Background: Promotion of mechanistic methods in New Zealand was with the introduction of FWD in 1993, NZIHT courses by T&T promoting the “precedent” method in the following years, and more formal momentum given by the first Mechanistic Design Workshop by Andrew Dawson in 2002 and his summary of outcomes including prioritisation needs and his pointers to future development:

The Mechanistic Design and Evaluation of Unsealed & Chip-Sealed Pavements

University of Canterbury Workshop, November 2002,
Hosted by Andrew Dawson

INTRODUCTION
Pavement Design is a process that has matured from experiential decisions by an engineer, through empirically-based procedures (which, in essence, sought to codify the engineer’s experience) to modern analytical methods. The latest manifestation of these is the (US) AASHTO 2002 procedure which seeks to analytically compute the effects of most of the factors which can affect pavement performance. This procedure will allow design to be done to a level of detail not previously possible in a routinely available design method.
BACKGROUND

Despite its sophistication, the AASHTO 2002 approach shares the same essential features as for any other analytical approach. The features are illustrated, diagrammatically, in Figure 1. Firstly candidate materials must be selected, characterised by laboratory and/or in-situ tests and this characterisation used to compute the values of certain stresses or strains (or, conceivably, other mechanical parameter) at critical points within the pavement. These parameters, together with their locations, have previously been selected as design criteria on the basis of an assessment of the failure mechanisms that must be designed against. The values required for those parameters have been computed on the basis of the level of performance required. The actual and required values of the design criteria may then be compared and the design declared successful or not. If it is not successful an alternative design or alternative materials must be selected or remediation measures applied.

This approach is no different from that employed for any other structural engineering design - for example in a concrete cantilever beam the key design criterion is likely to be the limiting tensile extreme fibre stress which can be tolerated at the root of the cantilever, on its top surface, due to bending moment in the beam. Although the above description and Figure 1 show the design process where a pavement design and materials are found in order to provide a desired level of performance, it is equally possible to use the analytical approach to determine the life of a pavement for which materials and cross-section are already known.
THE NEED

Increasingly, we want to use non-standard (recycled, alternative and marginal) aggregates to build our low-volume unsealed or chip-sealed pavements and we want to be more efficient in our use of conventional materials. There have been many developments in the last 20 years or so in laboratory testing, computational methods, instrumented trials and full-scale experiments (like CAPTIF) but, to date, we haven’t gone beyond empirical or chart-based methods. As a consequence, engineers don’t have the flexibility of specific designs for individual roads and materials or the possibility of fairly comparing alternative designs or remediations and of localising the approach to the specific situation.
MECHANISTIC DESIGN

The overall aim of the workshop is to investigate the potential for truly mechanistic design/evaluation of low-volume road pavements. Mechanistic credentials are claimed by many of the available design methods (e.g. ARRB) but I contend that, while analysis has often been done to interpolate and extend the design approach, truly mechanistic methods (as laid out in Figure 1) do not exist for low-volume pavements.

Neither do I mean by the phrase “Mechanistic Design” merely that a numerical analysis is performed. A numerical computation is a tool which can be used or mis-used depending on the model of the pavement and of the component materials which make up the pavement. However, if a truly mechanistic method is to be used then a computational procedure, probably involving some numerical technique, would seem inevitable. Charts may be able to codify computations in some circumstances, but care must be taken that they remain graphical means of performing a computation (like a nomograph) and not a means of hiding empiricism.

For there to be a truly mechanistic approach the designer needs to be using relationships between the loading and the responses of the pavement which describe, in theoretical and numerical terms, rational cause-and-effect linkages. This needs to draw on an engineering description, in stress-strain terms, of the materials from which the pavement is to be constructed. It is unlikely that, in the immediate future, it will be possible to achieve this goal without various adjustment factors (“fudges” !) because we won’t understand all the conditions which contribute to the exact scaling of the relationships.
Figure 1  Procedural Flow Chart for Analytical Pavement Design
PROBLEMS TO OVERCOME

It will be evident from Figure 1 that the problems to be overcome in bringing together a truly mechanistic design / evaluation method for low-volume pavements are in five areas:

1) FAILURE - We need to know the myriad ways in which low-volume pavements could conceivably fail and then to identify a key mechanical measure for each (often a stress or a strain at some point in the pavement) which will act as an indication of the performance being achieved.

2) DESIGN CRITERIA - Each key mechanical measure has to be computable and its limiting value determined as that which relates directly to the minimum acceptable standard of pavement performance with which it is associated.

3) MATERIAL CHARACTERISATION - The validity of the computational technique rests, to a large extent, on the veracity of the constituent materials’ stress-strain relationships. This means that our measurement techniques need to be accurate and that we need to evaluate the correct parameters.

4) EXTERNAL INFLUENCES - Most low-volume pavement materials change their response to a greater or lesser extent when the load level, the moisture content, the temperature or the speed of loading change. Non-linearity with applied stress (or with ambient stress) is now generally incorporated into the more advanced material descriptions.

5) COMPUTATIONAL ANALYSIS - A computational technique which reproduces the in-situ stress-strain field is needed.
LOAD EQUIVALENCY
Equivalency between traffic load level and number of passages of an axle loaded at a standard level is commonly expressed by the ‘fourth power law’. Several researchers have observed that this ‘law’ doesn’t hold for low-volume roads reflecting either that damage doesn’t follow such an equivalency pattern and/or that the power term is not 4 for these pavements. An appendix (see separate document entitled “Annex 1”) to these notes shows how equivalency should be formulated if pavement damage follows the form of some common types of material response models. It will be seen that the equivalency is not independent of the non-linearity of response of the material and varies over the life of the loading.

ENVIRONMENTAL LOADING
Finally, the loading by the environment should not be ignored. Frost-heave and soil swelling are mentioned in the separate “Table” document as specific distress modes caused by temperature and moisture respectively (Mechanisms 11 and 12). To these mechanisms should be added cracking of a seal coat due to tension induced by temperature effects, perhaps exacerbated by bitumen embrittlement due to uV aging and/or oxidation. These mechanisms need a separate analytical approach.
THE WORKSHOP

The overall aim of the workshop is to investigate the potential for truly mechanistic methods of analysis to be applied to the design and evaluation of low-volume road pavements. My hope is that we will be able to discuss in more detail the items discussed in the preceding pages so that we can go away having

1) agreed a basic framework for linking an engineering understanding of aggregate and soil layers to a mechanistic explanation of the pavement as a basis for design and for pavement evaluation,

2) defined the issues which need to be solved in order for such a fundamental and scientific understanding of the pavement to be implementable in day-to-day practice, and

3) laid out pointers as to how solutions to these might be achieved.

Such an approach would, eventually, allow performance-design not just performance-related design (just as the strength of steel and concrete are used as direct inputs to the analysis of a bridge deck). Ideally we’d like to take measures of soil and aggregate strength and stiffness, from laboratory or field testing, using them to calculate stresses and strains in the pavement and so predicting whether failure/distress will occur.
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 Nf 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: ( ) B \( \epsilon \) v = A/ N \([1\)

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.
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 centreline.

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-surfed 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 sublayer. 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 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 nonlinearity. 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.
7 DESIGN The preceding sections have described some of the issues addressed at the workshop. How might these be combined to form 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.
7 DESIGN continued

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).

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.
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.

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)
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 base 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

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. 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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