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EXAMINATION OF MECHANICAL PROPERTIES IN UNBOUND ROAD BASES

Kornél ALMÁSSY

Department of Highway and Railway Engineering
Budapest University of Technology and Economics
H–1521 Budapest, Hungary

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Abstract

The following document presents a study on the behaviour of unbound granular bases for 'thin asphalt' pavement structures. The Report presents the mechanical and practical properties of unbound granular materials with special regard on the determination of its the resilient modulus and the permanent deformation, and by proper material characterisation of the unbound bases.

To deal with the nonlinear behaviour, the mechanical attributes may examine by a Finite Element Model computer program.

To make the final result user friendly, it is more advisable to go for clever use of commonly accepted linear elastic approaches. The objective of the study is to investigate how the inaccuracies of linear elastic modelling of granular layers can be circumvented as best as possible.

In second phase the critical stresses and strains and the material characteristics are correlated to easy to measure parameters as surface deflections and layer thickness. These relationships form the basis of the development of predictive models for structural evaluation of unbound road bases in thin asphalt pavements.

For this study elaborated the following topics will be:

Keywords: thin asphalt, unbound granular base, nonlinear behaviour, permanent deformation, deflection, Falling Weight Deflectometer (FWD).

1. Introduction

1.1. General

Unbound granular materials can be applied in almost any road pavement structure, when it is used as a motorway base, or as a base of low-volume roads.

Therefore it is very important to know something about its behaviour. The main function of this base is to reduce the vertical compressive stress induced by traffic, in the sub-base and the subgrade, to a level at which no unacceptable deformation will occur in these layers.

This study focuses on the stress dependency of the base, being the most interesting and meaningful part of the structure. It presents the mechanical and practical properties of unbound granular materials with special emphasis on the determination of its resilient modulus and permanent deformation, which is achieved by proper material characterisation of the unbound bases.

A three layer model was developed to investigate the mechanical and material properties of unbound base. This model was tested by the computer program Bisar3PC and the American software Kenlayer. For modelling the nonlinear behaviour granular base, Kenlayer provided excellent opportunities, not in use of the Hungarian pavement designing procedure for the moment.

2. Falling Weight Deflection Testing

2.1. Introduction

Many structural evaluation procedures of road and airfield pavements use the Falling Weight Deflectometer (FWD) as a critical element of Non-destructive Deflection Testing (NDT). In principle the measurements can be applied on all types of pavements (asphalt concrete, cement concrete and block paving).

The FWD simulates moving truck wheel loads. In FWD testing an impulse loading is applied to the pavement. By means of falling weight deflections measurements it is possible to determine the structural condition of the pavement and the bearing capacity of the subgrade.

2.2. Fields of Interest

The FWD can be used in all phases of the pavement life of a road and airfield pavement structure:

- Testing at the road base or the binder course under construction provide more accurate results for the required thickness of the total of asphalt layers.
- Testing at in-service roads allows assessment of the residual pavement life and determination of appropriate maintenance and rehabilitation measures.

Non-destructive Deflection Testing is of importance for setting maintenance schemes for pavement maintenance (Pavement Management System-PMS). Usually the FWD is applied at the project level for assessing the structural condition, but it is also used at network level to evaluate the status of all roads.

2.3. The Measurement

2.3.1. Application

The FWD is a trailer mounted device that delivers a dynamic force impulse to the pavement. The principle of the measurement is quite simple. The equipment uses a weight that is lifted to a given height with a guided system and is then dropped to a foot plate resting on the pavement. Deflection sensors are used to record the vertical displacement at the surface of the pavement. These deflections are used to backcalculate layer module and determine the critical strains and stresses for maintenance, rehabilitation, and future design.

This requires additional data on layer thicknesses and material properties. Traffic counts and information on axle weights provide essential data for the residual structural pavement life.

In case of insufficient residual life the type and thickness of the strengthening measure can be computed. Usually, this strengthening can be transferred into maintenance and reconstructions measures depending on the overlay thickness, reuse of material and estimated traffic loads and intensities.

2.3.2. *About the Apparatus*

The majority of the FWDs is manufactured by the following three principal suppliers:

- Dynatest (Denmark)
- Carl Bro Pavement Consultants (formerly Phønix, Denmark)
- KUAB (Sweden)

They control together more than 95 percent of the world market.

Fig. 1 gives a schematic picture of the measurement.

