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Experimental characterisation and numerical modelling of the resilient behaviour of unbound granular materials for roads

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Abstract. This research paper deals with experimental characterisation and numerical modelling of the resilient behaviour of Unbound Granular Materials (UGMs) usually used in road construction. The first part of this paper describes the main results of an experimental program that was carried out to assess the mechanical properties of two local Unbound Granular Materials (UGMs) for construction purposes in road pavement. The second part of this paper is devoted to the numerical modelling of the resilient behaviour of UGMs used in flexible pavements. For this purpose, several nonlinear unbound aggregates constitutive models are implemented within an axi-symmetric finite element code developed to simulate the nonlinear behaviour of pavement structures. 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 constitutive models. Finally, conclusions of engineering significance are formulated.

Key words: Unbound granular materials, Experimental characterisation, Resilient behaviour, Finite elements.

1. Introduction

Low to moderate volume roads in Algeria cover more than 70% of the total road network of the country (Mamma, 2017). 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 (Huang, 2004), (COST 337, 2002).

In addition, for better understanding of the overall structural performance of flexible pavement structures, experimental tests where real traffic load conditions and in-situ physical and mechanical properties are accurately reproduced are needed. Several in-situ testing devices have been so far developed to predict the material behaviour of unbound granular materials (UGMs) under static or dynamic loading condition (Ekblad ,2008), (Ma et al., 2020), (Mneina & Shalaby, 2020), (Tutumluer, 2003).

However, due to cost considerations and difficulties to examine the influence of numerous factors affecting material behaviour using in-situ testing, laboratory tests have been developed to determine the mechanical behaviour of UGMs for pavement use. Repeated loading triaxial (RLT) tests (AASHTO, 2007), (EN 13286-7, 2004) have been extensively used to simulate the loading produced by the rolling wheel on granular materials when used as unbound granular layers in pavement construction under different conditions such as grading, density and moisture contents (Uthus et al., 2005).

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In this experimental study, the variable confining pressure (VCP) type RLT test (EN 13286-7, 2004), (Rondón et al., 2009) has been used as it offers the capability to apply a wide combination of stress paths by pulsing both the confining pressure and the vertical deviator stress. Such stress paths loading tests better simulate actual field conditions since in a pavement structure, the confining and loading stress acting on UGMs is cyclic in nature.

To analyse experimental tests results, several resilient strain behaviour models have been used by different investigators. These models have been categorized as either resilient modulus models or shear-volumetric strains models (Ekblad, 2008), (Lekarp et al., 2000). The K–G or shear-volumetric models are based on the decomposition of the principal stress and strain tensors, respectively, into two tensors: a deviatoric (or shear) and a volumetric.

In these models, nonlinear behaviour of aggregates is characterized through general equations in terms of stress dependent bulk and shear moduli. Basic assumptions are nonlinear elastic and isotropic material behaviour. The K–G or shear-volumetric model with three independent parameters (COST 337, 2002), (Jouve & Elhannani, 1994) is utilized in the present work to analyse the RLT test results and adequately incorporate the material nonlinearity due to the stress–stiffness dependency of the local aggregates. This model has the advantage to be in good agreement with RLT tests results and offers a reasonable compromise between accurate modelling and simplicity. Furthermore, based on extensive investigations of modelling laboratory RLT tests data, it has been found that shear-volumetric strain models give better predictions than resilient modulus models (COST 337, 2002).

In the first part of this research work, an extensive experimental study is carried out to examine the physical and mechanical properties of two local UGMs for road pavement use in Algeria. The study consists of three main stages:

1- Evaluation of physical properties in accordance with relevant tests standards.
2- Characterisation of the resilient deformations of the local UGMs and assessment of the effect of non-linear stress dependency on their behaviour.
3- Determination from RLT tests of the constitutive model parameters to be used at a later stage in nonlinear finite element analysis of flexible pavements utilizing the local tested UGMs.

The main objective of the second part of this research work is to examine the effect of nonlinearity in unbound materials on the response and the performance of a granular pavement. The UGMs nonlinear resilient model adopted by the AASHTO manual design guide is used (NCHRP, 2004). This model is then implemented within an axi-symmetric FEM code developed herein to evaluate its influence 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. Experimental study

In the following, the materials, their physical properties, the RLT testing equipment and procedure including sample preparation and testing program for the resilient behaviour characterization of the tested UGMs, are described. It should be noted that all tests used in the present work, were conducted in conformity with relevant European standards.

2.1. Materials

Aggregates used in this study were provided respectively from Cap-Djenet and Bordj-Bouarriridj deposits located in northern Algeria (Figure 1). These two deposits have great potential for high production levels of granular materials. 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 gray in color. 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 roadstones.

