

Consideration of the Non-Linearity of Unbound Granular Materials in the Design of Flexible Pavement Structures in Senegal

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Abstract The design of road structures in Senegal is based on the assumption of linear elastic behavior of the materials making up the various pavement layers. However, studies have shown that the experimental behavior of granular materials is rather nonlinear elastoplastic and is characterized by the reversible modulus M_r which depends on the state of stress. In this article, the study of nonlinearity was investigated with a view to its possible consideration in the design of flexible pavement structures. The results showed that the strains and stresses found are higher in the nonlinear case than in the linear case. Compared with the results obtained with the nonlinear model integrated into Alizé LCPC, the k - Θ model tends to overestimate vertical stresses and strains in sub-base layers. For subgrade layers, however, Alizé LCPC seems to overestimate them. These variations could be explained by discontinuities at layer interfaces and variations in the stiffness of materials in the pavement. However, the validation of nonlinear calculations with a view to improving road design in Senegal remains to be verified, in particular by experimental data.

Keywords: reversible modulus, linear, nonlinear, stress, strain, flexible pavement, design, materials

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1. Introduction

In the so-called rational approach to pavement design, the determination of the mechanical behavior of materials is generally done according to a linear elasticity calculation. This behavior is then described thanks to two constant parameters, namely the Young's modulus (E) and the Poisson's ratio (ν). In the case of untreated gravel, Young's moduli are determined by correlation studies due to lack of sufficient resources in laboratories in developing countries [1].

The experimental behavior of granular materials is rather nonlinear elastoplastic. This calls into question the input parameters of the various models that use a linear elasticity theory to describe a nonlinear plastic phenomenon. This is why the Mechanistic-Empiric design is presented as an alternative for a good consideration of the real behavior of granular materials under cyclic loads. The reversible modulus is used to measure the nonlinear elastic properties of granular materials. It is the modulus of elasticity of unbound materials based on the elastic deformation from mechanical tests under cyclic loading. Several models have been developed to determine the reversible modulus of granular materials as a function of

state and stress level, thus allowing to take into account the nonlinearity of these materials.

The reversible behavior that we are trying to study is the stable behavior reached after several thousand loading cycles. It corresponds to the "long term" behavior of the untreated gravel in a pavement in service, when it has undergone a certain number of loads due to placing and compaction (these are the most severe stresses) and to traffic. This procedure makes it possible to characterize untreated gravel for a wide range of uses (base or sub-base) [2].

2. Nonlinear Behavior of Granular Pavement Materials

The mechanical behavior of pavements is described by the elastic properties of the material, which are defined by Young's modulus and Poisson's ratio. It is found in previous work on pavement mechanics that the behavior of pavement layers can be nonlinear. To take this nonlinearity into account, the material is studied in its real behavior by replacing the constant linear modulus of elasticity by the reversible modulus (M_r) which is a function of the stress. The linear and nonlinear aspect is illustrated in figure 1.

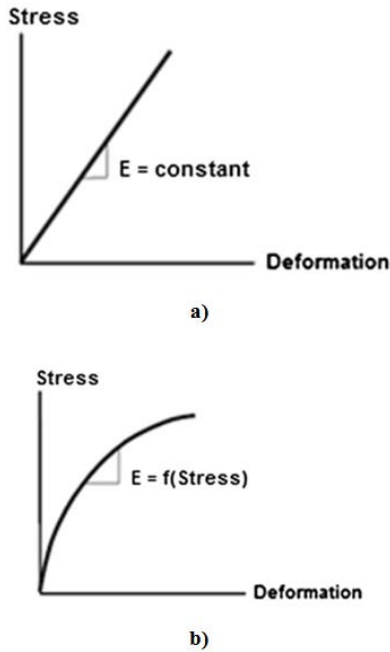


Figure 1. Study of the material behavior: **a)** linear **b)** nonlinear

For cyclic loading, the reversible modulus of the material is defined by the following relation:

$$M_r = \frac{\Delta\sigma_d}{\Delta\varepsilon_a} \quad (1)$$

$\Delta\sigma_d = \sigma_1 - \sigma_3$ is the deviatoric stress;

σ_1 is the main major stress;

σ_3 is the minor principal stress;

$\Delta\varepsilon_a$ is the reversible axial deformation;

There are several models in the literature for determining the reversible modulus of granular pavement materials.

The k- Θ model is most often used for the analysis of reversible behavior of granular materials and the study of the variation of stiffness with stress. [3,4,5] indicate that the reversible modulus is a function of the sum of principal stresses and propose a hyperbolic relationship commonly referred to as the k- Θ model.

$$M_r = k_1 \left(\frac{\Theta}{P_a} \right)^{k_2} \quad (2)$$

Θ is the sum of the principal stresses (kPa);

k_1 and k_2 are the parameters of the model;

P_a is the atmospheric pressure of normalization;

In spite of its simplicity in practice, this model has some shortcomings that have led some authors to make modifications.

