Abstract

The current version of the South African Mechanistic-Empirical Design Method for flexible pavements is based on technology and models developed during the 1970s and 1980s. Given the current level of concern regarding the validity of the results from the method and recent developments that are ready for implementation, it is an appropriate time for the revision of the design method. A framework for the ME-design method, the core components of the process and a number of characteristics desired from the revised method are presented. Resilient response models, the functionality required from the primary pavement response model, material yield strength models and damage models are discussed. The main challenges regarding the implementation and acceptance of the method in practice are listed.

1 INTRODUCTION

A mechanistic-empirical design (ME-design) method for flexible pavements has been available in South Africa since the 1970s. The detail of this method is presented elsewhere (Van Vuuren et al, 1974, Walker et al, 1977, Paterson and Maree, 1978, Maree and Freeme, 1981, Jordaan G J, 1993, Theyse et al, 1996:6 - 17). Although the latter versions of the method were published during the 1990s, the content of the method essentially remained the same as that of the earlier versions in terms of the load input, material characteristics, primary pavement response model and damage models or transfer functions. This method was implemented in a number of software packages since the late 1990s which exposed the method to a wide user group. The method therefore came under increasing scrutiny and criticism in the recent past (Jooste, 2004). This has progressed to the point where it has to be admitted that although the method is a valuable design tool, the current ME-design method is overly sensitive to changes in the input variables, leads to inadmissible and counter-intuitive results in some cases, provides unrealistic structural capacity estimates for certain pavement types and does not assess all materials equally, based on their true performance potential. In addition to these problems the method also focuses largely on the effect of load magnitude (stress condition) on the structural capacity of the pavement, ignoring the effects of construction and environmental conditions.

Given the current state of affairs, the future of mechanistic-empirical flexible pavement design in South Africa is in the balance. However, it is the opinion of the authors that substantial progress has been made in the recent past resulting in better models that may lead to a significant improvement in the results from the ME-design method. This paper provides a brief overview of the improved models that are available for immediate implementation and attempts to highlight certain of the challenges that still need to be faced in future. Only the main characteristics of the problems and solutions are highlighted.
without presenting much detail on how the solutions were developed or how the unresolved problems need to be solved. It is hoped that the process of ME-design and the models briefly outlined in the paper may be included in a revised version of the South African Mechanistic-Empirical Design Method (SAMDM) for flexible pavements in the near future.

2 A VISION OF THE FUTURE SAMDM

2.1 Supply and demand in the pavement engineering context

The pavement design problem is similar to most engineering designs in the sense that a facility needs to be designed that will provide an acceptable or appropriate level of service (supply) given the environment (demand) in which the facility operates. In the case of the pavement design problem, the “demand” may be further subdivided into the natural and traffic environment in which the road will operate while the structural capacity and functional service level of the pavement may be viewed as the “supply” from the pavement. The challenge of pavement design is to balance the supply and demand at an acceptable level of reliability or risk. If the methods for estimating the structural capacity, SC (supply) and design traffic, DT (demand) are inaccurate, a false balance will be achieved during the design process. In the view of the authors, the process for estimating the design traffic is therefore as much part of the ME-design process as the process for estimating the structural capacity of the pavement. A brief overview of recent developments in the characterisation of traffic loading is therefore included in the paper. Traffic data will become a direct input and traffic calculations an integral part of the revised ME-design method, especially when recursive analysis is introduced.

2.2 Bridging the gap between ME-design and pavement engineering practice

As mentioned earlier, the past focus of ME-design in South Africa has been the modelling of the primary pavement response in terms of deflections, strain and stress enabling the modelling of the long-term structural damage under repeated loading. This resulted in the ME-design process relying on inputs such the resilient modulus and Poisson’s ratio of the material in addition to having a transfer function calibrated for the particular distress mechanism associated with the material. Day-to-day engineering practice on the other hand, specifies and constructs roads based on a completely different set of parameters with very little correlation between the ME-design inputs and the common engineering parameters of the material.

In addition to this, the influence of field variables such as density and environmentally related parameters such as the temperature and equilibrium moisture content of pavement layers on the ME-design input and models were largely unquantified. Hopefully, the input variables and models that are used in future versions of the SAMDM will be correlated to basic engineering parameters with due consideration of environmental influences on the input variables and models used in the method.
2.3 Structural and functional performance simulation

Currently, the structural capacity of a pavement is defined by the SAMDM in terms of two terminal conditions. Firstly, in terms of a 20 mm rut on the surface of the pavement and secondly in terms of the fatigue cracking of the asphalt layers without stating the degree and extend of the fatigue cracking. The trend with “modern” ME-design methods (NCHRP, 2004; Ullidtz, 2002) is to move towards the quantification of the design risk by introducing variability and the simulation of the structural and functional deterioration of the pavement with time and increasing traffic. While the design risk may still be quantified based on the survival histogram approach from the 1993 AASHTO Design Guide (AASHTO, 1993) using stochastic simulation with a separation between the design traffic and structural capacity estimation processes, the simulation of structural and functional deterioration will require a recursive simulation scheme.

The following phases of development are therefore anticipated for the revised SAMDM:

- The initial revision of the method will introduce the quantification of the design risk or reliability using stochastic simulation applied to the processes for estimating the design traffic and structural capacity;
- This will be followed by the introduction of structural deterioration simulation using recursive analysis for rutting and fatigue damage;
- The final revision will introduce the simulation of functional deterioration linked to the structural deterioration models and additional models for riding quality deterioration.

2.4 Outline of the ME-design processes

Given the approach for the initial revision of the SAMDM, the concepts of an equivalent axle load and equivalent design traffic will be retained. The process for estimating the design traffic is illustrated in Figure 1 and will be separate from the structural capacity analysis for the initial revision phase.

In terms of the design traffic estimation, two types of input are required. The first being the heavy vehicle volume represented by the number of heavy vehicles counted in each of a number of heavy vehicle classes. Secondly an indication is required of the loading per heavy vehicle in each of these classes. The complexity of the heavy vehicle loading input will depend on the type of traffic data that are available. Given comprehensive axle load data, the loading for each of the heavy vehicle classes may be represented by the axle load distribution histograms for the steering, single, tandem and tridem axles of that vehicle class as shown in Figure 1. These axle load histograms are therefore the first possible entry point for information regarding the loading per vehicle. The axle load histograms are converted to an equivalent axle load per vehicle for each of the vehicle classes using a number of possible techniques of which the simplest (the exponential AASHTO load equivalency formulation) is shown in Figure 1. On the other hand, only classified vehicle count data may be available without axle load data, in which case an assumption will need to be made regarding the equivalent axle load per vehicle for each of the vehicle classes. This assumption should be guided by typical equivalent axle load per vehicle values calculated from the axle load spectra at locations with similar traffic and more particularly similar heavy vehicle loading characteristics. The equivalent axle load per heavy vehicle is therefore the second potential entry point for vehicle loading data in the absence of axle load data.
The average daily equivalent (ADE) traffic is calculated from the total equivalent traffic summed over the individual vehicle classes and divided by the duration of the traffic survey. If the traffic survey was done over a very short period of time, adjustment factors are used to adjust the ADE to an annual average daily equivalent (AADE) traffic. These adjustment factors make provision for weekday-weekend variations, seasonal variations and exceptional periods. The final two parameters required to adjust the AADE from the time of the traffic survey to the date of opening of the facility and to accumulate the design traffic (DT) for the structural design period are the growth rates for the heavy vehicle traffic and the equivalent loading per heavy vehicle.

