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RESEARCH ARTICLE

# Influence of Nonlinear Resilient Models of Unbound Aggregates on Analysis and Performance of Road Pavements

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***Abstract***

*Proper characterisation of the mechanical properties of unbound granular materials (UGM) is an essential issue in the analysis and design of flexible pavements. In particular, the resilient modulus of aggregates is a key input parameter in UGM characterization and prediction of pavement structural performance. In the present work, three UGM constitutive models are implemented within an axi-symmetric finite element code developed to simulate the nonlinear behaviour of pavement structures including two local aggregates of different mineralogical nature, typically used in Algerian pavements. The performance of these mechanical models is examined with regards to their capability of representing adequately, under various conditions, the granular material non-linearity in pavement analysis. In addition, deflection data collected by falling weight deflectometer (FWD) are incorporated into the analysis in order to assess the sensitivity of critical pavement design criteria and pavement design life to the three constitutive models. Finally, conclusions of engineering significance are formulated.*

***Keywords***

*nonlinear resilient models · unbound aggregates · RLT test results · FWD backcalculations · finite element simulation · pavement analysis · performance prediction*

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## 1 Introduction

Low to moderate volume roads in Algeria cover more than 70% of the total road network of the country [1]. These road structures typically consist of thin asphalt layers and structurally significant unbound base and sub-base granular layers over subgrade to carry the traffic load. It follows that accurate modelling of granular layer behaviour is of crucial importance for the analysis and prediction of pavement structural performance [2, 3].

The conventional way of designing a flexible pavement structure in road pavement engineering is to assume a constant stiffness over the granular layer thickness or to derive empirically this stiffness from a rigidity ratio depending on the mechanical properties of the subgrade soil (e.g. [2, 4]). However, experimental evidence [5–7] shows clearly that the resilient modulus of an unbound granular layer is a non-linear function of the stress levels acting at the various points of the aggregates.

Because of this material non-linearity and the sensitivity of the main pavement design criteria to the likely variations of the resilient modulus of the granular layers, numerical simulation of the non-linear mechanical behaviour of unbound granular materials (UGM) need be developed [3, 8, 9] .

Various constitutive models have been developed to this end [7, 9]. These resilient models can be categorized into two main classes; the resilient modulus models [6, 8, 11] and the shear-volumetric strain models [3–5, 10], frequently used for UGM nonlinear characterization in pavement engineering.

In the present study, three nonlinear constitutive models are tested. These are the $K-\Theta$ [2, 13], the NCHRP (often called modified universal) [8] and $K-G$ models [3, 4]. These models are then implemented within an axi-symmetric FEM code developed herein to evaluate their influence on nonlinear analysis and structural performance of road pavements. The finite element simulation results are validated using FWD backcalculated moduli to predict critical pavement response and design life. In addition, the sensitivity of pavement design criteria and estimated pavement design life to the likely variations of unbound granular material mineralogical nature is assessed and conclusions of engineering significance are formulated.

## 2 Analysed pavement structure and materials

In the following, the analysed pavement structure, the materials used and their mechanical properties are presented in detail.

### 2.1 Pavement structure

In order to study the influence of the three constitutive nonlinear models of unbound aggregates on critical responses and predicted pavement performance, a typical pavement structure has been selected. Details of the analysed pavement structure, with 5 m of subgrade soil over a rigid bottom under two thicknesses of unbound granular layers and a relatively thin asphalt layer, are summarised in Table 1.

**Tab. 1.** Layer thicknesses of studied pavement structure

| Asphalt (m) | Base (m) | Sub-base (m) | Soil (m) |
|---|---|---|---|
| 0.05 | 0.20 | 0.20 | 5.0 |

The structure represents, practically, a flexible pavement with structurally significant unbound granular layers commonly used as pavements subjected to low to medium traffic volumes. The pavement structure was analysed by varying the mechanical properties of two unbound granular materials with different mineralogical nature while keeping all other parameters unchanged. The subgrade soil was not considered as a stress dependent material; thus only the stiffness- stress dependency in the unbound granular material was investigated.

