen from flushed patch (about 20mm below top surface) showing stripped aggregate



2.4.2.3 Site 3: SH77 near Darfield

The site and core positions are shown in figures 2.41 to 2.45. As at the previous sites, samples (slabs in this case) taken from the flushed areas are thinner than those from the textured areas because it was not possible to recover the bottom half of the specimens intact (see figure 2.45). As previously found, bitumen had been clearly stripped from the seal aggregate which was essentially loose in the bottom of the core hole.

Figure 2.41 Site 3 flushed patches

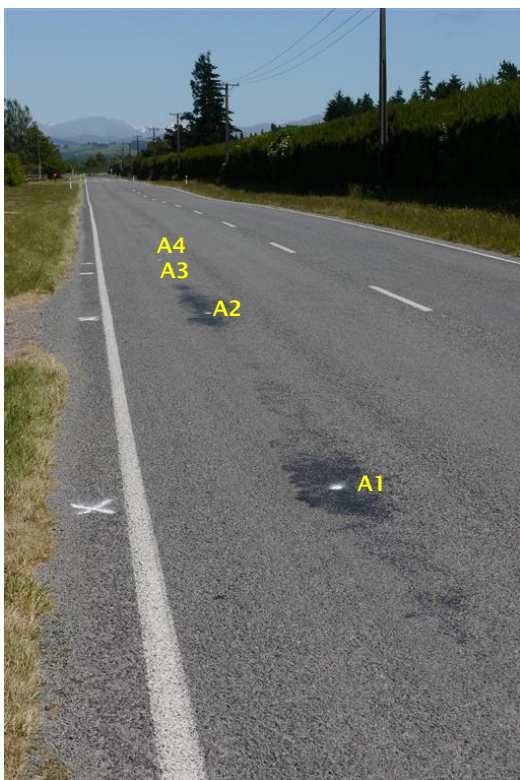


Figure 2.42 Site 3, texture adjacent to flushed patches (left) and flushed patch texture (right)

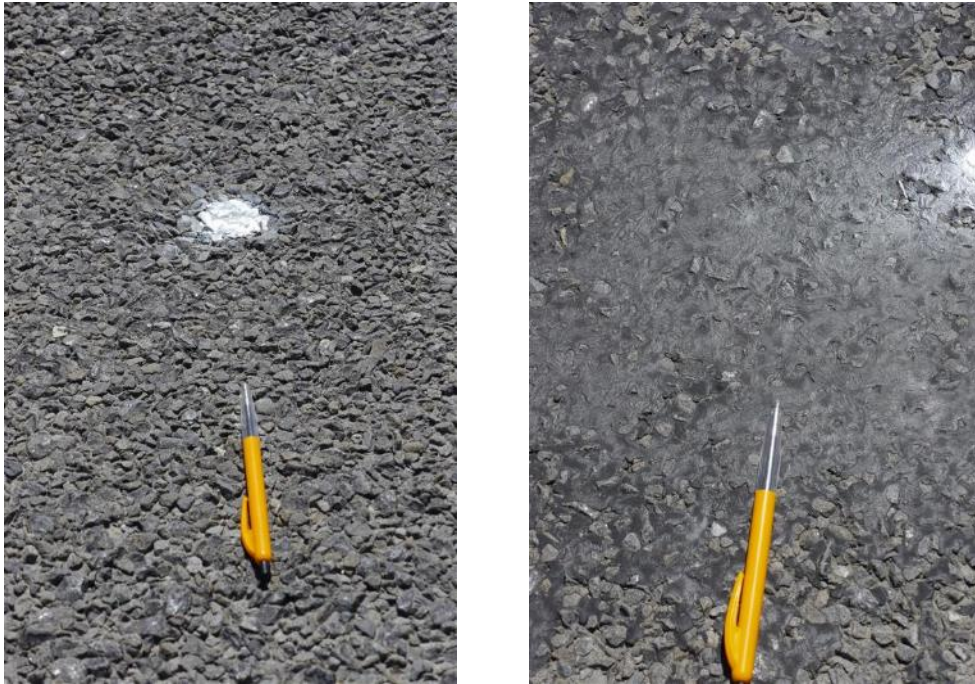


Figure 2.43 Site 3, profile of specimen from a textured area



Figure 2.44 Site 3, profile of specimen from a flushed area



Figure 2.45 Site 3, comparison of intact specimen thickness (illustrating extent of loss due to stripping)



2.4.2.4 Site 4: SH 77 near Darfield

Site 4 and samples taken from the site are shown in figures 2.46 to 2.52. As for the earlier sites, specimens taken from the flushed areas showed clear evidence of stripping at the base of the seal (figure 2.52).

Figure 2.46 Site 4, flushed patch, specimen locations marked



Figure 2.47 Site 4, texture adjacent to flushed patch



Figure 2.48 Site 4, specimen from flushed patch



Figure 2.49 Site 4, specimen taken adjacent to the flushed patch



Figure 2.50 Site 4, specimen from flushed patch



Figure 2.51 Site 4, specimen taken adjacent to the flushed patch



Figure 2.52 Site 4, material from the bottom of flushed patch specimen showing stripped aggregate



2.4.2.5 Site 5: SH2 Greytown

Site 5 and cores taken from the site are shown in figures 2.53 to 2.58. As at the previous sites stripping at the base of the seal in the flushed area was apparent and on this site there was clear evidence of the 'pumping' of basecourse fines through the seal layer (figure 2.54). Cores 7 and 8 (figure 2.55) had good seal texture and showed no signs of stripping damage. Core 8 was taken at the edge of the flushed patch and the onset of stripping was apparent as the asphalt layer failed during core extraction. Examination of the broken surfaces showed partial stripping of the aggregate. Only the top section of core 9 could be recovered intact due to stripping (figures 2.57 and 2.58).

Figure 2.53 Site 5, flushed patch and core locations



Figure 2.54 Site 5, flushed patch texture showing basecourse fines 'pumped' through the seal (evidence of water migration through the entire seal layer)



Figure 2.55 Site 5, specimens from textured area and partially flushed seal showing the full depth of the seal layer down to the (bitumen bound) basecourse. An asphalt layer is present two seal coats down from the surface. Stripping in this layer core 8 resulted in failure when extracting the core

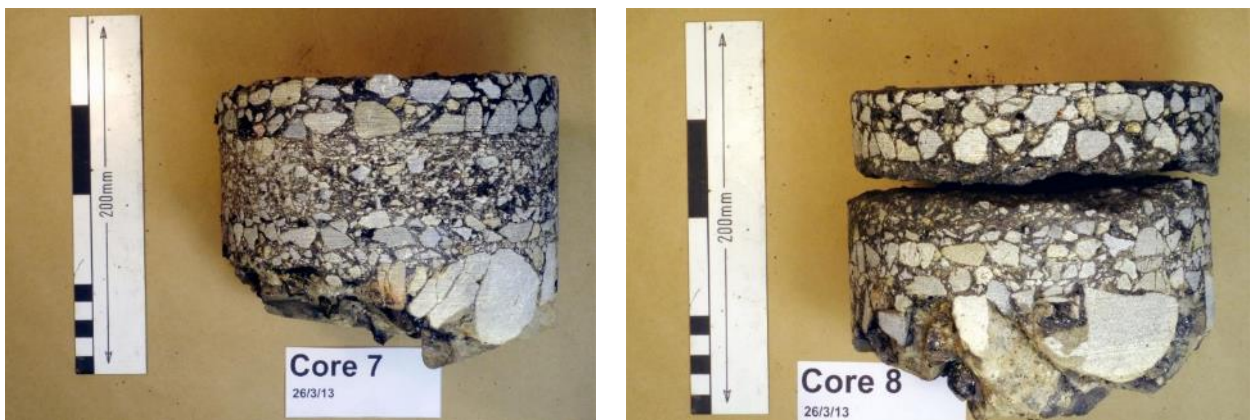


Figure 2.56 Site 5, top surface specimen from flushed patch



Figure 2.57 Site 5, base of specimen from flushed patch texture showing stripped seal

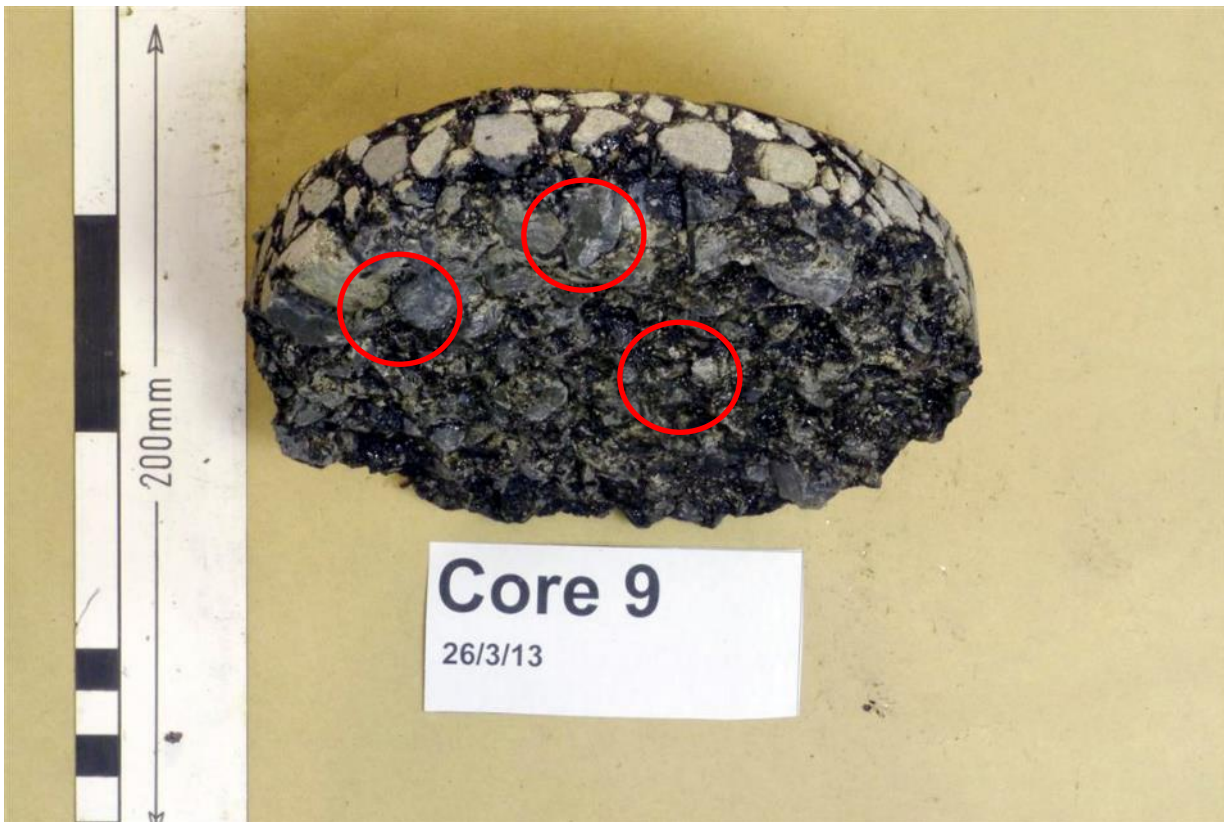


Figure 2.58 Site 5, base of specimen (core 9) from flushed patch showing stripped seal



2.4.2.6 Site 6: SH53 Kahutara Road

Site 6 and cores taken from the site are shown in figures 2.59 to 2.62. As at the previous sites, there was clear evidence of seal stripping beneath the surface of the flushed patches (cores 10 and 11), migration of basecourse fines through the seal was also widely apparent. As at the previous site a core from the non-flushed area (core 12, figure 2.62) also failed during extraction (sheared in an asphalt patching layer). The adjacent flushed area cores (10 and 11), could not be removed intact as the bottom sections were severely stripped. Close examination of the base of the top section that had broken off core 12, showed a small amount of stripped aggregate (much less than core 11) but clearly enough to weaken the asphalt layer and indicating that the stripping damage was developing outwards from the highly flushed patch.

Figure 2.59 Site 6, flushed patch (in shade at bottom right)



Figure 2.60 Site 6, vent holes and basecourse fines in flushed patch



Figure 2.61 Site 6, base of core 10 (flushed patch) showing stripping



Figure 2.62 Site 6, cores 11, 12 and 13 showing the progression of stripping damage from the heavily flushed patch (core 11) to good seal texture (core 13)



2.4.2.7 Site 7: SH58 Haywards Hill

Site seven is shown in figure 2.63 and the core positions (1–5) are marked. As with the other sites this too showed stripping at the base of the seal in the flushed area and loss of about 15mm of seal (cores 1 and 2). Cores 4 and 5 were extracted intact with basecourse attached. Core three also had stripped aggregate at the base but at the depth of the basecourse interface.

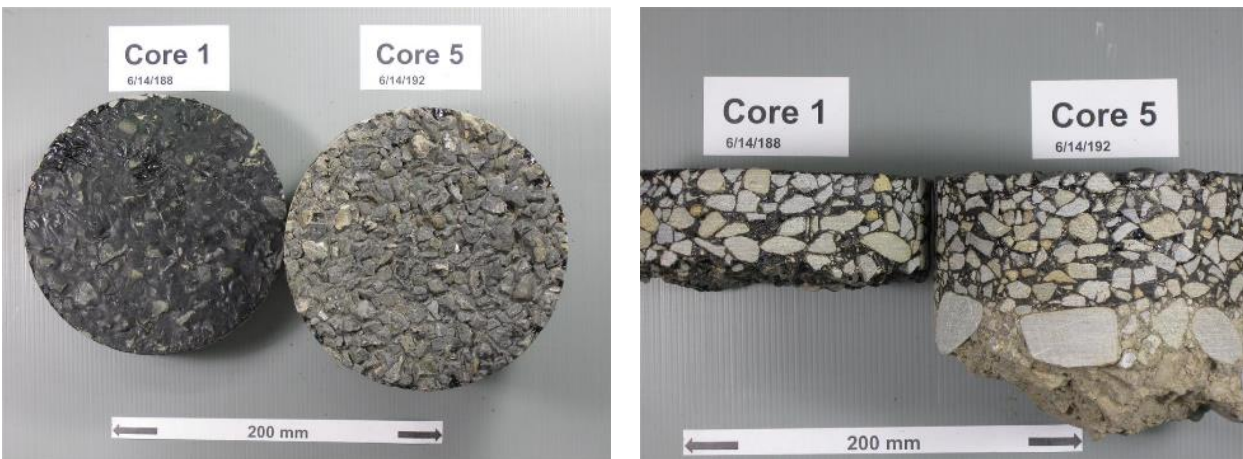
Figure 2.63 Site 7, showing the core positions



Figure 2.64 Site 7, flushed patch texture showing venting holes and adjacent well- textured seal



Figure 2.65 Site 7, cores 1 and 5 showing the loss of about 15mm of seal from core 1 (present as loose aggregate in the hole)



2.4.2.8 Analysis of field samples

Differences in the surface texture of the specimens examined could potentially have been caused by differences in the seal sequence at the particular positions at which the cores were taken, for example the presence of an asphalt patch under the flushed area but not the adjacent seal. The specimens were taken close together to minimise this possibility and the profiles carefully examined to detect differences. None were observed (at least in the intact cores). To further discount this possibility and also to eliminate the possibility that excess bitumen had been applied at the flushed patch positions, some of the cores were extracted using the method described in section 2.3.2 and bitumen contents and gradings compared (not all could be analysed due to budget constraints).

