Workshop Outcomes

1 INTRODUCTION
This paper records the Author’s view of the outcomes of the recent workshop in so far as they address the aim - to establish a fundamentally admissible mechanistic approach to the design of unsealed or thinly-sealed pavements. The accompanying ‘Briefing Paper’ was produced before the workshop and includes a review of the background, a definition of ‘mechanistic’ approach and a discussion of the elements which must be addressed correctly. It discusses a list of 23 failure mechanisms. These mechanisms are listed with others (see below) in the ‘Table of Mechanisms’ document. For each, some notes are provided indicating the methods (and associated difficulties) for:

- measuring relevant properties before design commences (typically in a laboratory test),
- measuring relevant properties in-situ either to check construction quality or to evaluate pavement condition,
- choosing an appropriate design criterion to associate with the failure method,
- setting an appropriate value for that criterion.

Whilst that briefing paper contained some details and sought to cover all the salient points, inevitably, it proved to be incomplete. Therefore, several parts of this paper go beyond the abstracting of information presented at the workshop and seek to elaborate somewhat.

The original list of 23 failure mechanisms has been amended a little and is summarised in Table 1. In order to maintained the original numbering, the 4 additional mechanisms identified have been given numbers starting at 24, but are placed in the Table at their logical positions. Other mechanisms may now bear revised descriptions.

2 FAILURE
Failure must be carefully defined for thinly-sealed and unsealed pavements. Most failures occur when performance limits are exceeded, i.e. the pavement becomes too rough, the ruts too deep, the skid-resistance too low, etc. Thus failure criteria must be set in terms of a permissible deterioration in serviceability rather than in terms of a critical stress or strain and are better known as ‘Performance Limit Criteria’. Furthermore, these criteria must not be broached until the pavement has carried the requisite number of vehicle loads. Such failure modes require that there is an understanding as to:

a) how the distress is generated by the loading, and,
b) very importantly, the manner in which it develops with time.

The first of these may, well, necessitate an understanding of structural failure mechanism. An increasing surface rut presents itself to the user (and pavement owner) as a serviceability issue, whereas its cause is probably structural inadequacy. So, although failure is not ultimately defined in terms of structural collapse, structural explanations and assessments are likely to be needed to compute whether the pavement can deliver the required performance.

However the second of the above points is equally necessary. The pavement designer or evaluator needs to know the pattern and scale of deterioration. For example, some pavements decay very slowly before a rapid final phase of serviceability loss (Fig 1, line C), while others deteriorate most rapidly in early life (Fig 1, lines A and B).
Table 1
Failure mechanisms considered

<table>
<thead>
<tr>
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<th>Description</th>
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<tr>
<td>1</td>
<td>Excess resilience of pavement (see also Mechanisms 6 and 7)</td>
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<td>2</td>
<td>Excess rutting from within granular layer due to granular material shear displacement</td>
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<td>3</td>
<td>Excess rutting from within granular layer due to compaction by traffic loading</td>
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<td>Excess rutting from within subgrade layer due to subgrade shear displacement</td>
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<td>Excess rutting from within subgrade layer due to combined action of subgrade and granular layer(s) due to complex stress interaction effects</td>
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<td>Excess rutting from within subgrade layer due to combined action of subgrade and granular layer(s) when subgrade is too resilient</td>
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<td>7</td>
<td>Pumping of subgrade into base course</td>
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<td>Frost action on susceptible subgrades or granular materials</td>
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<td>Wear due to stone displacement by tyre (‘gravel loss’)</td>
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<td>Wear due to stone ‘loss’ into soft subgrade (‘gravel loss’)</td>
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<td>Wear due to erosion of surface metalling by water (‘gravel loss’)</td>
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<td>Wear due to stone abrasion / attrition</td>
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<td>Wear due to studded tyre action</td>
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<td>19</td>
<td>Seal breakage due to traffic-induced flexure</td>
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<td>20</td>
<td>Seal breakage due to environmentally-induced shrinkage (thermal cracking)</td>
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<td>21</td>
<td>Seal breakage due to shoving / tearing / shearing</td>
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<tr>
<td>22</td>
<td>Inadequate surface condition- sealed surface too smooth due to aggregate texture loss</td>
</tr>
<tr>
<td>23</td>
<td>Inadequate surface condition- unsealed surface too slippery due to excess fines on surface (wet weather)</td>
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NB Shaded mechanisms largely set aside by workshop - see Section 3.1 and following
2.1 Performance Limit Criteria

Because the serviceability of the pavement is the user requirement, performance limit criteria must be set in the same terms - e.g. a permissible rut depth of x mm after \( N \) ESALs. However, when a structural failure mode underlies the deterioration in serviceability (as in the case of rutting) it is convenient to relate the tolerable loss in serviceability to a stress, strain or displacement measure at the same place in the pavement. Convenient, but not necessarily appropriate!