### 2.2 Materials and mechanical properties

A frequently used non-destructive in-situ device commonly used to estimate the elastic moduli of the constitutive layers of a pavement structure is the Falling Weight Deflectometer (FWD). The analysis of deflection data collected from FWD in situ tests provides a relatively rapid and reliable procedure to characterise the stiffness properties of constitutive layers of an existing pavement structure. In the present study, based on the FWD measured load and deflections, issued from an experimental pavement section built as part of an intensive project on local unbound granular materials [14], the elastic moduli of the analysed pavement structure have been backcalculated using the ELMOD software.

Various studies have been recently performed [1, 14, 15] to estimate laboratory measured resilient moduli of subgrade and base materials from FWD backcalculated moduli. The AASHTO suggests multiplying the backcalculated moduli for UGM and subgrade soils by appropriate adjustment factors to determine the $M_r$ values used for pavement design, in accordance with the values reported in Table 2 [18].

#### 2.2.1 Asphalt

The experimental results issued from the FWD test show that the FWD backcalculated modulus of the asphalt layer and Poisson's ratio are found to be equal to 5042 MPa and 0.35 respectively. Note that this value of asphalt layer elastic modulus is

**Tab. 2.** Ratio of laboratory $M_r$ to field backcalculated $E_{FWD}$ modulus values for unbound materials

| Layer type | Location | $M_r$ / $E_{FWD}$ |
|---|---|---|
| Aggregate | Between a stabilized and HMA layer | 1.43 |
| | Below a PCC layer | 1.32 |
| | Below an HMA layer | 0.62 |
| Subgrade | Below a stabilized subgrade | 0.75 |
| | Below an HMA or PCC layer | 0.52 |
| | Below an unbound aggregate base | 0.35 |

26% greater than the average value prescribed by the Algerian manual for pavement design [1].

#### 2.2.2 Unbound granular materials

***- Nonlinear elastic models considered in the study***

The resilient modulus of pavement foundation materials is a key input parameter in the analysis of flexible pavement structures (e.g. [8, 13, 15]). It is of paramount importance in UGM characterization and pavement structural performance prediction [19].

Many techniques including laboratory testing (e.g. [7, 20]), non-destructive in-situ investigations and correlations with empirical parameters (e.g. [6, 14]) were proposed to measure the resilient modulus of unbound granular materials. However, laboratory determination, in the form of Repeated Load Triaxial tests, has been regarded as the most accurate method of obtaining the resilient modulus property.

The resilient modulus ($M_r$) is defined as the ratio of the stress deviator ($\sigma_d$) to the resilient axial strain ($\varepsilon_r$).

$$M_r = \frac{\sigma_d}{\varepsilon_r}$$

where:

$$\sigma_d = \sigma_1 - \sigma_3$$

$\sigma_1$ Major principal stress
$\sigma_3$ Minor principal stress (confining pressure)
$\varepsilon_r$ Recoverable strain

Several Models [6, 9, 21] have been developed over the years that combine applied stresses and material characteristics to describe the nonlinear behaviour of granular materials under traffic loading.

- ***The K- $\theta$ model*** has been the most famous for characterizing the resilient response of granular bases and sub-base materials [21]. The resilient modulus (Mr) is given as follows:

$$M_r = k_1 x \theta^{k_2} \quad (1)$$

where $\theta$ is the first invariant of stress tensor given as follows:

$$\theta = \sigma_1 + \sigma_2 + \sigma_3 = \sigma_1 + 2x\sigma_3$$

$\sigma_2$ Intermediate principal stress
$k_1, k_2$ Material constants

Uzan [12] observed that the K - $\theta$ model did not summarize measured data well when shear stresses were significant, and proposed a three parameter model. This model is given as:

$$M_r = k_1 x \theta^{k_2} x \sigma_d^{k_3} \tag{2}$$

Witczak and Uzan [22] proposed a modification to the Uzan model by replacing the deviator stress term in Eq. (2) by an octahedral shear stress term. This octahedral shear stress model also considers the dilation effect that takes place when a pavement element is subjected to a large principal stress ratio $\sigma_1 / \sigma_3$.