Bitumen contents

The bitumen contents of the specimens are given in tables 2.13 to 2.16. Specimens from the non-flushed areas of seal had any retained basecourse material cut off before extraction. For the specimens from the flushed areas bitumen contents were performed separately on the top (intact) sections, and the 'loose' material from the base. As in most cases not all the loose material could be recovered in the field (without incorporating basecourse material), an overall mean was calculated for these specimens as the weighted mean of the top and bottom sections.

The bitumen contents of the top sections of the flushed cores were obviously much higher than the stripped material at the base. Overall, however, the results show the flushed and textured seals have very similar mean bitumen contents (around 8% by mass), which based on the discussion in section 2.2 indicates that the flushing observed was not due to excessive bitumen application. The flushed cores from site 1 had a noticeably lower overall bitumen content than that of the well-textured sites, 7% compared with 8.4%. This is still a relatively small difference (12%), and probably relates to the difficulty in obtaining a representative sample of the stripped aggregate without the inclusion of basecourse.

Table 2.13 Site 1, SH57 bitumen contents (% by mass)

Site	Section of specimen	Flushed			Textured			
		Specimen number			Specimen number			
		1	2	Mean	4	5	6	Mean
1	Top	11.7	10.6	11.2	-	-	-	-
	Bottom	2.8	2.6	2.7	-	-	-	-
	Weighted mean	7.0	7.0	7.0	8.2	8.6	8.3	8.4

Table 2.14 Site 2, SH2 Featherston bitumen contents (% by mass)

Site	Section of specimen	Flushed				Textured						
		Specimen number				Specimen number ^(b)						
		9 ^(a)	10	11	Mean	12	13	14	15	16	17	Mean
2	Top	-	10.1	9.9	10.0	-	-	-	-	-	-	-
	Bottom	-	6.8	6.4	6.4	-	-	-	-	-	-	-
	Weighted mean	8.6	7.8	7.5	8.0	8.6	8.8	8.7	7.4	7.5	7.8	8.1

^(a) Specimen recovered intact. ^(b) Specimens cut at the same height from the top as cores 10 and 11 (15mm).

Table 2.15 Site 3, SH77 bitumen contents (% by mass)

Site	Section of specimen	Flushed			Textured		
		Specimen number			Specimen number		
		A1	A2	Mean	A3	A4	Mean
3	Top	11.3	11.2	11.3	-	-	-
	Bottom	5.4	4.7	5.1	-	-	-
	Weighted mean	8.5	8.2	8.4	7.8	8.4	8.1

Table 2.16 Site 4, SH77 bitumen contents (% by mass).

Site	Section of specimen	Flushed			Textured		
		Specimen number			Specimen number		
		B1	B2	Mean	B3	B4	Mean
4	Top	9.8	10.8	10.3	-	-	-
	Bottom	3.4	3.5	3.5	-	-	-
	Mean	7.7	7.5	7.6	7.5	8.0	7.8

Aggregate gradings

Aggregate gradings for specimens taken from some of the sites are shown in figures 2.66 to 2.68. Gradings for the flushed cores with stripped aggregate are calculated from the weighted averages of gradings of the top and bottom parts of the core. The flushed core and textured core gradings for each site were averaged and plotted.

Although small differences were observed in some cases, these were similar in magnitude to the variations found between replicate specimens and no significant difference in the gradings was observed between flushed and textured specimens.

These results (together with the overall similar bitumen contents and visual assessment) indicate that the surfacing layers present were the same for both flushed and textured sampling points and the severe flushing observed was not due to patching or other anomalies.

Figure 2.66 Site 3, SH77 comparison of aggregate gradings for flushed and textured samples

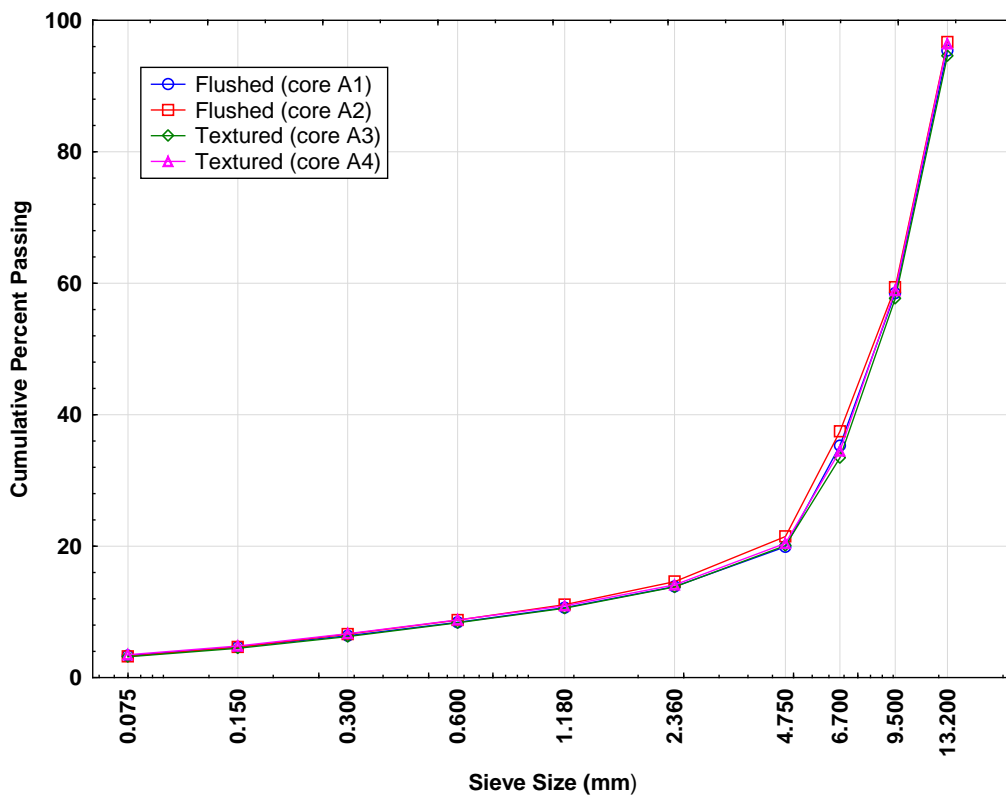


Figure 2.67 Site 2, SH2 comparison of aggregate gradings for flushed and textured samples

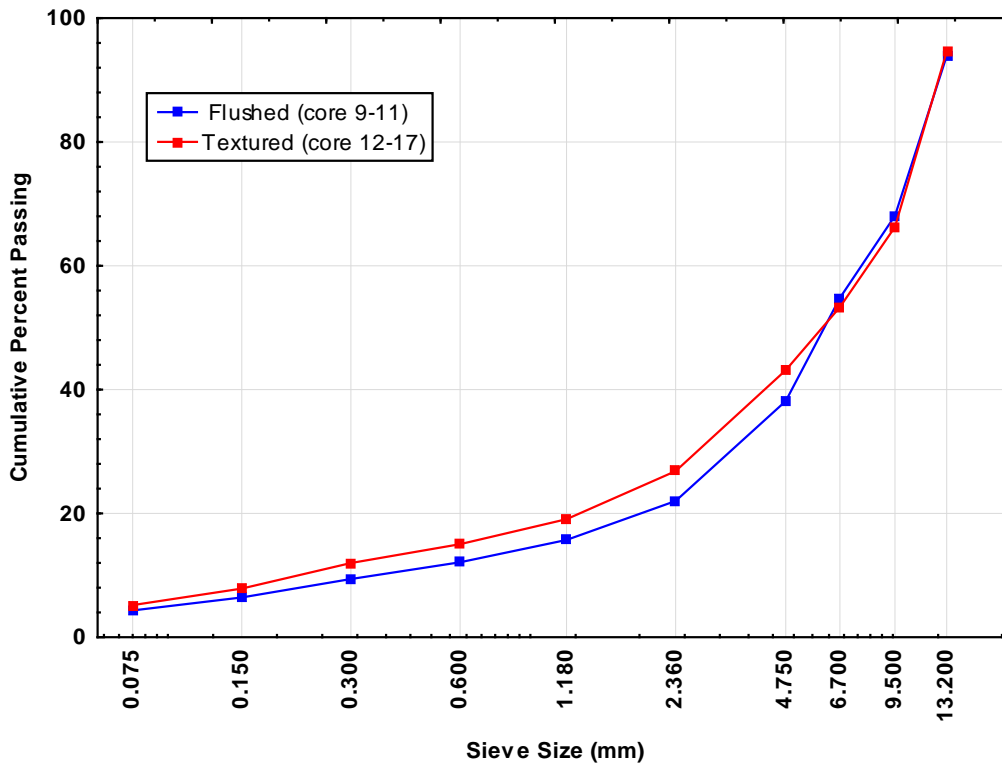
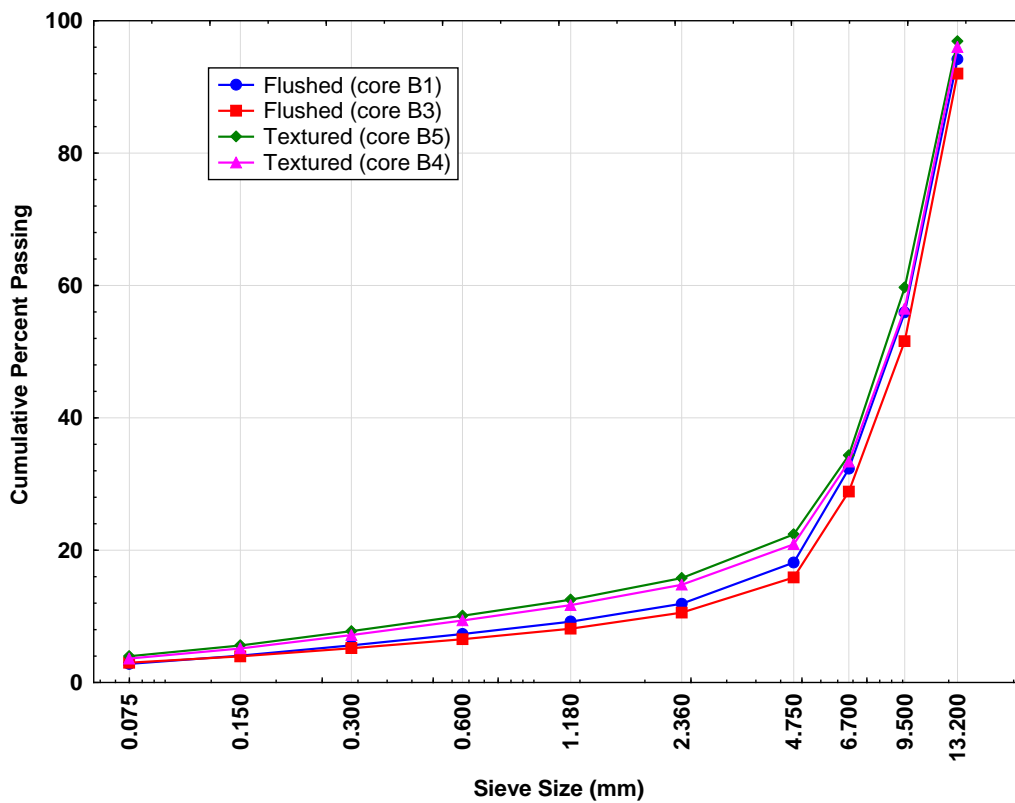


Figure 2.68 Site 4, SH77 comparison of aggregate gradings for flushed and textured samples



2.4.3 Conclusion

Water appears to be a major contributor to flushing. Blisters or holes caused by water vapour venting are almost always apparent in flushed seals and are evidence of serious problems with water ingress to the seal layer. The blisters are not likely to be caused by water vapour from moisture in the basecourse unless the basecourse is saturated with water or sufficient backpressure is generated in some other way. Blisters have also been observed on untrafficked artificial seals on steel plates where water must have entered the bitumen layer from above, possibly along aggregate faces or by adsorption onto fines in the bitumen. Unlike kerosene, water that is entrained in the bitumen is essentially immiscible and will vaporise as the temperature increases to produce a blister.

That seals are permeable to water is now well established. Water that enters the seal through a defect and is not able to drain will saturate the basecourse seal interface. In theory the M/4 basecourse grading is permeable to water but the vagaries of construction mean that this may not always be the case, or possibly the rate of water ingress through areas of the seal may be too great.

Water trapped in the base of the seal will vaporise in hot weather and vent to the surface. Mechanical entrainment of water by traffic into adjacent areas of bitumen and chemical disbonding and hydraulic stripping of the seal aggregate can occur. Bitumen is forced to the surface by water trapped at the basecourse surface and the seal flushes. The role of water in causing sub-surface stripping explains much of the variability observed in flushing patterns in the field.

2.5 Embedment into the seal layer substrate

Loss of surface voids can occur through embedment of the sealing chip into the substrate, ie the basecourse (see chapter 1, figure 1.1), or in some cases an asphalt layer such as a patch, smoothing coat or shape correction. The magnitude or rate of such processes and their overall contribution to flushing, have been little studied and quantitative data is lacking. Some embedment is of course desirable to allow the seal to resist shearing forces from traffic.

2.5.1 Embedment into an asphalt substrate

That embedment of chip into a soft asphalt substrate will occur is generally recognised. Marek (1971) measured embedment of chip into asphalt using a 10 tonne roller and found in the order of 40% of the voids in the chip were later lost.

Using a wheel tracking method, Ball and Patrick (1998) found the rate of texture loss in multi-layer seals over asphaltic concrete smoothing layers was significantly greater than those over seals only. The finding was based on very limited data and further work is needed to gauge the significance of the effect of asphalt patching or smoothing layers on flushing.

2.5.2 Embedment in the basecourse

The problem is illustrated in figure 2.69, which shows flushing in the wheel path of a one-year-old grade 3/5 first-coat seal. A few water vapour vent holes were observed but most of the texture loss appeared to be from chip embedment.

Potter and Church (1976) measured the embedment of 16mm size chip, first-coat seals, into a primed granular basecourse after 1.5 to 7 years with traffic levels of 1,200 to 2,000 vehicles per day. They found that embedment resulted in 20% to 40% of the voids being filled with basecourse material. Other measurements showed that intrusion of the basecourse was rapid (less than a single day with traffic of

100 to 2,600 vehicles per day) but on average changed little after three months. The authors believed that most embedment would occur over the first 12 months of the seal life.

Embedment is recognised in South African, Australian and New Zealand seal designs and binder rates are (theoretically), adjusted according to the substrate hardness (Transit New Zealand et al 2005). The substrate hardness is measured in millimetres using a ball penetrometer which measures the depth of penetration of a 19mm diameter steel ball when impacted by a 4.53kg weight (Choi 2009). No data, however, demonstrating the relationship between penetration value and actual chip embedment has been located. Marais (1969; 1979), *suggests* a relationship but no substantive data is presented.

Unfortunately it was not possible to investigate basecourse embedment further within the scope of this project; however, a method (similar to Austroads *AGPT-T253-06 Seal behaviour*), for study of texture loss in first-coat seals was developed that may be of use for future investigations (see figure 2.70).