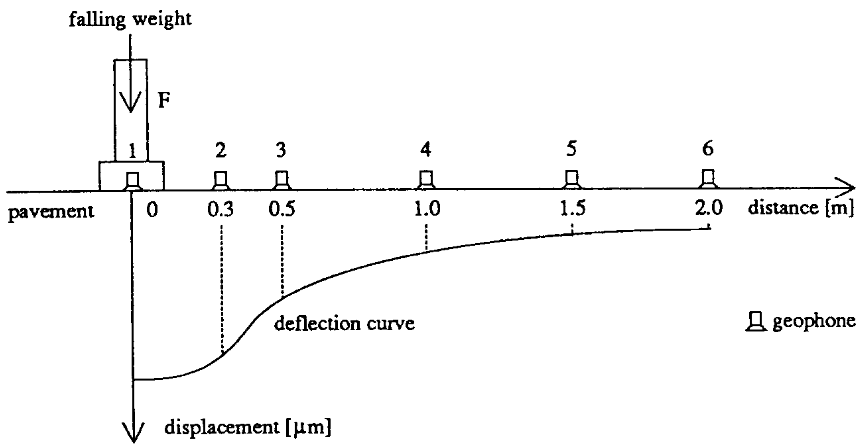

Fig. 1. Schematic picture of the measurement

The falling weight strikes a set of rubber buffers mounted to 300 mm circular foot plate, which transmits the force to the pavement. A thin ribbed rubber pad is always mounted under the footplate.

By changing the mass or the drop height or both, the impulse load can be varied. This load may be varied between 10 kN to 140 kN for regular types of FWD, but in some cases it may go up to 300 kN.

Six to nine deflection sensors (in the case of Dynatest geophones are used) measure the surface deflections caused by the impulse load.

The first deflection sensor is always mounted in the centre of the loading plate, while the remainder is positioned at various spatial distances up to 2.5 m from the load centre.

From all deflections recorded, peak values are stored and displayed. After lifting of the loading plate and the geophone beam, the whole setup may move to the next measuring point.

3. Pavement Structures in Model

3.1. The Dimension

The mechanical behaviour of this pavement structure was investigated by using a multi-layer model.

I divided the structure model into layers, the first is the asphalt layer, the second is the base layer which is subdivided into 5 sublayers (a, b, c, d and e), and the third is the subgrade (see *Fig. 2*).

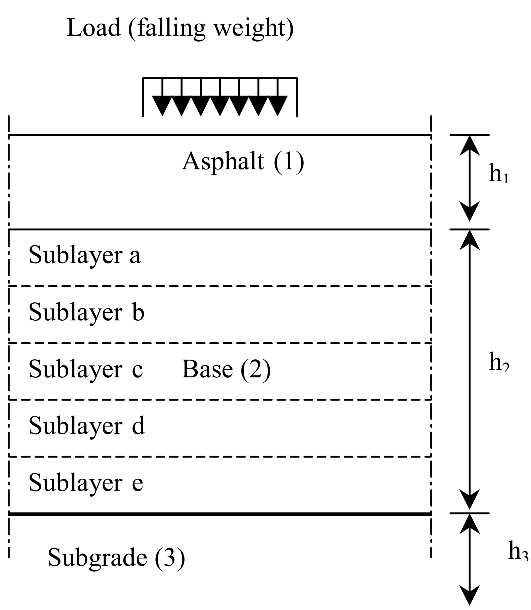

Fig. 2. The dimension structural modell

The load at the pavement surface is that of a typical FWD load. The idea of the 5 sublayers in the base will be elaborated in this chapter.

3.2. Load

Since it was the objective to use the results of the study for improved analysis of FWD data, the load in the model was a simulation of an FWD load. The load was set at 50 kN with a circular contact area with a diameter of 300 mm. This load not only resembles that of an FWD load (in terms of peak values) but also that of a wide base tyre with an axle load of 100 kN.

3.3. The Asphalt Layer

Wearing and bituminous base of the structure were not modelled separately. Only the thickness and the stiffness modulus of the total of the asphalt layers was varied. The following thicknesses were used 1 mm, 40 mm, 80 mm and 100 mm. The 1 mm case is typical for a sealed base.

The asphalt layer stiffness modulus was set to 2500 MPa, 5000 MPa and 10000 MPa. Poisson‘s ratio was fixed at 0.35.

3.4. The Base Layer

3.4.1. About the Unbound Granular Base Materials

The base layer is the most important part of this study. This layer transmits the loading to the subgrade, so the base plays a very serious role in the ‘force-game’. In this study the base consists of unbound granular materials, that may originate from various sources:

- conventional primary base course materials and natural gravel
- recycled base course materials and crushed rubble
- sands
- laterites

Only the first two types are considered in this study.

3.4.2. The Base Response Model

Unbound granular materials exhibit a stress dependent response. Usually the more stress is applied, the stiffer the material in the layer will respond. Various models are available that may describe this stress dependent behaviour. The following model is used mostly and is therefore applied in this study.