The subgrade strain criterion is an example of a convenient but fundamentally inappropriate criterion. This criterion uses resilient vertical strain at the top of the subgrade as a means of limiting rutting in the pavement as a whole. This is convenient because this strain may be readily computed using an elastic analysis of the pavement. However it is inappropriate because:

a) the resilient properties of unbound pavement materials are not closely related to the permanent deformation characteristics of the same materials.

b) even if they were, the criterion is set in one material, the subgrade, whereas rutting occurs as a consequence of plastic strains in all pavement layers. The ratio of plastic strains in the subgrade to strains in other layers depends on the relative thicknesses and relative properties of the materials.

c) the strain is assessed at one place whereas the rut results from the integration of induced stain over the full depth of the strain field. It might be argued that the two are simply related except that the modular ratio between the base and subgrade changes from pavement to pavement meaning that the pattern of stress changes and, hence, the pattern of strain change with depth in the subgrade is not constant from pavement to pavement.

d) the value of the criterion is defined in the form:

\[
\varepsilon_v = \left( \frac{A}{N} \right)^{B} \]

yet there is no mechanistic way of relating values \( A \) and \( B \) to the permissible rut depth so that, if a different rut depth is allowed, another empirical relationship must be developed.

Nevertheless some simplification is likely to be inevitable and, to a degree, some of the above criticisms may apply to more appropriate criteria. Thus a limit on shear stress in the base might be set as a certain percentage of the ultimate shear strength of the aggregate that makes up the base, as a means of precluding excess rutting within the base. Taking the same criticisms as above a) to d):-

a) Shear strength does have a closer relationship to permanent deformation - but it is not precise.

b) This criticism is satisfied - the criterion is set in the material where the failure mechanism is located.

c) To ensure that the criterion was assessed at the correct place, it would be necessary to evaluate the peak shear stress at many different points within the base and ascertain that the limit was satisfied at all points.

d) This criticism still stands, presumably the permissible percentage of ultimate shear strength must be changed if a different number of loadings is needed - but there is no mechanistic way of making such a change.

Furthermore, this more appropriate approach has only addressed one layer. The risk of rutting might be greater due to excess plastic subgrade strain - thus another criterion must, additionally, be selected which relates to this possible cause of premature pavement rutting and must then be evaluated to preclude performance failure originating from the subgrade layer. Perhaps there are other criteria that should also be selected and evaluated to cover other possible causes of pavement failure, as well.
2.2 Restriction Criteria

For any particular mode of pavement distress, an alternative definition of an objective, enumerated criterion is a limit value that precludes the failure type entirely. Thus, instead of requiring a relationship of the type shown in Equation 1 above, a maximum permissible stress or strain value is defined independent of the loading or pavement structure. This is a reasonable approach where it is economic to do so. It is normally taken when the distress mode is relatively easy to avoid as a consequence of material selection (or pavement design) made for other reasons. The effect is to ensure that pavement serviceability is limited by some other distress.

An example of such an approach is the adoption, in the UK, of a requirement that no material used in the pavement within 750mm of the pavement surface shall undergo a heave of greater than 12mm in the British Standard Frost Heave Test. With a relatively mild climate (compared to parts of N. America, Scandinavia and Russia), it is argued that this requirement doesn’t exclude too many materials and effectively removes consideration of frost heave from the design agenda. It is no longer necessary to compute the amount of frost penetration which might occur, nor the number of days it will remain in the pavement, because the material in the zone which might be frozen isn’t able to heave to any great degree.

Clearly, this ultimate limit type of criterion couldn’t be applied to all failure modes without risking gross over-design. However, something of that idea is inherent in the “perpetual pavement” idea and designing for resilient shakedown also takes this kind of approach (Newcomb et al 2001; Nunn & Ferne, 2001).

3 REVIEW OF FAILURE MECHANISMS

3.1 Excluded Mechanisms (outside scope)

Twenty three failure mechanisms were listed in the initial review, Table 1. Of these, seven were discounted as being beyond the immediate scope of this study. These included those involving individual particle wear (e.g. in generating dust or in losing surface skid resistance - Mechanisms 13, 15, 16 & 20) caused by direct contact with the vehicle tyre. It is argued that:

a) a mechanistic approach to describing such deterioration is not, at the present state of knowledge, possible, nor is it likely to be in the near future,

b) stone hardness measures (such as polished stone value) are available as a well-established empirical guide to particle deterioration (though not completely indicative),

c) they are completely non-structural and, hence, have no influence on the other failure mechanisms which are of interest.

Nevertheless, Mechanism 13 (dust loss) is of real concern to designers, especially as environmental and safety issues become more important, and a fundamental understanding of this problem will, increasingly, become a research priority.

Those discounted also included:

a) loss of surface condition due to “fatting up” (bitumen rising from chip seal layer(s) to the surface) - Mechanism 21,

b) excess fines on the surface causing wet-weather slipperiness - Mechanism 22,

c) excess loose stones on the surface giving inadequate skid resistance - Mechanism 23.

The last of these is believed to be addressed by limiting the ‘gravel’ loss problem (Mechanism 14) while the other two should be addressed by appropriate mix proportioning. ‘Fatting-up’ has been studied for asphalt concrete mixtures but seems to occur in chip seals as a consequence of multiple seal applications. Whilst the mixture appropriate for initial seal coats may be rather rich in bitumen (given the adherence of the bitumen to the adjacent un-bitumised layers), this proportion is
frequently too high for multiple seal layers. Then the layers combine to form, in effect, an asphalt
with a single-sized aggregate grading which compacts with time forcing bitumen to the surface.