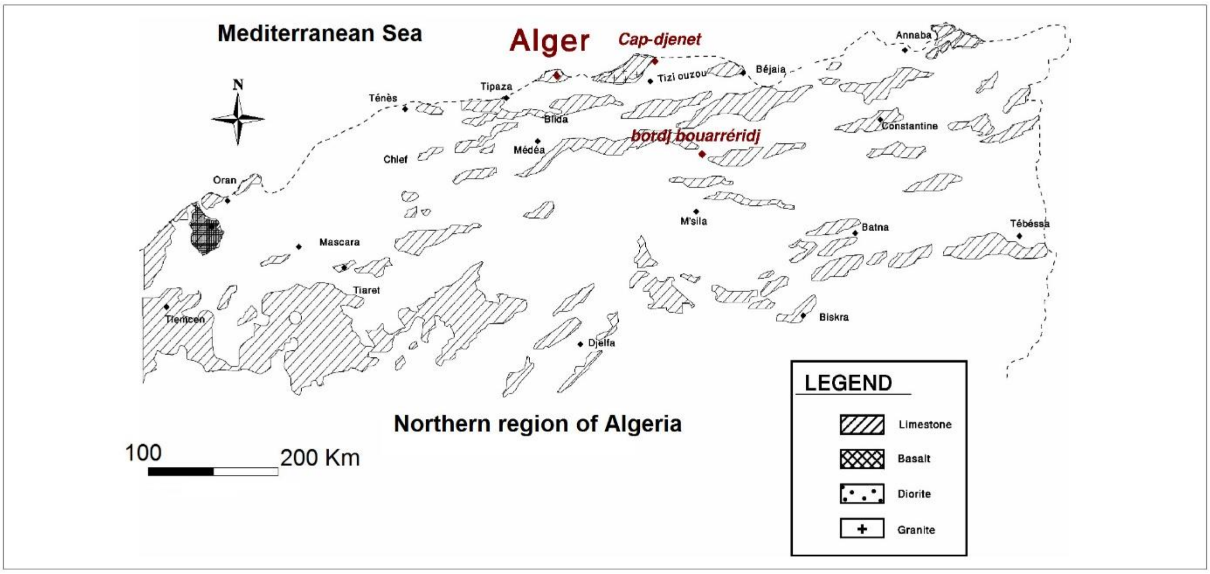


Fig 1. Map of aggregate materials deposits in Algerian northern region

2.2 Physical characterization

Granular materials consist of a large number of particles of different sizes. Previous investigations on their physical characterization show that such materials are dependent on maximum particle size and grain size distribution. Accordingly, unbound granular materials have been in the past empirically classified by maximum particle size and grain size distribution. On the other hand, in the French standard and in the European Standard EN 13285 (2010), (XP P18-545, 2004), the above characteristics are complemented by other tests results especially the Los Angeles (LA) and Micro-Deval (MDE) values. Because of their historical use and past acceptance, as well as document relationships to aggregate abrasion resistance and for empirical classification purposes, these two tests are included in the present work. Table 1, summarises the main state properties of the studied UGMs.

Table 1. Main state properties of the studied UGMs

Deposits	VB (%) *	Fines (%)	ES (%)	MDE (%)	LA (%)
CAP	2	9.5	55	14	12
BBA	1.3	7	55	24	23

**VB* methylene bleu value, *MDE* wet Micro-Deval value, *LA* Los Angeles value, *ES* sand equivalent ratio

The results indicated in Table 1 and reported in Fig. 2 show that the two local aggregates present acceptable state properties mainly in terms of LA and MDE values in accordance with the French standard XP P 18–545 (Paute et al., 1994), (Kolisoja, 1997).

In Figure 2, areas designated by the letters A to E represent various categories of mechanical resistance of UGMs aggregates corresponding to an empirical classification based on the values of LA and MDE. Its noted that UGMs CAP issued from eruptive crushed rocks gives better characteristic values than UGMs BBA issued from limestone crushed rocks.

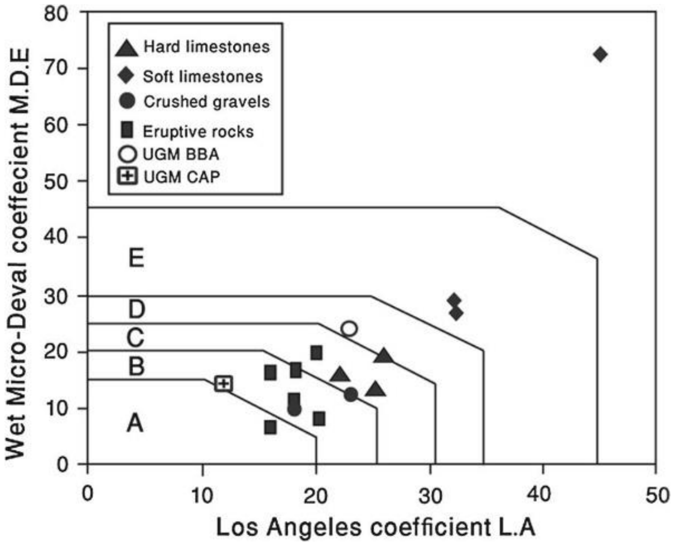


Fig 2. UGMs classification using Mechanical Index Tests data

2.3 RLT testing equipment and procedure

In order to study the behaviour of UGMs when used as unbound granular layers in pavement construction under different traffic and environmental state conditions, RLT tests can be used with advantage. The principal components of the RLT apparatus are illustrated in Figure 3 below.

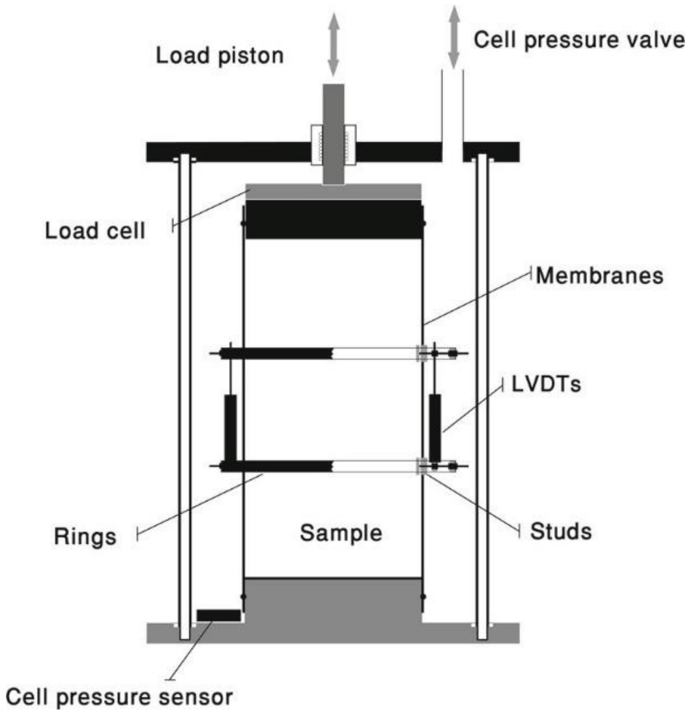


Fig 3. Triaxial cell and LVDTs

The axial load is capable of applying a vertical stress on samples of 16 cm diameter and 32 cm height. Example of triaxial cell and systems for measuring dis- placements using linear variable displacement transducers (LVDT).

