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Effect Of Nonlinearity In Granular And Subgrade Layers In Multi Layer Analysis Of Flexible Pavements

MayilVahanan.T.[1] and Dr.Murugesan.R.[2]

[1]Ph.D. scholar Anna University Chennai,
[2] Professor, Department of Civil Engineering, I.R.T.T.,
Erode - 625316, India.
coscmd@gmail.com, murugesan6raamin@gmail.com

Abstract

Most of the currently and flexible structural analysis models including IITPAVE is linear elastic analysis program. Under repeated wheel loads the pavement foundation materials in different layers do not behave linear elastically. The granular materials and subgrade soils are nonlinear with an elastic modulus varying with level of stresses. KENLAYER the part of KENPAVE computer program incorporated the nonlinear material properties by assumes a bilinear relationship between resilient modulus and deviator stress for granular and subgrade soil. Keeping the above in mind pavement response from KENPAVE and IITPAVE as per IRC:37-2012 were analyzed to determine the linear analysis of different combination of traffic and pavement composition. The results indicate that the non linearity yields 17.15% decrease in horizontal tensile strains at the bottom of bituminous layer and 22.99% increase in compressive strain on top of the subgrade layer and 24.29% increase in surface deflection using linear elastic analysis. The outcome of the study shows that nonlinear analysis is most accurate for the pavement responses and yet emerges with considerable different from the linear elastic solutions. Further improvement are attempted in nonlinear analysis to suit the Indian condition and to bring changes in IRC codes.

Keywords: IITPAVE, KENPAVE, Granular layers, Nonlinearity, Pavement responses.

1. INTRODUCTION:

Mechanistic concepts have been adopted for the analysis and design of flexible pavements in the recent years. Mechanistic analysis demands accurate material characterizations of pavement structural layers. Pavement foundation geomaterials in base/subbase and subgrade layers do not behave linear elastically under repeated wheel loads. Most of the currently used flexible structural analysis models including IITPAVE developed under the Research scheme of MoRTH assume that layer is homogeneous, isotropic, linear elastic with constant modulus of elasticity. But in reality, the materials are nonlinear, anisotropic and in homogeneous, and some are particulate; viscous and plastic deformations occur in addition to the elastic deformation. This nonlinear behavior is commonly characterized by stress dependent resilient modulus which is used as a fundamental input in the application of the layer theory in flexible pavements design. IITPAVE software is a multilayer elastic layer linear analysis program with input of thickness, elastic modulus and poissons ratio of various layers. The prime objective of this study is to compare pavement response provide by an nonlinear analyses from KENPAVE and IITPAVE based on IRC: 37-2012

1.1 Objective of the Research work:

1. To conduct linear analysis of various combination of traffic and pavement composition presented in the design charts of IRC: 37–2012 using KENPAVE for validation of KENPAVE results with IITPAVE.
2. To conduct KENPAVE nonlinear analysis of same combination using IRC guidelines for linear parameters whereas nonlinear coefficient of granular and subgrade layers are collected from nonlinear stress-dependent models.
3. To compare the effect of non linearity in horizontal tensile strain in bottom of bituminous layer, vertical compressive strain on top of subgrade and vertical deflection in the surface.

2. LITERNATURE REVIEW

Nonlinear / Finite element models have been applied extensively to analysis the pavement structures. In this section, the development of several nonlinear solution techniques including finite element methods currently (after 1990) used in pavement analysis are reviewed.

1. Crockford (1990) developed an unusual type of nonlinear resilient response model for characterization of granular layers and pavement evaluation in conjunction with the use of a falling weight deflectometer (FWD).
2. ILLI-PAVE is a commonly used finite element program developed at the University of Illinois (Raad and Figueroa, 1980) and MICH-PAVE program was developed at the Michigan State University (Harichandran et al.,1989) for the analysis of flexible pavements.
3. Brumton and De Almedia (1992) developed a finite element program named FENLAP for structural analysis pavements.
4. GT-PAVE finite element program (Tutumluer, 1995) had also taken into

account nonlinear material characterizations of granular materials and subgrade soils.