In summary, the input variables to the process of estimating the design traffic for the initial revision of the SAMDM are:

- The vehicle count ($C_i$) for each of the heavy vehicle classes;
- The power ($d$) of the AAHTO exponential load equivalency formulation when axle load data is available;
- The normalised axle load histograms or equivalent axle load per heavy vehicle ($E80_i$) for each of the heavy vehicle classes;
- The weekday-weekend ($F_{7/5}$), seasonal ($F_s$) and exceptional period ($F_E$) adjustment factors;
- The heavy vehicle growth rate ($h$); and
- The equivalent axle load per heavy vehicle growth rate ($v$).

During subsequent revisions, the axle load histograms from Figure 1 in combination with the contact stress for each load will input directly into the ME-analysis process shown in 0. It is unlikely that the ME-design method will progress to a point where the permanent deformation of the pavement system is modelled with hardening plasticity and the fatigue of the asphalt layers modelled with fracture mechanics. The constitutive material models and analysis tools required by such an approach are probably too complex for routine pavement design. It is therefore anticipated that the SAMDM will maintain a separation between the resilient response analysis of the pavement and the damage associated with repeated loading according to the diagram shown in Figure 2.
Figure 1: Design traffic estimation process
Figure 2: Outline of the mechanistic-empirical process for estimating pavement structural capacity
It must be realised that the details of many of the components of the process outlined in Figure 2 are still not finalised and will only be finalised during the revision process. However, the anticipated main components of the ME-analysis procedure are listed to the best of the authors’ ability:

- The geometry of the pavement;
  - The geometry will be defined by the thickness of the pavement layers;
- The material resilient response models calibrated for the main influential variables as indicated;
  - Hot-mix asphalt (HMA);
    - Temperature, load-pulse duration and ageing;
  - Stabilised material;
    - Binder type, binder content, ageing and possibly density;
  - Unbound material;
    - Stress regime, density and degree of saturation;
- Loading data;
  - Load magnitude and contact stress;
  - Load wander;
  - Vehicle speed;
- Primary pavement response model;
- Damage models calibrated for the main influential variables as indicated with the critical response parameter listed first;
  - Hot-mix asphalt (HMA);
    - Plastic strain – vertical elastic strain, temperature, and number of load repetitions;
    - Fatigue – horizontal tensile strain, stiffness and number of load repetitions;
  - Stabilised material;
    - Effective fatigue or stiffness reduction – strain ratio and number of load repetitions;
    - Crushing of cement stabilised material – stress ratio and number of load repetitions;
    - Plastic strain – stress ratio and number of load repetitions;
  - Unbound material;
    - Plastic strain;
      - Structural layers – stress ratio, density, saturation and number of load repetitions;
      - Subgrade – subgrade elastic deflection and number of load repetitions.

The details of each of the components of the processes for estimating the design traffic and structural capacity are discussed in subsequent sections of this paper.

3 QUANTIFYING THE TRAFFIC LOAD

Eventually the revised SAMDM will require the following loading characteristics:

- Load magnitude;
- Contact stress;
- Wander; and
- Vehicle speed.

The complex 3-dimensional nature of the contact stress at the tyre-pavement interface has been one of the focus areas of research in South Africa in the recent past. Many
publications are therefore available on this topic and the reader is referred to those publications for more detail (de Beer et al., 1997: 179 – 227, De Beer et al., 2004 and De Beer, 2006). Load wander and vehicle speed will be represented by appropriate statistical distributions. The vehicle speed distribution will be different for flat sections and climbing lanes.

The primary traffic input to the SAMDM will be the axle-load histograms representing the load magnitude for the steering, single, tandem and tridem axles of each of the vehicle classes in the vehicle classification system. Even if the loading per vehicle is represented by an equivalent 80 kN axle load (E80), the axle-load histograms are required to calculate the E80 value per heavy vehicle for each of the vehicle classes. These axle-load histograms will mostly be obtained from weigh-in-motion (WIM) surveys. The main challenges in terms of the characterisation of the axle-load histograms from WIM data are to account for the systematic and random error that may be present in the WIM data and to represent the axle-load histograms in a simplistic manner that will allow for the identification of recurring patterns in axle-load data within the vehicle classes and between routes with similar traffic environments.

Research by Prozzi et al. (2006) opened the opportunity to develop a statistical procedure that enables the correction of WIM data for systematic WIM error and random dynamic load variation. The procedure requires that mixed log-normal distributions are fitted to the observed axle-load histograms. The probability density function (pdf) of the log-normal distribution of the random variate $X$ is given in equation (Eq. 1).

$$f(x; \mu, \sigma) = \frac{1}{x\sigma\sqrt{2\pi}} e^{-(\ln(x) - \mu)^2/2\sigma^2}$$  \hspace{1cm} (Eq. 1)

where

- $\mu = \text{mean of } \ln(X)$
- $\sigma = \text{Standard deviation of } \ln(X)$

The expected value of $X$ is given by $E(X) = e^{\mu + \sigma^2/2}$ and the variance by $\text{var}(X) = (e^{\sigma^2} - 1)e^{2\mu + \sigma^2}$. A mixed log-normal distribution may be generated from a number of weighted individual log-normal distributions as long as the sum of the weights of the individual log-normal distributions equals one. The formulation for the mixed log-normal distribution consisting of $K$ individual log-normal distributions is given by equation (Eq. 2).

$$f(x; \mu_k; \sigma_k; W_k) = \sum_{k=1}^{K} \frac{W_k}{x\sigma\sqrt{2\pi}} e^{-(\ln(x) - \mu_k)^2/2\sigma_k^2}$$ \hspace{1cm} (Eq. 2)

where

- $\mu_k = \text{mean of the } k^{th} \text{ individual log-normal distribution}$
- $\sigma_k = \text{standard deviation of the } k^{th} \text{ individual log-normal distribution}$
- $W_k = \text{weight of the } k^{th} \text{ individual log-normal distribution}$

Figure 3 shows an example of a four part mixed log-normal distribution fitted to an observed normalised axle-load histogram. In general, a very good fit is obtained between observed axle-load histograms and a mixed log-normal distribution consisting of four individual log-normal distributions. The axle-load histogram is therefore reduced to a set of four means, standard deviations and weights resulting in a total of 12 unknowns. The
number of unknown parameters is, however, reduced to 11 because of the limitation that the sum of the weights should equal one. A convenient graphical summary of the mixed log-normal parameters for the yearly WIM data collected from 2003 to 2006 for a WIM station on a national route in South Africa is shown in Figure 4 for the tandem axles of long-vehicles (length exceeds 16.8 m). The standard deviation of each of the individual distributions is plotted against the average of the corresponding distribution. The size of the circle representing the data point is determined by the weight of the distribution. It is clear from the plot that the statistical parameters representing the yearly axle-load histograms remained fairly consistent for the period under investigation.