3.2 Mechanisms Subject to Restriction Criteria

Of the remaining mechanisms two further were excluded from mechanistic design requirements on
the basis that a ‘restriction criterion’ would apply. Pumping of subgrade fines into the base
(Mechanism 7) should be avoided by the use of geo-synthetics. Analytical calculation of the stress-
strain conditions which would pertain at the base-subgrade interface could be performed but there
seems no clear understanding as to the values which would give rise to concern nor how these
values would need to be set for different soil types. Thus the present approach of identifying likely
susceptibility on the basis of soil plasticity, and then providing a separating geosynthetic on the
basis of the potential fines migration which must be halted, seems a practical manner of approach,
for the time being. Clearly, this is an area in which further research could be taken. However, the
occurrence of this mechanism, except on a few well-known ‘problem’ soils is not common so the
present approach will probably satisfy most users.

The second mechanism listed was similarly excluded after some discussion. Rutting which occurs
as a consequence of base compaction (Mechanism 3) would suggest that inadequate compaction
was provided by the constructor. Thus a ‘restriction criteria’ can be set of x% of the density
achieved by a standard amount of compactive effort (as is typically done). The problem identified
with this approach is that the value appears to be non-constant for different materials. Some reach
their fully-compacted state easily, some with difficulty (where ‘fully-compacted’ means the density
achieved in the standard test - usually the modified Proctor method). Thus a value of say x=97%
might require a lot of effort on site so that traffic-induced compaction cannot be expected to cause
much more, or it could be easily achieved and further compaction under traffic, therefore, rather
likely. Furthermore, the constraints offered by laboratory moulds means that in-situ densities may
not relate very closely to the laboratory values. For this reason a site trial and a refusal density
compaction (to define the maximum density achievable and at what degree of ease) should always
be considered.

3.3 Fracture of seal coats

Mechanisms 17,18, and 19 concerning the fracture of seal coats were reckoned to be difficult to
address mechanistically at this time. Layered elastic analyses will not give sensible stress and
strain values for the seal coat as these layers do not fail in bending. More believable stress and
strain values can be obtained from finite element analyses, but obtaining representative material
properties for the seal layer remains challenging. ‘Superpave’ binder testing can help to assess
these, but the linkage between the component properties and the layer property would need
considerable study before they could be reliably defined. Finally, environmental loading would
need to be defined (Mechanism 18). Whilst air temperature in any particular location may be
determinable with a reasonable degree of accuracy, the conversion from this to seal-coat
temperature and to associated aging effects needs more study.

3.4 Subgrade Heave and Shrinkage

Mechanism 12, soil heave and shrinkage, did not receive much attention at the work-shop - perhaps
because of its relatively low significance in most locales, (though in certain places around the
world it is of major concern e.g. parts of Texas). However this rating of significance was a subject
of interest and is discussed later, in Section 9.

3.5 Frost Action

Frost heave (Mechanism 11) is a mechanism of concern in colder climates. When the climate is
not too cold, design can be undertaken to avoid the issue by constructing with non-susceptible base
materials (a restriction criteria approach) together with the use of either sufficient base thickness or,
where sub-grade is non susceptible, with susceptible material placed over capillary break. When a
subgrade is susceptible and the frost penetration is too great to avoid heave by placing a thick base
layer, the strategy adopted is to attempt to obtain subgrade and base uniformity both with respect to material (grading, compaction, etc.) and with respect to condition (water table, depth of cover). This will help to ensure that all heave takes place uniformly. As it is differential heave which causes most problems, overcoming this issue should solve most heave problems. Frost penetration can be predicted analytically by heat flow models, but the variability in climate from year to year makes local prediction maps based on a mixture of empiricism and computation likely to be just as reliable, except in very localised situations.

Two other frost actions are now introduced. The first is free-thaw damage of stones by which individual stone break due to repeated cycling of temperature - induced stresses (Mechanism 24). Once again this is a mechanism unsuited to a mechanistic analysis. Instead a free-thaw test on aggregate particles can be selected and a restriction criteria set to avoid this kind of aggregate damage.

Finally, and more importantly in heavily frost-affected areas, is the issue of Spring thaw (Mechanism 25). Computer methods are available to predict Spring-thaw moisture with time and such information could be linked with data on material variability with moisture content. At present most affected pavements operate on a load-reduction arrangement during Spring-thaw. However, better estimation of the pavement condition during the thaw and hence, the possibility of estimating the damage which might be caused by traffic at this time, would allow the pavement owner to charge on a damage-caused basis rather than imposing a no-travel ban on certain vehicles. Of course, this presupposes that damage due to a vehicle passage can be computed for a chosen pavement condition. This is considered later.

3.6 Resilience

The problems with excessive resilience (Mechanism 1) have already been listed in the briefing note and the workshop added little to this discussion. The effects of excessive resilience in decreasing transportation efficiency had not, necessarily, been previously recognised. Readers’ attention is directed to the recent study by Jamieson et al (2002) of Opus Labs in Wellington and older work by Douglas and Valsangkar (1992). Internationally, there is a drive to greater energy efficiency for environmental and economic reasons so, for this reason, reducing the flexibility of pavements may become more important.