The material constants k_1 and k_2 come from the following equation, which describes a relationship between the stiffness and stress:

$$M_r = k_1 * (\theta/\theta_0)^{k_2}. \tag{1}$$

Meaning: M_r = resilient modulus (KPa)
θ = sum of principal stresses (kPa)
θ_0 = reference stress (1 kPa)
k_1 = material parameter (MPa)
k_2 = material parameter (-)

For appropriate use of this model, we should use a pavement model that would allow variation of stiffness both in vertical and horizontal directions. Only finite element approaches are capable solving this problem. Since variation in vertical direction only may meet very well the objectives set in this study, other programmes may be used. Kenlayer is such a program.

The following chart shows the used materials and their k_1 and k_2 values. The layer thicknesses are set at 150 mm, 250 mm and 350 mm.

	Material constant k_1	Material constant k_2
Crushed masonry/concrete	22.6 MPa	0.44
Crushed rubble	36.1 MPa	0.34
Crushed masonry	6.3 MPa	0.49
Limestone	37.7 MPa	0.45

3.5. The Subgrade

The subgrade is modelled as a semi half space consisting of linear elastic material. This involves that stiffness will also not vary much with depth. This allows the simplification of the stiffness characteristic and the use of a linear elastic model for the subgrade. The following stiffness moduli were used 50, 100 and 200 MPa. Poisson's ratio was set to a constant value of 0.35.

4. The Used Softwares

4.1. Kenlayer

Kenlayer is an American computer program, so it is possible to give the units in inch and Psi or m, kPa and kN. SI-units were used in this study.

Kenlayer can be applied to layered systems under single, dual, dual-tandem, or dual-tridem wheels with each layer behaving differently, either linear elastic, nonlinear elastic, or viscoelastic. Maximum 19 layers and 24 load groups are allowed in this program [3].

4.2. Modelling Granular Layers

It is well known that most granular materials cannot take any tension. Unfortunately, when they are used as a base or subbase on a weaker subgrade the horizontal stresses at the middle or bottom of these materials are most likely in tension. Two methods have been incorporated in Kenlayer for nonlinear analysis. In method 1, the nonlinear granular layer is subdivided into a number of layers and the stresses at the middepth of each layer are used to determinate the modulus. If the horizontal stress, including the geostatic stress, is negative or is in tension, it is set to 0. This stress modification is necessary to avoid negative θ. In method 2, all the granular materials are considered as a single layer and an appropriate point, usually between the upper quarter and the upper third of the layer is selected to compute the modulus. Because of the point at the upper part of the layer, the chance of negative θ is rare, so no stress modification is needed. If θ turns out to be negative, an arbitrary or minimum modulus (EMIN) is assigned.

5. Design Model for Prediction of Permanent Deformation in Granular Road Base

5.1. Background of the Model

5.1.1. Origin of the Model

The South African Mechanistic Design Method (SAMDM) and certain other components of pavement design have been developed extensively over the last few decades. The research efforts in South Africa to determine a simplified mechanistic design method have been going on since 1974.

The method includes the whole field of road pavement structure designing (material and pavement behaviour, design traffic, desired service level, etc.). The concept is also involved in pavement life prediction. Design models have been developed for each consistent layer of the pavement structure. Now, in this study we are only interested in the unbound granular base layer.

The process starts off with the load and material characterization. The standard design load for South Africa is a 40 kN dual wheel load at 350 mm spacing between centres of the two tyres and a uniform contact pressure of 520 kPa due to the legal axle load of 80 kN allowed on public roads. The only criterion provided for granular base layers was that the working stress should be limited to 70% of static shear strength or that safe working stresses should be determined from repeated loading triaxial tests.

The above mentioned design procedure gives satisfactory results for the South African situation. However, the principal design conditions are not the same as in the Dutch situation. Subgrade and subbase are usually much stiffer in South Africa than in the Netherlands. Besides that material quality in South Africa is of a higher level leading to better compaction and consequently to better behaviour and performance. Also temperature and climatic conditions are far from equal. However, the basic model seems applicable since it allows input of ‘local’ data as material parameters and traffic induced stresses. In this way possibilities exist to copy the basic structure and to fine-tune the model to the Dutch situation. A similar approach may be repeated to calibrate the model to the Hungarian conditions. So, the purpose of this study is to apply the South African Model, to detect transfer problems and to develop an appropriate adjusted model.

5.1.2. The Basic Parameters

The South African researchers developed the concept of the ‘safety factor’ against shear failure for granular materials in their design method. The safety factor against shear failure is based on the Mohr–Coulomb theory for static loading and represents the ratio of the material shear strength and the applied stress causing shear. The safety factor for granular materials can be written as follows:

$$F = (K * (\sigma_{1f} - \sigma xx2m))/(\sigma zz2m - \sigma xx2m), \tag{2}$$

where: F = safety factor
K = constant 0.8
σ_{1f} = failure stress
$\sigma zz2m$ = vertical stress under load mid-depth granular layer
$\sigma xx2m$ = radial (horizontal) stress under load mid-depth granular layer.