![Figure 3: Mixed log-normal distribution fitted to an observed axle-load histogram](image)

![Figure 4: Statistical parameters representing yearly mixed log-normal distributions](image)

Once the statistical parameters representing the axle-load histogram are known, the systematic and random error in the observed WIM data may be corrected according to equations (Eq. 3) and (Eq. 4). Figure 5 and Figure 6 show the corrected axle-load histogram for a 5 percent random and systematic error respectively for the data from Figure 3. The correction for the random error reduces the variation in the axle load histogram but the statistical moments of the corrected distribution remain similar to that of the observed distribution. The correction for the systematic error causes a significant reduction in the observed axle loads if there is a positive bias in the calibration of the WIM.
\[ \mu_{\text{Static}} = \mu_{\text{WIM}} - \ln(1 + E) \]  
\[ \sigma^2_{\text{Static}} = \sigma^2_{\text{WIM}} - \sigma^2_{\text{Error}} \]  

(Eq. 3)  

(Eq. 4)  

where

- \( \mu_{\text{Static}} \) = mean of the logarithm of the actual static load
- \( \mu_{\text{WIM}} \) = mean of the logarithm of the observed dynamic WIM load
- \( E \) = percentage systematic error of the WIM equipment
- \( \sigma^2_{\text{Static}} \) = variance of the logarithm of the actual static load
- \( \sigma^2_{\text{WIM}} \) = variance of the logarithm of the observed dynamic WIM load
- \( \sigma^2_{\text{Error}} \) = variance of the random error

Figure 5: Corrected axle-load histogram for 5 percent random error

Figure 6: Corrected axle-load histogram for 5 percent systematic error
The effect of WIM error on the design traffic estimate was subsequently investigated at the hand of a comprehensive set of WIM data collected from 1998 to 2005 at a WIM station on a national route in South Africa. Figure 7 (a) shows the number of days counted per year and Figure 7 (b) the heavy vehicle volumes for three vehicle classes based on the length of the vehicle. The yearly axle load histograms were used together with the annual vehicle count to calculate the E80s per vehicle as well as the annual E80s for each class based on the AASHTO load equivalency law with an exponent of 4.5. These results are shown in Figure 8 for zero systematic and random error. The process was also completed for 5 percent systematic and random error.

A summary of the cumulative E80 is shown in Figure 9 calculated for different combinations of systematic and random error with the legend indicating the systematic error first followed by the random error. It is clear that the random error has very little effect on the cumulative E80. The systematic error on the other hand, has a significant effect. Extrapolating the cumulative E80 to the end of a 20 year period using the polynomial models shown in Figure 9 results in design traffic estimates of 69 million assuming zero systematic error and 57 million for 5 percent systematic error, a difference of 20 percent. While this may not appear to be a significant difference in terms of the design traffic it implies that the road will be over-designed by about 20 percent if the 5 percent systematic WIM error is ignored or unknown. The magnitude of the difference in design traffic will be more pronounced for load sensitive pavements and higher WIM errors. A very limited
recent study in South Africa indicated that the systematic WIM error may be as high as 10% at certain WIM stations.

Figure 9: Cumulative E80s

In summary, recent developments in the measurement of the tyre-pavement contact stress and statistical procedures for “fingerprinting” and correcting axle-load histograms for WIM error have improved the characterisation of traffic loading for the purpose of pavement design. Although the example provided still relies on the concept of equivalent traffic it will have similar impact in a recursive ME-design procedure which directly utilises the axle-load histograms. It is strongly recommended that these improvements should be incorporated in the revised SAMDM.

4 ESTIMATING THE STRUCTURAL CAPACITY

The main components of the process shown in Figure 2 for estimating structural capacity are subsequently discussed.

4.1 Resilient response models

The type of resilient response models that are required for ME-design is determined by the behaviour of the material type under consideration, the significant factors affecting the behaviour of the material and the constitutive models that are available in the primary pavement response model. The future constitutive models required for the primary pavement response model are discussed in a later section. It is also unlikely that constitutive models other than Hooke’s linear-elastic model will be available during the early revision phases. The discussion of resilient response models is therefore focused on models that can be implemented based on the linear-elastic model. The terminology used in this paper as well as certain potential problems regarding the interpretation of test results with the aim to obtain the stiffness modulus of materials are, however, discussed first.

Figure 10 (a) shows a typical example of the uni-axial stress-strain response of a specimen under monotonic loading and Figure 10(b) under cyclic or dynamic loading. There are two options to describe the "stiffness" of the material. The first is to use a chord modulus ($M_c$) from the origin of the stress-strain curve to any point on the curve. The second is to use a tangent modulus ($M_t$) starting with the initial tangent modulus ($M_{ti}$) at the
origin of the curve progressing to the tangent modulus at any point on the curve. The chord modulus is also referred to the resilient modulus \((M_r)\) in the case of unbound material and the dynamic or complex modulus \((E^*)\) in the case of asphalt although the shape of the hysteresis curve is slightly different for asphalt tested under forced vibration.

![Typical stress-strain response curves](image)

**Figure 10: Typical stress-strain response curves**

The stress-strain behaviour of the material is clearly non-linear in both cases. Given different test conditions such as the confinement pressure for unbound material and temperature for asphalt material, the response of the material may also be different as illustrated in Figure 11. The behaviour of unbound material is therefore said to be stress-dependent under both monotonic and dynamic loading and that of asphalt, temperature dependent. It is, however, important to note that a distinction is made between non-linearity and stress or temperature dependency, concepts often confused in engineering. A truly linear-elastic material may also behave in a stress or temperature dependent manner.

![Effects of test conditions on the stress-strain response curve](image)

**Figure 11: Effects of test conditions on the stress-strain response curve**

The temperature and strain-rate dependent (visco-elastic) behaviour of asphalt is in fact modelled in the AASHTO 2002 ME-design method using a linear-elastic chord modulus model that is calibrated for the effect of temperature and load-pulse duration (NCHRP, 2004: RR-11). Similarly the stress-dependency of unbound material may be modelled using a chord modulus that is calibrated for the effect of confinement and shear stress.
Such an approach is illustrated in Figure 12 where the stiffness modulus depends on the prevailing temperature or stress conditions.

The problem associated with a stress or temperature dependent linear-elastic model is that it still allows stress conditions beyond the yield strength of the material as no limit is placed on the maximum allowable stress and the linear stress-strain curve extends to infinity. This is in contrast to the actual material response illustrated in Figure 11 (a). An alternative to this approach is to use either a hypoelastic model which is a truly non-linear elastic model or a perfect plasticity model. Hypoelastic and plasticity models follow the actual non-linear stress-strain response of material and therefore do not allow inadmissible stress states. Figure 13 shows the stress states calculated at a number of locations in an unbound granular base layer using linear-elastic, stress-dependent linear-elastic and plastic models. (Balay et al, 1997: 834). Stress states plotting on the left-hand side above the slanted failure line are not admissible. Although the stress-dependent model is an improvement on the single stiffness modulus linear-elastic model, it is still not on par with the plasticity model in terms of preventing inadmissible stress results.

The tangent modulus model developed by Duncan and Chang (1970: 1629 – 1653) may be classified as a hypoelastic model. The Duncan and Chang model is often used in geotechnical engineering but not in pavement engineering. Duncan and Chang used a
hyperbolic model suggested by Kondner (1963: 115 - 143) to model the non-linear stress-strain response of monotonic tri-axial test results on loose material which does not exhibit a clear failure point on the stress-strain curve. Pavement materials on the other hand are normally well compacted and exhibit a clear failure or yield point on the stress-strain curve. The Duncan and Chang model does not fit the response of dense pavement materials well. An alternative tangent modulus model has recently been developed (Theyse, 2007: 98). Considering only the loading portion (Eq. 5) of this model, the model follows the stress-strain and stiffness modulus data well up to the failure point, as illustrated by the example in Figure 14.

\[ M_t = \left[ 1 + \left( \frac{\sigma_d}{\sigma_d^f} \right)^b \left( \frac{1}{1 - \left( \frac{\sigma_d}{\sigma_d^f} \right)^b} \right) \right]^{1/b} k p_a \left( \frac{\sigma_3}{p_a} \right)^n \]  

(Eq. 5)

where

- \( M_t \) = tangent modulus (MPa)
- \( \sigma_d \) = deviator stress (kPa) = \( (\sigma_1 - \sigma_3) \)
- \( \sigma_d^f \) = deviator stress at failure (kPa)
- \( \sigma_3 \) = minor principal stress (kPa)
- \( p_a \) = reference pressure (101.3 kPa)
- \( b, k, n \) = model parameters

Figure 14: Modelling test variable dependency with a linear-elastic model

Hypoelastic models require a piecewise linear solution and the implementation of hypoelastic tangent modulus models and perfect plasticity will only be possible once a finite-element (FE) primary pavement response model is available. Until such time, the problem of inadmissible stress results will not be fully solved.