An approximately 10mm high bund made of modelling putty (eg plasticine) is formed around the seal patch to be investigated. The bund is filled with a fast setting epoxy resin (eg Araldite K219 which cures in about 15–20 minutes), to hold the chip in place and the sample is cut from the surface. The bund and excess seal surface around the patch can be easily broken off later to leave the test patch. The base of the patch is washed and brushed to remove attached basecourse and reveal the embedment texture which can be measured using sand circle test sand or similar.

Figure 2.69 Flushing in the wheel path of a one- year- old first- coat seal (SH73 Annavale)

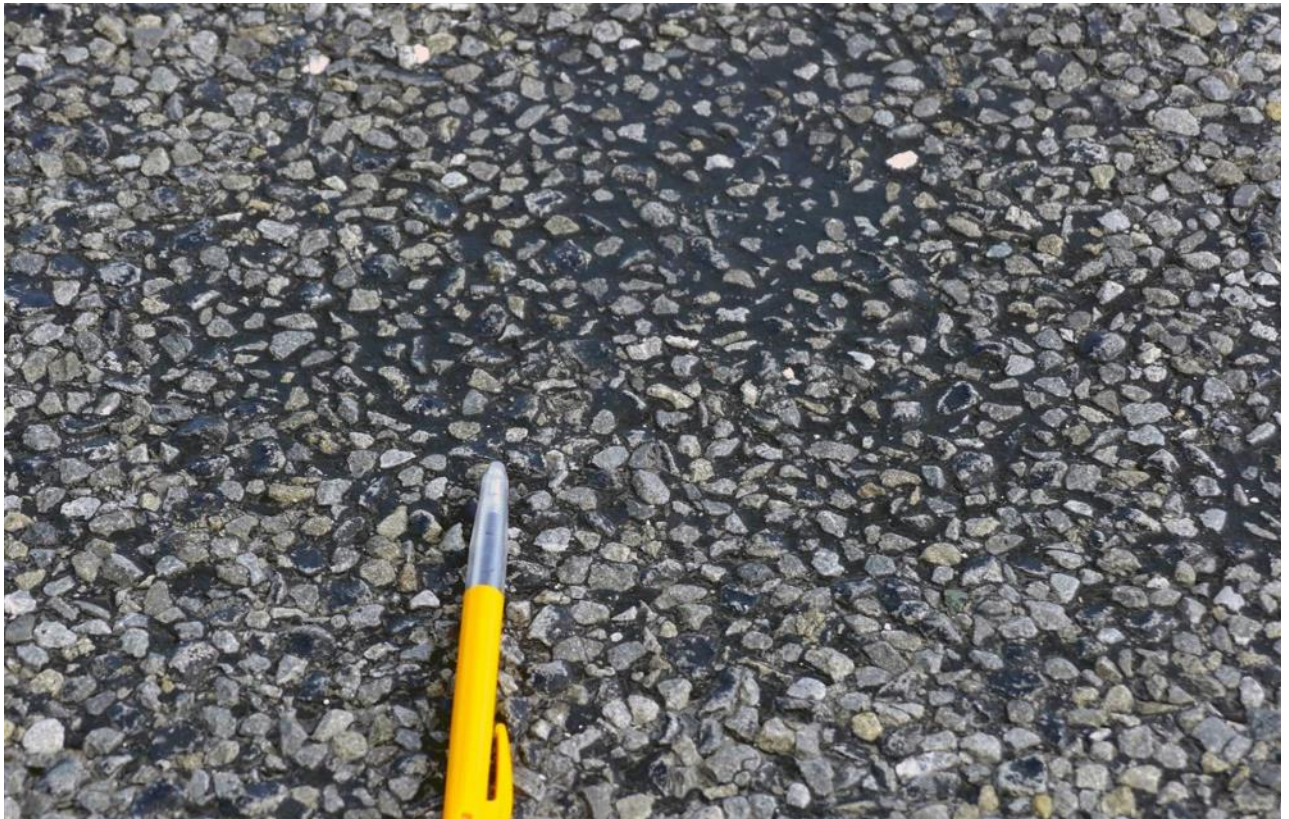


Figure 2.70 Method to examine first- coat seal basecourse embedment. Left, securing the chip with an epoxy resin. Right, cutting around the specimen that can now be removed without distorting the sample patch



2.5.3 Conclusion

The only published study on texture loss from embedment into the basecourse suggests that 20% to 40% of seal layer void volume may be occupied by basecourse penetration. How applicable this data is to New Zealand seals is not known, nor whether embedment continues in the longer term and should be investigated further.

2.6 Low binder viscosity

It is sometimes asserted in the literature that low viscosity binders contribute to flushing, although often flushing and bleeding effects are not clearly differentiated. In practice low binder viscosity most often arises from use of excessive concentrations of cutter (usually kerosene) or sealing with cutback bitumen late in the season when the road temperatures are low and evaporation rates are reduced. Evaporation rates have been studied by Dickinson (1998) and others in Australia (Meydan 1997) and are strongly temperature dependent. A field study from seals in Lower Hutt showed that up to 20% of the added kerosene remained in the seal binder even after five years (Herrington et al 2006).

A low viscosity binder is likely to contribute to runoff into gutters or the road shoulder and pick-up and tracking of bitumen on tyres (Herrington et al 2010). It may also potentially produce a greater and more rapid reorientation of the sealing chip and reduction in seal layer volume than would otherwise be the case. The effect of binder viscosity on the rate at which texture loss through reorientation occurs has been studied previously as discussed below.

Ball and Patrick (1998) took seal samples from six sites in the Wairarapa and then trafficked them in the laboratory using a small, rolling car tyre exerting a tyre footprint pressure of 207kPa. The sites consisted of five grade 3, and one grade 2 single coat seals and varied in age from 3.5 to 10 years.

The samples were rolled at 40°, 50° and 60°C and the change in texture measured. The binder was also recovered and viscosity measurements made at the rolling temperatures.

In some cases the tests at 40°C indicated a slower rate of change of texture than the tests at 50° and 60°C (where the binder viscosity is lower), but inconsistencies were observed and no clear relationship between bitumen viscosity and rate of texture loss was found. The results were inconclusive due to the small change in texture observed and the experimental errors involved in the measurements.

In subsequent work (Ball 2005) field trials were constructed at several locations around New Zealand. At each site sequential sections of seal were constructed with different binder grades following standard practice and the rate of change in texture monitored for three years. At each site this allowed comparison of binder effects alone, with all other variables essentially the same. None of the sites were first-coat seals. Details of the sites are given in table 2.17. The rate of change (B) of the texture depth normalised for differences in chip sizes, with the log of elv was calculated. Results obtained for each of the three sites are plotted in figure 2.71.

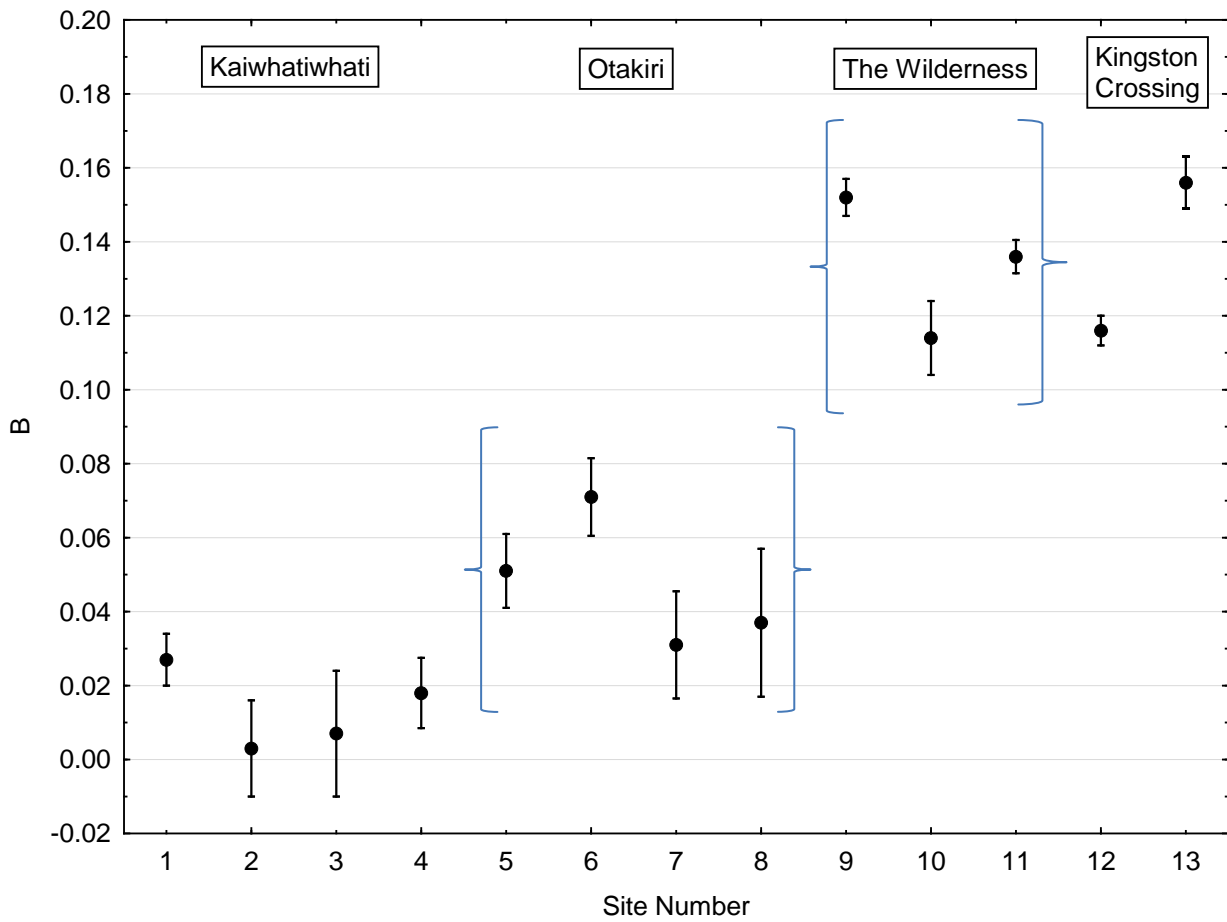
It can be seen that there is an overlap of the slopes on each site with no evidence of a significant difference associated with binder grade. There is, however, a significant difference between sites. Contrary to what may have been expected the southern sites (number 9 to 15) lost texture faster than the sites in the North Island. The traffic volume on the sites ranged from an AADT of 450 to 1,450, ie relatively lightly trafficked and all used a single coat seal.

Table 2.17 Details of trial sites for studying the effect of binder grade on texture change

Trial site		Site no.	Binder	Initial elv	Final elv	B for T (mean) values
Kaiwhatiwhati SH38 RS 0	RP 0/15.549 – 15.932	1	130/150+ 2% SBS + 2pph kero	181 097	2 193 289	0.027±0.014
	RP 0/15.932 – 16.312	2	130/150 + 2pph kero	181 097	2 193 289	0.003±0.026
	RP 0/16.312 – 16.695	3	80/100 + 4pph kero	181 097	2 193 289	0.007±0.034
	RP 0/16.695 – 17.145	4	CRS2 70% 130/150 + 2% cutter	181 097	2 193 289	0.018±0.019
Otakiri SH34 RS 0	RP 0/4.129 – 5.130	5	180/200 + 1pph kero	132 969	1 679 119	0.051±0.02
	RP 0/5.130 – 6.130	6	130/150 + 3pph kero	132 969	1 679 119	0.071±0.021
	RP 0/6.130 – 7.130	7	80/100 + 5pph kero	132 969	1 679 119	0.031±0.029
	RP 0/7.130 – 8.031	8	80/100 + 3pph kero	132 969	1 679 119	0.037±0.040
The Wilderness SH94 RS 115	RP 115/4.34 – 4.57	9	180/200 + 3pph kero	78 709	1 302 848	0.152±0.010
	RP 115/4.95 – 5.34	10	130/150 +3pph kero	78 709	1 302 848	0.114±0.020
	RP 115/4.57 – 4.95	11	80/100 +3pph kero	78 709	1 302 848	0.136±0.009
Kingston Crossing SH94 RS 16	RP 16/3.91 – 4.02	12	180/200 + 3pph kero	68 780	1 110 435	0.116±0.008
	RP 16/3.28 – 3.68	13	130/150 + 3pph kero	68 780	1 110 435	-
	RP 16/3.09 – 3.28	14	80/100 + 3pph kero	68 780	1 110 435	0.156±0.014

Source: Ball (2005)

Figure 2.71 Rate of texture loss for trial sites



Source: Ball (2005)

Wellington City Council conducted a trial in February 2000 to investigate the effect of binder grade on texture loss (Stevenson 2000). Grade 3/5 racked-in seals were constructed on two sites using a range of binders. The texture depth of sites two years later is given in table 2.18. The ratio of the wheel path texture to that of untrafficked seal between the wheel paths is different between sites because of differing traffic levels but very consistent within each site. This trial confirms the findings of Ball (2005) that cutback binders and even unmodified bitumens (within the 80/100 to 180/200 grade range) do not appear to affect the rate of change of texture or time to flush of a chipseal.

Table 2.18 Wellington City Council field trial results

Site 1: Burma Road, Khandallah	Texture depth after two years (mm)		
	Wheel path	Between wheel paths	Wheel path/between wheel paths
130/150	2.80	3.84	0.73
80/100	3.56	5.09	0.70
180/200	3.71	4.93	0.75
180/200+ 2pph kero	3.48	4.62	0.75
130/150 + 2pph kero	3.51	4.70	0.75
80/100 +2pph kero	3.55	4.88	0.73
Site 2: Ironside Road, Johnsonville			
130/150	2.89	3.36	0.86
80/100 +2pph kero	2.51	2.98	0.84

Source: Stevenson (2000)

2.6.1 Conclusion

These results indicate that over the range of sealing grade binders and at typical kerosene contents the range of binder viscosities achieved has produced no measurable influence on texture change (ie chip reorientation and possibly embedment in the underlying seals). Higher kerosene contents, however, may potentially have an effect (as may much stiffer binders such as epoxy modified bitumen (Herrington and Bagshaw 2014) but in general using harder penetration grades such as 80/100 in place of 180/200 will not reduce the rate of flushing.

2.7 Thermal expansion of the bitumen volume

Thermal expansion of the binder volume on a hot day is sometimes anecdotally claimed as a cause of flushing. Research on thermal expansion as a mechanism for flushing has not been reported in the literature but a simple calculation) shows that this theory is not tenable.

Based on the void/depth relationship given by Dickinson (1990) a hypothetical seal with the following properties was considered:

- chip ALD = 11mm
- seal height = $1.1 \times \text{ALD} = 12.1\text{mm}$
- total voids = 4.63Lm^{-2} .

If the binder reaches 60% of chip height then (following Dickinson), 40% of the voids in the seal layer are filled. The binder volume is thus 40% of the total voids or 1.85Lm^{-2} .

For a change in binder temperature from, for example 15°C to 60°C , then using the standard volume expansion factors (*ASTM D4311/ D4311M 2009 Standard practice for determining asphalt volume correction to a base temperature*), the corresponding increase in binder volume is $1.0288 \times 1.85 = 1.90\text{Lm}^{-2}$.