This model is called Universal Model and is given as follows:

$$M_r = k_1 \times \theta^{k_2} \times \tau_{oct}^{k_3} \tag{3}$$

where $\tau_{oct}$ is the octahedral shear stress given as:

$$\tau_{oct} = \frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_1 - \sigma_3)^2}$$

The parameters $k_1$, $k_2$, and $k_3$ are multiple regression constants evaluated from resilient modulus test data.

- ***The NCHRP model*** [8] introduced in the Mechanistic Empirical Pavement Design Guide ( MEPDG), estimates the resilient modulus using a generalized constitutive model for Level 1 analysis for the nonlinear stress-dependent modelling of both the unbound aggregates and fine-grained soils. The resilient modulus used in the 2004 MEPDG design procedure, is as follows:

$$M_r = k_1 \times P_a \times \left(\frac{\theta}{Pa}\right)^{k_2} \times \left(\frac{\tau_{oct}}{Pa} + 1\right)^{k_3} \tag{4}$$

where $P_a$ is the atmospheric pressure.

- ***The K-G model*** is the third nonlinear resilient model utilized to simulate the non-linear resilient behaviour of the two granular layers used in the present investigation. The resilient modulus expression is as follows [3,23]:

$$E = \frac{9G_a\left(\frac{p}{Pa}\right)^{1-n}}{3 + \frac{G_a}{K_a}\left[1 - \beta\left(\frac{q}{p}\right)^2\right]} \tag{5}$$

with:

$$\beta = (1-n)\frac{K_a}{6G_a},$$
$$q = \sigma_1 - \sigma_3,$$
$$p = \frac{\sigma_1 + 2\sigma_3}{3}$$

In the above expressions, $p$ and $q$ represent respectively the volumetric and deviator stresses. The stresses $\sigma_1$ and $\sigma_3$ are the major (axial) and the minor (confining) principal stresses respectively. The parameters $K_a$, $G_a$, $n$ are material constants and $P_a$, *a* conventional pressure equal to 100 kPa.

This three parameter model ($K_a$, $G_a$, $n$) was selected for the sake of simplicity and for its relative capacity to adequately represent the high deviatoric shear stress found in thin asphalt pavements . It assumes that the shear and volumetric strains are linked and that the material is isotropic. It offers various advantages and a reasonable compromise between simplicity and accurate modelling [21].

- ***RLT test results of the two aggregates considered in the study***

Aggregates used in this study were provided respectively from Cap-Djenet and Bordj-Bouarreridj deposits located in Northern Algeria. These two deposits have great potential for high production levels of granular materials [7].

Aggregates issued from Cap-Djenet (CAP) deposit are of basaltic origin. They were produced from volcanic eruptions. Basalt is a rough stone, fairly light in weight and grey in colour.

Aggregates issued from Bordj-Bouarreridj (BBA) deposit are limestones, produced from crushing sedimentary rocks composed mainly of calcium carbonate ($CaCO_3$). Most limestones are hard and durable. They typically produce strong aggregates with low water absorption and are suitable for road stones.

The main results of the RLT tests for the two local unbound granular materials are summarised in Table 3. Non-linear regression analysis is carried out to determine model parameters of nonlinear resilient constitutive models for both UGM BBA and CAP. The adequacy of resilient modulus prediction was assessed in this study using the coefficient of determination, $R^2$, which represents the proportion of variation in the predicted variable that is accounted for by the regression model and takes on values ranging from zero (i.e. no correlation) to one (i.e. perfect correlation).

The main results of the regression analysis including the coefficient of determination ($R^2$) are presented in Table 3, below.

The results showed that the coefficient of determination, $R^2$ values for all nonlinear resilient models ranged between 0.81 and 0.98.