An additional concern regarding the calibration of stiffness modulus models for pavement materials is the analysis of dynamic response results with static theory. This problem applies equally to the analysis of dynamic laboratory tests and falling-weight deflectometer (FWD) field tests. The dynamic response of a laboratory specimen or a pavement system is determined by the stiffness, damping and inertia of the specimen or system. Any displacement measured during such a dynamic test includes the effects of stiffness,
damping and inertia. If analysed with static theory which only provides for the stiffness of
the system, the effects of damping and inertia are assigned to the stiffness of the system
resulting in an overestimate of the stiffness. This aspect was already highlighted by
Lourens in 1991 following a study on the dynamic response of pavements which lead him
to the conclusions cited below.

“It follows therefore that the behaviour of a structure is likely to be misinterpreted under
dynamic loading if not analysed by a rigorous method which takes full cognisance of its
mass and damping characteristics” (Lourens, 1991:7) and “The calculations in this report
suggest that many aspects of conventional back analysis methods for roads, as used up to
now, are suspect” (Lourens, 1991:7).

The impact of using static theory to analyse dynamic response data is illustrated with an
example. A finite-element model was created of a pavement with the characteristics listed
in Table 1. A dynamic analysis was performed using a haversine load pulse of different
duration and a set of surface deflections generated in each case. These dynamic
deflections were back analysed using static theory with the results given in Table 1.
Unfortunately the pavement used in this study is not representative of typical South African
pavements given the thickness of the asphalt layer. The response of the system is
dominated by the thick and stiff base layer and the thick subgrade resulting in the largest
back-calculation error of 35 and 108 percent respectively for these two layers for a 30 ms
load pulse which is representative of the FWD load-pulse duration.

<table>
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<th>Table 1: The impact of analysing dynamic response data with static theory</th>
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<tr>
<td><strong>Pavement characteristics</strong></td>
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<td><strong>Layer thickness (mm)</strong></td>
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Differences in the initial modulus of unbound material derived from monotonic loading tri-
axial test results and the resilient modulus derived from dynamic tri-axial test results have
also been observed (Theyse, 2007). Figure 15 shows a summary of the initial and resilient
modulus results for a crushed stone material at comparable combinations of low and high
density (LD and HD) and saturation (LS and HS). The resilient modulus results exceed the
initial modulus results by far. Again, static theory was used to analyse the dynamic test
results therefore attributing the effects of damping and inertia to the stiffness of the
material. Although the “dynamic” modulus may be used in a quasi-dynamic analysis,
Lourens (1991) indicated that results obtained from such an approach are not correct.

In summary, the resilient response characteristics of pavement materials which is one of
the main inputs to the SAMDM are not yet fully understood and often incorrectly modelled
but a design solution has to be put in place without delay. A resilient modulus approach,
calibrated for the appropriate parameters will therefore be adopted for the initial revision of
the SAMDM but will require critical assessment during subsequent revisions. Three broad
material types have been identified of which the mechanical properties and behaviour are
sufficiently different to warrant separate resilient response models. These materials are
hot-mix asphalt, stabilised material and unbound granular material. The resilient modelling
options anticipated for these materials during the initial revision phase are briefly discussed.

![Static tri-axial test initial modulus data](image1)

![Dynamic tri-axial test resilient modulus data](image2)

(a) Initial modulus
(b) Resilient modulus

**Figure 15: Comparison of the static initial modulus and dynamic resilient modulus of a crushed stone**

4.1.1 Models for hot-mix asphalt
The resilient response of hot-mix asphalt (HMA) is primarily visco-elastic (strain-rate dependent) with the temperature of the material and the binder characteristics having significant influences on the response of the material. Included under the binder characteristics would be the binder type and age or condition. The resilient response models for HMA should reflect this behaviour and therefore strictly requires a temperature dependent visco-elastic formulation and constitutive model in the primary response model. Although visco-elastic constitutive models are available in integral transformation solutions of multi-layer systems such as VEROAD (Hopman et al, 1997: 693 – 705), the AASHTO 2002 ME-design guide does not use an explicit visco-elastic model but rather a linear-elastic model that is calibrated for the effect of load-pulse duration, temperature and binder viscosity (NCHRP, 2004: RR-11). Given the limited HMA test data available in South Africa the revision of the SAMDM will rely heavily on HMA data and models from overseas research. It is therefore anticipated that the models from the AASHTO 2002 ME-design guide will be used for at least for the initial revision of the SAMDM.

Equations (Eq. 6) and (Eq. 7) provide the formulation for the stiffness modulus and Poisson’s ratio from the AASHTO 2002 ME-design guide. These models were calibrated against a comprehensive data base for the purpose of the AASHTO ME-design guide and predictive models are available for the mixture specific model parameters in terms of basic mixture parameters. These predictive models will be implemented in the initial revision of the SAMDM. These models provide for the effects of load-pulse duration, temperature and binder viscosity (age/condition) on the resilient response of HMA.

\[
\log(E_{ac}) = \delta + \frac{1}{1 + e^{\beta + \gamma \left[\log(t) - \alpha (\log(\eta_b) - \log(\eta_c))\right]}}
\]

(Eq. 6)

\[
\nu_{ac} = 0.15 + \frac{0.35}{1 + e^{c + NE_{ac}}}
\]

(Eq. 7)
where

\[ E_{ac} = \text{asphalt stiffness modulus} \]
\[ T = \text{load-pulse duration} \]
\[ \eta = \text{binder viscosity at the temperature of interest} \]
\[ \eta_{Tr} = \text{binder viscosity at a reference temperature} \]
\[ \alpha, \beta, \delta, \gamma, c = \text{mixture specific model parameters} \]
\[ \nu_{ac} = \text{Poisson’s ratio for asphalt} \]
\[ a, b = \text{mixture specific model parameters} \]

4.1.2 Models for stabilised material
The formulation of the resilient response models for stabilised material has not been finalised. The resilient response of this material group may vary from the stiff, brittle characteristics of cement stabilised material to almost visco-elastic behaviour for mixtures with high foamed and emulsified bitumen contents. At low cement and bituminous binder contents the resilient behaviour of this material may approach that of unbound granular material which depends on density, saturation and stress conditions. One aspect certain to affect the resilient response of stabilised material is the time-related changes in the resilient response characteristics of this material. Unfortunately very little data are available to calibrate resilient response models for these time-related changes and this aspect will have to be one of the research priorities of the revision process. Factors that will need to be considered are:

- The effect of early curing soon after construction on the resilient response of the material;
- The effects of long-term ageing on the resilient response of the material. These effects will be different for cement and bituminous stabilisation with the ageing of bitumen expected to result in an increase in the stiffness of the material while carbonation of the cement reaction may lead to a reduction in the stiffness of the material;
- The effects of repeated loading on the resilient response of the material. The stiffness of the material is expected to reduce because of load-associated damage.