The constant K has been set to 0.8 in this study. Lower values must be used if the layer is situated in a zone where moisture may accelerate distress. Per material at least three static triaxial tests to failure were performed, the σ_{conf}-level was different in these tests. The result of the three tests shows that σ_{1f} increases with σ_{conf} [4]. This increase of σ_{1f} is explained by the failure criterion of Mohr–Coulomb which results to the following expression:

$$\sigma_{1f} = ((1 + \sin\phi) * \sigma_{\text{conf}} + 2 * c * \cos\phi)/(1 - \sin\phi) \text{ [4]}, \tag{3}$$

where: c = cohesion of the material [kPa]
ϕ = angle of internal friction of the material [°]
σ_{conf} = confining pressure [kPa]

The more traffic will travel over the road to be designed, the tougher the limits are that have to be set to the safety factor. In the South African approaches several levels of reliability are used for the various classes of the road. In case of low volume roads the level of reliability was set to 70% loading to a z-score of 0.524. The following formula was developed in South Africa:

$$\text{Log}\,(N) = 2.605122 * F - 0.62792 * z + 4.510819\ [2], \tag{4}$$

where N = number of load applications
F = safety factor
z = z-score of standard normal distribution (for 70% reliability $z = 0.524$)

The coefficients in *Eq.* (4) are the result of combining result from calculations with results of field trials with the Heavy Vehicle Simulator (HVS). These coefficients might need adjustment for countries than South Africa.

5.1.3. The Complications and the Alternative Solutions

Very often, the structural analysis of a pavement with a granular base and subbase will result in tensile stresses in the granular base, resulting in almost no resistance against shear failure predicted by the mechanistic method. This is particularly applied to the linear elastic approach allowing tensile stress to be developed in the granular bases. The linear elastic model and the resulting Mohr circle for such a case are illustrated in *Fig. 3*.

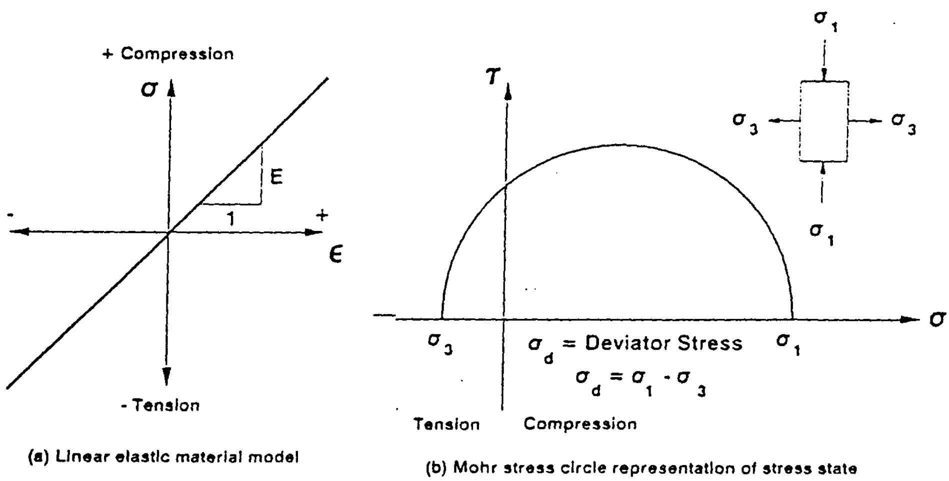

Fig. 3. The Mohr circle where tensile stress is allowed

To circumvent the drawback of the linear elastic approach the procedure proposed in the software code Kenlayer [3] was used. This procedure subdivides the granular layer into several linear elastic sublayers.

For all sublayers the *Eq.* (1) applies the stress dependency concept.

Based on the stress dependency model, the load induced stresses and overburden of the pavement structures. Stiffness moduli are computed for each sublayer via iteration.

The idea does not allow any tensile stress to develop in unbound base materials. If a tensile minor principle stress is calculated (negative value), the value is set equal to zero. What this implies in practice is that the granular layer will only carry loading in compression. Therefore the Mohr circle is modified as you can see in *Fig.4*.