It is shown that NCHRP model gives the highest values of the coefficient of determination for both studied UGM, when compared to the K - Θ and the K - G models. The NCHRP model is thus more accurate in simulating the measured responses. Such results are in line with previous findings obtained in other investigations [8, 9].

Based on the model regression parameters reported in Table 3, the three above considered constitutive models may be used at a later stage for the nonlinear finite element analysis of flexible pavements utilizing the two local tested aggregates. Moreover, these models may be advantageously used to determine more realistic average values of characteristic resilient modulus $M_r^c$ than those issued from empirical classification of unbound granular materials [5, 7].

As an example, the Algerian manual for pavement design utilizes the characteristic modulus for comparison of stiffness properties of unbound granular materials obtained from Eq. (5) (with value of shear ratio ($q$ / $p$) equal to 2 and mean normal stress ($p$) equal to 250 kPa). The following results were found for the characteristic resilient modulus.

**Tab. 3.** Model parameters for unbound granular materials used in the present study.

| UGM | K - θ | | | NCHRP | | | | K - G | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | $k_1$ (MPa) | $k_2$ | $R^2$ | $k_1$ (MPa) | $k_2$ | $k_3$ | $R^2$ | $K_a$ (MPa) | $G_a$ (MPa) | $n$ | $R^2$ |
| BBA | 28.30 | 0.51 | 0.91 | 3.14 | 0.70 | -0.41 | 0.98 | 172 | 186 | 0.45 | 0.81 |
| CAP | 23.68 | 0.50 | 0.85 | 2.58 | 0.77 | -0.58 | 0.95 | 86 | 172 | 0.36 | 0.82 |

$$M_r^c = 678\,MPa \text{ for UGM BBA.}$$

$$M_r^c = 467\,MPa \text{ for UGM CAP.}$$

It is clearly seen that the aggregates issued from limestone crushed rocks (UGM BBA) are much more rigid than those issued from crushed eruptive rocks (UGM CAP). Similar conclusions can be obtained by substituting appropriate values of $p$ and $q / p$ in Eqs. (1) and (4) for the K - $\theta$ and NCHRP models.

It is to be noted that the in-situ FWD tests were performed on the pavement test section utilizing UGM BBA as base and subbase granular materials. Thus, the field backcalculated modulus $E_{FWD}$ value must be multiplied by an adjustment factor equal to 0.62 as indicated in Table 2 [18].

### 2.2.3 Subgrade soil

The mechanical properties of the subgrade soil were not considered variable in the present study; the value of 63 MPa for the resilient modulus of the subgrade soil is estimated on the basis of the FWD backcalculated subgrade soil modulus (using ELMOD software) converted to an equivalent laboratory $M_r$ value using an adjustement factor equal to 0.35 (see Table 2). Poisson's ratio is taken equal to 0.35.

Note that the above value of the subgrade resilient modulus is rather close to the minimum value of subgrade modulus corresponding to subgrade soil type S2 defined in the Algerian manual for pavement design.

## 3 Numerical modelling of pavement structures

The continuous trend towards improved computing facilities coupled with increasing knowledge of the mechanical properties of materials has enhanced the crucial need for the development of finite element codes for the non-linear analysis and design of pavements, both in mainframe and personal computers.

### 3.1 Finite element modelling

The finite element method is particularly efficient for modelling the non-linear behaviour of pavement structures as it can easily accommodate variability in material properties, changes in pavement geometry and modifications in applied loading. In pavement engineering, pavement structures are often modelled as axi-symmetric systems. The finite element domain is modelled using 8-node rectangular ring elements; each node having two degrees of freedom associated with the nodal displacement components in the vertical and the radial directions [23, 24]. The elements each contain four Gaussian points at which stresses and strains are calculated. The mesh is automatically made for a structure by superposed layers of elements, the material parameters being constant for each layer. The heights of these horizontal layers are constant or in geometrical progression in some groups, and the computer code does the same by columns of elements. The structure is meshed as shown in Tables 4 and 5. The letters R and P respectively stand for a regular mesh and a mesh in geometrical progression.