The effect of the change in temperature from 15°C to 60°C has been to increase the binder volume from 1.85 to 1.90 Lm⁻². This will increase the binder rise up the chip from 60% to approximately 62%. The rise in bitumen height is equivalent to 2% of 12.1mm or 0.24mm. The change in texture associated with a change in binder temperature from 15°C to 60°C is therefore approximately only 0.2mm. Thermal expansion of the bitumen is thus too small to have any significant bearing on seal flushing.

Thermal expansion would also of course be reversed in cold weather (ie the binder would contract) and is likely to make only a negligible contribution to chipseal flushing.

3 Modelling flushing in chipseals

3.1 Introduction

This chapter presents the outcomes from data analysis carried out to investigate the relationships between flushing and variables known to cause flushing. The report also presents the development of a multi-stage model to predict flushing. The primary goal of the flushing modelling process undertaken in this research was to develop a data-driven prediction model based on pavement management systems data collected through New Zealand's Long-Term Pavement Performance (LTPP) programme. The data analysis and the model development were performed by analysing pavement condition data from the state highway and the local authority LTPP databases. The objectives of the presented data analysis and model development were to:

- investigate the relationship between flushing and variables known to cause flushing
- identify the combination of factors that provide the best indication of flushing potential of a chipseal surface
- determine a threshold to identify when a pavement surface is flushed
- develop a data-driven model to predict the initiation and progression of flushing on a chipseal pavement.

3.2 Data set description

3.2.1 The Long-Term Pavement Performance data set

The New Zealand LTPP programme records highly accurate pavement condition data from selected state highway and local authority pavement sections. Data from the state highway and local authority LTPP programme databases was selected for analysis to determine the effects of variables known to cause flushing. The data set consisted of pavement condition data from 58 pavement sections from the state highway LTPP programme and 82 pavement sections from the local authority LTPP programme. The data for the state highway pavement sections has been collected annually since 2001 and the local authority pavement data has been collected annually since 2003. The analysis presented in this chapter used state highway LTPP data from nine survey years (2001 to 2011) and local authority LTPP data from seven survey years (2003 to 2011).

The LTPP database recorded flushing on a section by measuring the area percentage of the pavement surface that was displaying flushing. Flushing was recorded in three severity categories: low, moderate/medium and high. The low flushing severity category consisted of sections that had separate spots of flushing distributed sparsely on the chipseal surface, as shown in figure 3.1a. The moderate or medium flushing severity category consisted of sections with flushing spots joined together and the aggregates visible on the chipseal surface, as shown in figure 3.1b. The high flushing severity category consisted of sections where the aggregates were completely covered by bitumen, as shown in figure 3.1c.

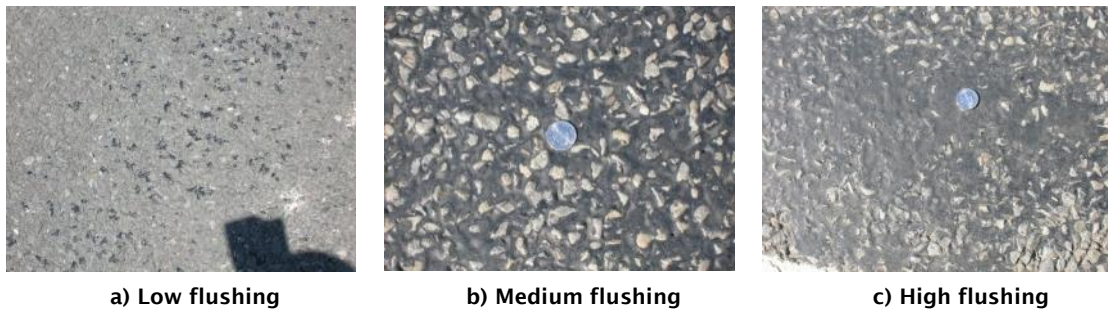
Figure 3.1 Illustration of flushing ratings used in the LTPP database

Table 3.1 shows the pavement condition measures (variables) recorded in the LTPP database, which were investigated as part of the data analysis. These variables had been identified as contributing to flushing in previous research (Alderson 2008; Ball et al 1999; Ball and Patrick 1998; Lawson and Senadheera 2009; Park 2007; Weissmann and Martino 2009). The variables were divided into three categories: pavement composition characteristics, traffic-related factors and the effects of other pavement defects.

Table 3.1 Factors likely to affect flushing

Variable	Description	Type
Pavement composition characteristics		
Surface age (years)	Age of chipseal layer. In the case of multiple resurfacings, the age of the last surfacing was used	Discrete
Surface thickness (mm)	Full thickness of the surfacing layer which has resulted from multiple resealing	Continuous
Grade of aggregates	Grade of aggregates that makes up the chipseal	Discrete
Number of chipseal layers	The number of chipseal layers in the pavement resulting from multiple resealing	Discrete
Material type	Material type of the surfacing layer, eg two-coat seal, void fill seal	Discrete
Polymer modified binder (PMB)	The presence of PMB in the chipseal surfacing layer (0 = no, 1 = yes)	Binary
Traffic- related factors		
Number of vehicles per day (vpd)	Number of vehicles as measured by average annual daily traffic (AADT)	Discrete
Percentage of heavy commercial vehicles (HCVs) (%)	Percentage of vehicles (from total AADT) classified as HCVs	Discrete
Climatic factors		
Temperature (°C)	Average air temperature	Continuous
Rainfall (mm)	Average yearly rainfall	Continuous
Humidity (Hpa)	Mean vapour pressure	Continuous
Pavement defects		
Rutting (mm)	Depth of rutting on the pavement surface	Continuous
Roughness (m/km)	Surface roughness, measured by the International Roughness Index (IRI)	Continuous

3.2.2 Study methodology

The following method was followed in the analysis of LTPP data:

- 1 The first step of the data analysis process was to perform factor analysis to determine the factors (independent variables) that had an effect on flushing. Correlation matrices were created to identify factors (from table 3.1) most suitable for correlation analysis.
- 2 Independent variables were plotted against flushing (dependent variable) using scatter plot graphs. This task was carried out using SPSS Statistics 19 (IBM Corporation 2010). The strengths of the relationship between flushing and each of the identified factors were analysed using Pearson correlation and a one-way analysis of variance (ANOVA) test. The correlation coefficient, R^2 values and standard errors were calculated.
- 3 Partial correlations were carried out on the data sets to investigate the combined effects of independent variables on flushing. This procedure was intended to reveal inter-relationships between factors that caused flushing.
- 4 Regression analysis was then performed on the factors identified from the above steps to determine their strength in predicting flushing. Flushing was modelled in two phases: one model was used to predict whether flushing had initiated on a pavement, and a second model was used to predict the progression of flushing.

3.3 Data analysis results

The following section presents results from factor analysis of variables known to cause flushing as identified in table 3.1. Correlation matrices were used to identify the variables that were most suited to be included in the correlation analysis.

3.3.1 Factor analysis of variables causing flushing

Effects of pavement characteristics, traffic-related factors and other pavement defects

Factor analysis was performed to identify the independent variables that were most likely to impact on flushing and to identify multi-collinearity between variables in order to select the most suitable variables for detailed analysis. The pavement characteristics included in factor analysis were age of the surfacing layer (Surfage) and thickness of the surfacing layer (Surfthick). The traffic-related factors included were traffic volume (AADT) and percentage of HCVs. The other pavement surface defects identified as likely to influence flushing were rutting and surface roughness, where measurements for left wheel path and right wheel path rutting (Rutlwp and Rutrwp) and roughness (LwpIRI and RwpIRI) were selected for analysis. Table 3.2 shows the resulting correlation matrix from factor analysis.

Table 3.2 Correlation matrix for pavement composition and traffic-related factors – LTPP data set

	TotalFlushing	Surfthick	Surfage	AADT	HCV	Rutlwp	Rutrwp	LwpIRI	RwpIRI
TotalFlushing	1.000	0.770	0.474	-0.172	0.307	0.210	-0.170	0.109	-0.214
Surfthick	0.770	1.000	0.309	-0.136	0.210	0.117	-0.241	0.133	-0.157
Surfage	0.474	0.309	1.000	-0.111	0.324	0.078	-0.109	0.129	0.085
AADT	-0.172	-0.136	-0.111	1.000	-0.318	0.124	0.190	-0.026	-0.156
HCV	0.307	0.210	0.324	-0.318	1.000	0.080	-0.165	-0.120	-0.100
Rutlwp	0.210	0.117	0.078	0.124	0.080	1.000	0.436	-0.021	-0.094

	TotalFlushing	Surfthick	Surfage	AADT	HCV	Rutlwp	Rutrwp	LwplRI	RwplRI
Rutrwp	-0.170	-0.241	-0.109	0.190	-0.165	0.436	1.000	-0.203	0.088
LwplRI	0.109	0.133	0.129	-0.026	-0.120	-0.021	-0.203	1.000	0.568
RwplRI	-0.214	-0.157	0.085	-0.156	-0.100	-0.094	0.088	0.568	1.000

As can be seen in table 3.2, both pavement characteristics (surface thickness and surface age) had strong correlations to flushing (TotalFlushing), thus both of these factors were further evaluated in detail. Of the traffic-related factors included in the factor analysis, only HCV showed any significant relationship to flushing. Rutting and roughness also had a correlation to flushing. Thus, all of these factors (shown in red bold font) were included in further analyses.

Effects of climatic factors

The correlation matrix for climatic effects, namely humidity (MeanHumidity), rainfall (MeanRain) and temperature (MeanTemp) is shown in table 3.3. Temperature and humidity showed correlations to flushing and these two factors were investigated further. Rainfall showed a weak correlation to flushing, with a correlation coefficient of -0.029, thus rainfall was not investigated further.

Table 3.3 Correlation matrix for climatic factors

	TotalFlush	MeanHumidity	MeanRain	MeanTemp
TotalFlush	1.000	-0.257	-0.029	-0.483
MeanHumidity	-0.257	1.000	0.251	0.827
MeanRain	-0.029	0.251	1.000	0.078
MeanTemp	-0.483	0.827	0.078	1.000

3.3.2 Correlation ANOVA causing flushing

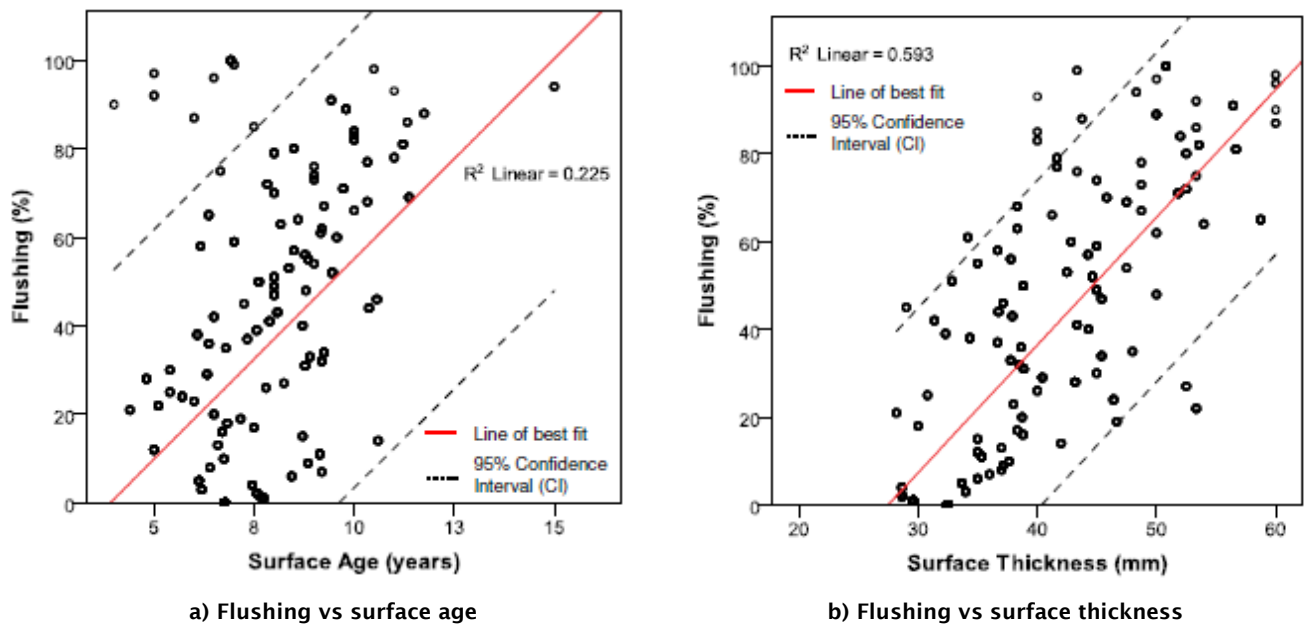
3.3.2.1 Pavement composition characteristics analysis

Surface age and surface thickness

Pavement surface age and surface thickness were identified as being two pavement composition characteristics that influenced flushing greatly. Figure 3.2 presents the correlations between (a) flushing and surface age and (b) flushing and surface thickness. The correlation between flushing and surface age was positive and moderately strong ($R^2 = 0.23$) (Cohen 1988). The correlation between flushing and surface thickness was also positive and strong ($R^2 = 0.59$).

The correlation between flushing and surface age also aligned with past findings, where older chipseal surfaces were found to exhibit more flushing (Alderson 2008; Kodippily et al 2014).

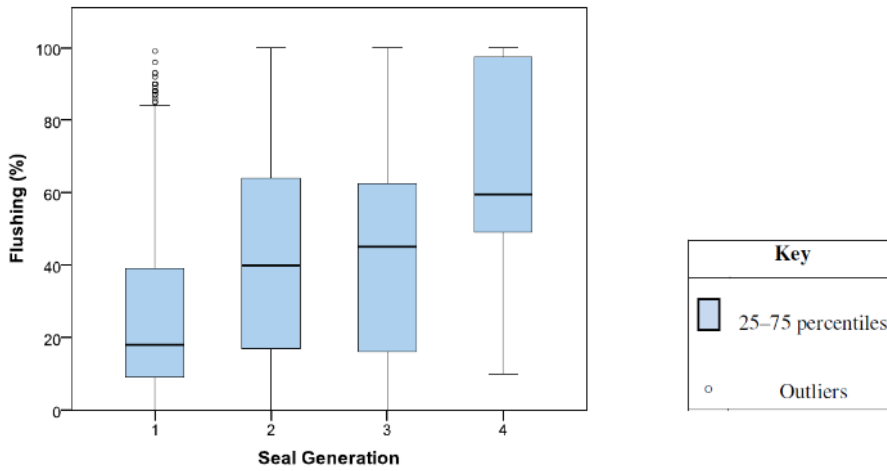
Figure 3.2 Correlation between flushing and pavement surface characteristics



Pavement seal generation and flushing

The pavement sections included in the data analysis were classified into the generation of the seal, which was determined based on the maintenance that had been performed on the pavement sections. The seal generation was assigned as 1, 2, 3 or 4, where 1 indicated first generation seals which were pavements consisting of the original seal from the time the pavement was first constructed, 2 indicated second generation seals which were pavements that had been resurfaced once, 3 indicated third generation seals which were pavements that had been resurfaced twice, and 4 indicated fourth generation seals which were pavements that had been resurfaced three times. Descriptive analysis of the data set showed that surfaces classed as second generation or higher had increased levels of flushing compared with first or new generation surfaces. Figure 3.3 shows the flushing measurements in each seal generation group where there was a clear difference in the mean flushing values between first generation seals and higher generation seals (p value < 0.005). Of the data set analysed for seal generation effects, only 0.50% of data points were identified as outliers.