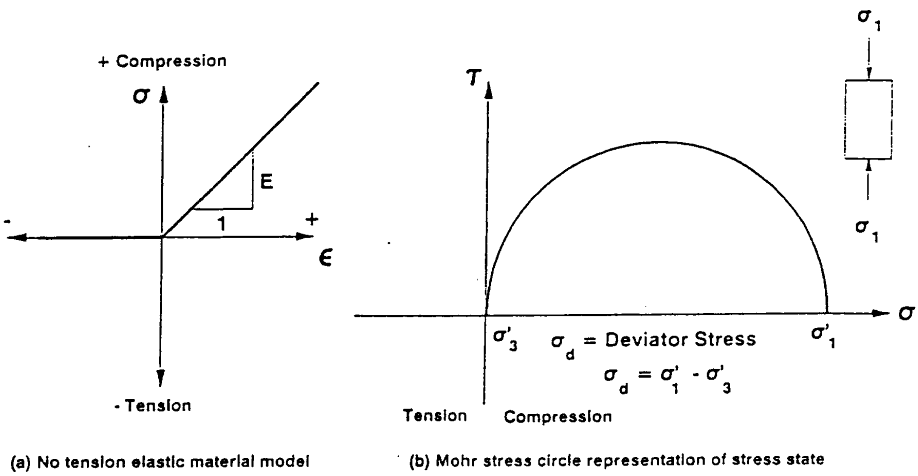

Fig. 4. The Mohr circle where is only allowed compression

Although the Kenlayer procedure tries to minimize the occurrence of the tensile stress in the granular layer, it does not provide absolute certainty. On the contrary, in most cases tensile horizontal stresses are computed middepth in the granular base. This is hardly the case in South Africa. This seems odd, but it can be explained easily. In South Africa the subbases are usually built up with a stiff material, whereas in the Netherlands usually soft subbases are applied. In this case the tensile stress will be computed in the granular layer. This will not be true in case of stiff subbases.

6. Results

6.1. Vertical Stress

Statistical analyses were made by using SPSS to find accurate relationships between the dependent variables vertical and horizontal stresses in the centre of the granular base and the independent variables. The set of independent variables consisted of easy-to-measure data only. Not all independent input variables appeared to be discriminative or suitable to explain the correlationship.

The regression equations can be applied for all base course material. Not only base course materials vary in characteristic and behaviour, also other materials are used in base course. For that reason an attempt was made to see whether results of all base course materials analysed could be combined (see *Fig.* 5).

The correlation coefficient shows that an accurate predictive equation could be developed to estimate the vertical stress from deflection and thickness parameters.

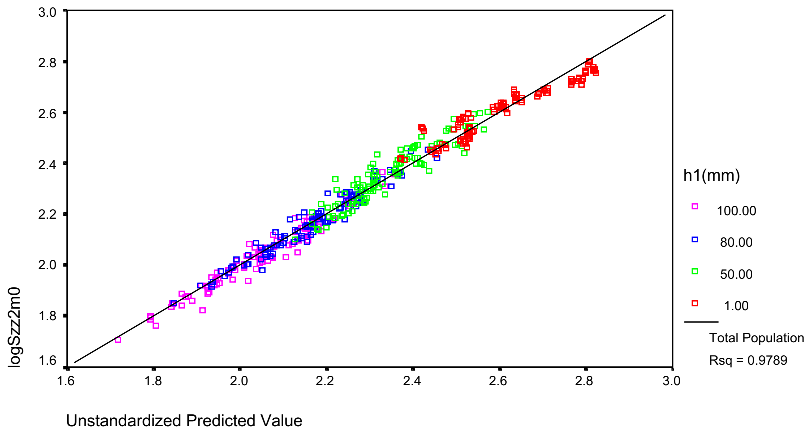

Fig. 5. Goodness of fit for base course materials combined

6.2. Horizontal Stress Directly from Kenlayer

6.2.1. Known Base Course Material

Unlike the vertical stresses more problems were expected with the tensile (negative) horizontal stresses. The problem was that the horizontal stresses in the centre of the base layers are most of them in tensions. This involves that it is not possible to use the logarithmic regression of the negative values. For that reason a log-linear relationship was investigated. The independent variables were all converted into logarithmic input variables whereas the dependent variable, the horizontal stress, was entered as a linear variable.

Fig. 6 presents a graphical result. This graph shows the goodness of fit between the actual stress (vertical axis) and the predicted stress (horizontal axis).

As you can see from the results the crushed masonry performed worst in the regression analyses probably because it is the weakest material.