A shortcoming of the k- Θ model is related to the fact that the effect of stresses on the modulus is estimated only by the sum of the principal stresses. This model does not predict the volumetric strain and as such can only be applied if the confining stress is less than the deviatoric stress.

Since the k- Θ model is not sufficient to describe the behavior of granular materials, [6] introduces a modification in the k- Θ model. The introduction of deviatoric stress as an additional component with the

consideration of the effect of shear behavior shows a good correlation with the test results.

$$M_r = k_1 \left(\frac{\Theta}{P_a} \right)^{k_2} \left(\frac{\sigma_d}{P_a} \right)^{k_3} \quad (3)$$

Θ is the sum of the principal stresses;

σ_d is the deviatoric stress;

k_1 , k_2 and k_3 are the parameters of the model.

Uzan's model seems to be in line with all aspects that characterize the behavior of granular materials. By considering the sum of stresses and deviatoric stress, Uzan's model takes into account the shortcomings of the k- Θ model, which does not include the effects of shear, and therefore better matches the experimental results. This problem was especially emphasized when the confining stress values applied to the sample were higher than the deviatoric stresses applied during the test.

A generalized model is proposed by [7] to characterize the reversible modulus of untreated gravel and fine soils. This equation (4) combines the hardening effect of the sum of the principal stresses and the softening effect of the shear stress. Thus the value of k_2 must be positive and that of k_3 negative.

$$M_r = k_1 \cdot P_a \left(\frac{\Theta}{P_a} \right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1 \right)^{k_3} \quad (4)$$

Θ is the sum of the principal stresses;

τ_{oct} is the octahedral shear stress;

$$\tau_{oct} = \frac{1}{3} \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2} \quad (5)$$

k_1 , k_2 and k_3 are the parameters of the model with $k_1 > 0$, $k_2 > 0$ and $k_3 < 0$.

3. Finite Element Modeling of Pavements

The finite element approach provides the best method for analyzing multi-layered pavement systems [8]. Two-dimensional, three-dimensional, or axisymmetric finite element models have different formulations and consider different stress and strain components. Compared to the multilayer elastic method, the finite element method is better because it can rigorously account for anisotropy, material nonlinearity, and various boundary conditions [9].

However, these two-dimensional models have limitations. They cannot take into account non-uniform tire contact pressures and multiple rolling loads. To overcome these limitations, three-dimensional finite element approaches are becoming increasingly popular. With 3D analyses, the responses of flexible pavements are studied under spatially varying tire contact pressures, it is also possible to capture the effect of material nonlinearities as well as the effect of the combination of loads including different types of non-symmetric loading. However, the 3D finite element approach can be difficult and often takes more time.

Finite element modeling for solving linear or nonlinear structural problems can be done in three phases.

- Preprocessor phase: this step consists in defining the mathematical model representative of the physical model of the structure, specifying the calculation options (2D, 3D, etc.), the discretization of the different parts of the structure (nodes, types of elements, etc.), the mechanical models to be associated with the mesh (behavioral model and associated mechanical properties) and the system to be solved (boundary conditions, loading, etc.);
- Calculation phase: resolution of the discretized problem (calculation and assembly of the elementary stiffness matrices, resolution of the linear or nonlinear system, etc.);
- Post-processor phase: analysis and processing of results (displacements, stresses, deformations, etc...).

In the following, we will use the finite element code CAST3M[®] which is a modeling program widely applied to the analysis of road pavements.

3.1. Presentation of the CAST3M[®] Finite Element Code

CAST3M[®] is a software package for calculating structures using the finite element method and more generally for solving partial differential equations using the finite element method. It has been developed at the Department of Mechanics and Technology (DMT) of the French Atomic Energy Commission (CEA).

The main characteristic of CAST3M[®] is that it is extremely adaptable to the multiple applications of each user. The development of CAST3M[®] is part of a research activity in the field of mechanics whose object is to define a high level instrument, which can be used as a support for the design, dimensioning and analysis of structures and components, in the nuclear field as well as in the classical industrial sector. In this perspective, CAST3M[®] integrates not only the calculation processes themselves but also the model building functions (pre-processor) and the results processing functions (post-processing). CAST3M[®] is a program that the user can adapt to his needs to solve his own problems.

CAST3M[®] has a command language consisting of a series of operators allowing the user to manipulate data and results in the form of objects by giving them names: this is the Gibiane language using a text editor (any one). Then launch the CAST3M[®] application on the created file.

Solving mechanical problems requires the knowledge of boundary conditions, loading conditions and the behavioral law that relates deformations to stresses. Most commercial finite element software offers, in addition to a large choice of behavioral laws, the possibility to implement one's own law when no model in the provided library is able to adequately represent the behavior of the material used. This alternative is offered with the CAST3M[®] software.