The loading (Figure 4) is carried out in exercising cyclically and simultaneously a confining pressure σ3 (applied via pneumatic pressure) and an additional dynamic vertical stress q (deviator stress) (Figure 5a). Under cyclic loading, UGMs present resilient strains which are recovered after each cycle, and permanent strains which accumulate with the number of cycles (Figure 5b).

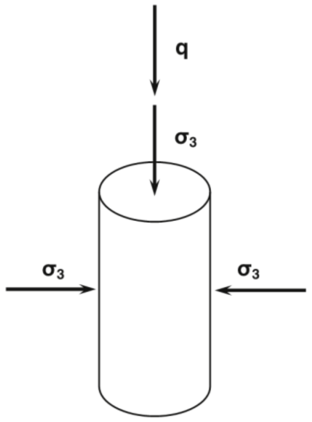


Fig 4. Stress state in triaxial configuration

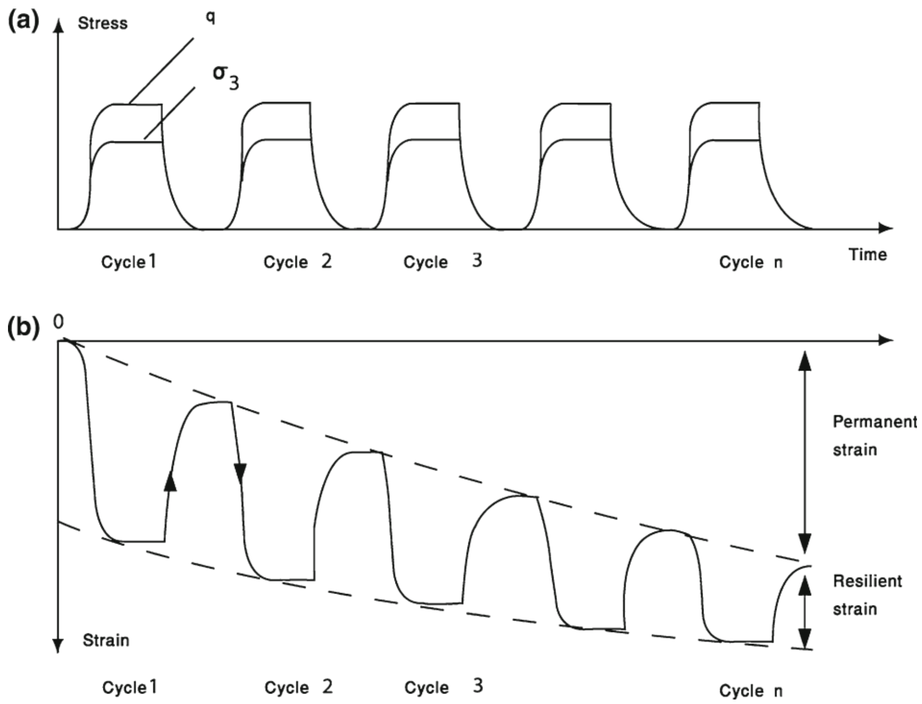


Fig 5. Applied stresses and measured strains in RLT tests: (a) applied stress history; (b) components of strain under cyclic stresses

The testing procedure consists in the following main tests. All tests were conducted in accordance with relevant French and corresponding European standards.

- Grain size distribution

Both unbound granular materials BBA and CAP are sieved in the laboratory in the following four fractions:

0–3, 3–8, 8–15, 15–25 mm in accordance with French standards NF P 98–125 (2009). It is observed from Figure 6 that the associated grading curves of the tested materials are practically similar and follow closely the average standard grain distribution curve.

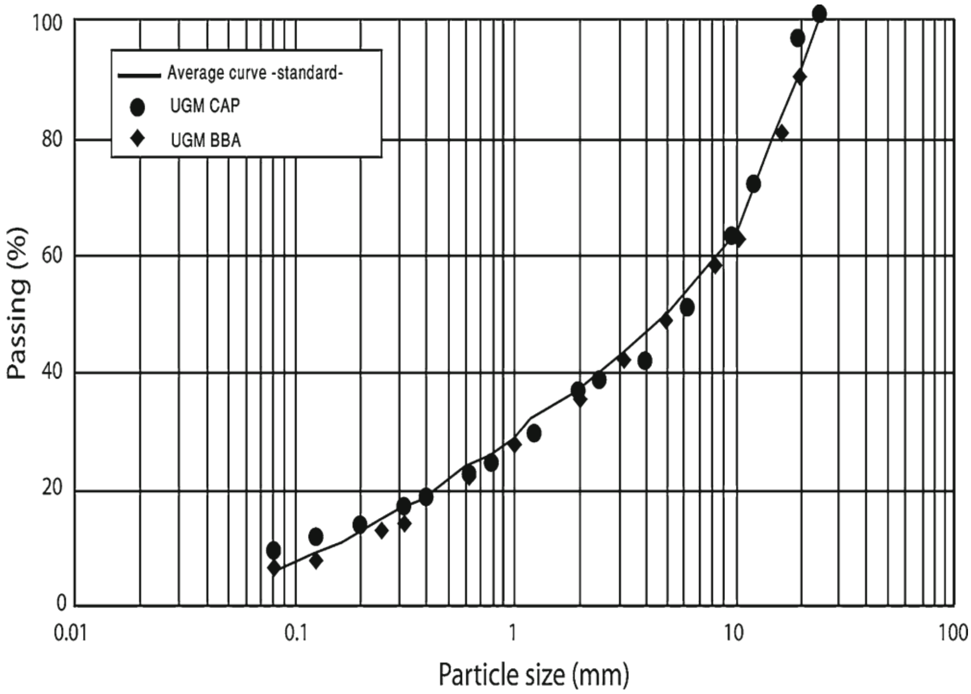


Fig 6. Particle size distributions

- Modified proctor test results

Modified proctor tests are performed on the two local UGMs to obtain the densities to which the materials should be compacted in the RLT tests. The results of UGMs identification are summarised in Table 2.

Table 2. Modified Proctor test results

UGMs	MDD (t/m^3)	OMC (%)
CAP	2.27	7
BBA	2.28	5

It is seen from above results that the aggregate samples are characterised by practically the same Maximum Dry Density (MDD) but with slightly different optimum moisture content (OMC (%)).