**Tab. 4.** Vertical mesh

| Layers | Number of elements | Mesh type |
|---|---|---|
| Asphalt | 3 | R |
| Base | 5 | R |
| Subbase | 5 | R |
| Subgrade | 6 | P |

**Tab. 5.** Horizontal mesh

| Zones | Number of elements | Mesh type |
|---|---|---|
| 1 | 3 | R |
| 2 | 3 | R |
| 3 | 4 | R |
| 4 | 8 | P |

Near the load where the stress and strain gradients are large, the mesh should be fine whereas at greater distances from the applied load, the mesh can be coarser. Because of axi-symmetry, both geometrical and material, only the region to one side of the load centre line was considered.

The mesh is fixed at the bottom allowing no lateral movement and rollers on the sides allow vertical displacement to take place. For illustration purposes, a schematic representation of a F.E model used for pavement analyses is presented in Fig. 1 (not to scale).

The pavement structure was subjected to a circular load which has radius of 17.5 cm and uniform pressure of 676 kPa.

Table 6 summarises constitutive material properties of pavement structure layers including BBA granular base and sub-base materials for the three UGM nonlinear models considered in present investigation.

Note: A constant Poisson's ratio of 0.35 is considered for all layers (i.e. same value used in ELMOD software).

The values of linear model moduli for UGM BBA in base and sub-base layer were found to be equal to 600 and 189 MPa respectively in accordance with conventional procedures used in pavement design [1, 25]. Similarly, corresponding values for

**Tab. 6.** Material properties of pavement layers used for studied UGM constitutive models (UGM BBA)

| Layer | Adjusted FWD modulus (MPa) | Linear model modulus (MPa) | Constitutive models |
|---|---|---|---|
| A.C | 5042 | 5042 | Linear elastic |
| Base | 285 | 600 | Linear elastic: $M_r = 600$ MPa<br>Nonlinear elastic:<br>- K - $\theta$ model (Eq. (1))<br>- NCHRP model (Eq. (4))<br>- K - G model (Eq. (5)) |
| Sub-base | 285 | 189 | Linear elastic: $M_r = 600$ MPa<br>Nonlinear elastic:<br>- K - $\theta$ model (Eq. (1))<br>- NCHRP model (Eq. (4))<br>- K - G model (Eq. (5)) |
| Subgrade | 63 | 63 | Linear elastic |

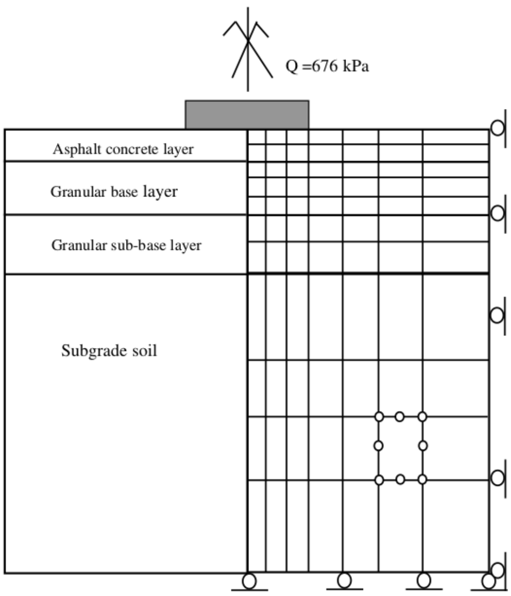


**Fig. 1.** Schematic representation of a F.E model used for pavement analyses (not to scale)

UGM CAP were found to be equal to 400 and 158 MPa respectively.

For nonlinear calculations purposes, values of regression parameters to be used for the tow tested UGM are given in Table 3 for each of the nonlinear resilient constitutive models considered in the present work.