5. The KENLAYER part of KENPAVE computer program provided the solution for an elastic multilayer system under a circular loaded area and was developed by Huang (1993) at University of Kentucky. This program handled multiple wheels, iterations for nonlinear layers, and viscoelastic layers. To deals with nonlinearity, KENLAYER divided the layers into a number of sublayers and the stresses at the mid-height were used to compute the modulus of each layer.
6. Thompson and Garg (1999) introduced an "Engineering Approach" to determine critical pavement responses based on the superposition of single wheel pavement response.
7. Wang (2001) investigated the response of flexible pavement structures with materials, model dimensions and different loadings using three-dimensional finite analysis.
8. Erlingsson (2002) conducted three-dimensional finite analysis of a heavy vehicle simulator used to test low volume road structures (COSMOS^{TM}).
9. Nesnas et al. (2002) used the ABAQUS^{TM} solver to study three-dimensional model for the prediction of surface crack opening due to temperature variations.
10. Sukumaran (2004) presented a three dimensional analysis model of airport flexible pavements using ABAQUS^{TM}. The discussed issues were construction of mesh,mesh refinement, element aspect ratios and material nonlinearities.
11. Saad et al. (2005) examined the dynamic response of flexible pavement structures to single wheel traffic load using ADINA^{TM} three-dimensional model.

3. METHODOLOGY OF THE STUDY

3.1 Data Collection

In the present paper study the flexible pavement for design traffic of 10 msa, 50 msa and 150 msa with CBR of 10% as per plate -7 of IRC:37-2012 has been considered. For linear analysis elastic modulus of subgrade correlated with effective CBR is given by Equation:

$$M_{R\ subgarde} = \left.\begin{aligned} &10 * CBR \\ &= 17.6 * (CBR)^{0.64} \end{aligned}\right\} \begin{aligned} &\text{For CBR 5} \\ &\text{For CBR >5 ……..} \end{aligned} \qquad (3.1)$$

$$M_{R\_granular} = 0.2 * h^{0.45}\, M_{R\ subgarde} \quad \text{…….} \qquad (3.2)$$

Where, h= thickness of granular sub base and base, mm

The elastic modulus of bituminous layer mix type BC & DBM for VG40 bitumen @ temperature 35°c is 3000Mpa and poisson's ratio of granular bases and sub-base is recommended 0.35 as per IRC: 37-2012 codes.

3.2 Material properties of Non-linear layers

3.2.1 Granular materials:

It is well known that granular materials and subgrade soils are nonlinear with an elastic modulus varying with the level of stresses. The nonlinear material properties, which have been incorporated in **KENLAYER** with a simple relationship between resilient modulus and the first stress invariant, can be expressed as

$$E = K_1 \theta^{K_2} \qquad \ldots\ldots. \qquad (3.3)$$

In which K_1 and K_2 are experimentally derived constants and θ is the stress invariant, which can be either the sum of three normal stresses, σ_x, σ_y and σ_z, or the sum of three principal stresses σ_1, σ_2 and σ_3

$$\theta = \sigma_1 + \sigma_2 + \sigma_3 = \sigma_x + \sigma_y + \sigma_z \qquad \ldots\ldots. \qquad (3.4)$$

Including the weight of a layered system gives

$$\theta = \sigma_x + \sigma_y + \sigma_z + \gamma z\ (1+2\ K_0) \qquad (3.5)$$

In which γ is the average unit weight, z is the distance below surface at which the modulus is to be determined, and K_0 is the coefficient of work pressure at the rest.