- Sample preparation

The specimens are prepared with a vibro-compression method (EN 13286-52, 2004). For the two UGMs, all specimens for both UGM are compacted to density equal to 97% of maximum dry density and a moisture content w in such a way that: w = OMC -3%. These density and moisture contents values are typical of those encountered in road pavements in Algeria.

- RLT testing programme

Tests were conducted following the test procedure for measurement of resilient deformations described by the European Standard NF EN 13286-7 (EN 13268-7, 2004). The number of cycles of loading was set at 20000. The programme included, for each UGMs, a series of four tests conducted at constant moisture content and under each of the different stress paths illustrated in figure 7 below.

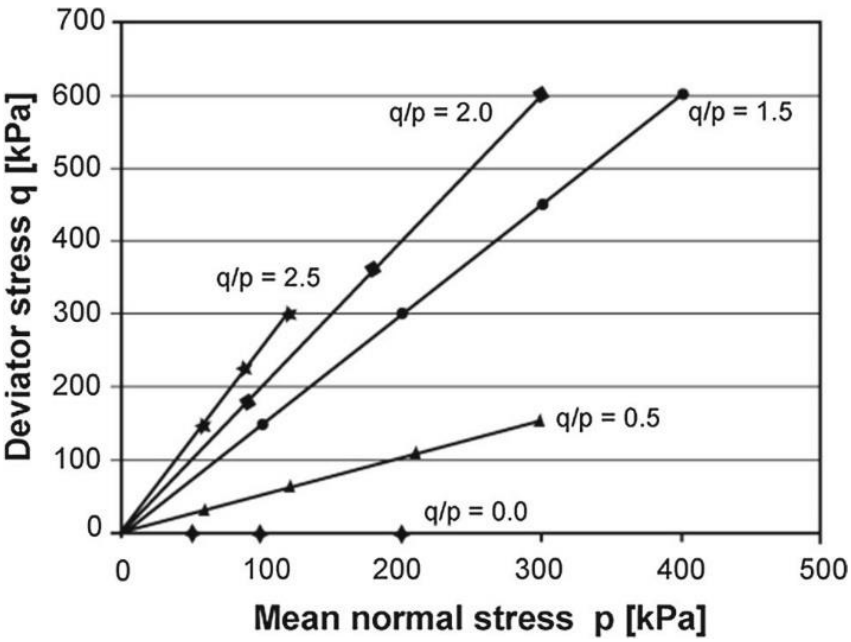


Fig. 7 Stress paths schedule

3. Analysis of experimental results

3.1 Modelling of RLT Tests Data

To analyse the tests results, nonlinear expressions for the volumetric and deviatoric strains derived from a shear-volumetric strain model with three independent parameters were considered (COST 337, 2002), (Jouve & Elhannani, 1994). This model (often referred to as the K–G model) is used to characterise nonlinear behaviour due to stress–stiffness dependency of the granular materials in terms of bulk and shear moduli. This model has the advantage to be in good agreement with RLT tests results and offers a reasonable compromise between accurate modelling and simplicity. Furthermore, based on extensive investigations of modelling laboratory RLT tests data, it has been found that shear-volumetric strain models give better predictions than resilient modulus models (COST 337, 2002).

The volumetric and shear components of the stress and strain parameters used in the K-G model can be written as:

$$\mathrm{K} = \frac{\mathrm{p}}{\varepsilon_{\mathrm{v}}} \text{ ; } \mathrm{G} = \frac{\mathrm{q}}{3\varepsilon_{\mathrm{q}}}$$

with:

$$\mathrm{p} = \frac{\sigma_1 + 2\sigma_3}{3}; q = \sigma_1 - \sigma_3$$

$$\varepsilon_v = \varepsilon_1 + 2\varepsilon_3; \varepsilon_q = \frac{2}{3}(\varepsilon_1 - \varepsilon_3)$$

ε_v and ε_q, is the resilient volumetric and deviatoric (or shear) strains, respectively; K, is the bulk modulus and G, is the shear modulus. The stresses σ_1, σ_3 are major (axial) and the minor (confining) principal stress, respectively. The same notations apply to the principal strains ε_1 and ε_3. Based on the nonlinear elastic properties of the K-G model and assuming material isotropy, the bulk and shear moduli can be determined as follows:

$$K = K_a \left(\frac{p}{p_a}\right)^{1-n} \left[1 - \beta \left(\frac{q}{p}\right)^2\right]^{-1} \quad (1)$$

$$G = G_a \left(\frac{p}{p_a}\right)^{(1-n)} \tag{2}$$

with:

$$\beta = (1-n)\frac{K_a}{6G_a} \tag{3}$$

In the above expressions, the parameters K_a , G_a , n are material constants and p_a is the atmospheric pressure, equal to 100 kPa conventionally. It is also seen from equations 1 and 2 that the parameters K_a, G_a have the same units as the elastic bulk and shear moduli respectively. It follows that the material constants K_a and G_a will have pressure units. Moreover, the volumetric and deviatoric strain expressions, as a function of the stress invariants, can be written as:

$$\varepsilon_v = p_a^{1-n} p^n \left[\frac{1}{K_a} - \frac{\beta}{K_a}\left(\frac{q}{p}\right)^2\right] \tag{4}$$

$$\varepsilon_q = p_a^{1-n} P^n \left[\frac{1}{3G_a}\left(\frac{q}{p}\right)\right] \tag{5}$$

The values of the model parameters are systematically determined by using a generalisation of the least square regression method applied to both volumetric and deviatoric strains (Jouve & Elhannani,1994). The accuracy on the two types of measurements can be summarised by the global correlation index:

$$\rho = 1 - \frac{1}{\sqrt{2}}\sqrt{(1-\rho_v)^2 + \left(1-\rho_q\right)^2} \tag{6}$$

where ρ_v and ρ_q represent the partial correlation indices associated with volumetric and deviatoric strain. The application of the nonlinear regression method to RLT tests data carried out on the local materials has provided the model parameters summarised in Table 3.