### 3.2 Method of resolution

The application of the finite element method [24] to the analysis and design of flexible pavement structures characterised by nonlinear elastic behaviour may be briefly summarised as follows:

$$\begin{aligned} \{\varepsilon\} &= [B]\,\{U\} \\ \{\sigma\} &= [D]\,\{\varepsilon\} \end{aligned} \tag{6}$$

where:

$\{\varepsilon\}$ Resilient strain tensor

$\{\sigma\}$ Resilient stress tensor

$\{U\}$ Nodal displacement vector

$[B]$ Strain displacement matrix

$[D]$ Elasticity matrix

$$\{F\} = \sum_i \int_V [B]^T \{U\}\, dV \tag{7}$$

and

$$\{F\} = [K]^e \{U\} \tag{8}$$

with:

$$[K]^e = \sum_i \int_{V_I} [B]^T [K]^e [B]\, dV \tag{9}$$

In these equations:

$[K]^e$ is the secant stiffness matrix of the structure, which depends on the stresses.

$\{F\}$ is the nodal force vector, which is given by the load due to the half-axle of a vehicle.

To calculate $\{U\}$, the direct iteration method [4, 23, 24] was used as follows:

$$\{U^n\} = \left[K^{n-1}\right]^{-1} \{F\} \tag{10}$$

and

$$\{\sigma^n\} = \left[D^{n-1}\right] [B]\, \{U^n\} \tag{11}$$

This method is illustrated in Fig. 2. The initial values are initial stresses due to self-weight. The program continues to iterate until sequential displacement computations agree with some specified tolerance.

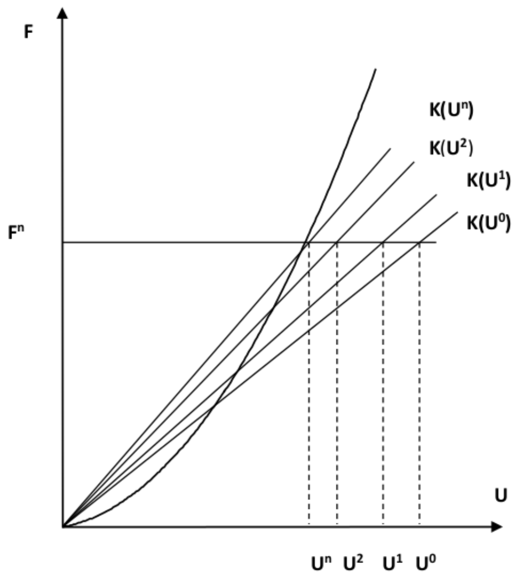


**Fig. 2.** Illustration of the resolution method

## 4 Results of non-linear analyses and discussion

The main results of the numerical analyses were summarised in terms of values of

1 the design criteria generally used in pavement engineering:
   - The deflection at the surface, which is, to some extent, an indication of the structure ability to bear repeated traffic loads.
   - The horizontal tensile strain at the bottom of the bituminous layer usually related to risks of asphalt layer cracking by tensile fatigue failure.
   - The vertical strain at the top of the soil usually related to risks of rutting of pavement.

2 the number of load repetitions to failure obtained from either the fatigue cracking or the rutting distress models.

Radial strain at the bottom of Asphalt layer and vertical strain at the top of Subgrade are two critical responses which are also used to control fatigue and rutting of flexible pavements respectively [1, 2]. Error in computation of these two responses results in erroneous prediction of the pavement performance (e.g. [26, 27]).

Several fatigue and rutting models have been developed to relate the asphalt modulus and the measured strains to the number of load repetitions to pavement failure [2, 28]. Most of the fatigue cracking and the rutting failure models usually take the following forms:

$$N_c = f_1 \times \varepsilon_t^{f_2} \times E_1^{f_3} \tag{12}$$

$$N_r = f_4 \times \varepsilon_v^{f_5} \tag{13}$$

where:

$N_c$ Allowable load repetitions to prevent the cracking fatigue of the asphalt layer

$N_r$ Allowable load repetitions to prevent the rutting at the top of subgrade soil due to accumulated pavement deformation.