4.1.3 Models for unbound material
The resilient response behaviour of unbound granular material depends on the density and degree of saturation of the material as well as the stress regime to which the material is subjected. The resilient response models for unbound material for the initial revision of the SAMDM are listed in equations (Eq. 8) and (Eq. 9). The resilient modulus model was developed by Theyse (2007: 212) and the Poisson’s ratio model by Bonaquist and Witczak (1997: 786). Model calibration factors are available from the original research.

\[ M_r = p_{am} \cdot 10^{k_c} \cdot \frac{SD^c \cdot S^{k_c}}{10^{k_c}} \left( \frac{\sigma'}{p_{am}} \right)^{k_c} \quad \text{(Eq. 8)} \]
\[ \nu = p_1 - p_2 \left[ \frac{\sigma_d}{\sigma_d} \right] + p_3 \left[ \frac{\sigma_d}{\sigma_d} \right]^{p_4} \quad \text{(Eq. 9)} \]
where

- $M_r$ = resilient modulus (MPa)
- $p_{rain}$ = reference pressure (101.3 kPa)
- $SD$ = solidity (dry density as ratio of the apparent density, %)
- $S$ = degree of saturation (%)
- $\theta'$ = effective bulk stress (kPa)
- $\sigma_d$ = deviator stress (kPa)
- $K_0$, $K_{SD}$, $K_s$, $K_f$ and $K_2$ = regression model parameters
- $\nu$ = Poisson’s ratio
- $\sigma_{df}$ = deviator stress at failure (kPa)
- $\nu_i$ = model parameters for $i = 1$ to 4

The resilient modulus model is based on the model by Uzan (Uzan, 1985: 54, Uzan et al, 1992: 334 - 350, Uzan 1999: 112) but introduces the density and saturation level of partially saturated unbound granular material in the model formulation. It has to be noted that the bulk stress term is in terms of effective stress including the residual compaction stress, overburden stress, suction pressure and external load stress. These aspects will be discussed further under the discussion of the primary pavement response model. The model was calibrated for the combined data from a selection of unbound granular material ranging from sand to crushed stone with the same level of accuracy as the calibration of the model for the individual materials. The model presented in equation (Eq. 8) therefore provides a predictive model for the resilient modulus of unbound granular material with due consideration of the density, saturation and effective stress conditions. Figure 16 (a) shows results from the predictive model for an effective bulk stress of 650 kPa and deviator stress of 500 kPa at three solidity levels ranging from a value representative of sand (65 percent) to a value representative of crushed stone (86 percent).

4.2 Primary pavement response models

The primary pavement response model for the initial revision of the SAMDM will be an integral transformation solution of a multi-layer, linear-elastic system (Maina and Matsui, 2004). Given the resilient response models from the preceding section the primary pavement response model will have to provide for the following adaptations of the linear-elastic model:
• A density, saturation and stress-dependent model for unbound material; and
• A temperature and load rate dependent model for hot-mix asphalt.

This will be achieved using sub-layering in the integral transformation solution of the multi-layer, linear-elastic system, effectively implementing vertical stress-dependency.

In the case of the resilient response models for unbound material, the effective bulk stress, deviator stress and yield or failure stress terms will require feedback from the primary pavement response model and an iterative solution will be implemented. The density and saturation dependency will be implemented as part of the pre-processing effort of the primary pavement response model. The models for the unbound material are based on effective stress and the following stress components will be considered:

• The vertical and horizontal overburden stress at rest;
• The equal-all-round suction pressure in the partially saturated material;
• Residual compaction stress; and
• The stress induced by the external load.

The overburden stress will be calculated from the density of the material and the lateral earth pressure coefficient at rest, the compaction stress from research by Uzan (1985: 54) and the suction pressure from a density and saturation dependent suction pressure approximation developed by Theyse (2007: 138). The suction pressure approximation is given by equation (Eq. 10) and is calibrated from tri-axial yield strength results. The model fitted to suction pressure results for clay (data from Vanapalli and Fredlund, 2000: 195 – 209) and a dolerite subbase material from South Africa (Theyse, 2007: 146) is shown in Figure 17.

\[ p_{suc} = \frac{\rho S}{e^{\omega/SD}} \]  
\[ \text{(Eq. 10)} \]

where

- \( p_{suc} \) = suction pressure (kPa)
- \( S \) = degree of saturation (ratio of voids filled with water to total voids)
- \( E \) = base of the natural logarithm
- \( \rho, \omega \) = model parameters

![Approximate effective suction confinement](image)

(a) Suction pressure for clay

![Suction pressure curve](image)

(b) Suction pressure for a dolerite subbase

**Figure 17: Suction pressure approximations for clay and natural gravel**
The models for HMA do not require feedback from the primary pavement response model but thermally coupled stress analysis will be considered. The effective stress for unbound material and the thermally coupled stress for HMA will be accounted for according to the vector formulation given in equation (Eq. 11).

\[
\sigma = E\varepsilon + \sigma_0
\]  

(Eq. 11)

where

- \(\sigma\) = stress vector
- \(\varepsilon\) = strain vector
- \(E\) = stiffness matrix according to Hooke’s law
- \(\sigma_0\) = initial stress vector

Given the problems highlighted during the discussion on the material resilient response models, the primary pavement response model will have to evolve to the point where the following material and analysis models can be accommodated:

- 3-dimensional stress dependency;
- Hypoelastic models calibrated from static test results;
- Perfect plasticity models calibrated from static test results;
- Contact non-linearity at pre-defined cracks; and
- Dynamic analysis including the effects of stiffness, damping and inertia.

It will only be possible to include these models in a finite-element solution. A finite element primary pavement response model will therefore have to be implemented in the subsequent revisions of the SAMDM. Finite Element (FE) analysis tools will also have to be developed for the dynamic analysis of dynamic laboratory and field tests such as the FWD test.

4.3 Damage models

Two modes of structural damage will be considered for the initial revision of the SAMDM. The plastic strain of all the pavement layers will be modelled and will contribute to the rut on the pavement surface. The reduction in the effective stiffness of HMA and stabilised pavement layers caused by a reduction in the effective depth of the layer resulting from fatigue damage under repeated loading will be modelled in addition to the plastic strain of these layers. Damage models will be calibrated for each of the three main material types HMA, stabilised material and unbound granular material.

Critical, load related response parameters from the primary pavement response model will provide the input to the damage models but the damage models will also include the effects of field variables such as temperature, density and saturation.

4.3.1 Damage models for HMA

The revision of the SAMDM will rely on the HMA damage models from the AASHTO 2002 ME-design guide given the lack of similar models calibrated for South African materials. The AASHTO 2002 ME-design guide plastic strain damage model for HMA is given by equation (Eq. 12) while equation (Eq. 13) provides the formulation of the fatigue damage model.
\[
\frac{\varepsilon_p}{\varepsilon_r} = k_i \beta_{f1} 10^{-3.1552} T^{1.734} \beta_{f2} N^{0.39937} \beta_{f3}
\]  \hspace{1cm} (Eq. 12)

\[
N_f = 0.00432 C \beta_{f1} \left( \frac{1}{\varepsilon_t} \right)^{3.291} \beta_{f2} \left( \frac{1}{E} \right)^{0.854} \beta_{f3}
\]  \hspace{1cm} (Eq. 13)

where

\[\varepsilon_p = \text{vertical plastic strain}\]
\[\varepsilon_r = \text{vertical resilient strain}\]
\[T = \text{temperature}\]
\[N = \text{number of load repetitions}\]
\[\beta_{fi} = \text{plastic strain model calibration factors, } i = 1 \text{ to } 3\]
\[k_1 = \text{plastic strain model calibration factor}\]
\[\varepsilon_t = \text{tensile resilient strain}\]
\[E = \text{dynamic stiffness modulus}\]
\[\beta_{fi} = \text{fatigue model calibration factors, } i = 1 \text{ to } 3\]
\[C = \text{fatigue model calibration factor}\]