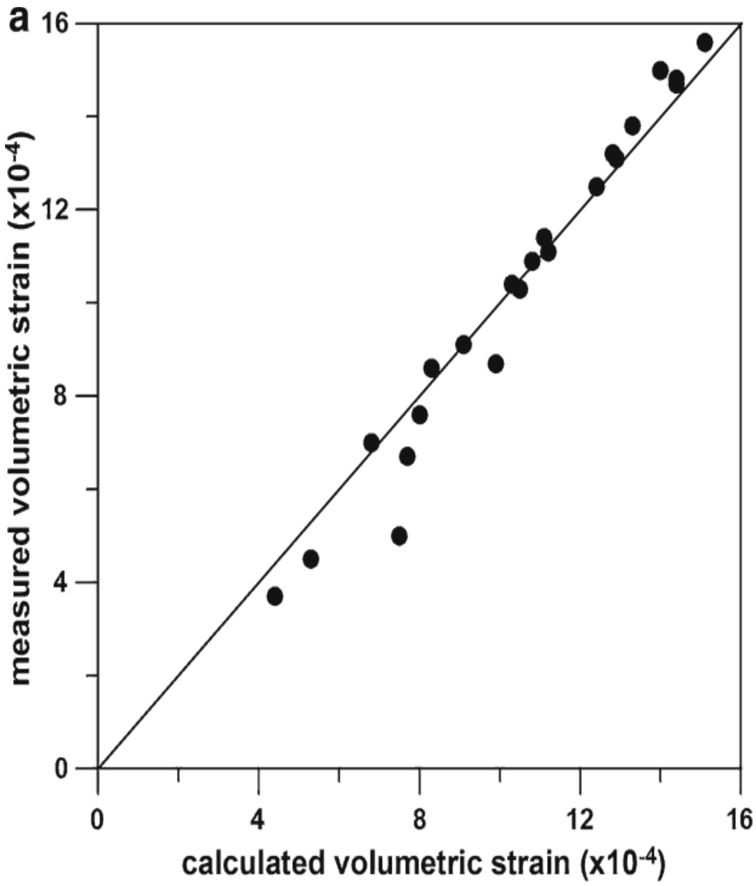
Table 3. K-G model parameters

UGMs	K_a (MPa)	G_a (MPa)	n
CAP	86	117	0.454
BBA	172	186	0.357

These model parameters are then used in Equations. 4 and 5 to calculate both volumetric and deviatoric strains for the CAP and BBA UGMs.

It is clearly seen from both Figures 8 and 9 (associated, respectively, with UGMs BBA and CAP) that the agreement between measured and calculated strains (volumetric as well as deviatoric) is very good with values of global correlation indices found as follows:

- UGMs CAP: ρ = 0.80; UGMs BBA: ρ = 0.81



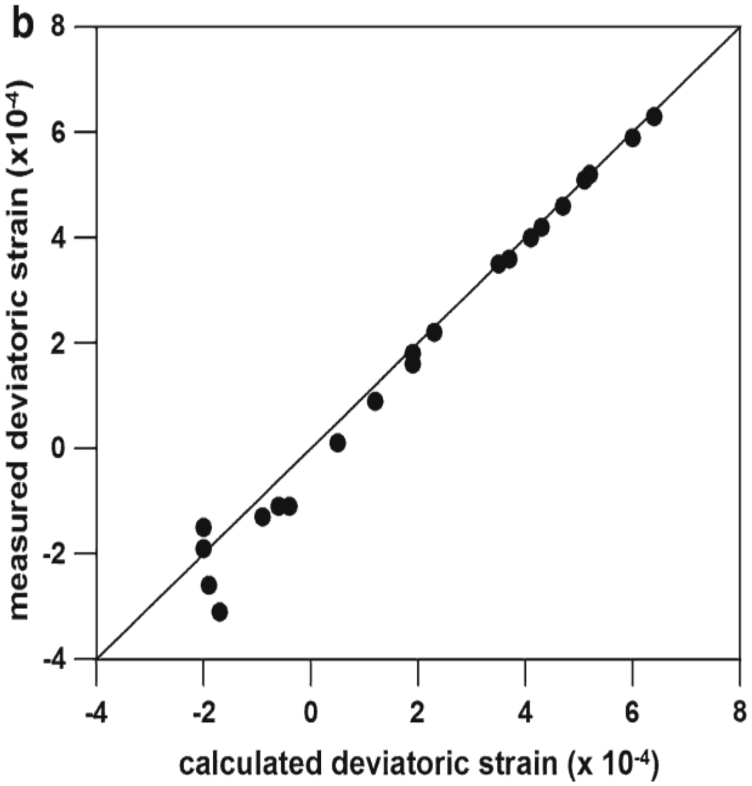


Fig. 8 Comparison between measured and calculated strain for UGMs BBA. a Volumetric; b deviatoric