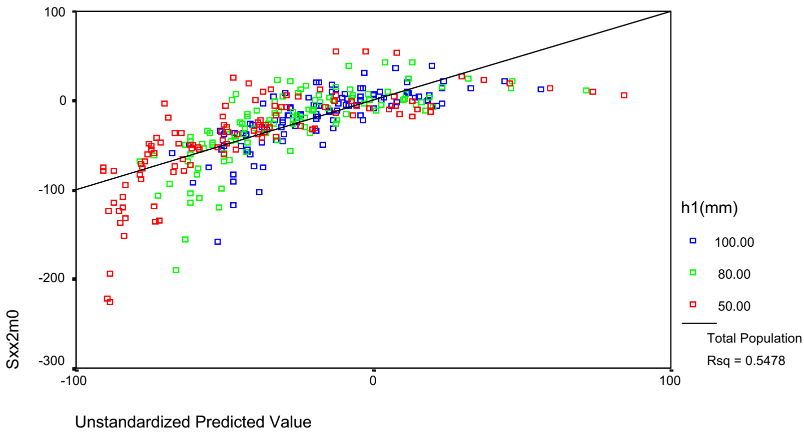

Fig. 6. Goodness of fit for base course materials combined

6.2.2. Unknown Base Course Material

The very thin asphalt layers were exluded from the analyses because they could not be well modelled together with the other thicknesses of the asphalt layers. Even then no good correlationship could be developed for all base course materials. (See *Fig. 6*).

The figure shows that no accurate general relationship could be developed to predict the horizontal stress middepth in the granular base course.

Another problem is that in the majority of the cases, tensile stresses were calculated leading to unrealistic low safety factors in the South African approach.

The communication with the developers of the South African model revealed, that they suffered too much under the tensile stresses. They disclosed that as a result of this had shifted the safety factor upward as having it taken necessary from their experience.

It was obvious that the plan was not feasible, so another solution had to be found to eliminate the negative values, and to get realistic safety factors.

6.3. The Horizontal Stress via an Alternative Approach

6.3.1. Known Base Course Material

Section 6.2 revealed that in determination of reliable horizontal stresses in granular layers you will be faced with problems even when the change of stiffness modulus with stress in the vertical direction is properly modelled. The program Kenlayer will come up with tensile horizontal stresses in the lower half of the granular layer.

In reality this will hardly be the case. Except for some confinement and residual stresses, there will be no tensile horizontal stresses.

The strains, however, represent in the lower half rather accurately the actual behaviour, since the strains are coupled to bending of the layered system. For this reason an alternative approach was set up to develop a procedure of assessment of the horizontal stress middepth in the road base, based on calculated strains at that point.

The alternative approach uses the vertical stresses as computed by Kenlayer as principal input data. The horizontal stress is defined as a function of the vertical stress, as presented below.

$$Sxx2m_0 = f * Szz2m_0.$$

The conversion factor f is assumed to be dependent on the ratio of the horizontal strain and vertical strain middepth in the road base.

The calculations for the four road base materials mentioned above showed that the ratio varied between -0.6 and -0.2. At -0.6 structural capacity of the road base is at it weakest, therefore the lower limit of the conversion factor is set to zero for this strain ratio. At -0.2 and higher ratios the structural capacity of the road base is assumed to be good. From this ratio and higher, the conversion factor is set equal to the coefficient of earth pressure.

Several soil mechanics and pavement design references fix this coefficient to a value of 0.5 or 0.6. It is convenient to consider that the horizontal stress is equal to the vertical stress times an arbitrary constant K which depends on the stress or deformation conditions but does not vary with depth if the same conditions are applied to the material all the depths [5]. In this study the coefficient is set to a value of 0.5.

Based on the two fixed points the following relationship exists between the conversion factor f and the strain ratio.

$$f = 1.25 * (\varepsilon_h/\varepsilon_v) + 0.75.$$

The strain ratio can easily be calculated in design approaches via Kenlayer. For in-service roads, however, we need to predict this ratio based on deflections and layer thicknesses. Logarithmic regression ($\log(-Exx2m/Ezz2m)$) was applied where the strain ratio was transformed to.

The next figure (*Fig. 7*) displays the results graphically for one of the materials investigated.

6.3.2. Unknown Base Course Material

The following table lists the principal output data of the statistical analyses performed on all base course materials. *Fig. 8* presents the goodness of fit graph.

The picture divides very spectacularly the results into two parts. One group contains the crushed masonry – the weakest material – and the other group contains