According to the AASHTO 2002 ME-design guide formulation of the plastic strain model, the plastic strain of HMA material is assumed to be proportional to the elastic or resilient strain. Fundamental theory of plasticity (Chen and Mizuno, 1990), local (Maree, 1978 and Theyse, 2000: 285 – 293) and international research (Huurman, 1997 and Van Niekerk et al., 1998) on unbound material as well as international research on HMA (Long et al, 2002) relate the plastic strain of materials to either deviatoric (shear) stress or a ratio relating the deviatoric (shear) stress to the yield strength of the material. The validity of the vertical elastic strain as the critical response parameter for HMA plastic strain therefore needs to be assessed critically during subsequent revision of the SAMDM and probably needs to be replaced with a shear stress based approach. Such an approach was used in the calibration of HMA plastic strain models for the Californian ME-design method, CalME (Ullidtz et al, 2006). The damage model formulations in equations (Eq. 12) and (Eq. 13) are also valid for total plastic strain and fatigue damage after “N” load repetitions. Although these formulations may be implemented in a recursive analysis scheme, incremental damage model formulations providing the damage for a load repetition increment “ΔN” are more convenient for recursive implementation. Incremental damage models will therefore need to be calibrated for subsequent revisions of the SAMDM.

### 4.3.2 Damage models for stabilised material

The formulation of the damage models for stabilised material are not final yet but based on observation from laboratory tests, full-scale accelerated pavement tests and in-service pavements, provision will be made for the following modes of distress or damage:

- Plastics train;
- Effective fatigue or stiffness reduction; and
- Crushing failure of cement stabilised material.

It is anticipated that the critical response parameter for the plastic strain damage model of stabilised material will be a stress ratio relating the working stress to the yield strength of the material. This requires the calibration of a yield strength model for stabilised material that includes the binder type, binder content and probably the density of the material. Sufficient data are available for the calibration of such a model for the initial revision phase but the data are only valid for a condition of 28 days curing. The long-term changes in the yield strength of stabilised material will have to be quantified during subsequent revisions of the SAMDM. The current practice of assigning the yield strength properties of the parent aggregate to a cement stabilised layer after the initial stiffness reduction phase is questionable unless there is a complete chemical reversal of the cementitious reaction. This aspect needs to be resolved and yield strength models need to be calibrated to account for the long-term changes in the yield strength of stabilised material.

Although the existence of a stiffness reduction phase is accepted for cement stabilised layers it is still debated for bituminous stabilised layers. Data from in-service pavements should be utilised to confirm or deny the stiffness reduction phase of bituminous stabilised layers during the initial revision of the SAMDM. Until such time, the current stiffness reduction models for cement and bituminous stabilised layers will be utilised. These models relate the duration of the stiffness reduction phase to the ratio between the working strain and the strain-at-break of the material. The strain-at-break is determined from a monotonic load, flexural beam test shown in Figure 18. Figure 19 illustrates the effect of binder content on the strain-at-break results for a recycled, previously cement stabilised ferricrete (Long and Theyse, 2001). Although the scatter in the data is significant there is a general increase in the strain-at-break with increasing bituminous binder content. It must be stressed that the stiffness reduction models merely indicate a change in the long-term resilient response of stabilised layers for recursive analysis and do not represent a terminal structural condition.

![Figure 18: Setup and typical result from the monotonic load flexural beam test](image)

(a) Test setup  
(b) Typical test result
The current crushing failure models for cement stabilised layers (de Beer, 1993) will be retained for the initial revision of the SAMDM.

4.3.3 Damage models for unbound granular material

Although the dominant mode of distress of unbound material is plastic strain or permanent deformation, a separation is made between the damage models for the subgrade and structural layers. A study by Theyse (2001) using Heavy Vehicle Simulator (HVS) data from a wide selection of test sites in South Africa showed that high shear stresses are induced in the subgrade of road pavements under extremely high aircraft type loads. Under normal road traffic, however, the shear stress in the subgrade of road pavements has dissipated to the extent that a proportional relationship may be assumed between the plastic and elastic strain of the subgrade. The distinction between the pavement subgrade and unbound structural layers is therefore made based on the level of working shear stress under loading. Theyse (2001) found that for all the sections tested with dual-wheel loads up to 100 kN a cover of 300 mm was sufficient to ensure the proportional relationship between the plastic and elastic strain of the subgrade. More appropriate decision criteria have subsequently been suggested by Thompson et al (2006) based on the ratio (SSR) of the working subgrade deviator stress and the subgrade unconfined compressive strength (UCS). These criteria are listed in Table 2. The SSR should be maintained below 0.6 to ensure that the working shear stress has been dissipated sufficiently to maintain the proportional relationship between the subgrade plastic and elastic strain.

<table>
<thead>
<tr>
<th>Subgrade damage potential</th>
<th>Low</th>
<th>Limited</th>
<th>High</th>
</tr>
</thead>
<tbody>
<tr>
<td>SSR</td>
<td>&lt; 0.6</td>
<td>0.6 – 0.75</td>
<td>&gt; 0.75</td>
</tr>
</tbody>
</table>

Given that the above criteria are satisfied, the subgrade permanent deformation model developed by Theyse (2001) may be applied to the ME-design analysis of the subgrade. Equation (Eq. 14) gives the formulation of the subgrade permanent deformation model with the subgrade elastic deflection as the critical response parameter. The model is illustrated in Figure 20 for a range of subgrade permanent deformation levels.
\[
\log N = C - \left( \frac{\delta_s}{a} \right)^{\frac{1}{\beta}}
\]  

(Eq. 14)

where

\[ N = \text{number of repetitions to reach a certain level of subgrade permanent deformation} \]
\[ \delta_s = \text{subgrade elastic deflection (micron)} \]
\[ a, b, \text{and } C = \text{model parameters having different values for different levels of permanent deformation} \]

**Figure 20: Subgrade permanent deformation damage model**

Given the current well-known subgrade vertical strain criteria by Dorman and Metcalf (1965: 69-84), Theyse (2001) also investigated the relationship between subgrade permanent deformation and the vertical strain at the top of the subgrade. A comparison of the relationships between 7 mm subgrade permanent deformation and two potential critical response parameters, the subgrade vertical strain and subgrade elastic deflection are shown in Figure 21. The subgrade elastic deflection provides a better prediction of the subgrade permanent deformation as it combines the effect of the external load, the cover provided by the structural layers and the response of the full depth of subgrade material affected by the load, not only the upper portion as represented by the subgrade strain.

**Figure 21: Relationships between subgrade permanent deformation and two potential critical response parameters**
In the region of high shear stress close to the road surface a different approach is required. The original work done in South Africa in this regard was spearheaded by Maree (1978) relating the plastic strain of the unbound material to the ratio between the shear strength of the material and the working shear stress imposed on the material, which he called the factor of safety. The permanent deformation damage model based on the factor of safety unfortunately yields unrealistic structural capacity estimates for unbound material. Subsequent work by Wolff (1992) developed S-N type damage models for different levels of plastic strain from HVS data. This work could, however, not account for the effect of material density or moisture content changes as these data were not available for all the HVS tests investigated by Wolff.