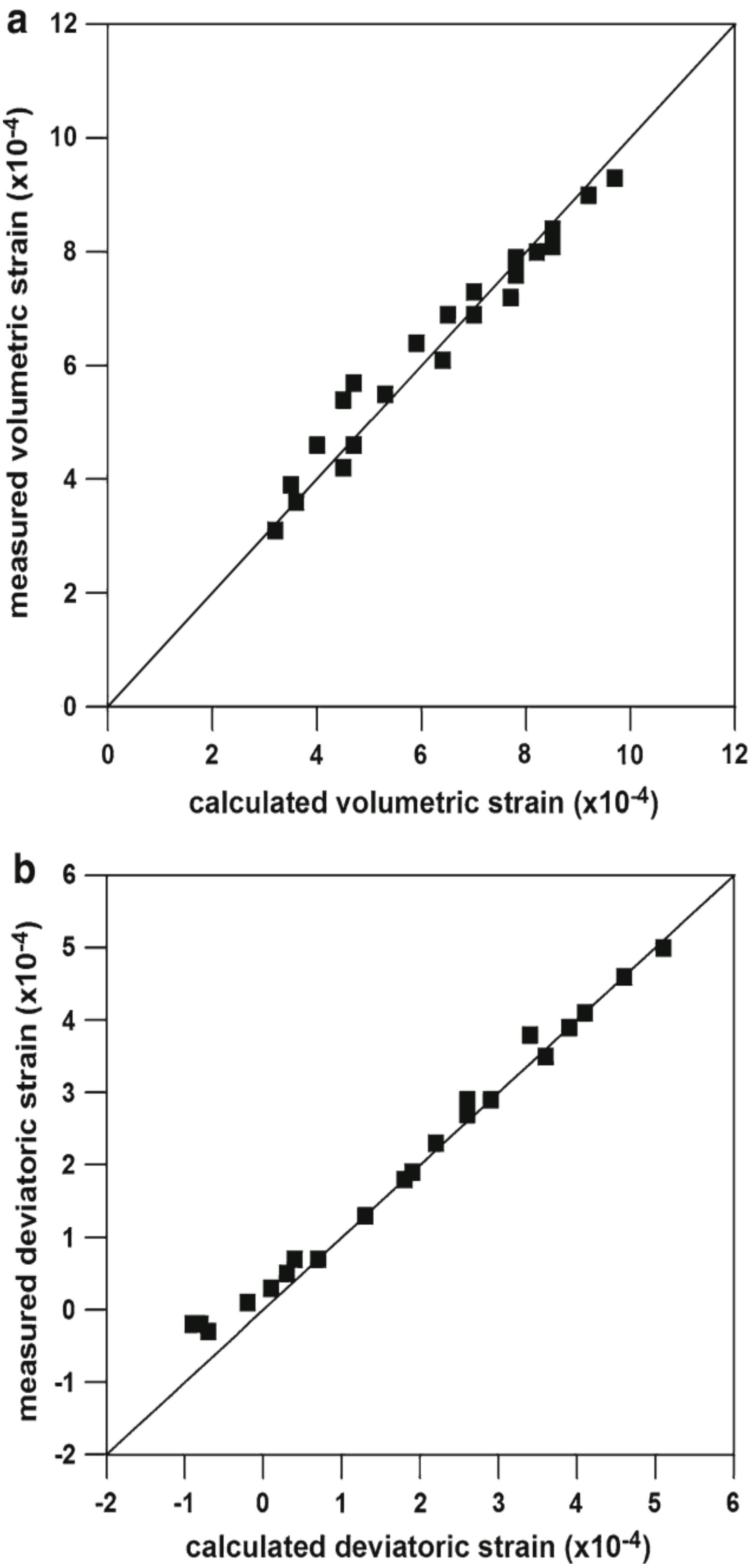


Fig. 9 Comparison between measured and calculated strain for UGMs CAP. a Volumetric; b deviatoric

4. Numerical modelling of the nonlinear resilient behaviour of UGMs

Structural analysis of flexible pavements has been and still is currently performed using multi-layer elastic theory. However, for thinly surfaced pavements subjected to low to medium volumes of traffics, the importance of nonlinear stress-strain behaviour of UGMs requires the use of more sophisticated numerical models for structural design and performance of such pavements.

The main objective of this section is to examine the effect of nonlinearity in unbound materials on the response and the performance of a granular pavement. The UGMs nonlinear resilient model adopted by the AASHTO manual design guide is used (NCHRP, 2004). This model is then implemented within an axi-symmetric FEM code developed herein to evaluate its 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.

4.1 Analysed Pavement Structure

In order to study the effect of the behaviour nonlinearity in granular layers on pavement response and performance, a typical pavement structure has been selected. Details of the analysed pavement structure are summarized in Table 4.

Table 4. Layer thicknesses of studied pavement structure

Asphalt (m)	Base (m)	Subbase (m)	Soil
0.05	0.20	0.20	5

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.

4.2 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 characterize 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 (Azzouni, 2010), the elastic moduli of the analysed pavement structure have been backcalculated using the ELMOD software.

Various studies have been recently performed (CTTP, 2001), (Ji et al., 2014) to estimate laboratory measured resilient moduli of subgrade and base materials from FWD backcalculated moduli. The AASHTO suggests multiplying the backcalculated moduli for UGMs and subgrade soils by appropriate adjustment factors to determine the resilient moduli (Mr) values used for pavement design, in accordance with the values reported in Table 5 (AASHTO, 2008).

Table 5. Ratio of Laboratory Mr to Field Backcalculated EFWD

Layer type	Location	Mr/EFWD
Aggregates	Between a stabilized and HMA layer	1.43
	Below a PCC layer	1.32
	Below an HMA layer	0.62
Subgrade soil	Below a stabilized subgrade	0.75
	Below an HMA or PCC layer	0.52
	Below an unbound aggregate base	0.35

4.2.1 Asphalt

The experimental results issued from the FWD test show that the 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 26% greater than the average value prescribed by the Algerian manual for pavement design (CTTP, 2001).

4.2.2 Unbound Granular Materials

The resilient modulus of pavement granular materials is a key input parameter in the analysis of flexible pavement (NCHRP, 2004). It is of paramount importance in UGMs characterization and pavement structural performance prediction (Almássy, 2002). Many techniques including laboratory testing (Sandjak & Tiliouine, 2014), (Rondón et al., 2009), non-destructive in-situ investigations and correlations with empirical parameters (Gonzalez et al., 2007), 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 (σ_d) to the resilient axial strain (ε_r).

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

with: $\sigma_d = \sigma_1 - \sigma_3$

Several models (Ekblad, 2008), (Lekarp et al., 2000) have been developed over the years that combine applied stresses and material characteristics to describe the nonlinear behavior of granular materials under traffic loading. The K-θ model has been the most famous for characterizing the resilient response of granular bases and sub-base materials (Zienkiewicz and Taylor, 2005). The resilient modulus (Mr) is given as:

$$M_r = k_1 \theta^{k_2} \tag{7}$$

where θ is the first invariant of stress tensor given as follows:

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

where, σ_2= Intermediate principal stress, k_1, k_2= Material constants.