3.2. Loading and Boundary Conditions

Traffic loads are applied to the pavement by tires that exert forces on the tire-pavement contact surface. The reference load used for road design in Senegal is 130 kN. However, in our simulations, we consider the half-axle,

that's to say 65 kN. This loading is shown in figure 2 and will be distributed and applied on two finely meshed footprints located on the surface layer. The configuration of the domain size of the pavement structure, the simulation of the traffic load, and the mesh refinement are the most important factors [10]. For the boundary conditions, horizontal displacements are locked in the transverse directions and vertical and horizontal displacements are locked in the lower part of the subgrade.

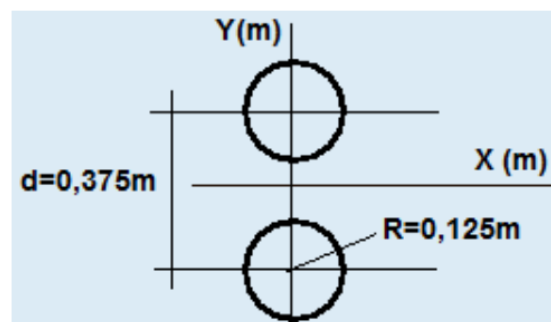


Figure 2. Standard axle load of 65 kN (LCPC, 1994)

4. Methodology

This part is reserved for the methods to be adopted for the determination of material characteristics and model parameters used as input parameters in finite element modeling.

4.1. Characteristics of the Study Materials

The materials used in this study are untreated gravel from the quarries of Diack (basalt), Bakel (sandstone) and Sindia (laterite). These quarries are all located in Senegal. These materials have been previously tested at the Laboratory of Mechanics and Modeling (L2M) of the Engineering Sciences Department of the Iba Der Thiam University of Thiès, at the Cheikh Anta Diop University of Dakar, at the University of Paris-Est, and at the University of Wisconsin in the United States of America.

Diack basalt was widely used as a surface course for bituminous pavements. However, with the increase in traffic volume and axle loads, this material is increasingly used as a base or sub-base layer for flexible pavements. It is often used in the form of untreated gravel 0/31.5 or in the form of Grave Bitumen 0/20 with high performances. This material is well known and constitutes a good reference material in Senegal [1].

As for laterite, it still constitutes the bulk of the road network in Senegal. The plasticity of this material varies widely, from one deposit to another, and even within the same deposit (Chairman, 1988). Often, laterites are so plastic that they can only be used as pavement material after stabilization [8]. Previous studies of this material show that it has geotechnical characteristics that allow it to be used as a sub-base for flexible pavements.

The Bakel sandstones, covering the entire northern region of Senegal, could constitute substitute materials for basalt and laterite to reduce the overexploitation of these two materials and the reduction of project costs in this region.

Particle size analysis and sedimentometry were performed on these materials. The particle size curves and characteristics are summarized in figure 3 and Table 1. The values of the uniformity coefficients $C_u > 2$ prove that the particle size curves are all spread out and consequently, these materials have the advantage of presenting high densities, low permeabilities and are easily compacted [11].

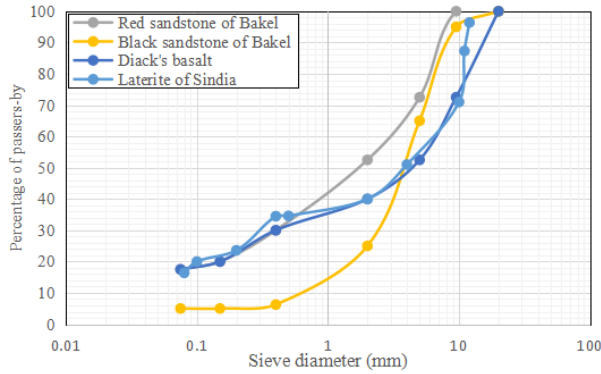


Figure 3. Sieve size curve of the study materials

Table 1. Particle size characteristics of the materials

Material	Granulometric characteristics			
	Cc	Cu	fine	< 2 mm
Diack's basalt	3,84	66	8	18
Laterite of Sindia	0,34	207	17,86	41,6
Red sandstone of Bakel	0,008	150	17	40
Black sandstone of Bakel	2,28	7	4	25

Compaction is carried out at 95% of the Modified Optimum Proctor (OPM) which corresponds to the CEBTP (Experimental center for buildings and public works) road specifications for untreated gravel base layers in Senegal.

The compaction ability of untreated gravel is evaluated from the modified Proctor test, according to the French standard [12]. Table 2 shows the compaction and bearing capacity parameters of the materials.