Figure 3.3 The amount of flushing with respect to seal generation

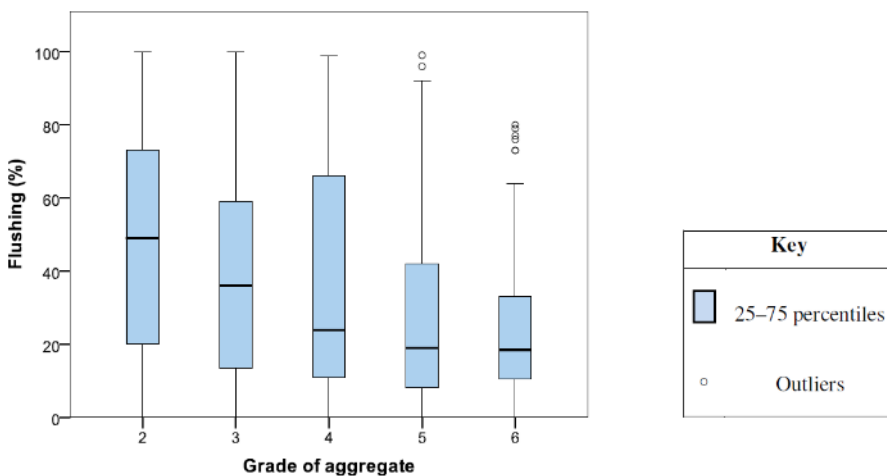


Note: The whiskers extend to 1.5 x inter-quartile range (IQR) from the box
 1 – first generation; 2 – second generation; 3 – third generation; 4 – fourth generation

Flushing and aggregate grade

The aggregate grades making up the LTPP data set were grade 2 (19mm), grade 3 (16mm), grade 4 (14mm), grade 5 (9.5–5.0mm) and grade 6 (6.7–3.0mm). The relationship between flushing and aggregate grade is shown in figure 3.4, and as can be seen, the amount of flushing is lower on surfaces that have smaller sized aggregates. This trend indicated that aggregate size was a factor that needs to be included in the flushing prediction model development. Of the data set analysed for aggregate grade effects, only 0.05% of data points were identified as outliers.

Figure 3.4 The relationship between flushing and the grade of aggregates



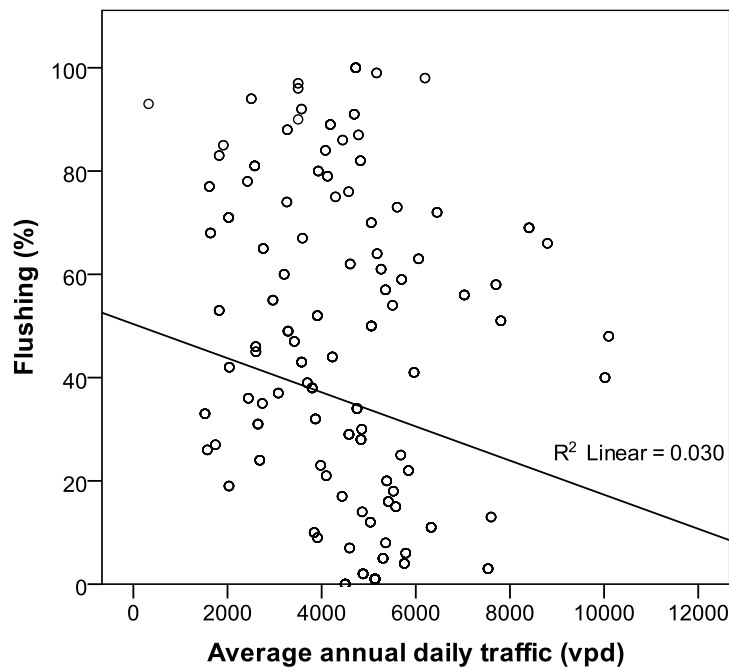
Note: The whiskers extend to 1.5 x inter-quartile range (IQR) from the box

3.3.2.2 Traffic loading and flushing

The correlation between flushing and traffic volume was assessed as shown in figure 3.5. The results show the flushing/traffic volume relationship to be negative, and the strength of the correlation was low ($R^2 = 0.03$) and not statistically significant. The weak correlation between flushing and traffic volume was

unexpected, as traffic volume typically contributes to the deterioration of pavement texture. Partial correlations between flushing and traffic volume, and surface thickness and surface age were tested to determine if pavement composition characteristics had an impact on the flushing/traffic volume relationship. However, neither surface thickness nor surface age was found to impact on the relationship between flushing and traffic volume. In addition to the traffic volume, the effect of HCVs on flushing was also explored as HCV volume was identified from factor analysis to impact on flushing. Correlation analysis revealed a positive correlation between flushing and HCVs, but the R^2 value of this correlation was only 0.09, which indicated that HCV only accounted for a small percentage of the variance in flushing.

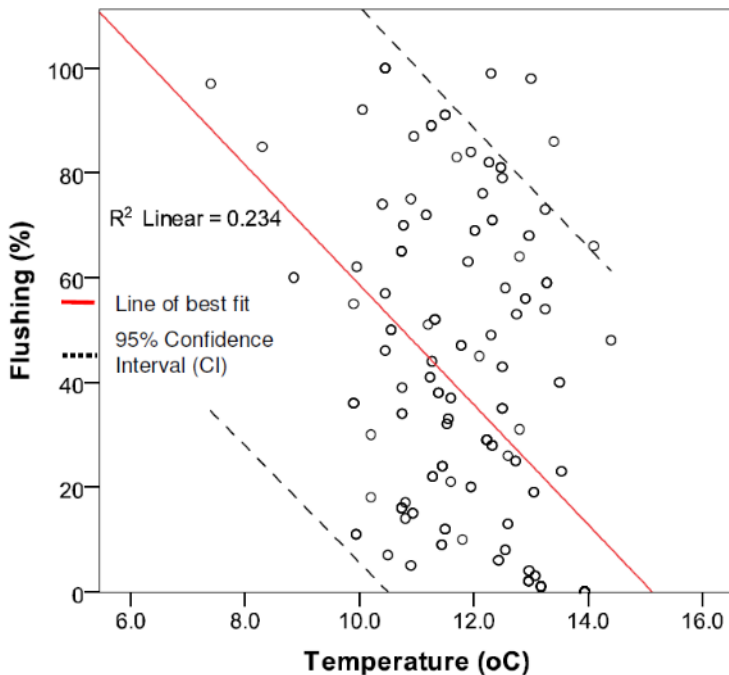
Figure 3.5 Relationship between flushing and traffic volume



The lack of a strong correlation between flushing and traffic volume was attributed to the design of the LTPP sites. The traffic volumes measured on the LTPP sites were between 42vpd and 24,360vpd, and these traffic volumes would have covered the range of traffic volumes expected on these LTPP sites. The LTPP pavements would have been constructed to satisfactorily withstand their expected traffic volumes and as a result flushing development due to heavy loading would have been minimised. If the LTPP pavements were exposed to significantly higher traffic volumes than those observed in the data set, traffic volume could become a significant variable causing flushing. In order to incorporate the effects of chipseal design characteristics, surface aggregate size should be incorporated into regression analyses.

3.3.2.3 Climatic factor analysis

The climatic factors investigated with respect to flushing were air temperature ($^{\circ}\text{C}$) and humidity (measured by mean vapour pressure, HPa). Correlation analysis of flushing and air temperature showed a negative correlation which was moderate in strength between the two variables ($R^2 = 0.23$). This correlation is shown in figure 3.6.

Figure 3.6 Correlation between flushing and mean air temperature

The negative correlation between flushing and temperature was contradictory to what was expected. A positive correlation was expected between flushing and temperature as binder behaviour with respect to temperature indicated that softer binder was associated with high temperatures, which can lead to more flushing. Furthermore, the lack of a stronger correlation between flushing and temperature was surprising, as available literature (Lawson and Senadheera 2009) indicated temperature was likely to have a significant effect on the occurrence of flushing. It was determined that the effects of temperature were being altered to an extent by factors such as the amount of bituminous binder in the seal layers which determines the extent to which binder migration can occur, or the presence of harder binder which can minimise temperature-related flushing. These factors would have been a part of the seal design process which would have assigned binder quantities and grades according to the expected temperatures of a site. As a result of this seal design process, temperature-related flushing would have been minimised. Moreover, it was concluded that the temperature data available in the LTPP database, which presented the average air temperature of the pavement sites, was inadequate at accurately representing the correlation between flushing and temperature, and the temperature of the pavement surface would better represent the effects of temperature on flushing. However, pavement surface temperature data was not available in the LTPP database and so the effects of pavement surface temperature could not be explored in this data analysis. Additionally, the relationship between flushing and humidity was found to have a weak R^2 value of 0.07. The effects of humidity on chipseal pavements would have also been taken into account in the design process of the seals. For the above reasons, temperature and humidity were not included in the regression model.

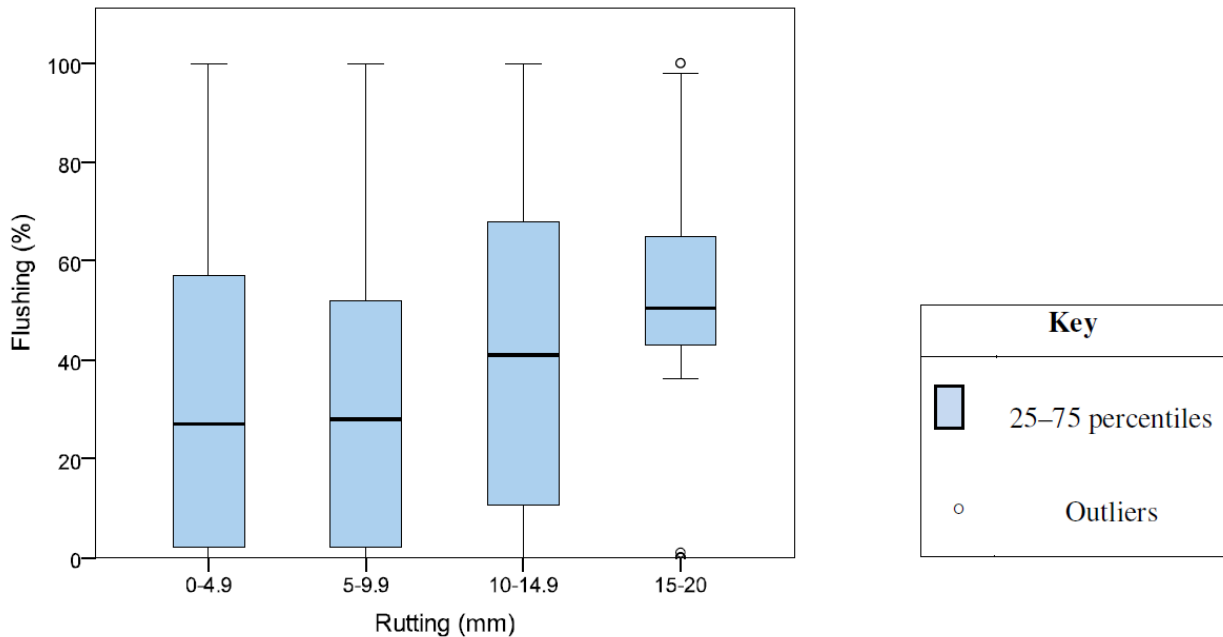
3.3.2.4 Effects of other pavement defects

Rutting

Pavement defects identified to have an impact on the occurrence of flushing were rutting and surface roughness. Correlation analysis for the flushing/rutting relationship revealed a weak positive correlation ($R^2 = 0.04$) between flushing and rutting measurements. Knowledge of pavement behaviour indicates that rutting affects flushing where the presence of large ruts can contribute to binder accumulation, particularly during construction. This creates an area rich in binder, which can lead to flushing occurring

as a secondary defect. To investigate the flushing/rutting relationship further the rutting values were categorised into different groups. The resulting box plot is presented in figure 3.7 which shows an increase in the median flushing values with an increase in rut depths, although the trend only becomes significant when large rut depths are present (greater than 10mm). The result in figure 3.7 confirmed that rutting was a factor contributing to flushing.

Figure 3.7 Relationship between flushing and rutting



Note: The whiskers extend to 1.5 x inter-quartile range (IQR) from the box

Roughness

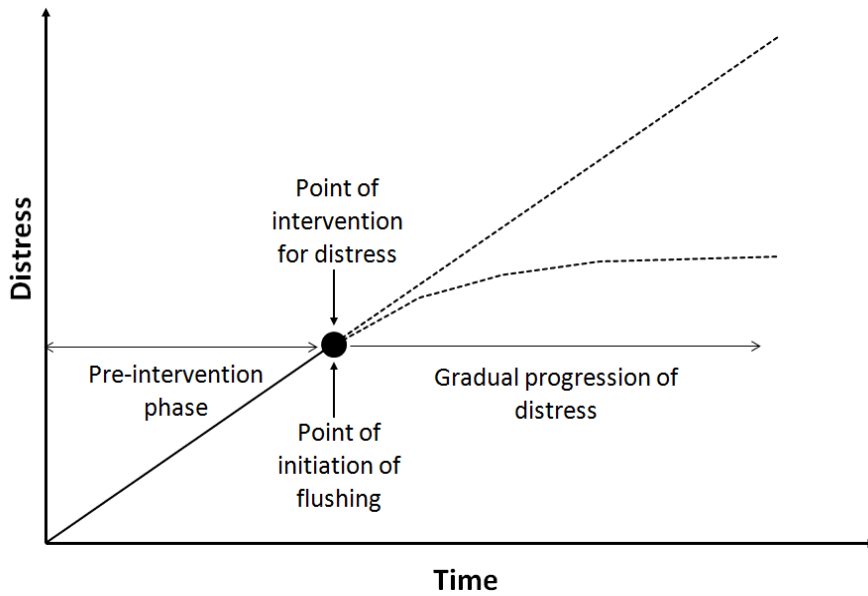
Correlation analysis of the flushing/roughness relationship of LTPP data revealed a weak correlation between the two variables, where the R^2 value of the relationship was 0.05. A stronger correlation was expected between flushing and roughness as a pavement that had not flushed would retain its roughness. It is likely that the obtained result of the flushing/roughness relationship was the product of a lack of severe flushing on all the sites in the data set. Severe flushing would have a more significant impact on roughness than low or moderate flushing; however, only a few sites in the LTPP data set had severe flushing, and most of the sites that were analysed exhibited lower severity flushing. Due to this pattern of flushing in the data set any negative correlation between flushing and roughness was unlikely to be translated in the roughness results.