Recent research in South Africa by Theyse (2007) on partially saturated, unbound granular material introduced the effects of material density and moisture content in the damage models of these materials based on a stress ratio approach similar to the factor of safety approach. The model provides for the suction pressure generated in the partially saturated material based on the soil-water characteristic curve of the material and is based on an effective stress formulation. The following components are therefore required by this design procedure:

- A yield strength model for unbound material including the effects of confinement, material density and moisture content;
- The soil-water characteristic curve (SWCC) of the material is required to determine the suction pressure in the partially saturated material;
- A plastic strain damage model relating the stress ratio, number of load repetitions and plastic strain of the material.

The yield strength model of which the formulation is given by equation (Eq. 15) was calibrated with good accuracy and high precision on a material-by-material basis from static tri-axial results as is illustrated by the example in Figure 22. The model could unfortunately not be calibrated very accurately for the combined data from all the materials using only effective stress, density and degree of saturation, indicating that there is still some material effect that needs to be accounted for. A general yield strength predictive model could therefore not be developed and requires further investigation. The model may, however, be calibrated and used on a material-by-material basis with good accuracy.

\[
\sigma_1^y = \frac{e^{\omega SD}}{e^{\omega S}} \left( \sigma_3 + \frac{\rho S}{e^{\omega SD}} \right)^c - \frac{\rho S}{e^{\omega SD}} \quad \text{if } \sigma_3 \geq -\frac{\rho S}{e^{\omega SD}}
\]

\[
\sigma_1^y = \text{Undefined if } \sigma_3 < -\frac{\rho S}{e^{\omega SD}}
\]

(Eq. 15)

where

- \(\sigma_1^y\) = yield strength (kPa)
- \(S\) = degree of saturation (ratio of voids filled with water to total voids)
- \(SD\) = solidity (ratio of the volume filled with solids to the total volume)
- \(E\) = base of the natural logarithm
- \(\sigma_3\) = minor principal stress or confining pressure for the tri-axial test (kPa)
- \(a, b, c\) = effective stress yield strength model parameters
- \(\rho, \omega\) = suction pressure model parameters
The suction pressure approximation used in equation (Eq. 15) is given by equation (Eq. 16).

\[
p_{\text{suc}} = \chi [u_a - u_w] \approx \frac{\rho S}{e^{\omega S / \rho S}}
\]

(Eq. 16)

where

\( p_{\text{suc}} \) = suction pressure (kPa)

\( [u_a - u_w] \) = matric suction (kPa)

\( \chi \) = Bishop parameter related to the degree of saturation

![Yield Stress Plot](image)

**Figure 22: Illustration of the calibration accuracy of the yield strength model for a given material**

The SWCC required by the yield strength model was approximated and “back-calculated” from monotonic loading tri-axial tests using a process pioneered by Heath (2002). The SWCC could only be verified with independent measurements for one material (fine grained sand). The independent matric suction measurements as well as the SWCC approximation for the sand are shown in Figure 23. It was not possible to repeat the direct matric suction measurements for coarse pavement materials. Techniques should be investigated to measure matric suction for coarse material including modified and stabilised material.
Once accurate estimates of the yield strength and stress ratio of partially saturated unbound granular material were possible at any combination of solidity and saturation, a single plastic strain damage model could be calibrated for the combined plastic strain data of 6 crushed stone products from commercial quarries. The prediction accuracy of the model for the combined data is on par with that of the models for the individual materials. The models were calibrated from dynamic tri-axial plastic strain test results. The model incorporates the effect of the stress ratio, material density (solidity) and degree of saturation. Figure 24 shows an example of an individual material model for crushed sandstone. The data and model are shown for different combinations of high density (HD), low density (LD), high saturation (HS) and low saturation (LS) showing the dramatic effect of the degree of saturation on the plastic strain response.

The model formulation is provided by equation (Eq. 17) and the model for crushed stone at 86 % solidity and 40 % saturation is shown in Figure 25 on a normal scale, indicating that there is very little structural capacity when the working stress approaches the yield strength of the material (stress ratio approaches 1). Unfortunately a similar combined model could not be calibrated for the combined data of the natural gravels that were tested.
because of the huge variation in material response. It is recommended that the model for crushed stone material should be implemented as a plastic strain damage model during the initial revision of the SAMDM. The plastic strain data and models for natural gravel should be explored further during subsequent revision phases to investigate the development of a single plastic strain model for natural gravel by incorporating appropriate material properties in the model.

\[
\log(N) = 10^{I - a \frac{S^d}{S_D} S^{\alpha} S^{\beta}} \quad \text{(Eq. 17)}
\]

where

- \( N \) = number of load/stress repetitions to induce a certain level of plastic strain
- \( S \) = degree of saturation
- \( SD \) = solidity
- \( SR \) = stress ratio
- \( I, a, b, d \) and \( m \) = regression model parameters

![Crushed Stone Products
All plastic strain levels](image)

**Figure 25: Plastic strain S-N damage model for crushed stone**

The damage model formulation for the subgrade and unbound structural layers are again not well suited to implementation in a recursive scheme. Incremental damage models will be developed for recursive analysis during the subsequent revision phases of the SAMDM.

5 **IMPLEMENTATION CHALLENGES**

The preceding discussion covers the core components of the envisaged ME-design method but there are many aspects related to the successful implementation of the method that are not discussed. These aspects are varied and there are many implementation challenges that may only be realised during the revision of the SAMDM. This section provides a short summary of the obvious challenges that will have to be met to ensure the successful implementation of the revised method and to gain acceptance from engineering practice.
5.1 Application to new and rehabilitation design

The revised ME-design method will have to be applicable to new and rehabilitation design of light pavement structures for low-volume roads and heavy-duty pavements for high-volume roads carrying many heavy vehicles. The design method will have to be integrated with the guideline documents on new design and rehabilitation investigation and design although the sound engineering practice such as the rehabilitation investigation process currently contained in these documents should not be affected.

One of the main differences between new and rehabilitation design regarding the revised SAMDM will be the way in which the available materials are characterised for rehabilitation design which will rely heavily on in-situ testing as opposed to probably more laboratory characterisation for new design. Special consideration will have to be given to the sampling, testing and material characterisation for new and rehabilitation design in terms of sample size and the type of tests required as well as the procedures used for the analysis of the test results. If possible, an attempt will be made to provide the engineer with guidelines on the type and frequency of tests required to characterise the input to the SAMDM with a certain level of confidence. It should also be kept in mind that non-standard laboratory testing of materials is not automatically excluded from rehabilitation projects.

5.2 Risk and performance simulation

One of the main aims of the immediate revision of the SAMDM is to introduce stochastic simulation of the design traffic and structural capacity estimates to enable a rigorous analysis of the design reliability or risk using a survival histogram approach. It is anticipated that this will be achieved through Monte-Carlo simulation. Each of the input variables in the design traffic and structural capacity estimation processes will therefore be represented by a random variate. Provision will be made for a number of basic input distributions such as triangular, normal and log-normal distributions.

The subsequent revision phases will make provision for the recursive analysis of structural damage and eventually the simulation of functional performance aspects such as riding quality. The computational requirements of such a design system will be enormous and a balance will have to be reached between the value of the simulation process and the computational effort required to do the simulation.

5.3 Traffic related challenges

Although statistical techniques have been developed to account for systematic and random WIM error, the magnitude of these two errors for WIM installations in South Africa is still unknown. It is likely that these two types of error will also vary significantly from one installation to the other, depending on the calibration of the equipment and site conditions.