In this study, several rounds of Californian Bearing Ratio (CBR) testing were conducted by [13,14] by first compacting the sample at its optimum moisture content and then at different Proctor curve moisture contents. The CBR obtained at 95% OPM show that only the CBR of the Diack basalt satisfies a base course. According to CEBTP (1984), the base course is subjected to significant stresses, the materials that constitute it must have sufficient qualities. However, the recommended CBR at 95% of the OPM must be at least 80. The CBR of all the materials allow them to be used as a sub-base layer because it is higher than 30 [15].

In this three-dimensional modeling, a linear behavior is considered for the asphalt layer and the subgrade with Young's moduli of 1300 MPa and 30 MPa respectively. The choice of the modulus for the wearing course is not arbitrary. Dione (2015) studied different pavement types with moduli of 2300 MPa and 1300 MPa in the wearing course. The first modulus is not the one usually used in Senegal. 1300 MPa represents the minimum modulus to

avoid pavement failure [13]. It represents a reference modulus for a wearing course because the use of very high moduli leads to the use of low thicknesses and therefore to undersizing [16]. The 2300 MPa modulus is obtained by iteration. Indeed, several moduli were tested starting with 1300 MPa. The gradual increase of this modulus up to 2200 MPa leads to a failure in the wearing course [13]. According to the researcher, the failure disappears once a value of 2300 MPa is reached for a flexible pavement with a base layer of untreated gravel and an 8 cm asphalt concrete wearing course. Hence the choice of a wearing course thickness of 8 cm throughout the modelling. As for the base and sub-base layers, their behavior is considered to be nonlinear.

Table 2. Compaction and bearing capacity characteristics of materials

Materials	Compaction characteristics		Carrying capacity characteristics
	$W_{opt}(\%)$	$\gamma_{dmax}(kN/m^3)$	CBR (%) at 95% of OPM
Diack's basalt	4,2	22,05	86
Laterite of Sindia	9,66	19,7	54,8
Red sandstone of Bakel	6	21	50
Black sandstone of Bakel	4	20,45	68

[17] performed preliminary simulations in order to optimize the numerical modeling to be undertaken in the following. Indeed, the more discretized the model studied, the longer the simulation time. These numerical simulations concern a pavement structure consisting of a wearing course in asphalt concrete, a base course consisting of Diack basalt, a sub-base course consisting of Sindia laterite and a support soil. It is then necessary to verify beforehand that the dimensions of the model, the mesh level of the structure and the chosen boundary conditions allow to optimize the computation time of the solution while preserving the quality of the model response. Following these preliminary simulations, the pavement structure with the following characteristics is selected:

- Length*width=30R*30R and of depth 24R (R=0,125m is the radius of the circular footprint of the wheel);
- Number of CUB8 elements equal to 11200;
- Number of loading steps equal to 50.

The implementation of the Uzan model and those of the k- Θ and NCHRP models is used for pavement base modeling by integrating the model parameters into the tangent stiffness matrix equation. In equation 6, the parameters of the Uzan model are incorporated.

$$D_T = \left[\frac{-k_2 \left(\frac{\theta + \theta_t}{P_a} \right)^{-k_2-1} \left(\frac{\sigma_d}{P_a} \right)^{-k_3}}{k_1 P_a^2} + \frac{-k_3 \frac{15\sqrt{2}}{3} \left(\frac{\theta + \theta_t}{P_a} \right)^{k_3} \left(\frac{\sigma_d}{P_a} \right)^{-k_3-1}}{k_1 P_a^2} \right] \sigma_1 + \left[\frac{\left(\frac{\theta + \theta_t}{P_a} \right)^{-k_3} \left(\frac{\sigma_d}{P_a} \right)^{-k_3}}{k_1 P_a} \right] \quad (6)$$

Θ is the sum of the principal stresses;
 σ_d is the deviatoric stress;

k_1 , k_2 and k_3 are the parameters of the model;

P_a is the atmospheric pressure of normalization;

Figure 4 shows the computational algorithm used for the implementation of the different models.

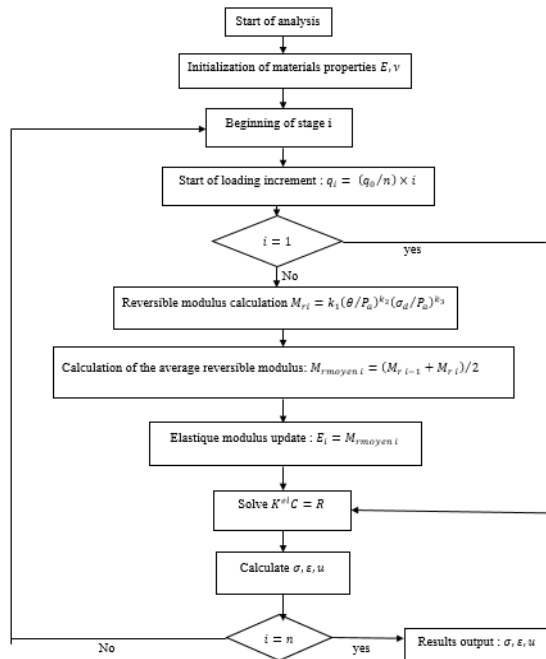


Figure 4. Flow chart of the nonlinear analysis [8]

The nonlinear analysis consists of several iterations. A linear analysis is performed in each iteration, after which the reversible modulus of each finite element is improved as needed. The iteration is repeated until the reversible moduli of all elements stabilize.