3.7 Rutting

Excessive rutting has been, and continues to be, one of the chief concerns of those responsible for engineering thinly sealed and unsealed pavements. There is increasing recognition that rutting within the aggregate layer is an important element of the overall distress (Mechanism 2). The shakedown approach, which seeks to obtain a pavement which does not undergo long-term rutting but settles down to a behaviour which is entirely resilient, is very attractive. The early work by Sharp (1983,1987) derived the limits to the imposed stress (for the in-pavement situation) on the basis of the largest residual stresses which can be generated in the pavement without failure occurring anywhere. He used a static analysis of the pavement to achieve his analysis, analytically determining that combination of imposed wheel load and generated in-situ stresses which produced stresses which are everywhere within the Mohr-Colomb failure criterion. More recent work has concentrated on determining the limit to the surface load which can be applied by performing repeated load triaxial tests and observing the stress limits for stabilising behaviour (Fig. 2) (Werkmeister et al, 2001 & 2003). A pavement stress analysis (by linear elastic or finite element methods) is then performed and the loading or pavement design adjusted until no stress occurs in the aggregate greater than the limit determined from the triaxial test. Unfortunately this approach takes no recognition of the in-situ stress which might be generated (and which might be beneficial for the pavement behaviour), so there is a need to pull these two approaches together.

The use of a permissible stress framework defined by stress loci is a similar means of limiting rutting in the base layer(s) (Mundy 2002). These loci might be set by back-analysis of successful (and unsuccessful) pavements together with analytical models which will certainly require some
non-linear capability (either Finite Element or Linear Elastic with non-linearity adjustments (see Section 5).

\[ \text{Deviatoric stress} [\text{kPa}] \]
\[ \text{Cell pressure} \sigma_3 [\text{kPa}] \]

**Figure 2** Permanent strain behaviour (a) and the stress boundaries (b) associated with the different strain behaviours (after Werkmeister et al, 2001)

Rutting in the aggregate layers is undoubtedly connected to the influence on aggregate behaviour provided by the subgrade soil and the same is true in the reverse direction - the aggregate has a big influence on rutting in the subgrade. There is a fairly straightforward affect, doubtless - as the modular ratio between the two layers increases, so load spreading in the aggregate improves, the stress imposed on the subgrade reduces, and subgrade rutting becomes less important. However, there are also more complex relationships. The ability of a layer to deform plastically probably influences the development of residual stresses in the pavement which, in turn, affect stiffness, and hence, the distribution of traffic-related stresses (see Mechanism 5).

Furthermore it may be possible for the subgrade to be too resilient (Mechanism 6) but this was felt by workshop attendees only to be an issue for the pavement if it resulted in plastic damage of the
overlying base (for example, causing loosening of it). In most cases, ensuring that the pavement, as a whole, did not flex unduly (see Mechanism 1 discussed above) was likely to be sufficient.

Subgrade rutting (Mechanism 4) is only an issue in inadequate pavements. The fundamental purpose of the pavement construction is to reduce the stresses imposed on the subgrade so that the subgrade doesn’t rut. One might, therefore, infer that subgrade rutting is the first failure made to address. Designing the pavement to achieve this goal is, thus, a major issue and is discussed further in Section 7.

3.8 Gravel Loss

The previous two sections have dealt with what might be termed structural failure. Most pavements are structurally adequate, at least for a while, so day-to-day failure is seen in the occurrence of excessive or premature wear. First to be considered is ‘gravel loss’.

The main structural layer of the pavement is provided by the granular base with, perhaps, a sub-base. This can wear away with time. On unsealed roads the direct action of vehicle tyres may displace stones to the pavement shoulders (Mechanism 14). Loss of aggregate from the surface may also be caused by erosion due to water action in wet weather (Mechanism 27). On all pavements loss may occur as aggregate is pressed into a soft subgrade such that the properties of the subgrade between the stones governs the behaviour rather than stone-to-stone action (Mechanism 26). This last mechanism is usually countered by the use of a geosynthetic separator to maintain aggregate cleanliness. The inclusion of a separator therefore acts as a form of restriction criterion.

3.9 Roughness

The other wear problem faced by the pavement engineer is that of developing roughness. Some rutting is usually predicated by an increase in roughness but that rutting is non-uniform from place to place leading to an increasingly uneven longitudinal profile (Mechanism 8). This means that roughness prediction on the basis of a fundamental engineering approach will be difficult as roughness must depend, to a considerable degree, on the non-uniformities in the pavement construction and pavement properties. Perhaps a statistical approach based on the variability of measured properties could be useful here.

It is known that roughness development is also caused by the non-uniform loading imposed by moving vehicles which tend to ‘bounce’ along the pavement. Bounce of the vehicle as a whole leads to long wave-length unevenness while suspension flutter (‘wheel hop’) tends to cause corrugations (Mechanism 10). Corrugations are particularly prevalent on bends, hills and on the approaches to junctions where shear between the wheel and pavement surface is greatest. This shear leads to a snatching action as the wheel hops, thereby helping to wear or laterally displace material away from the surface at the points in time when the wheel is not so intimately in contact with the surface. Corrugations are mostly seen in unsealed pavements. However some short-wavelength irregularity can be apparent in sealed pavements due to material displacement and/or differential compaction under bouncing wheels.