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

$$M_r = k_1\,\theta^{k_2}\,{\sigma_d}^{k_3} \tag{8}$$

Witczak and Uzan (1988) proposed a modification to the Uzan model by replacing the deviator stress term in (8) 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 σ_1/σ_3. This model is called Universal Model and is given as:

$$M_r = k_1 \times \theta^{k_2} \times {\tau_{oct}}^{k_3} \tag{9}$$

where τ_{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 Expanded universal model (NCHRP, 2004), (AASHTO, 2008) introduced in the AASHTO 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 2008 MEPDG design procedure, is as:

$$M_r = k_1\, P_a\, (\frac{\theta}{Pa})^{k2}\, (\frac{\tau_{oct}}{Pa} + 1)^{k3} \tag{10}$$

where Pa is the atmospheric pressure.

In this paper, the expanded universal model is used to assess the effect of nonlinearity in granular layers on pavement response and performance.

The main results of the regression analysis including the coefficient of determination (R^2) are presented in Table 6.

Table 6. Model Parameters for the Studied UGMs

UGMs	k_1 (MPa)	k_2	k_3	R^2
CAP	2.58	0.77	-0.58	0.95
BBA	3.14	0.70	-0.41	0.98

It is shown that the expanded universal model gives high values of the coefficient of determination for both studied UGM.

Based on the model regression parameters reported in Table 6, the considered constitutive model may be used at a later stage for the nonlinear finite element analysis of flexible pavements utilizing the two local tested aggregates. Moreover, this model may be advantageously used to determine more realistic average values of resilient modulus than those issued from empirical classification of unbound granular materials (Paute et al., 1994).

It is to be noted that the in-situ FWD tests were performed on the pavement test section utilizing UGMs BBA as base and sub-base granular materials. Thus, the value of the field backcalculated modulus EFWD must be multiplied by an adjustment factor equal to 0.62 as indicated in Table 5.

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 Mr value using an adjustment factor equal to 0.35 (see Table 5). 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 guide for pavement design.

4.3 Finite Element Modelling of Pavement Structure

The finite element method is particularly efficient for modelling the nonlinear 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- rectangular ring elements; each node having two degrees of freedom associated with the nodal displacement components in the vertical and the radial directions (Tiliouine & Sandjak, 2001), (Zienkiewicz and Taylor, 2005). 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 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 Figure10 (not to scale).

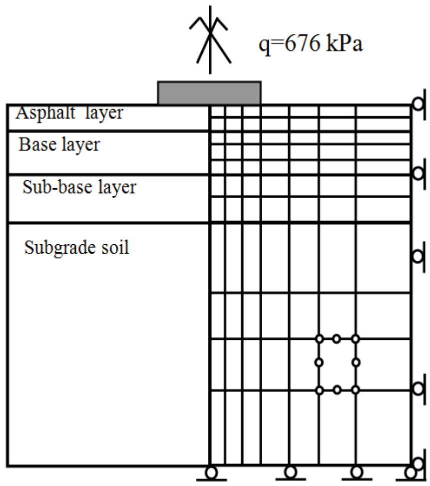


Fig. 10 Comparison Schematic representation of a F.E model (not to scale)

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

Table 7 summarises constitutive material properties of pavement structure layers including BBA granular base and sub-base materials for the three UGMs nonlinear models considered in present investigation.

Table 7. Material properties of pavement layers

Layers	Adjusted FWD Modulus (MPa)	Linear model modulus (MPa)	Constitutive models
Asphalt	5042	5042	-Linear elastic
Base	285	600	-Linear elastic - Nonlinear (10)
Subbase	285	189	-Linear elastic - Nonlinear (10)
Subgrade	63	63	-Linear elastic

4.4 Results of Nonlinear Analysis and Discussion

The main results of the numerical analyses were summarised in terms of values of the design criteria generally used in pavement engineering and the number of load repetitions to failure obtained from either the fatigue cracking or the rutting distress models.

The three design criteria taken into account are:

- 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.

Radial strain at the bottom of Asphalt layer and vertical strain at the top of subgrade soil are two critical responses, which are also used to control fatigue and rutting of flexible pavements respectively (CTTP, 2001), (Huang, 2004). Error in computation of these two responses results in erroneous prediction of the pavement performance (Bocz, 2009), (Sadrnejad et al., 2011), (Zhang et al., 2020).

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 (Ekwulo and Eme, 2009).

Most of the fatigue cracking and the rutting failure models usually take the following forms:

$$N_c = f_1\, {\varepsilon_h}^{f_2}\, E_1^{f_3} \tag{11}$$

$$N_r = f_4\, {\varepsilon_v}^{f_5} \tag{12}$$

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, ε_h: Maximum horizontal tensile strain at the asphalt layer, ε_v: Compressive vertical strain at the top of subgrade soil, E_1 is 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. 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 (Huang, 2004), (Murana & Olowosulu, 2013), 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

The values of the three design criteria and design life (for fatigue cracking and rutting distress models) reported in Tables 8 and 9 were computed using the linear and the nonlinear expanded universal model for granular base and sub-base layers of the analysed pavement structure. Validation by FWD backcalculations, of finite element simulation results using the UGMs BBA has been performed and corresponding results are presented in Table 8. In addition, for the sake of clarity, variation of design criteria and design life for the linear and nonlinear models based on UGMs CAP are summarized in Table 9. 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 nonlinearity of the resilient behaviour of the unbound aggregates. 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 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 design life).

Furthermore, it is noted from Tables 8 and 9 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 nonlinear 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.

Table 8. Validation by FWD Back -calculations of F.E. simulation Results (UGMs BBA)

Design criteria	FWD	Linear model	Nonlinear model
W (mm)	0.870	0.725 (0.83)	0.792 (0.91)
ε_h (10^{-6})	270	136.20 (0.50)	183 (0.68)
ε_v (10^{-6})	842	707 (0.84)	824 (0.97)
N_C	433 332	4 119 670 (9.50)	1 558 521 (3.59)
Nr	79 526	173 673 (2.18)	87 604 (1.10)

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

Table 9. Validation of Design Criteria based on UGMs CAP

Design criteria	Linear model	Nonlinear model
W (mm)	0.831	0.892 (1.07)
ε_h (10^{-6})	188	238.6 (1.26)
ε_v (10^{-6})	821.60	912 (1.11)
N_C	1 426 217	650 917 (0.46)
Nr	88 756	55 620 (0.63)

Values in parentheses represent ratios of F.E nonlinear simulation results to corresponding linear values.