An iterative macro is created, the material properties change at each iteration, in relation to the stress state of

each element, until arriving at the convergence criterion as for example that of [18] presented in equation (7).

$$\frac{\sum_{i=1}^n |E_{newi} - E_{oldi}|}{\sum_{i=1}^n E_{oldi}} \times 100 \leq 5 \quad (7)$$

n : number of elements in the layer

E_{newi} : module of element i for the iteration

E_{oldi} : modulus of element i for the previous iteration

4.2. Determination of material parameters

In the finite element modeling under Cast3M[®], which is the subject of this article, only the parameters k_i ($i=1, 2, 3$) of the different models (k- Θ , Uzan, NCHRP) will be used. Triaxial repeated loading tests for the study of reversible behavior were performed by [1] on untreated gravel at the University of Wisconsin in the United States of America. The determination of the parameters k_i of these models was done from the regression of the results of cyclic triaxial tests. The chosen fitting method, based on the least squares method, is not very time consuming and allows a large number of calculations. For all the adjustments, we used an option (solver) of the Excel spreadsheet, allowing, on the basis of the method of least squares, to seek the set of parameters which minimizes the deviations between the experimentally measured values and the values calculated by the model.

To determine the Young's moduli of the subgrade materials, triaxial repeated loading tests for the study of reversible behavior were performed by [1] on untreated gravel. The nonlinear elasticity model K- Θ was calibrated on the results of these tests to determine the parameters k_1 and k_2 . These allow the calculation of a reference value for the Young's modulus according to NF P 98-235-1 ($q=600$ kPa, $P=300$ kPa).

Table 3. Material parameters (linear case)

Layer	Nature of the Layers	Thickness (m)	Density (kg/m ³)	E(MPa)	Poisson's ratio ν
Surface	Asphalt concrete	0,08	-	1300	0,25
Base	Diack's basalt	0,25	2417	352	0,35
Sub-base	Laterite of Sindia	0,25	2020	274	0,35
Subgrade	-	2,42	-	30	0,35

Table 4. Model parameters k- Θ

Layers	Nature of the layers	Thickness e_i (m)	Linear model		Nonlinear model	
			ν	E (MPa)	k_1 (MPa)	k_2
Surface	Asphalt concrete	0,08	0,25	1300	-	-
Base	Diack's basalt	0,25	0,35	-	147	0,53
Sub-base	Laterite of Sindia	0,25	0,35	-	229	0,49
Subgrade	-	2,42	0,35	30	-	-

Table 5. Parameters of the Uzan model

Layers	Nature of the layers	Thickness e_i (m)	Linear model		Nonlinear model		
			ν	E (MPa)	k_1 (MPa)	k_2	k_3
Surface	Asphalt concrete	0,08	0,25	1300	-	-	-
Base	Diack's basalt	0,25	0,35	-	93717	1,00	- 0,32
Sub-base	Laterite of Sindia	0,25	0,35	-	133017	1,05	-0,53
Subgrade	-	2,42	0,35	30	-	-	-

Table 6. NCHRP Model Parameters

layers	Nature of the layers	Thickness e_i (m)	Linear model		Nonlinear model		
			ν	E (MPa)	k_1 (MPa)	k_2	k_3
Surface	Asphalt concrete	0,08	0,25	1300	-	-	-
Base	Diack's basalt	0,25	0,35	-	1224	0,93	- 0,52
Sub-base	Laterite of Sindia	0,25	0,35	-	2097	0,98	-0,96
Subgrade	-	2,42	0,35	30	-	-	-

Table 7. Vertical stresses and strains above the layers (linear case)

Layer	Stress (MPa)	Deformation ($\mu\epsilon$)
Surface	- 0,398	- 448,13
Base	- 0,341	- 384,89
Sub-base	- 0,008	- 182,32
Subgrade	- 0,024	- 493,66

Table 8. Material parameters of structure 1

Layers	Nature of the layers	Thickness	Linear model		NCHRP model		
			ν	E (MPa)	k_1 (MPa)	k_2	k_3
Surface	Asphalt concrete	0,08	0,25	1300	-	-	-
Base	Diack's basalt	0,25	0,35	-	1224	0,93	-0,52
Sub-base	Laterite of Sindia	0,25	0,35	-	2097	0,98	-0,96
Subgrade	-	2,42	0,35	30	-	-	-