The workshop attendees suggested that a shear strength criterion might be applied to the material forming unsealed pavement surfaces as a simple means of limiting the occurrence of corrugations. Although a different loading mechanism it would seem that the accumulation of localised deformations within the surfacing is not too dissimilar to that experienced in the rutting of granular layers. Thus, the shakedown approach discussed above (Section 3.7 and Fig. 2) might be used in outline such that the stresses imposed by the hopping wheel were in Range A on Fig. 2 - indicating that the granular material would be non-deforming.

Potholing (Mechanism 9) is also driven by loading and can be repeated along a pavement at the points where vehicles or wheels bounce downwards thereby amplifying the loading. However the workshop attendees identified aggregate separation and material saturation as key influences. When an aggregate segregates it frequently loses some of its strength as the packing becomes less
optimal. Furthermore, the section that is now low on fines can hold water rather easily and, in an unsealed pavement, will not readily shed surface run-off. Such a section may well be surrounded by a section of pavement with excessive fines - thereby further assisting in maintaining saturation of the open-graded part by preventing it from draining. Such a weak and saturated section is ill-equipped to carry the imposed traffic loading and aggregate is rapidly displaced. In a sealed pavement this is preceded by the excessive flexure, shear and subsequent breakage of the seal, hence letting water in more readily. Thus potholing may be seen as a progressive failure mechanism with a relatively small defect attracting unfavourable conditions and responses which accelerate ultimate failure.

The workshop discussion suggested that there was a need to define the grading limits outside of which potholing could be anticipated. This would give a basis for design and quality control which was not possible at present.

4 CONDITION

The materials which make up the pavements depend for their mechanical performance on their

- mineral composition
- grading
- compaction
- operating stress condition (see also Section 5)
- moisture

The first two of these are inherent to the material but the last three depend on the conditions of use of the material. To some extent these are controlled by the pavement constructor and designer but the pavement user, the climate and the pavement maintenance regime may all influence pavement condition and, hence, its performance. For an analytical design to be successful it must replicate the condition that the pavement will have during service.

4.1 Compaction

With respect to compaction some allowance will need to be made for stiffness change due to compaction during the pavement’s life. Ten or 20% increases in granular material modulus are often observed in laboratory testing and greater stiffness change may occur in-situ where vehicles impose (in effect) a kneading compaction and generate (unquantified) locked-in horizontal residual stresses. Stiffness properties in-situ are unlikely to match those measured in the laboratory even if density and other parameters match because the different compaction methods will have an influence. So the adopted stiffness values will, inevitably, require some ‘correction’ based on experience of in-situ properties. Density and compaction method may also be expected to affect (e.g.) shakedown limits (see Fig. 2b) although the extent of the effect is not known.

At the evaluation stage, stiffness may be determined semi-directly (e.g. by FWD) and the effect of compaction is, thus, subsumed into the measurement. Further compaction at evaluation stage (unless in a quality control context) is unlikely so stiffness may then be assessed with some confidence. Because the relationship between permanent deformation characteristics and density is rather slight (but see Lekarp et al 2000), density change effects will probably be ignored in the immediate future. Further research may change this.

4.2 Stress Condition

Soils and granular materials have stress-dependent properties. Thus they vary their behaviour as a consequence of their:

- loading from different types of traffic,
- in-situ (residual stress conditions),
- position in the pavement (depth and distance from the wheel load).
In the past the effect of different types of traffic has been accumulated and related to a standard load. This is considered in Section 8 in particular. Suffice it to say at this point that there is an increasing view that faster computational equipment should allow pavements to be analysed relatively quickly for the range of loads they are likely to experience such that the load response under each may be assessed individually.

The issue of residual stresses generated in the pavement by compaction and past trafficking could be important for design as mentioned above (see Section 3.7). Unfortunately, the measurement of lateral stresses has proved intractable. For the time-being, it is likely that an assumed pattern of stresses will need to be assumed. They cannot be as great as passive conditions -

$$\sigma_h < \sigma_v (1+\sin\phi')(1-\sin\phi')$$

otherwise the pavement would fail in horizontal compression, but are expected to be considerably greater than the ‘at-rest’ conditions. Elasto-plastic stress analysis with an appropriate yield condition can give an idea of possible stress conditions in-situ during loading (e.g. Jouve and Guezouli, 1996) and, by adopting a resilient unloading, a pattern of locked in stress may be deduced. Whether they are a good reflection of reality is, however, something else!

Position in the pavement will have a large effect upon the stresses experienced by an element of pavement or subgrade. This is the subject of Section 5.

4.3 Moisture Condition

Moisture has a huge impact upon the resilient response, the permanent deformation behaviour, on the wear and on the environmental damage properties of the pavement. Yet of all the conditions it is the least controllable. When the pavement is too wet, strength and associated properties diminish rapidly. When too dry, there may be insufficient inter-particle suction to hold materials together and, once again, bulk strength and associated properties deteriorate.

The unpredictability and variability (due to seasonal and individual weather events) have tended to push designers towards a ‘worst-mean’ approach. This may lead to over-design but has generally served designers well except when exceptional events occur. The Spring-thaw issue faced in countries with very cold winters is an example of a case where extreme moisture leads to a separate design approach for those conditions. In principle, there is nothing to stop designers completing an analytical assessment of their proposed pavement for several cases covering the range of moisture conditions likely to be encountered during the pavement’s life and then summing the damage at each condition (see Section 8 for more details about damage accumulation). This could be a practical way ahead for mechanistic structural design of granular pavements. In this case it would be necessary to characterise the pavement materials across the range of likely moistures to be experienced in-situ. This pre-supposes:

- a greater testing budget,
- a knowledge of the likely range that will occur. Greater information on in-situ moisture is becoming available (e.g. US LTPP studies) and existing data / older studies need re-visiting to establish a data-base of information. Moisture prediction methods are available\(^1\).