3.4 Predicting the initiation of flushing

In pavement distress modelling it is important to determine when a distress gets to the point where intervention is needed to minimise the progression of that distress. Identifying the point at which intervention is needed can allow for appropriate monitoring and planning of suitable maintenance methods to address the distress. The point of intervention can vary depending on the type of distress being modelled, such as the instance when the distress first appears on the pavement compared with when the distress reaches an unacceptable level. In the case of flushing, a pavement can have a small amount of flushing and still be structurally sound and safe for use by traffic, as in the 'pre-intervention'

phase in figure 3.8. Once flushing becomes more widespread on the pavement surface so that the safety performance and structural integrity of the seal layers are compromised, the pavement must be monitored for maintenance. Thus, when modelling flushing the point at which intervention is needed can be considered as the point at which flushing initiates (as shown in figure 3.8) and is referred to as the point of flushing initiation.

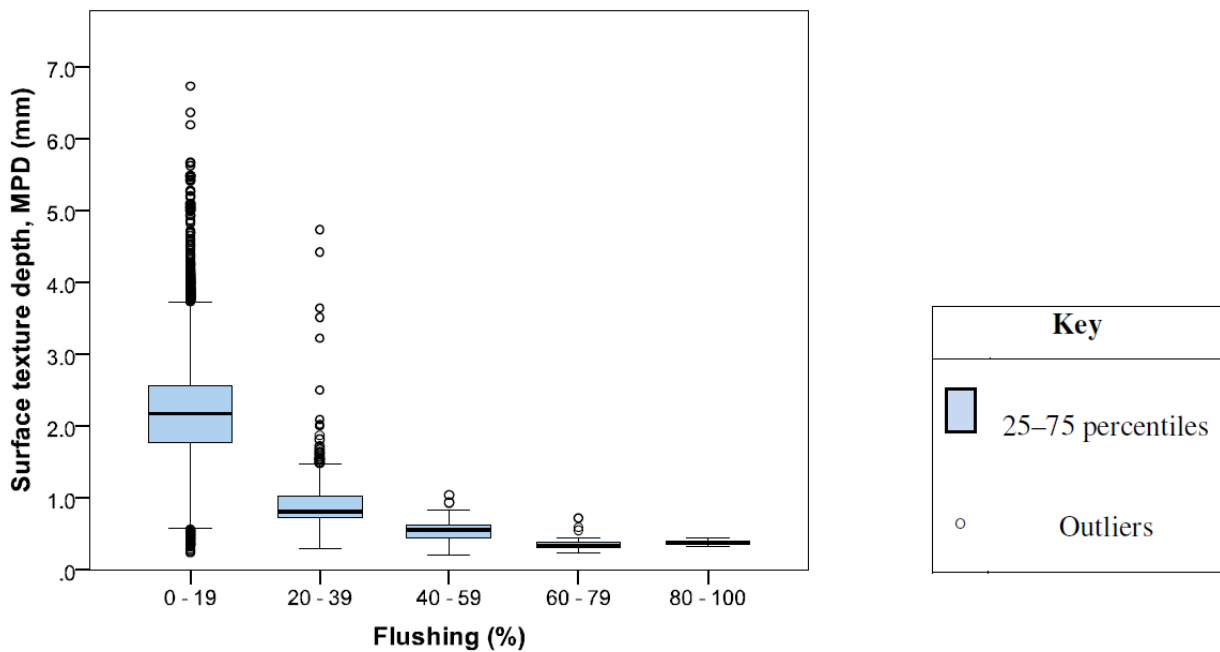
Figure 3.8 Deterioration phases of flushing



3.4.1 Model development

The point of flushing initiation was modelled using logistic regression. The percentage of flushing recorded on each pavement site in the LTPP database was converted to a binary variable, *FlushingInitiated*, where 1 was used to indicate that flushing had initiated on a pavement and 0 was used to indicate that flushing had not initiated. The threshold for the initiation of flushing was determined by examining the relationship between flushing and surface texture depth as given by mean profile depth on pavement surface data collected from the state highway network in the Napier and Hawke’s Bay regions of New Zealand. Historical pavement texture data from six years of surveying (from 2004 to 2009) was collected from the RAMM database and analysed to determine the relationship between flushing and surface texture. This relationship is shown in figure 3.9. The greatest change in surface texture occurred when flushing was between 0% and 20%. When flushing reached higher than 20% the change in surface texture was slower. The point of deterioration of surface texture is an important maintenance trigger which also indicates the presence of noticeable levels of flushing. Hence, a flushing threshold value of 20% was chosen as the point at which flushing initiates on a given pavement site.

Figure 3.9 Relationship between flushing and surface texture

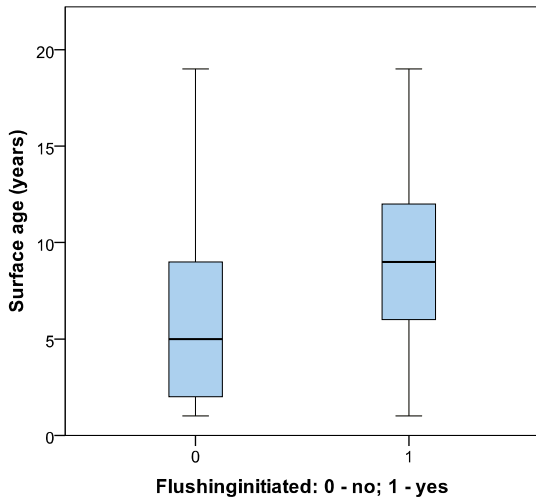


Note: The whiskers extend to 1.5 x inter-quartile range (IQR) from the box

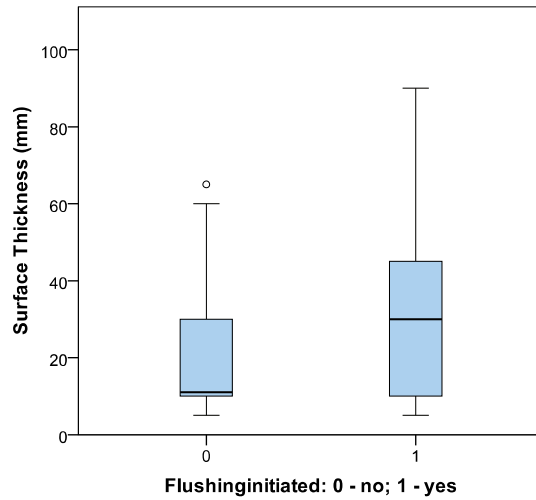
In order to make the flushing initiation prediction model more applicable to a wide range of pavements, the LTPP data from all survey years from the state highway and local authority LTPP databases was combined. The combined data set was then separated into two subsets by allocating the data entries randomly to the subsets and making sure the subsets were similar in size. One subset was used for model development and the other subset was used for model validation. The subset used for the logistic model development contained 827 data entries and the subset used for model testing contained 812 entries.

The variables included in the logistic regression were determined by investigating the effect that each potential independent variable had on flushing initiation. Figure 3.10 shows the effect of surface age (in years), surface depth (in millimetres), rutting (in millimetres) and number of HCVs on flushing initiation. Seal generation as shown in figure 3.10c indicates the generation of the seal, where 1 indicates first generation seals, 2 indicates second generation seals, 3 indicates third generation seals and 4 indicates fourth generation seals. It was clear from figure 3.10 that surface age, surface thickness and rutting had an effect on flushing initiation, where for all three variables the difference in the mean between pavements with and without flushing initiating was statistically significant (p value < 0.001). The number of HCVs was seen to not have a notable effect on flushing initiation.

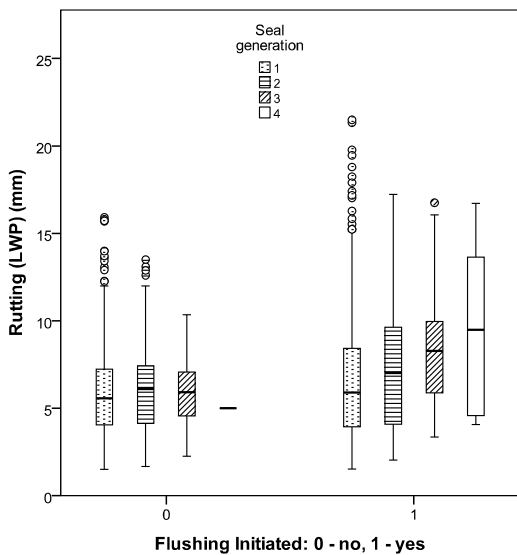
Figure 3.10 Effect of independent variables on flushing initiation



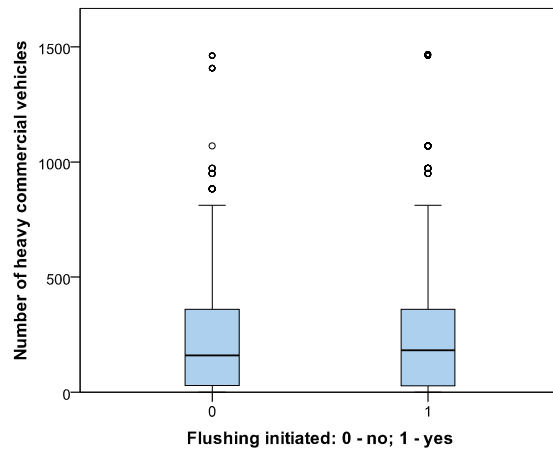
a) Surface age



b) Surface thickness



c) Rutting



d) Number of HCVs

Logistic regression was performed using a forward stepwise method. The independent variables used in the regression were surface age (surfafe), surface thickness (SurfDepth), rutting (RutLANE), and number of HCVs. The model coefficients from the logistic regression are shown in table 3.4, and as can be seen, surface age and surface thickness were statistically significant variables in the model (p value < 0.001). Rutting and the number of HCVs were not included in the final model as the significance of the contribution of rutting (p value = 0.654) and the number of HCVs (p value = 0.347) to the model was low. The contribution of each of the independent variables was as expected, as shown by the positive constants in column 'B' in table 3.4, where flushing was expected to increase with increasing surface age and surface thickness.

Table 3.4 Model coefficients from logistic regression to model initiation of flushing

Variable	B	SE	Wald	df	p value	Exp(B)
SurfDepth	0.046	0.005	77.134	1	< 0.001	1.047
surfage	0.293	0.025	132.594	1	< 0.001	1.340
Constant	-2.913	0.265	120.994	1	< 0.001	0.054

Based on the coefficients in table 3.4 the logistic model for initiation of flushing is as shown in equation 3.1:

$$p(\text{FlushingInitiated}) = \frac{1}{1 + e^{-(0.293\text{surfage} + 0.046\text{SurfDepth} - 2.913)}} \tag{Equation 3.1}$$

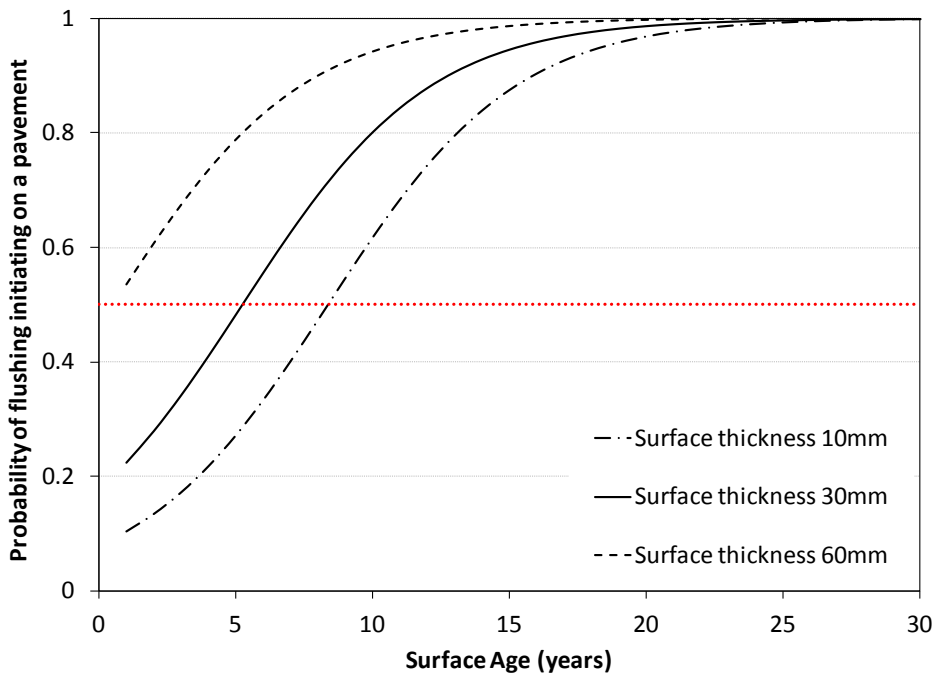
Where, $p(\text{FlushingInitiated})$ is the probability that flushing has initiated

surfage is the age of the pavement surface in years

SurfDepth is the thickness of the pavement surfacing layer in millimetres.

The graphical representation of the logistic model format to indicate the initiation of flushing is shown in figure 3.11, which presents the probability of flushing initiating on a given pavement surface with respect to surface age and surface thickness. The output in figure 3.11 shows that the time at which flushing initiates varies significantly for different surface thicknesses.

Figure 3.11 Model outcome to predict the probability of flushing initiation



3.4.2 Test of logistic model accuracy

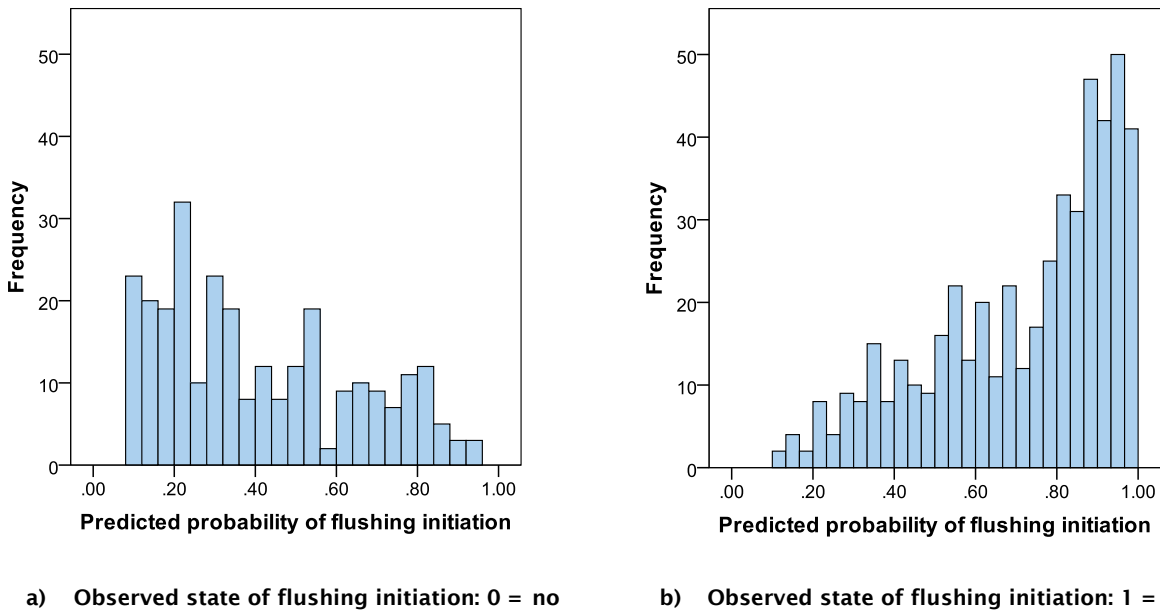
The prediction accuracy of the logistic model is shown in table 3.5. The predicted values for ‘Selected cases’ show the model predictions for the data set used to develop the model, and the predicted values for ‘Unselected cases’ show the model predictions for the data set used to validate the logistic model. The overall prediction accuracy of the logistic model was established by determining the percentage of the model-validation data set for which flushing initiation was predicted correctly. The developed logistic

model is predicting at an accuracy of 76% (75.5% from table 3.5). The model is much more accurate when predicting flushing initiation for a pavement where flushing had initiated (82% from table 3.5) than on a pavement where flushing had not initiated (64% from table 3.5). A graphical representation of the actual and the predicted results of flushing initiation for the data that was used to validate the logistic model is shown in figure 3.12. As can be seen in figure 3.12, the majority of the data entries were predicted accurately by the model, where on the plot of pavements that did not have flushing initiating (figure 3.12a) the bars were clustered to the left side of the plot as expected and on the plot of pavements that had flushing initiating (figure 3.12b) the bars were clustered to the right side of the plot.