Other challenges related to traffic data and specifically axle load information is the extrapolation from data collected over very short survey durations to structural design periods in excess of 10 to 15 years. In other cases no traffic and axle load data will be available for the design location and the design engineer will have to rely on data from a site that he believes is representative of the design location. A traffic stratification system is therefore required. A prerequisite for the successful development and implementation of such as system is the availability of traffic data from a comprehensive set of permanent traffic survey sites that covers all the potential design scenarios from a traffic point of view.
A programme for sampling at permanent traffic surveys sites therefore needs to be developed and implemented for South Africa. The data from such a programme need to be captured in a design traffic information system and made available to design engineers with the appropriate traffic data analysis tools. Such an information system should include traffic volume, axle load and tyre inflation pressure.

A secondary information system is required to relate the axle load and tyre inflation pressure data to tyre-pavement contact stress utilising the data from stress-in-motion surveys done in the research environment until stress-in-motion becomes a routine measurement.

5.4 Material characterisation

The problems related to using inappropriate analysis tools for deriving the required design input from test results have already been touched on. Appropriate analysis tools should be developed for the analysis of laboratory and field test results with aim to derive design inputs. Protocols are also needed to ensure the repeatability and reproducibility of the non-standard laboratory and field tests required for material characterisation.

Although the resilient response models presented in this paper may seem very complicated, the calibration of the models is often less complicated than expected. The density, saturation and stress-dependent model for unbound material only distinguishes between different materials based on the density and saturation of the material with the model parameters for effective bulk stress and deviator stress being the same for the range of unbound material that was tested. While the laboratory calibration of the model will retain this distinction between materials based on density and saturation these terms will collapse into a single term for the calibration of the model using FWD data with the model parameters for effective bulk stress and deviator stress remaining fixed. The back-calculation process therefore reduces to finding the correct value for the combined density and saturation term. This term will differ from layer to layer and therefore distinguishes between layers of different unbound material.

5.5 Environmental and construction impacts

All the material models presented in the paper depart from the assumption that field variables related to the natural environment (temperature and moisture content) and construction variation (density, binder content, etc.) are known. A system of environmental information and predictive relationships between the environmental conditions and pavement variables needs to be established. This system also needs to capture the spatial variation of construction variables.

5.6 Pre-calibrated material models

The revised SAMDM will allow for project specific calibration of the models but in the absence of the necessary data to perform such calibrations, default input values and pre-calibrated material models will have to be provided based on the data sets that were used to develop the formulation of the models. A material information system containing pre-calibrated material models and the analysis tools for model calibration at a project level needs to be developed.

5.7 Validation and calibration of the method

Finally, the success of the revised SAMDM will be determined by the accuracy of the method in terms of the unbiased assessment of the structural capacity of different
pavement types. This accuracy can only be determined by evaluating the structural capacity estimates obtained from the method to a benchmark of observed structural capacities under in-service conditions for a range of different pavement types and environmental conditions. A performance based information system is therefore required to capture the actual structural capacity of different pavement types under in-service conditions, to provide a starting point for designs that may be refined using the ME-design method and to validate and, if required, calibrate the ME-design method.

6 CONCLUSION AND RECOMMENDATIONS

The current version of the South African Mechanistic-Empirical Design Method for flexible pavements is based on technology and models developed during the 1970s and 1980s. Given the current level of concern regarding the validity of the results from the method and recent developments that are ready for implementation, it is an appropriate time for the revision of the design method. A framework has been created for the revision of the SAMDM. The core components of the ME-design process have been discussed in addition to a number of characteristics desired from the revised method and implementation challenges that will need to be addressed. The design method will have to apply to new and rehabilitation design, bridge the gap between ME-design and day-to-day pavement engineering practice, be unbiased in terms of estimating the structural capacity of different pavement types based on the true performance potential of these pavements and will have to yield realistic, validated design results. Design risk or reliability simulation as well as time-based performance simulation will be introduced using stochastic simulation and recursive analysis respectively.

Significant advances in the characterisation of traffic loading data have been noted. Recent developments in the measurement of the tyre-pavement contact stress and statistical procedures for “fingerprinting” and correcting axle-load histograms for WIM error have been presented to improve the characterisation of traffic loading for the purpose of pavement design. It is strongly recommended that these improvements should be incorporated in the revised SAMDM.

A general discussion on resilient response modelling revealed that the resilient response characteristics of pavement materials, which is one of the main inputs to the SAMDM, are not yet fully understood and often incorrectly modelled. Yet, a design solution has to be put in place without delay for the initial revision of the SAMDM. A resilient modulus approach, calibrated for the appropriate parameters will therefore be adopted for the initial revision of the SAMDM but will require critical assessment during subsequent revisions. Three broad material types have been identified of which the mechanical properties and behaviour are sufficiently different to warrant separate resilient response models. These materials are hot-mix asphalt, stabilised material and unbound granular material. The resilient modelling options anticipated for these materials during the initial revision phase are presented and include the effects of the significant influential variables appropriate to each of the main material types.

An integral solution of a multi-layer, linear-elastic system will be retained as the primary pavement response model for the initial revision of the SAMDM. The following adaptations of the linear-elastic model are, however, required:

- A density, saturation and stress-dependent model for unbound material; and
- A temperature and load rate dependent model for hot-mix asphalt.
This will be achieved using sub-layering in the integral transformation solution of the multi-layer, linear-elastic system, effectively implementing vertical stress-dependency during the initial revision phase.

Given the problems highlighted in the discussion on the material resilient response models, the primary pavement response model will have to evolve to the point where the following material and analysis models can be accommodated:

- 3-dimensional stress dependency;
- Hypoelastic models calibrated from static test results;
- Perfect plasticity models calibrated from static test results;
- Contact non-linearity at pre-defined cracks; and
- Dynamic analysis including the effects of stiffness, damping and inertia.

It will only be possible to include these models in a finite-element solution. A finite element primary pavement response model will therefore have to be implemented in the subsequent revisions of the SAMDM. FE analysis tools will also have to be developed for the dynamic analysis of dynamic laboratory and field tests such as the FWD test.

A selection of damage models are presented for implementation during the initial revision of the SAMDM. The damage models will be calibrated for the appropriate critical response parameters appropriate to each of the material types as well as the significant influential variables. The method will make provision for the modelling of the plastic strain contribution from all pavement layers including HMA layers, stabilised layers, unbound structural layers and the pavement subgrade.

Elastic strain will be used as the critical response parameter for modelling the plastic strain of HMA layers during the initial revision. The validity of the assumed correlation between the elastic and plastic strain of HMA needs to be evaluated and if found to be incorrect, the use of a more appropriate critical response parameter will have to be investigated. Stiffness reduction and fatigue damage models will be provided for stabilised and HMA material. The effects of long-term changes in material characteristics on the stiffness, yield strength and permanent deformation resistance of all materials will have to be addressed to enable realistic damage modelling.

The successful implementation of the revised SAMDM will rely on the availability of appropriate and accurate input to the design method. Some of the input to the design method will be project specific but certain of the required inputs will have to be obtained from general information systems. A number of information systems required for the successful implementation of the design method have been identified. These systems will have to be developed, populated and made available to engineering practice for design purposes and include:

- A design traffic information system;
- An environmental information system that also contains predictive relationships to derive field variables such as the temperature of HMA layers and moisture content of unbound layers from the environmental variables. This system also needs to provide input on the construction associated spatial variability of field variables such as layer thickness and density;
- A materials input information system. This system needs to contain all the pre-calibrated material and damage models used in the ME-design method in addition to the basic material properties of the materials used in the calibration of the models. The
system also needs to provide the necessary analysis tools required for the project specific calibration of the material models.

7 ACKNOWLEDGEMENTS

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8 REFERENCES


KEY WORDS