For those non-structural wear mechanisms, the incorporation of water condition in such a prescriptive manner seems less feasible, at least at present. For these modes of distress, incorporation by reference to experience, albeit quantified, seems more likely.

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\(^1\) G. Salt of Tonkin & Taylor has information.
5 NON-LINEARITY

It is generally recognised that the non-linear and stress-dependent resilient properties of compacted aggregates have a significant effect on the behaviour of the granular layers that they form. Modulus almost invariably rises with increasing all-round (“hydrostatic”) stress. It often rises with increasing deviatoric stress as well, although decrease with increasing deviatoric stress is not uncommon. These two effects, superimposed on

a) the stress change with depth due to self-weight,

b) the pore pressure (or, more likely, the pore suctions) which probably become more positive with depth,

c) lateral stress developed by previous compaction and trafficking, and

d) the traffic-induced stresses which decrease with depth and radius from the point of loading,

create a highly non-uniform modulus field for a material which is, otherwise, constant in property. Thus modulus changes with depth and, less obviously, with radial distance from the loading centre-line.

It is unfortunate that linear approaches have been used for so long that they have become, de facto, the “correct” way of analysing pavements. To a large extent this is probably because non-linearity is much less important in the analysis of pavements comprising a thick asphalt or concrete surface. However, this is clearly not the case for thinly-surfaced or unsealed pavements (see Plaistow & Dawson, 1995, for example).

In practice, most linear-elastic analytical (LEA) techniques seek to replicate some of this variability in modulus by sub-layering the granular material, assigning different modulus values for each sub-layer. Chosen appropriately (which requires some experience and self-critique), this technique can get close to the true variation with depth. Improvements and greater confidence comes when used iteratively - i.e. the stresses calculated by an analysis are used to compute the moduli at key points and the analysis repeated with the new moduli - until a harmonious set of stresses and moduli is computed. Nevertheless this still leaves radial inconsistency. LEA methods, by definition, have constant modulus values in the horizontal direction which will be the source of significant error, especially nearer the surface where the radial rate of change in stress will be the greatest.

There appear to be three means of dealing with this:

a) ignore it,

b) perform several LEAs with the moduli values in any one analysis being representative of those at a particular radius. Moduli and stress are harmonised at the radius of interest by iteration. A composite analysis is then ‘sewn together’ using each analysis to provide a concentric ring of stresses, strains and deflections at the radius which each represented,

c) perform a non-linear finite element (FE) analysis incorporating non-linear constitutive material models for the base and subgrade.

Approach (b) is probably now not warranted as FE approaches become much more practical to perform on desktop PCs. Computer hardware speed and software usability continue to increase suggesting that FE approaches will become routine in the near future. One such approach was seen at the workshop (see presentation by G van Blerk, for example, though not available at time of writing).

There was a considerable discussion about the importance of incorporating anisotropy in the analysis (as is the case with CIRCLY). No firm conclusion was reached on this point, but the relative importance of non-linearity was highlighted.
6 SUBGRADE

6.1 Shakedown

So far, most of the text has concentrated on the aggregate base layer(s). In many regards the same issues apply to the subgrade. Non-linearity is also of some importance although somewhat diminished compared to the base since the subgrade’s greater depth in the pavement means that stress pulses and stress gradients are both smaller thereby limiting the impact of the soil’s non-linearity. Permanent deformation characteristics are certainly not the same for the subgrade soils as they are for granular pavement layers, but similar principles will apply. Shakedown testing doesn’t seem to have been performed in the same way as for granular materials but there is some evidence to support the idea of a threshold deviatoric stress which must not be exceeded if significant permanent deformation is to be avoided.

6.2 Subgrade Strain Criteria

Most approaches at present adopt a resilient vertical subgrade strain criterion to prevent rutting in the whole of the pavement. There appeared, at the workshop, to be general agreement that this was inappropriate but there was less agreement on its replacement. The author of this paper is of the view that relating any resilient strain to plastic strains or to deflections is fundamentally flawed. Others are willing to accept a relationship between resilient and plastic strain, but only in the same material (the subgrade soil in this case). Others would prefer to use a resilient subgrade deflection criterion on the basis that deflection and rutting are both displacements, not strains.

The paper provided after the workshop by van Blerk (see accompanying ‘Subgrade Strain Criteria’ document) shows that the criteria will vary depending on the performance target adopted and on the relationship adopted with respect to the number of applications of load.