Table 3.5 Prediction accuracy of the logistic model for flushing initiation

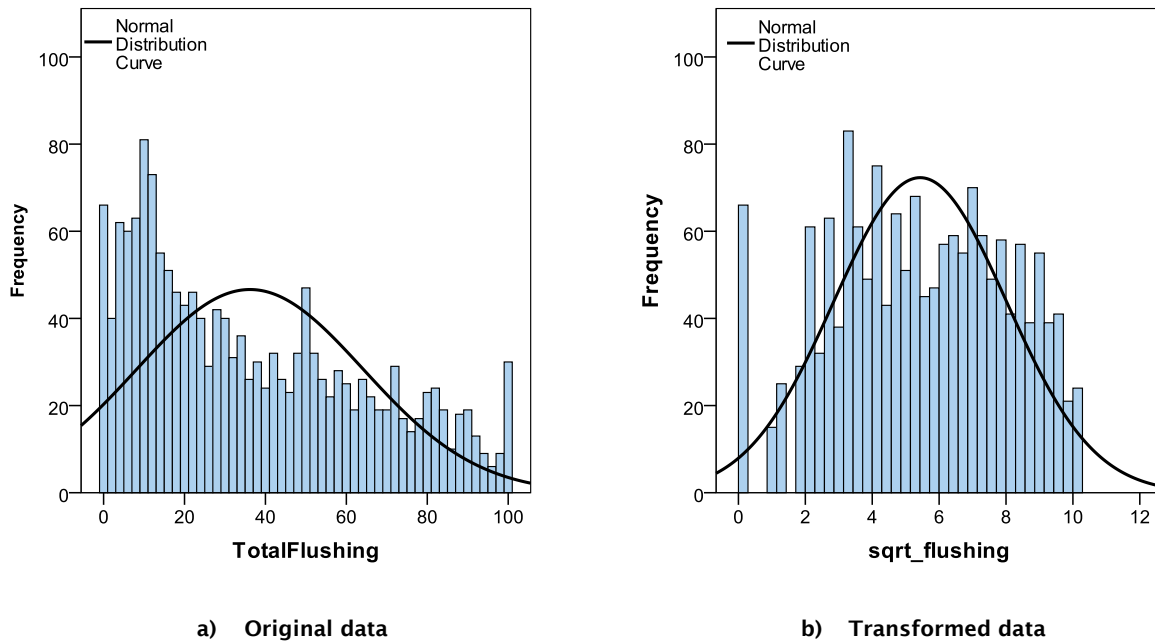
Observed		Predicted					
		Selected cases			Unselected cases		
		FlushingInitiated		Percentage correct	FlushingInitiated		Percentage correct
		0	1		0	1	
FlushingInitiated	0	194	126	60.6	190	106	64.2
	1	109	398	78.5	93	423	82.0
Overall percentage				71.6			75.5

Figure 3.12 Predicted probabilities for flushing initiating on LTPP pavement sections



3.5 Predicting the progression of flushing

The following section presents the results obtained during the development of the flushing progression model. The independent variables used in the development of the flushing progression model were pavement surface thickness, surface age, percentage of HCVs and rutting. The progression of flushing on the LTPP sites was modelled using a forward stepwise linear regression model. Analysis of the distribution of flushing measurements in the data set revealed that the flushing data distribution was right skewed and violated normality assumptions of linear regression (figure 3.13), thus the dependent variable TotalFlushing was transformed by obtaining the square root of TotalFlushing.

Figure 3.13 Distribution of original dependent variable TotalFlushing data and the transformed data

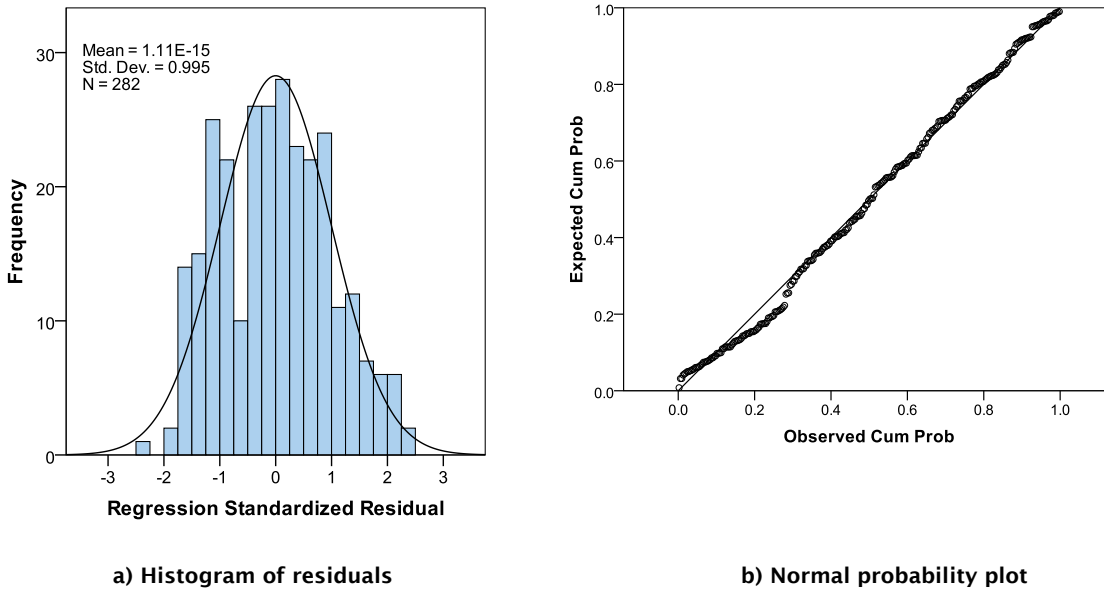
Analysis of flushing measured on seals of different generations revealed there were clear differences in flushing measurements between first generation or new seals compared with second and higher generation seals. Flushing on first generation seals was notably lower than on higher generation seals and this variation was taken into account in the flushing progression model by developing two variations of the linear model. The data set for the analysis was separated based on the surface generation, where set 1 consisted of data from first generation seals and set 2 consisted of data from second and higher generation seals, and two linear models were developed for the two seal generation types. The regression model coefficients for first generation seals are shown in table 3.6. The variables that were found to be significant contributors to the linear model (p value < 0.001) were surface age (surfage), surface thickness (SurfDepth) and percentage of HCVs (pc_heavy). The R^2 value of the linear model was 0.35, which indicated the model had moderate statistical robustness. Analysis plots of the residuals of the linear model for first generation seals are shown in figure 3.14. The normally distributed residuals in the histogram and the normal probability plot showed that normality and linearity assumptions of linear regression were satisfied for the data set.

Table 3.6 Linear regression model coefficients for first generation seals

Model	Unstandardised coefficients		Standardised coefficients	t	p value	95% confidence interval for B	
	B	Std error	Beta			Lower bound	Upper bound
(Constant)	-0.509	0.584		-0.871	0.384	-1.66	.641
surfage	0.280	0.025	0.546	11.080	< 0.001	.230	.329
pc_heavy	0.141	0.020	0.349	6.985	< 0.001	.101	.181
SurfDepth	0.160	0.051	0.155	3.158	0.002	.060	.260

Note: Dependent variable: sqrt_flushing

Figure 3.14 Evaluation of residuals from linear regression for first generation seals



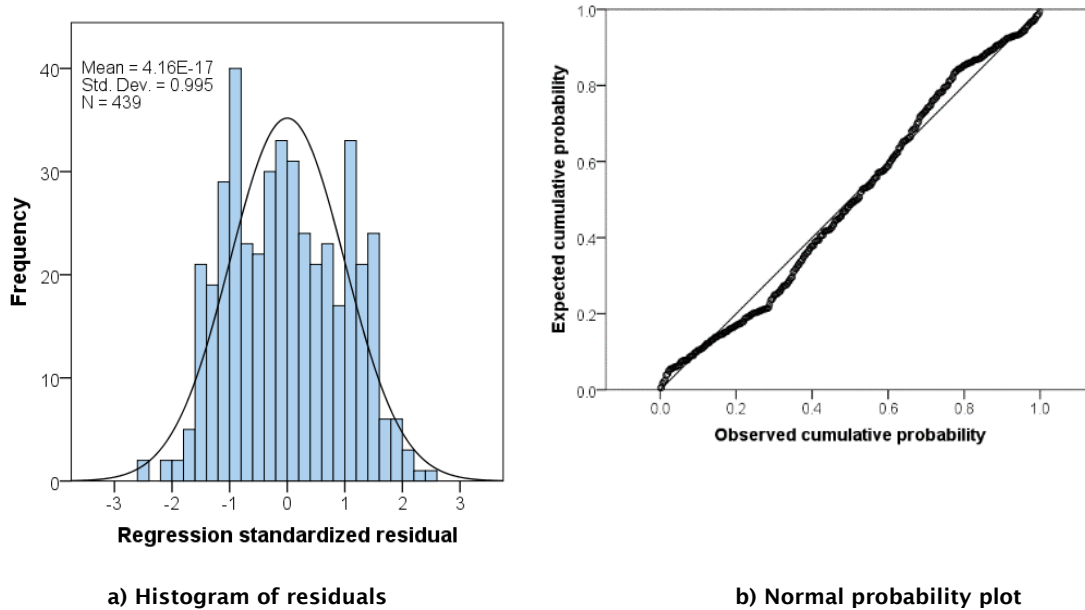
For pavements with seals in the second or higher generation category the linear regression coefficients are shown in table 3.7. The independent variables found to be significant contributors to the linear model (p value < 0.001) were surface age (surface), surface thickness (SurfDepth), rutting (RutLANE) and grade of aggregates (Chipsizes). The R^2 value of the linear model for second and higher generation seals was 0.64, which was statistically robust. Results from the analysis of residuals for the linear model for second and higher generation seals are shown in figure 3.15. The normality and linearity assumptions of linear regression were also satisfied for the second and higher generation seals model.

Table 3.7 Linear regression model coefficients for second and higher generation seals

Model	Unstandardised coefficients		Standardised coefficients	t	p value	95% confidence interval for B	
	B	Std. error	Beta			Lower bound	Upper bound
(Constant)	0.108	0.474		0.227	0.821	-.824	1.04
surface	0.416	0.017	0.742	24.193	< 0.001	.382	.450
SurfDepth	0.040	0.005	0.288	8.453	< 0.001	.031	.049
RutLANE	0.110	0.029	0.111	3.838	< 0.001	.054	.167
Chipsizes	0.170	0.065	0.094	2.615	0.009	.042	.299

Note: Dependent variable: sqrt_flushing

Figure 3.15 Evaluation of residuals from linear regression for second and higher generation seals



The model developed for predicting flushing progression on LTPP pavements is shown in equations 3.2 and 3.3.:

First generation seals:

$$\text{Flushing} = [0.280\text{surface} + 0.160\text{SurfDepth} + 0.141\text{pc_heavy}]^2 \quad (\text{Equation 3.2})$$

Second and higher generation seals:

$$\text{Flushing} = [0.416\text{surface} + 0.040\text{SurfDepth} + 0.110\text{RutLANE} + 0.170\text{Chipsizes}]^2 \quad (\text{Equation 3.3})$$

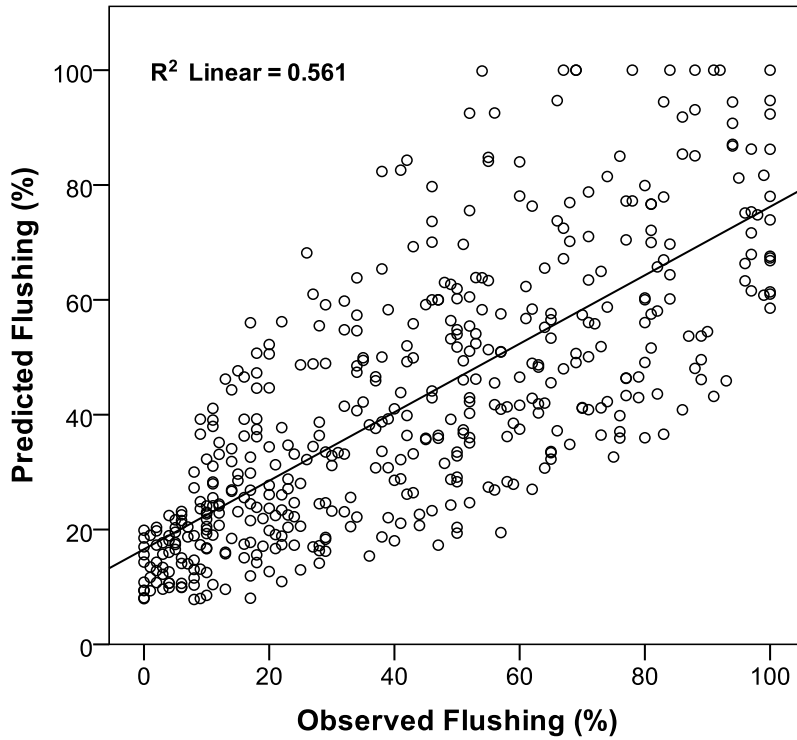
Where,

- Flushing* is the percentage of flushing on the pavement
- surface* is the age of the pavement surface
- SurfDepth* is the thickness of the pavement surfacing layer (in mm)
- RutLANE* is the average rutting on the pavement (in mm)
- pc_heavy* is the percentage of heavy commercial vehicles
- Chipsizes* is the grade of aggregates on the pavement surface.

3.5.1 Testing of flushing progression model

The model developed to predict the progression of flushing was tested using the second subset of LTPP pavement data. As the model for first generation seals only showed moderate robustness, this model was not tested further and testing was only performed for the second and higher generation seals model. Figure 3.16 shows the predicted flushing values against the observed flushing values for pavements with second and higher generation seals. The R^2 value of the relationship of observed flushing and model predicted flushing was 0.56. The linear model had statistically significant strength in its predictions of flushing for second and higher generation seals, as shown in figure 3.16. The R^2 value of the observed versus predicted flushing plot was lower than the R^2 value for the model itself, but given that the model was applied to a completely separate data set, the obtained prediction still showed statistically significant strength.