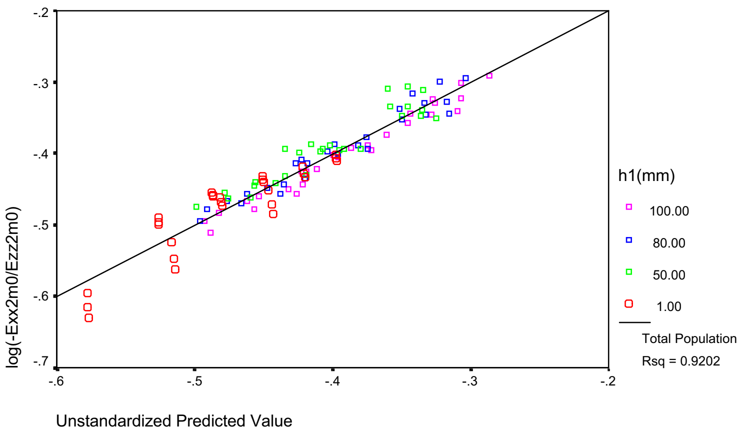

Fig. 7. Goodness of fit for base course material crushed rubble

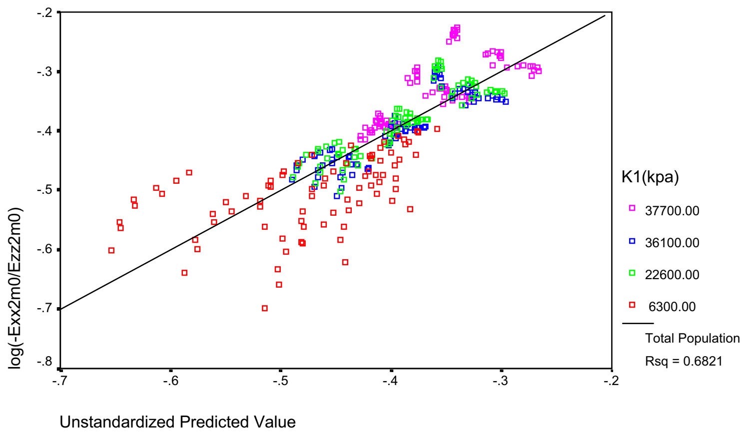

Fig. 8. Goodness of fit for base course materials combined

the remaining three materials. It is obvious, that the crushed masonry caused the bad regression result, so in a next step this material was erased from analyses.

The next graph (*Fig. 9*) presents the main results. Clearly a more accurate relationship is found between the strain ratio and deflection and layer thickness parameters, although still some scatters may be observed.

Fig. 9 shows that the lowest volume of $\log(-Exx2m/Ezz2m)$ is -0.5 which is a transformation of strain ratio of -0.32. *Figs.*5–6 showed that in case of crushed masonry the lowest value of $\log(-Exx2m/Ezz2m)$ could be down to -0.7 (strain ratio -0.20). The corresponding values of the conversion factor f are 0.35 and 0.5. The higher this factor, the higher the safety factor in the South African model

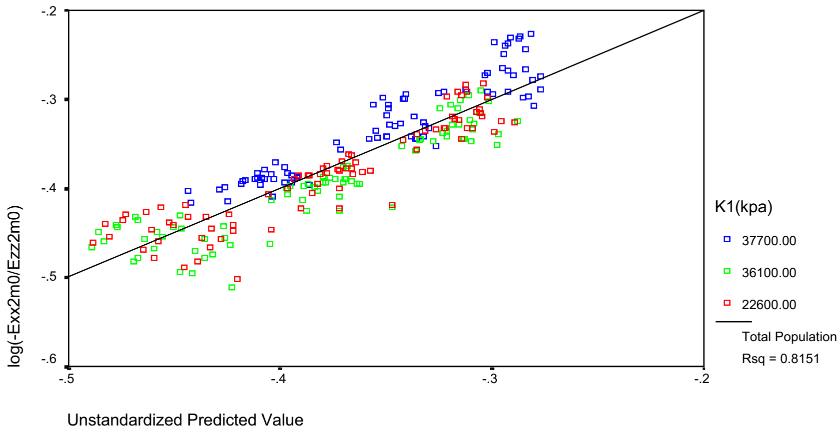

Fig. 9. Goodness of fit for three base course materials combined

is. If we apply the relationship developed without crushed masonry for a case with crushed masonry, we will underestimate the safety factor. This involves that we make a conservative, safe design when the 'without' relationship is used for the 'with' case.

6.4. Adjusted Predictive Model

Based on the results of the sections, the safety factor can be determined. Due to the modified alternative model in each case positive and more realistic value were found. This is the point where this approach departs from the South African model. The following graph (*Fig. 10*) presents some results.

Relationships are presented between the layer thickness h_1 and the safety factor (F) with varying E_1 and layer thickness $h_2 = 0.25$ m and with constant $E_3 = 100$ MPa.

The graph (see *Fig. 10*) reveals very well the differences between the thick and less thick layers or the stiff or less stiff materials.

In the case of the limestone which is the stiffest material among the three others not always the highest safety factor value expected was obtained.

The reason behind this is that the used cohesion (c) and angle of internal friction (ϕ) were not sufficient for this material.