For example, the Austroads approach is sometimes said to assume a ‘7.14 power law’ for the prediction of rutting damage. Strictly speaking this isn’t true. What the Austroads design approach gives is a subgrade strain criterion to prevent premature rutting which states:

\[
N_{std} = \left( \frac{8511}{\epsilon_{vp}} \right)^{7.14} \quad \text{[3]}
\]

where \(N_{std}\) is the number of (equivalent) standard mass axles to be carried and \(\epsilon_{vp}\) is the permitted resilient vertical strain at the top of the subgrade produced by the standard axle. When a LEA analysis is performed, \(\epsilon_{v}\), the actual resilient vertical strain at the top of the subgrade, will rise proportionately with the magnitude of the wheel load, thereby indicating a 7.14\textsuperscript{th} power law relationship between \(N_{actual}\) and \(N_{std}\) if \(\epsilon_{v} = \epsilon_{vp}\). This breaks down, however, when a non-linear analysis is performed. Nor does it work so readily in reverse - i.e. if one assumes a more conventional 4\textsuperscript{th} power law to compute the number of equivalent standard axles, \(N_{std}\) then the use of the above equation leads to values of \(\epsilon_{vp}\) reducing at a 7.14\textsuperscript{th} power ratio rate with respect to \(N_{actual}\) but only at a 1.785\textsuperscript{th} power law rate with respect to load level:

\[
\epsilon_{vp} = \frac{8511 \left( \frac{P_{std}}{P} \right)}{N^{0.14}} \quad \text{[4]}
\]

where \(N\) is the number of applications of axle load \(P\) and \(P_{std}\) is the load of a standard axle. Of course, the actual \(\epsilon_{v}\), which is generated by a value of axle load, \(P\), greater than the standard value, \(P_{std}\), will increase beyond that value of \(\epsilon_{v}\) due to an axle of standard load capacity. If a LEA analysis is in use then the increase in \(\epsilon_{v}\) will be proportional to the increase in \(P\). Thus to ensure
that $\varepsilon_i = \varepsilon_{ip}$, $N$ must be reduced beyond that already achieved by use of the 4th power law. IN this way an apparent 7.14th power law might be generated.

This issue of load equivalency is discussed further in Section 8.

7 DESIGN

The preceding sections have described some of the issues addressed at the workshop. How might these be combined to from an appropriate design? It is suggested that an appropriate analytical design procedure might be as follows (using Figure 1 as a pro-forma):

1) Determine the relevant moisture content range(s) over which the aggregate base and subgrade layers will operate during their design life (Section 4.3).

2) Characterise the materials for their non-linear resilient properties and for their shear strength and for permanent deformation characteristics, the last of these using a shakedown framework (presentations of Ross Peploe and response by Andrew Dawson).

3) Express the base and subgrade moduli in terms of non-linear materials models. Many models have been used by researchers, but sophistication is probably not warranted, at this time, due to other approximations. So the “k-Θ” of Uzan (1985) models are suggested with constant Poisson’s ratio values in any one layer.

4) Use a FE analysis incorporating these non-linear models to analyse a potential pavement sequence. Determine the stresses throughout the pavement and the surface deflection. There is still some uncertainty in estimating the horizontal stresses in order to make the analysis truly representative (Section 4.2).

5) Compare the calculated surface deflection with the value set to ensure efficient trafficking. If the deflection is too great (see Mechanism 1), make one or more of the following changes
   a) provide a thicker aggregate layer,
   b) provide a stiffer aggregate layer,
   c) stabilise the subgrade to make it stiffer.

The shape of the deflection bowl can be used to determine from which layer the overall softness of response derives, and thus the appropriate selection of the above three remediation approaches can be judged. As a first approximation the elastic solution for a homogenous half-space may be used to deduce the equivalent single modulus value which would give the same central surface deflection. If this modulus is much less than that of the aggregate base, the subgrade is having too great an influence and solutions (a) or (c) are sensible. If not then solution (b) is suggested.

6) Assuming Step (5) is completed satisfactorily, the stresses in the aggregate may be compared with the permissible stress states deriving from a shakedown interpretation of permanent deformation testing of the aggregate base material (Werkmeister et al, 2001 and 2003). They may also be compared with stress loci boundaries based on shear strength (Mundy 2002). It is not completely clear how these two approaches may be reconciled. However, the stress loci approach may allow the results of the rather time-consuming permanent deformation testing to be reliably extended to a broader range of stress cases.

If the stresses measured exceed the permissible ones the following adjustments must be considered:
   a) a base aggregate with higher resistance to permanent deformation must be used in the places where failure was indicated (usually nearer the surface). Thus the addition of a strong surface aggregate is a special case of this solution method,
b) a thinner base of the same aggregate (this will only be appropriate where the subgrade is strong/stiff and will restrain lateral movement within the aggregate - e.g. on hard rock subgrades),

c) use of a bound overlay (asphalt or concrete). This, effectively, down-rates the aggregate.

7) Assuming Step (6) is completed satisfactorily, the stresses/strains/deflections in the subgrade may be compared with the permissible values of these criteria. Section 6.2 has already described the subgrade strain approach in outline. An alternative approach using the shakedown concept (as in Step (6) above) might be more suitable, but insufficient work has been done to make a recommendation here. If inadequate performance is indicated then a stiffer or thicker aggregate layer over the subgrade is required.

The remaining failure mechanisms are then dealt with by restriction approaches (see Section 2.2) or by other means (see Section 3 and below). Having achieved a permissible design it must be tested at different conditions - in particular wetter conditions (see Section 4.3). In a preferred approach, ‘partial’ designs would be completed, as outlined by the steps above, for different moisture and traffic conditions. The damage accrued at each condition would then be summed and the overall damage compared with the permissible level. This summation of damage is discussed in the next section.