The variations within the pavement section (using the two UGMs) of the horizontal strain and the vertical strain are plotted in Figs. 11, 12 respectively.

Figs. 11 and 12, clearly show that using aggregate from different mineralogical nature can affect both horizontal and vertical strains specially along to the granular layers.

It is observed from the nonlinear simulation results that the three design criteria and pavement design life are very sensitive to changes in UGMs 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 granular materials (UGMs BBA) can reduce significantly the tensile strain at the bottom of the asphalt layer and hence increase the pavement fatigue life.

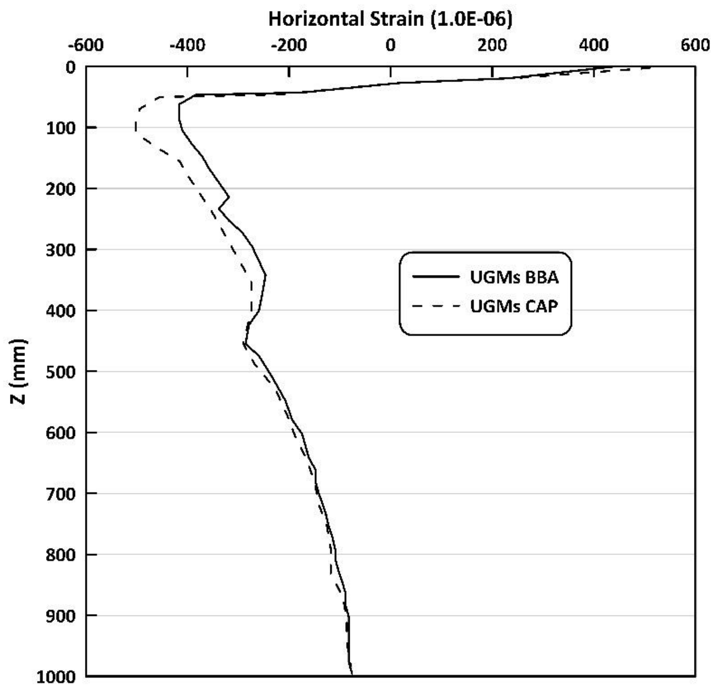


Fig. 11 Variation of the horizontal strain with depth using the two UGMs

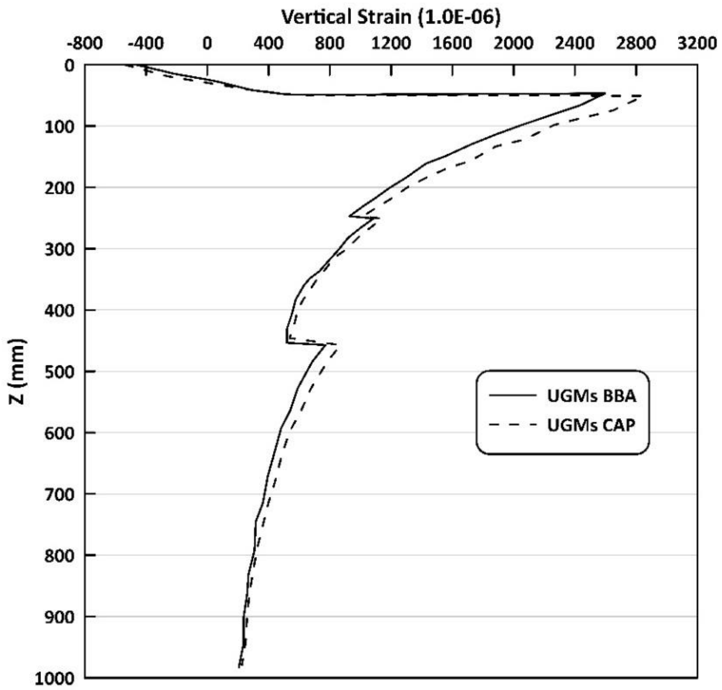


Fig. 12 Variation of the vertical strain with depth using the two UGMs

5. Summary and conclusions

In the first part of this Article, several aggregate specimens from two different deposits within Northern Algeria have been tested in order to assess their suitability as base and subbase materials in road pavement construction.

An experimental investigation including tests on engineering main state characteristics in accordance with empirical classification as well as RLT tests to characterise nonlinear resilient behaviour under cyclic loading, has been carried out. The applicable and relevant standards were employed in all conducted tests.

From the experimental results obtained in this investigation, the following main conclusions may be drawn:

Aggregates issued form both CAP and BBA local deposits have great potential to be used as base course materials for road pavements subjected to low to moderate volumes of traffic in Northern Algeria.

- RLT tests results show that limestone aggregates from BBA deposit present significantly better stiffness characteristics than the aggregates of basaltic origin from CAP deposit especially in terms of resilient modulus, contrarily to the results obtained from empirical ranking tests.
- Density and moisture content as well as stress dependency have shown significant effects on resilient behaviour of studied aggregates, in conformity with results from previous investigations on other granular materials reported in specialised literature.
- Parameter values of the studied mechanical model to be eventually used at a later stage in non-linear finite element analysis of pavement structures in Northern Algeria, have been determined from RLT tests results by using a generalisation of the least square regression method applied to both volumetric and shear strains.
- Measured and calculated strains (volumetric as well as shear) are in excellent agreement for both tested aggregate specimens indicating that the mechanical model used in the

present study reflects with very good approximation their nonlinear resilient behaviour.

In the second part of this Article, the expanded universal nonlinear resilient model for unbound granular materials has been implemented in a numerical code based on axi-symmetric finite elements to study the nonlinear resilient behaviour of unbound granular materials and the structural performance of road pavements. Numerical investigations have been carried out to assess the influence of the nonlinear resilient model on the behaviour of two local unbound aggregates of different mineralogical nature typically used for road construction in Algeria.

Simulation results show that 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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