Table 9. Material parameters of structure 2

Layers	Nature of the layers	Thickness e_i (m)	Linear model		NCHRP model		
			ν	E (MPa)	k_1 (MPa)	k_2	k_3
Surface	Asphalt concrete	0,08	0,25	1300	-	-	-
Base	Red sandstone of Bakel	0,25	0,35	-	905	0,79	-0,42
Sub-base	Laterite of Sindia	0,25	0,35	-	2097	0,98	-0,96
Subgrade	-	2,42	0,35	30	-	-	-

Table 10. Material parameters of structure 3

Layers	Nature of the layers	Thickness e_i (m)	Linear model		NCHRP model		
			ν	E (MPa)	k_1 (MPa)	k_2	k_3
Surface	Asphalt concrete	0,08	0,25	1300	-	-	-
Base	Black sandstone of Bakel	0,25	0,35	-	701	0,89	-0,48
Sub-base	Laterite of Sindia	0,25	0,35	-	2097	0,98	-0,96
Subgrade	-	2,42	0,35	30	-	-	-

5. Results and Discussion

For the impact of nonlinearity, the results of the 3D linear and 3D nonlinear models are compared.

The results obtained further illustrate what has been found in several studies, namely the diffusion of stresses with depth. In all cases, stresses and strains decrease with depth. For all models (k- Θ , Uzan and NCHRP), vertical deformation in the nonlinear case is greater than in the linear case (Figure 5). It averages 377 $\mu\epsilon$, 119 $\mu\epsilon$ and 95 $\mu\epsilon$ more for the k- Θ , Uzan and NCHRP models respectively. However, the largest discrepancy is noted with the k- Θ model. This discrepancy could be explained by a significant decrease in deviatoric stress σ_d (Uzan model) and octahedral τ_{oct} (NCHRP model). With the Uzan model, when k_2 or k_3 is negative, and Θ or σ_d approaches zero, reversible modulus values can become very high. The same authors also showed that with the NCHRP model, when k_2 is negative and Θ tends towards zero, reversible modulus values become very high. In our study, we kept the coefficient k_2 coefficient and a k_3 coefficient.

Consequently, the increase in the reversible modulus would explain the low values of strains and stresses obtained with the Uzan and NCHRP models by comparison with those obtained with the k- Θ model.

The nonlinear nature of the materials observed at high stress levels could explain the difference in strain and stress, which decreases and tends to cancel out towards the platform layer. Figure 6 illustrates the evolution of these deviations as a function of depth. For all the models studied, the difference decreases with pavement depth. With the diffusion of stresses in the depth of the pavement layers, this nonlinear character would disappear as we approach the subgrade. Consequently, taking into account the nonlinearity of granular materials in the subgrade would be essential for the proper design of flexible pavement structures.

By making a comparative study of the results obtained with the three models (k- Θ , Uzan, NCHRP), a significant difference of the k- Θ model compared to the other models is observed (Figure 6). For example, the variation $\Delta\epsilon_v$ for the vertical deformation above the platform is 709.04 $\mu\epsilon$, 420 $\mu\epsilon$, 483.138 $\mu\epsilon$ for the k- Θ , Uzan, and NCHRP models respectively.

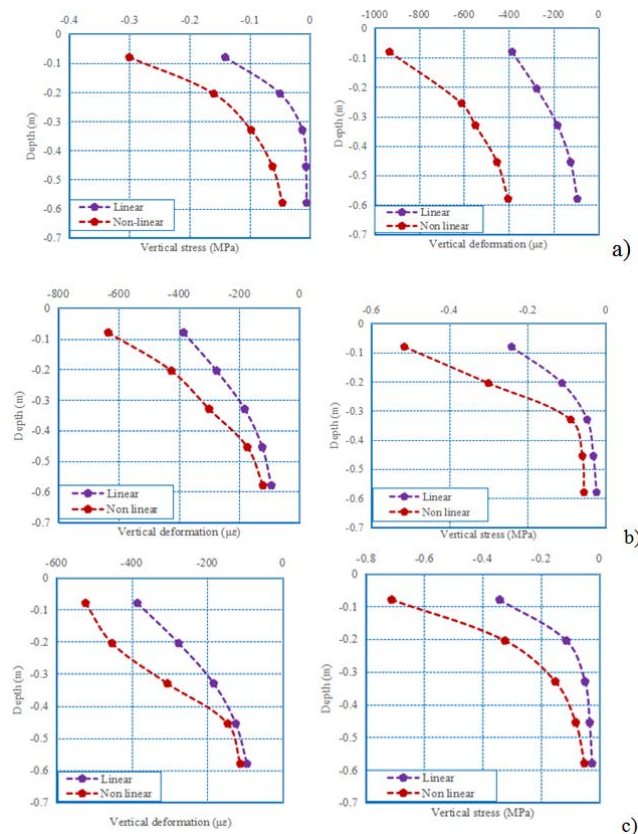


Figure 5. Variation of strain and vertical stress with depth: a) k- Θ model b) Uzan model c) NCHRP model.