To describe non-structural wear effects, the workshop suggested that the “Mathematical Model of Pavement Performance” (MMOPF) proposed by Ullidtz (1998) be investigated further.

8 DAMAGE

Pavement design aims to select materials and a pavement cross-section which will provide adequate performance under many passages of a range of traffic loadings. Each traffic passage causes some damage and, cumulatively, this causes pavement failure. The general assumption is that this damage accumulation is linear - i.e. the Palmgren-Miner rule is valid. This is discussed in detail in Annex 2, where it is shown that non-linearity of the pavement behaviour means that simple addition of damage cannot, in reality be achieved. Furthermore, the power law approach, which underwrites so much pavement load assessment, is intimately tied up with the damage accumulation model - if damage cannot be simply summed then it is likely to become impossible to state an equivalency between two load levels, as the stage at which each level of loading is applied will become significant. For example, if 100 passes of a heavy vehicle followed by 10000 of a light vehicle will not generate the same damage as 10000 passes of a light vehicle followed by 100 of a heavy vehicle (which experience suggests, indeed, might be the case) then the damage accumulation will be very load case specific and design much more difficult to achieve.

In the light of this, an approximation method of handling damage accumulation will be likely to be required for some time to come. The separation of early and late damage would, however, seem to be a development which should be seriously considered. The recent work by Alabaster et al (2002) suggested that a power law for rate of damage accumulation after an initial settling-down period was warranted. Kinder and Lay (1988), in effect, achieve something similar, but with one equation, and this may be a better way of progressing:

\[
\text{rut or damage} = K \left( \frac{P}{P_{\text{std}}} \right)^m N^{n/a} \quad \ldots [5]
\]

where K, m and a are pavement or material parameters and P, P_{std} and N are defined as previously. Alternatively a compaction-wear model has recently been described (Arnold et al, 2003):
rut or damage = \(
K \left( \frac{P}{P_{\text{std}}} \right)^m (C + W.N) \)

...[6]

where \( C \) is the pavement or material compaction parameter and \( W \) the wear parameter.

Both approaches model early and late damage. Without this separation of the two elements, load equivalency values become large and very sensitive to the criteria applied. Thus in the data made available by Korkiala-Tanttu at VTT in Sweden (see documents VTT-data and correspondence ‘Leena-John-Andrew’) the power law varied from circa 4 to 9 for the same data depending on whether the critical rut depth was 2.5mm or 20mm, whereas separating out the initial damage from wear gives more constant power law values around 3. A third alternative is given by Theyse in the presentation by him to the workshop (for one load level):

\[
\text{rut or damage} = mN + a(1 - e^{-bN}) \]

...[7]where \( m \), \( a \) and \( b \) are material or pavement parameters.

One further effect to be incorporated is that of vehicle wander. Not only does this spread the damage so that accumulation of damage in one place will not be as rapid as it would otherwise have been, but it also provides a ‘kneading’ action which, to some extent, mitigates previous damage. No mechanistic approach is available to model this so, for the time being, a correction factor approach is probably indicated.

9 EVALUATION

Although most of this document has (explicitly) covered design, some final comments on evaluation are warranted. Much of the information provided has implicit value for evaluation. By its nature non-destructive evaluation can only assess stiffness characteristics. Non-linearity could be investigated by the application of different load levels using a FWD and this should be more routinely done. As a simplification the modular ratio between base and subgrade may be used as a guide to pavement quality. If this is tied together with a resilient subgrade strain/stress/deflection approach (see Section 6 above) then some indication of propensity for subgrade rutting may be achievable. For the bas only full-scale trial trafficking is likely to give an indication of rutting likelihood within the base, but the use of the DCP, the stress locus approach and the monitoring of existing rut development might, together, be used in some (as yet undefined) way to assess base rutting potential.

10 DISCUSSION AND CONCLUSIONS

This paper has sought to bring together the workshop’s discussions, presentations and their implications. With a desire to move towards performance-related specifications and maintenance strategies there is a need to understand the granular pavement better and to apply that understanding in an appropriate manner. Section 7 has outlined how the design might be achieved. Other failure mechanisms were addressed in Sections 2 and 3. Some can be readily dealt with by restricting the materials used although some (e.g. Section 3.3) require new methods of analysis which haven’t yet been addressed.

In practical terms it seems likely that some prioritisation of effort is required in design. It was suggested that there was a need to rank the significance of each failure mode and of the factors controlling it so that the detail expended in the design effort could match the need. Significance might be judged by answering questions such as:

- How likely is this variable to change?
- Over what range might this variable vary?
- If this variable changes, what are the implications for the pavement design?
• What tolerance can be achieved in construction compared with the dimensional change a particular refinement might generate?
• If this type of failure were to occur in practice, how costly / difficult would it be to rectify?
• At what condition / time / position is this failure mechanisms most / least likely?
• What is the variability of material supply, of construction and of material condition?

On this basis design effort and attention to detail might be better directed.

Clearly some of the analytical design of granular pavements according to a fundamentally acceptable framework is not yet achieved and many difficulties remain. However, the workshop has made some real progress and, it is hoped, this paper will assist in providing pointers to future development. Even with the implementation of the ideas discussed, validation and scaling will remain necessary and the definition of appropriate values for the scaling parameters remains a critical task.

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