$\varepsilon_t$ Maximum tensile strain at the asphalt layer

$\varepsilon_v$ Compressive vertical strain at the top of subgrade soil

$E_1$ The elastic modulus of the asphalt layer

$f_i$, $_{i=1,\ldots,5}$ regression distress model parameters depending on material type, definitions used to identify failure limits and climatic as well as traffic conditions [2].

The design life of a flexible pavement is the minimum number of load repetitions required to cause either fatigue or rutting failure. The performance model considered in this study is the model proposed by Asphalt Institute [2, 29] with the following $f_i$ values for the regression coefficients:

$$f_1 = 0.0796$$
$$f_2 = -3.291,$$
$$f_3 = -0.854,$$
$$f_4 = 1.365E-09,$$
$$f_5 = -4.477$$

### 4.1 Sensitivity of design parameters to UGM constitutive models

The values of the three design criteria and design life (for fatigue cracking and rutting distress models) reported in Tables 7 and 8 were computed using the linear and the three nonlinear constitutive models for granular base and sub-base layers of the analysed pavement structure. Validation by FWD backcalculations, of finite element simulation results using the various constitutive models for UGM BBA has been performed and corresponding results are presented in Table 7. In addition, for the sake of clarity, variation of design criteria and design life for the linear and nonlinear models based on UGM CAP are summarized in Table 8. Only the mineralogical nature and hence the stiffness of the unbound granular materials was varied. All the other geometrical and mechanical parameters of the pavement layers were kept unchanged.

- It is seen that design parameter values of the pavement structure are in general affected by the changes in the constitutive models used to simulate the resilient behaviour of the unbound aggregates. Nevertheless, it may be noted that a good agreement exists among critical responses computed using the NCHRP and K - G constitutive models.

- Responses computed assuming linear elastic behaviour for the base layer, present significant differences as compared to those based on nonlinear behaviour of the granular layers. For the case at hand, consideration of nonlinearity resulted in a 36% higher tensile strain at the bottom of asphalt layer and a 18% higher vertical strain over the subgrade, than the corresponding values obtained using linear elastic analysis. It is

**Tab. 7.** Validation by FWD backcalculations of F.E simulation results based on various constitutive models (UGM BBA).

| Models | FWD | Linear | K - Θ | NCHRP | K - G |
|---|---|---|---|---|---|
| **W (mm)** | **0.870** | **0.725** (0.83) | **0.746** (0.85) | **0.792** (0.91) | **0.794** (0.92) |
| $\varepsilon_t\left(10^{-6}\right)$ | **270** | **136.20** (0.50) | **144.30** (0.53) | **183** (0.68) | **186** (0.69) |
| $\varepsilon_v\left(10^{-6}\right)$ | **842** | **707.20** (0.84) | **781** (0.93) | **824** (0.97) | **837** (0.99) |
| $N_C$ | 433 332 | 4 119 670 (9.50) | 3 406 386 (7.86) | 1 558 521 (3.59) | 1 477 311 (3.40) |
| $N_r$ | 79 526 | 173 673 (2.18) | 111 361 (1.40) | 87 604 (1.10) | 81 675 (1.02) |
| **Design life** | **79 526** | **173 673** | **111 361** | **87 604** | **81 675** |

* Values in parentheses represent ratios of F.E simulation results to corresponding FWD backcalculated values.

**Tab. 8.** Variation of design criteria and design life based on UGM CAP

| ***Models*** | ***Linear*** | ***K - Θ*** | ***NCHRP*** | ***K - G*** |
|---|---|---|---|---|
| **W (mm)** | **0.831** | **0.838** (1.01) | **0.892** (1.07) | **1.07** (1.35) |
| $\varepsilon_t\left(10^{-6}\right)$ | **188** | **214.60** (1.14) | **238.60** (1.26) | **290.70** (1.74) |
| $\varepsilon_v\left(10^{-6}\right)$ | **821.60** | **862.70** (1.05) | **912** (1.11) | **1051** (1.40) |
| $N_C$ | 1426217 | 922669 (0.65) | 650917 (0.46) | 339814 (0.24) |
| $N_r$ | 88756 | 71 332 (0.80) | 55 620 (0.63) | 29 472 (0.33) |
| **Design life** | **88 756** | **71 332** | **55 620** | **29 472** |

important to note that these differences in the tensile strains and vertical strains will in turn result in exponentially amplified differences when predicting pavement performance (especially for fatigue cracking fatigue design life).