The k- Θ model appears to overestimate the vertical deformations. The values obtained with the Uzan and NCHRP models would be closer to reality. However, the validation of the nonlinear calculations remains to be verified in particular by experimental data.

$$\Delta\sigma_v = \sigma_v(\text{linear}) - \sigma_v(\text{non-linear})$$

$$\Delta\varepsilon_v = \varepsilon_v(\text{linear}) - \varepsilon_v(\text{non-linear})$$

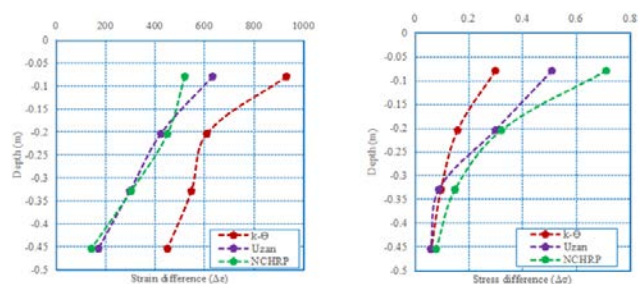


Figure 6. Variation in stress and strain difference with depth

In this part, a study of the comparison of the stresses and strains undergone by three pavement structures is carried out with a view to the possible use of Bakel sandstone in sub-base layers of flexible pavement structures. What differentiates these three structures are the characteristics of the materials that constitute their base layers. The base layers of the first, second and third structures are made of Diack basalt, Bakel red sandstone and Bakel black sandstone, respectively. A slight difference of the deformations and stresses of structure 2 compared to structure 1 taken as reference is observed, contrary to structure 3 where the vertical deformation deviates more from that of the reference structure (figure 7). In general, there is a slight difference in the stresses

and strains of the structures whose base layers are made of red and black Bakel sandstone, compared to the Diack basalt based structure, which is a reference material in road engineering in Senegal (figure 8). These sandstones, after further study, could serve as an alternative to laterite and basalt and reduce project costs.

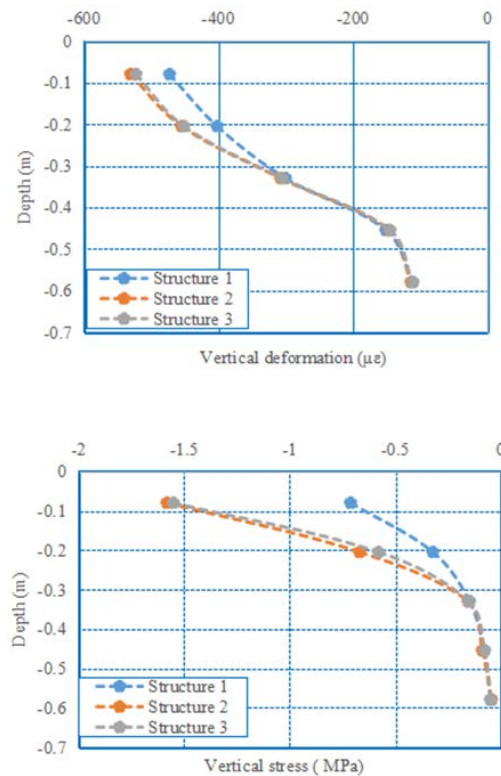


Figure 7. Vertical strains and stresses in the different structures

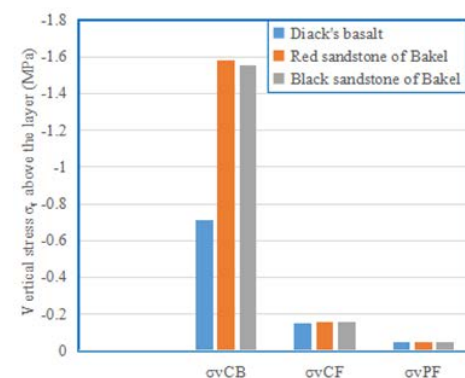
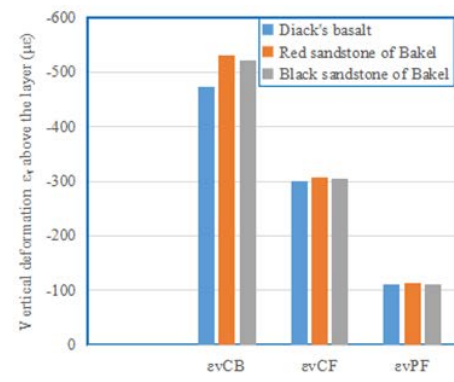


Figure 8. Variation of vertical strain and stress according to the material used in the base layer

5. Comparative Study with the Regulatory Method for the Design of Flexible Pavements

The regulatory method for designing flexible pavements is based on two criteria. The first is related to rutting since the vertical elastic deformation at the top of the subgrade must not exceed a certain limit depending on the traffic. The second criterion is related to the fatigue of the asphalt concrete under flexural tension cycles.