When the safety factor is computed, then the following equations (see chapter 5) can be used to calculate the number of load applications (predicted life period):

$$\text{Log}\,(N) = 2.605122 * F - 0.62792 * z + 4.510819\ [7],$$

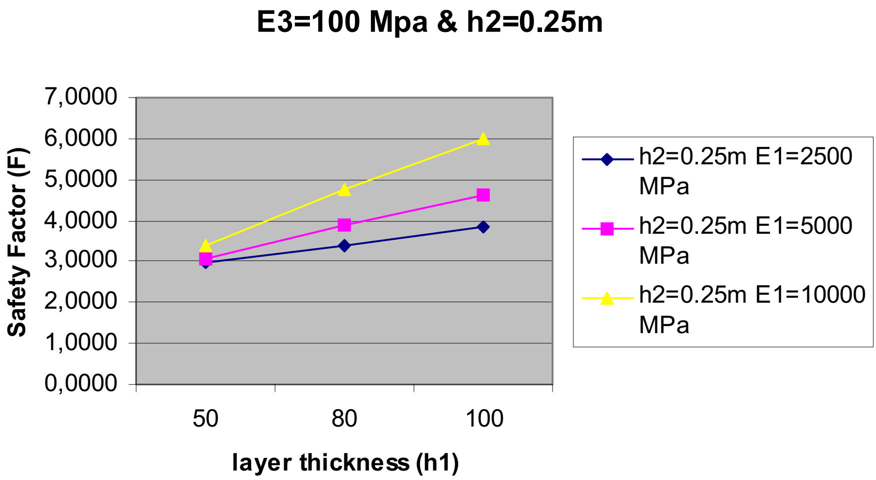

Fig. 10. Safety factor when E_3 and h_2 are constant and h_1 and E_1 are variable

where N = number of load applications
F = safety factor
z = z-score of standard normal distribution.

There are problems to overcome in using the equation. The constants in the equation were determined for the South African structural conditions.

Another point is that the basic South African model, the safety factor was allowed to vary between 0 and 2. The figures above show that this factor varies between 2.3 and 7 via the alternative approach. This absolutely makes it necessary to adjust the equation and to calibrate it to the local conditions.

A first adjustment step that can be made is to adapt the original South African predictive model to the 'new' safety factors. If we assume that the range of the South African factor from 0 to 2 is equal to the range of the 'new' factor from 2 to 8, then the following relationship exists.

$$F_{\text{South Africa}} = 1/3 * (F_{\text{new}} - 2).$$

If we assumed that scatter around the equation is the same, then the South African equation can be rewritten as follows:

$$\text{Log}\,(N) = 0,068 * F - 0.62792 * z + 2.774.$$

7. Conclusions and Recommendations

7.1. Conclusions

The study presented in this report leads to a number of main conclusions:

- The used software code Kenlayer is very efficient to determine the mechanical and structural behaviour of the non-linear elastic granular materials. Stress dependent granular layers can be simply modelled by using the well known $K-\theta$ model. Care should however be taken in computing horizontal stresses in the modelled granular layer. Especially in the lower half of the granular layer tensile stresses are found even when this layer is modelled as a stress dependent layer. The vertical stresses computed by Kenlayer meet the expectations.
- The determination of the so-called safety factor in the South African predictive model for permanent deformation of unbound granular road bases is more complex than it was explained in the original source. The safety factor is driven by the horizontal and vertical stress mid-depth in the layer under analyses and its failure characteristics. Especially the determination of the horizontal tensile stress cannot be performed via simple modelling to arrive at useable safety factors.

7.2. Recommendations

- More research efforts should be put into the accurate determination of stress in unbound granular layers. These approaches should account for that in the lower half of these layers no or hardly any tensile stress should be found. It is recommended to modify multi-layer programmes for this purpose rather than to develop finite element programmes. Anisotropy is regarded as a viable option.

References

[1] HORNSTRA, A., *Dimensioneren Lichte Verharding Report*, KOAC-WMD, Apeldoorn, NL, 2000.
[2] HUANG, Y. H., *Pavement Analysis and Design*, Englewood Cliffs, NJ, USA, 1993.
[3] HUURMAN, M., *Permanent Deformation in Concrete Block Pavements*, PhD-thesis, Delft University of Technology, Delft, NL, 1997.
[4] SCOTT, F. R., *Principles of Soil Mechanics*, Palo Alto, Massachusetts, USA, 1963.
[5] SWEERE, G. T. H., *Unbound Granular Bases*, PhD-thesis, Delft University of Technology, Delft, NL, 1990.
[6] THEYSE, H. L. – de BEER, M. – RUST, F. C., *Overview of the South African Pavement Design Analysis Method*, Divisional Publication DP-96/005 Transportek, CSIR, Pretoria, South Africa, 1996.