Figure 3.16 Model predictions for flushing progression on second and higher generation seals



The predictive strength of the model for first generation seals was expected to be low because the mechanisms that cause flushing on first generation seals can vary much more than the mechanisms that cause flushing on second and higher generation seals.

4 Conclusions

4.1 Physical mechanisms

Flushing is a complex process involving multiple physical mechanisms that may be operating simultaneously and contributing to the loss of surface texture in different proportions (which may also change over time). Detailed conclusions have been presented at the end of each section. Overall conclusions are presented below.

Factors making a major contribution to flushing are:

- aggregate abrasion and breakdown leading to a reduction in the size of the sealing chip and the build-up of fines in the seal void volume
- compaction and reorientation of the seal layer under traffic reducing the available void volume in the seal layer
- water venting and sub-surface stripping in seal layers due to water trapped at the seal-basecourse interface and probably principally arising by ingress through the seal surface.

Factors having no or making only a minor contribution to flushing are:

- thermal expansion of the bitumen (the effect is minor)
- excess bitumen application. Use of high, non-standard bitumen application rates will obviously act to fill seal void volume, but this appears to be the exception rather than the rule
- binder viscosity, which at least over the range of standard sealing grades, will have a major effect on bleeding and tracking of bitumen but does not affect the rate of seal texture loss.

There is insufficient information to draw conclusions about embedment into the basecourse. It is recognised that embedment occurs but very little quantitative work on this mechanism has been carried out, and the extent or variability of the effect is unknown.

4.2 Modelling flushing

A revised flushing model was created based on LTPP data to predict the time of initiation of flushing and the trend in the progression of flushing.

The model was found to have an accuracy of 74% when used to predict the initiation of flushing on a separate set of data. The trend in the progression of flushing was modelled using a linear model format and flushing progression was modelled with variations for new or first generation seals, and second and higher generation seals. The linear model was statistically strong (R^2 of 0.45 for new or first generation seals and 0.628 for second and higher generation seals). The developed linear model was tested using a separate set of LTPP data and the model predictions revealed that the developed model was robust at predicting the progression of flushing.

5 Recommendations

5.1 Physical mechanisms

The work presented in this report illustrates the fact that a number of approaches are needed as there is no single simple method to prevent flushing in chipseals:

- 1 A test based on the MD test should be included in the NZTA M/6 specification for sealing chip to control aggregate breakdown based on aggregate source.
- 2 The permeability/drainage of the basecourse needs to be improved to prevent build-up of water at the base of the seal. This may require a rethink of the M/4 grading envelope or simply perhaps more rigorous enforcement of construction requirements.
- 3 The reasons why seals 'leak' need to be urgently addressed. Is it inherent in the technology or can construction practices be changed to minimise leakage?
- 4 Highly polymer, or crumb rubber, modified binders or epoxy bitumen (or similar very high strength thermosetting polymer modified bitumen), may act to minimise chip reorientation and seal layer compaction but this needs to be confirmed through trials. Recent research has shown that compared with conventional binders, thermosetting epoxy binders may have sufficient strength to resist chip embedment and reorientation. Reactive epoxy binders may also react with the aggregate surface and resist sub-surface stripping.
- 5 Build in more seal void volume through appropriate seal design, minimising bitumen application and maximising development of a stone-on-stone skeleton (mimicking open graded porous asphalt or SMA mixes). There is a limit to what can be achieved here as the seal must also waterproof the basecourse and retain chip under traffic stresses.

5.2 Modelling flushing

Data from accelerated pavement deterioration studies using different seal types should be used to investigate the accuracy of the model and improve its performance.

It is recommended that the items of pavement data currently collected as part of the LTPP programme be extended to include data relating to the soil moisture environment of a pavement, particularly the dry density, wet density and water content of soils. The soil moisture data items were identified as an important predictor of flushing from previous flushing modelling attempts but were unable to be included in the modelling work presented here. Having soil moisture data available from future LTPP surveys will likely be very useful in the development of further distress prediction models.

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Appendix A: Effect of cutting seal samples on measured aggregate fines contents

An experiment was conducted to determine if removal of seal samples by coring or subsequent cutting of the specimens to remove adhering basecourse or unwanted seal layers may generate enough fine aggregate material to significantly affect the aggregate grading results.

A 60mm thick, square slab of seal from SH77 with no adhering basecourse was cut in half, as shown below (figure A1.1). One half (B3), was extracted as normal, the other was again cut in half to give samples B3a and B3b. B3a was extracted and B3b was cut into quarters (B3c) and the four sections combined and extracted. The ratio of cut face length was 6:4:6 for B3, B3a and B3c respectively. The ratio of cut face length to unit area was 3 (6/2), 4 (4/1) and 6 (6/1) for B3, B3a and B3c respectively.

Aggregate gradings for the extracted seals are given in table A.1. Differences between the three specimens are small. The difference between B3 and B3c, with the greatest difference in cut face to area ratio is within the range found for replicate extractions and gradings reported in section 2.3. The results indicate that the cutting involved in taking cores and sample preparation does not significantly affect the grading results.

Figure A.1 Cut faces for SH77 seal slab

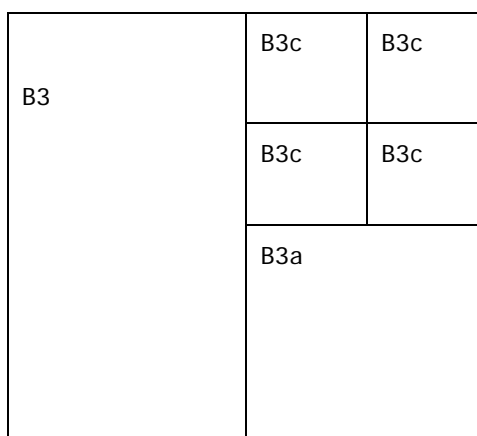


Table A.1 Comparison of aggregate gradings for differently cut seal specimens

Sieve size (mm)	% mass passing			
	B3	B3a	B3c	B3c-B3
13.2	96.9	97.3	97.5	0.6
9.5	59.7	60.2	61.0	1.3
6.7	34.3	34.7	35.2	0.9
4.75	22.4	21.9	22.2	-0.2
2.36	15.8	15.5	15.3	-0.4
1.18	12.5	12.3	12.1	-0.4
0.6	10.1	10.0	9.7	-0.4
0.3	7.8	7.7	7.4	-0.4
0.15	5.6	5.6	5.3	-0.3
0.075	4.0	4.0	3.7	-0.2

Appendix B: The Micro- Deval (MD) test

The MD test was developed in France from the earlier Deval test in the 1960s (Tourenq 1971; Mahmoud and Masad 2007). The test is standardised both in the USA (ASTM 6928-10 *Resistance of coarse aggregate to degradation by abrasion in the Micro-Deval apparatus*) and Europe (EN 1097-1:2011 *Tests for mechanical and physical properties of aggregates. Part 1: Determination of the resistance to wear (Micro-Deval)*). The two specifications differ slightly (but significantly) in terms of the sample size and grading of the test aggregates used. Many other test variants also exist including those used in Quebec and Ontario (Rogers 1998).

The general principal is that the test aggregate is soaked in water for a period then rotated in a drum with water and 5kg of 9.5mm steel balls for two hours at 100rpm. The fine material produced is weighed and the loss from the test sample expressed as percentage of the initial sample weight.

In the European standard method the test aggregate is 10 to 14mm size, in the US method three different sample sizes are allowed (9.5 to 19mm, 4.75 to 12.5mm, or 4.75 to 9.5mm) depending on the nominal largest aggregate size. In the European method aggregate fines passing a 1.16mm sieve are used to calculate the percentage lost, in the US method it is a 1.18mm sieve. These differences prevent direct comparison of results from the two the methods. The two methods are compared in table B.1 together with the Opus Research modification used in the current work (see section 2.3.5).

The MD test is closely related to the Los Angeles abrasion and impact test (AASHTO T96); in fact the LA test was derived from the original Deval test in the 1920s (Fowler et al 2006). The LA abrasion test involves rotating 5kg of aggregate with 12 46.8mm diameter steel balls at 30rpm for 500 revolutions. Material passing a 1.7mm sieve is calculated as a percentage loss. The test creates much more breakdown of the aggregate than the MD test and is carried out in the dry. The LA abrasion test is the most commonly used aggregate strength test in the USA (Gransberg et al 2010).

Table B.1 Comparison of the ASTM, EN and Opus Research versions of the MD test methods

Property	ASTM D6928	EN 1097- 1	Opus Research
Steel balls used	5kg , 9.5mm dia.	5kg , 9.5mm dia.	25 x 9.5mm and 1 x 53.9mm
Water soaking time	>1 hr at 20±5°C	None	1 hr at 20±5°C
Container size	Internally smooth cylinder, 198mm id x 173.5mm internal height	Internally smooth cylinder, 200mm id x 154mm internal length	Cylinder with protrusions, 198mm id x 173.5mm internal height
Aggregate sample weight	1,500g	500g	300g
Sample grading (sieve size and weight)	19.0–16.0mm 375g 16.0–12.5mm 375g 12.5–9.5mm 750g Or 12.5–9.5mm 750g 9.5–6.3mm 375g 6.3–4.75mm 375g Or 9.5–6.3mm 750g 6.3–4.75mm 750g	10–14.0mm With 30–40% passing 11.2mm Or 60–70% passing 12.5mm	9.5–13.2mm (NZTA M/6 grade 3)

Property	ASTM D6928	EN 1097- 1	Opus Research
Test conditions	120min, 105min or 95min at 100rpm depending on sample grading, 2L water	120min at 100rpm, 2.5L water	27min at 100rpm, 0.39L water
Sieve size for fines produced	1.18mm	1.16mm	Gradings compared

B1 Applications of the MD test

B1.1 Asphalt mix

There has been a significant amount of work on the MD test in Canada and the USA. The test, its precision, correlation with other aggregate strength and weathering tests (such as the LA abrasion test and sodium sulphate soundness test) and relationship to field performance in asphalt mixes and for coarse aggregates used in basecourse have been reviewed by a number of authors (Cuelho et al 2007; Wu et al 1998; Fowler et al 2006).

Most studies have found the MD test to be a good indicator of aggregate field performance (in asphalts) relative to other strength and durability test procedures (Rogers et al 1991; Richard and Scarlett 1997; Fowler et al 2006; Brandes and Robinson 2006; Rangaraju et al 2005; Tarefder et al 2003; Kandhal and Parker 1998; Wu et al 1998; Rogers 1998). For example Cooper et al (2003) found the MD tests had a 64% success rate in predicting performance based on 104 Ontario aggregates. Lang et al (2007) found the MD test to distinguish good aggregate from poor ones with 69% accuracy. In a study of 20 different aggregates Hossain et al (2008) found that the MD test differentiated good and bad performing aggregates at least 70% of the time. However, a few studies have found the MD test not to be a good predictor of field performance (Cooley et al 2002; Hunt 2001).

A caveat with these studies is that 'good' or 'bad' performance in terms of aggregate strength and durability is assessed qualitatively based on practitioner experience with behaviour in the field. Such an approach is usually satisfactory for identifying extreme behaviour (good or bad) but is less effective in detecting small differences between materials.

The precision of the MD test is good compared with other aggregate strength and durability tests such as the LA abrasion test and the sodium (or magnesium) sulphate tests. An inter laboratory study on 58 different materials using a Canadian (Ontario) variant of the test (500g sample size) had a coefficient of variation of 3.2%, inter-laboratory studies with eight laboratories gave coefficients of variation ranging from 2.5% to 10% depending on the mass loss – presumably using the Ontario method (Rogers et al 1991; Rogers 1998). Another study (using AASHTO T327, which is equivalent to the ASTM D 6928 MD test), found for 52 aggregate sources a coefficient of variation of 2.8% (Jayawickrama et al 2007).

A2.2 Chipseals

There is very little published on the use of the MD test to assess the strength and durability of sealing chip. In Britain researchers have compared the MD test to the older aggregate abrasion value (AAV) test (EN 1097-8 *Tests for mechanical and physical properties of aggregates*). The AAV test has been used in Britain for many years to assess the strength properties of surfacing aggregate in chipseals and hot-rolled asphalt. The method is also used in Canada.

Aggregates 10.4–14mm size are mounted in a flat tray using resin, with the flat side uppermost (in the same way that specimens for the PSV test are prepared). The specimens are pressed against a rotating

steel disc and an abrasive sand is introduced to grind away the surface of the aggregate. The mass loss from the specimens is used to calculate an AAV value.

A comparative study of 26 UK aggregate sources using the AAV and MD tests concluded that the MD test was to be preferred (Woodside and Woodward 1998). The MD test uses a wet aggregate which is more realistic than testing dry aggregate especially as the presence of water had a significant effect on losses compared with the tests run in the dry condition. The observation that wet aggregates are more susceptible to abrasion losses has also been made by other authors even as long ago as 1929 (Lovegrove et al 1929). The MD test was also found to be more sensitive to the presence of small proportions of weak particles in otherwise strong aggregate mixtures. The correlation between the AAV and MD test results was not particularly good ($r^2 = 0.64$). Similarly in Canada, Rogers (1998) found that the MD test did not correlate well with the AAV test for asphalt aggregates and the MD test was superior to the AAV in predicting field performance.

Gransberg et al (2010) suggested that MD test could usefully be included in a chipseal aggregate specification (in addition to the LA abrasion test) on the basis that it simulated abrasion under wet conditions in the field; however no data on the relationship of the MD test results to field performance was presented. The MD test results did not correlate well to the LA abrasion test results, which was consistent with earlier findings (Cooley et al 2002).

Appendix C: X- ray tomography study of seal layer air voids

In 2014, a study was undertaken to measure the change in air void composition of multi-layer seal cores before and after tracking. The study has been published in the *Journal of Computing in Civil Engineering* (Kodippily et al 2014) as 'Computed tomography scanning for quantifying chipseal material volumetrics'. The study's abstract is included below.

Abstract

In the reported study the viability of using Computed Tomography (CT) scanning for assessing flushing defects in thin sprayed seal (chipseal) surfacings was explored. The study was undertaken to investigate the micromechanical interactions that occur within chipseal layer materials in order to examine their relationship to the origination of flushing, using CT scanning techniques. In particular, the effectiveness of using image analysis techniques to analyse the changes in air voids that occur within a chipseal layer during loading was investigated.

The presented study was based on laboratory testing of chipseal pavement samples (cores) from inservice pavements in the Auckland and Waikato regions of New Zealand. The cores, of 200 mm diameter and thicknesses ranging from 32.4 mm to 44.5 mm, were subjected to varying levels of lateral cyclic loading using a wheel tracking machine and the deformation that had occurred on the surface of the cores was measured. Two small specimens were extracted from each loaded core, one specimen from the wheel tracked area of the core and the other specimen from the untracked area of the core. The specimens were scanned using a CT scanner and the resulting scan images were analysed using image analysis techniques to determine the distribution of air voids within each specimen. The air voids within the tracked and untracked specimens of each core were compared to examine the changes that had occurred to the distribution of air voids during loading. The results from the study showed that image analysis is an effective tool to analyse air voids within a chipseal layer.