We consider a pavement structure subjected to very low traffic, the equivalent NE number of heavy vehicle passages chosen is equal to 10^5 . The pavement structure considered consists of a wearing course in asphalt concrete, a base course in untreated gravel (all compacted to the optimum moisture content) and the whole resting on a subgrade.

In Senegal, and taking into account the current design techniques directly derived from the "rational" methods, the design criterion with respect to rutting consists only in limiting the value of the vertical elastic deformation at the top of the subgrade [19]. This criterion is in the form : $\varepsilon_z^{adm} = A \cdot (NE)^{-0,222}$ ($A=0,012$ for pavements with medium to heavy traffic, and $A=0,016$ for low traffic, NE represents the number of equivalent axles).

Table 11. Material parameters for verification calculation with Alizé

Layers	Nature of the Layers	Thickness (m)	Density (kg/m ³)	E(MPa)	Poisson's ratio ν
Surface	Asphalt concrete	0,08	-	1300	0,25
Base	Diack's basalt	0,25	2417	481	0,35
Sub-base	Laterite of Sindia	0,25	2070	320	0,35
Subgrade	-	∞	-	30	0,35

The vertical deformation values obtained above the base, foundation and subgrade layers with the Alizé and Cast3M[®] calculation codes are summarized in Table 12.

Table 12. Vertical deformation (en $\mu\epsilon$) above the layers

Layer	Linear analysis	Nonlinear analysis		
	Alizé LCPC	k- Θ	Uzan	NCHRP
Base	809,40	932,54	634,23	522,37
Sub-base	332,80	548,45	300,05	305,20
Subgrade	682,80	402,70	121,57	111,52

In the base and sub-base layers, the values obtained with the Alizé-LCPC software are higher than those obtained with the Uzan and NCHRP models. However, the opposite is true of the k- Θ model. With the latter, these deformations represent 123 $\mu\epsilon$ and 215 $\mu\epsilon$ more in the base and sub-base layers respectively. The k- Θ model appears to overestimate vertical deformations in the pavement base layers. Indeed, this overestimation of vertical deformations with the k- Θ model could be explained by the insufficient prediction of unbound granular materials used in subbase layers (Ba, 2012) with this model, unlike the Uzan and NCHRP models. In subgrade layers, the deformations obtained with the Alizé software are higher, with very high deviations from the Uzan (561 $\mu\epsilon$ more) and NCHRP (571 $\mu\epsilon$ more). These variations may be due

to changes in material behavior at the interface between layers, or to reduced stiffness.

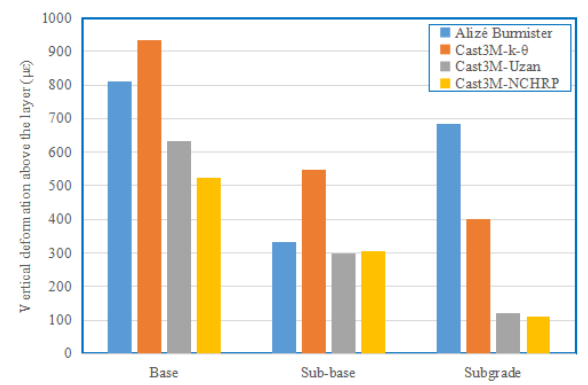


Figure 9. Vertical deformations above the layers according to the model used

6. Conclusion

Currently, design methods are based more and more on the finite element method and sophisticated calculation codes. This approach is essential for the analysis of pavement behavior and for understanding the failure mechanisms involved. The current design method used in Senegal assumes that granular materials have a linear elastic behavior characterized by the Young's modulus and Poisson's ratio. However, studies have shown that this behavior is rather elastoplastic. The results found in our study showed that the strains and stresses obtained with the Uzan and NCHRP models are higher than those from the linear model, in contrast to the k- Θ model. These results require some caution as one might think that the predictions of the linear model seem to be on the safe side and that the degradation of pavement structures would be unrelated to the non-accounting for nonlinear behavior. Moreover, the comparative study with the regulatory method of pavement design with the Alizé software would confirm this. Compared to this regulatory method, the Uzan model tends to underestimate the vertical deformations, contrary to the k- Θ model which tends to overestimate them. Therefore, the validation of the non-linear calculations remains to be verified in particular by experimental data.

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