- Furthermore, it is noted from Tables 7 and 8 that allowable load repetitions computed based on Asphalt Institute transfer functions for predicting fatigue cracking and rutting design life, are notably different when using linear and nonlinear models of granular layers (especially for the fatigue cracking failure mode). It follows that conventional design methods usually based on multi-layer linear elastic theory significantly overestimate the pavement design life. This clearly illustrates the need of using proper constitutive modelling for characterization of non-linear aggregate behaviour and prediction of pavement response on the one hand, and the crucial importance of developing well calibrated distress models especially for predicting fatigue cracking design life on the other hand.
- For comparison purposes, the evolution of the granular layer resilient modulus with depth using the three non-linear constitutive models has also been investigated. Fig. 3 shows the values of resilient modulus versus depth on the load axis within the granular layer. It is to be noticed that the variation of resilient modulus with depth follows a nonlinear form. Noticeable differences at the upper part of the granular layer between the resilient modulus values calculated using the three models are observed. However, these differences decay with depth increase and become, as expected, very close at the lower part of the granular layer.

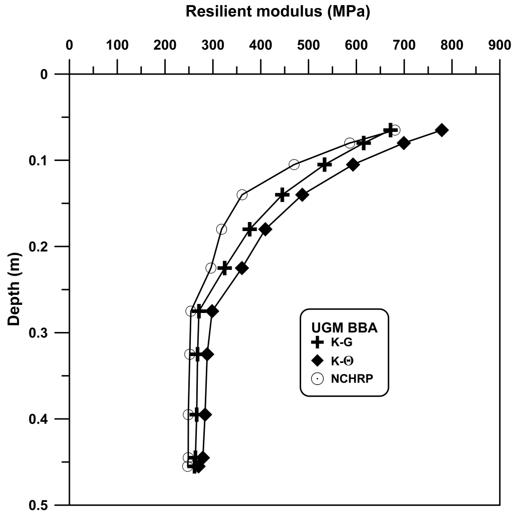


**Fig. 3.** Evolution of granular layer resilient modulus with depth

### 4.2 Effects of UGM mineralogical nature on design criteria and structural performance

It is observed from the results reported in Tables 7 and 8 that the three design criteria and pavement design life are very sensitive to changes in UGM mineralogical nature. In particular, the horizontal strain at the bottom of the asphalt layer is the most sensitive design parameter to variations in the mechanical characteristics of the unbound granular materials. In addition, it is clearly seen that the use of stiffer aggregates (UGM BBA) can reduce significantly the tensile strain at the bottom of the asphalt layer and hence increase the pavement fatigue life.

## 5 Summary and conclusions

In the present study, three constitutive nonlinear models have been implemented in a numerical code based on axi-symmetric finite elements to study the nonlinear resilient behavior of unbound granular materials and the structural performance of road pavements. Numerical investigations have been carried out to assess the influence of nonlinear resilient models on the behaviour of two local UGM of different mineralogical nature typically used for road construction in Algeria.

Simulation results show, among others, that good agreement is observed between critical pavement responses computed using NCHRP and K - G model. In addition, computed critical pavement design criteria and predicted pavement design life using linear and nonlinear analysis are found to be substantially different. This clearly demonstrates the importance of nonlinear characterization of base and sub-base granular aggregates for accurate pavement design and the need to develop well calibrated transfer functions for performance prediction especially for predicting fatigue cracking design life.

The simulation results also show that the use of stiffer aggregates can reduce significantly the tensile strain at the bottom of the asphalt layer and hence increase the pavement fatigue life which could reduce the construction cost of road pavements. This is especially important in flexible road pavements subjected to low to moderate volumes of traffic.

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