Thompson and Elliott (1985) developed simple regression equations or algorithms for crushed stone base is represented by Eq. 3.3 with K_1 = 62.1 Mpa, K_2 = 0.33 and K_0= 0.60

3.2.2 Fine-Grained soils:

The resilient modulus of fine –grained soils decreases with the increase in deviator stress σ_d In laboratory triaxial tests, $\sigma_2 = \sigma_3$, so the deviator stress is defined as

$$\sigma_d = \sigma_1 - \sigma_3 \qquad \ldots\ldots. \qquad (3.6)$$

In layered system, σ_2 may not be equal to $\sigma_{3.}$ Including the weight of layered system yields

$$\sigma_d = \sigma_1 - 0.5(\sigma_2 + \sigma_3) + \gamma z\ (1- K_0) \qquad \ldots\ldots. \qquad (3.7)$$

KENLAYER uses the three normal stresses, σ_x, σ_y and $\sigma_{z,}$ to replace the three principal stresses σ_1, σ_2 and σ_3 in Eq. 3.7

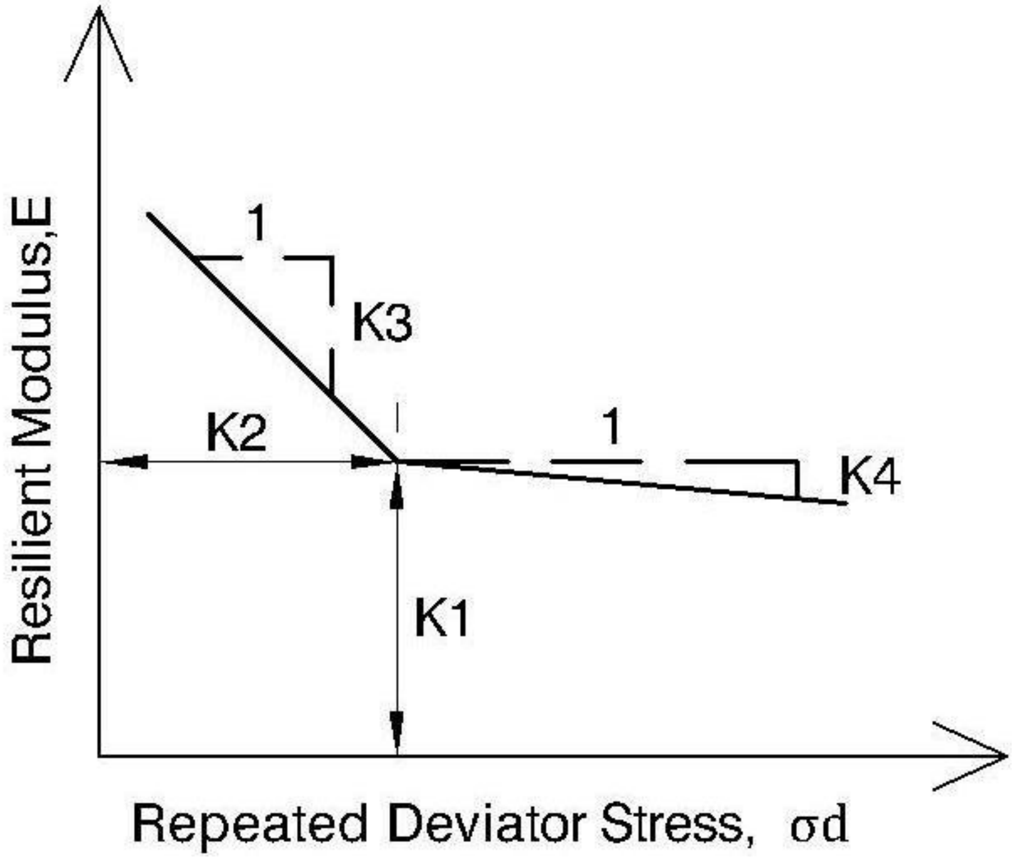


Fig 1: General relationship between resilient modulus and deviator stress of fine – grained soils

Fig 1 shows the general relationship between resilient modulus and deviator stress of fine – grained soils obtained from laboratory repeated –load tests. The bilinear behavior can be expressed as

$$E = K_1 + K_3 (K_2 - \sigma_d) \text{ when } \sigma_d < K_2 \quad \ldots\ldots. \qquad (3.8)$$

$$E = K_1 - K_4 (\sigma_d - K_2) \text{ when } \sigma_d > K_2 \quad \ldots\ldots. \qquad (3.9)$$

In which K_1, K_2, K_3, and K_4 are material constants.

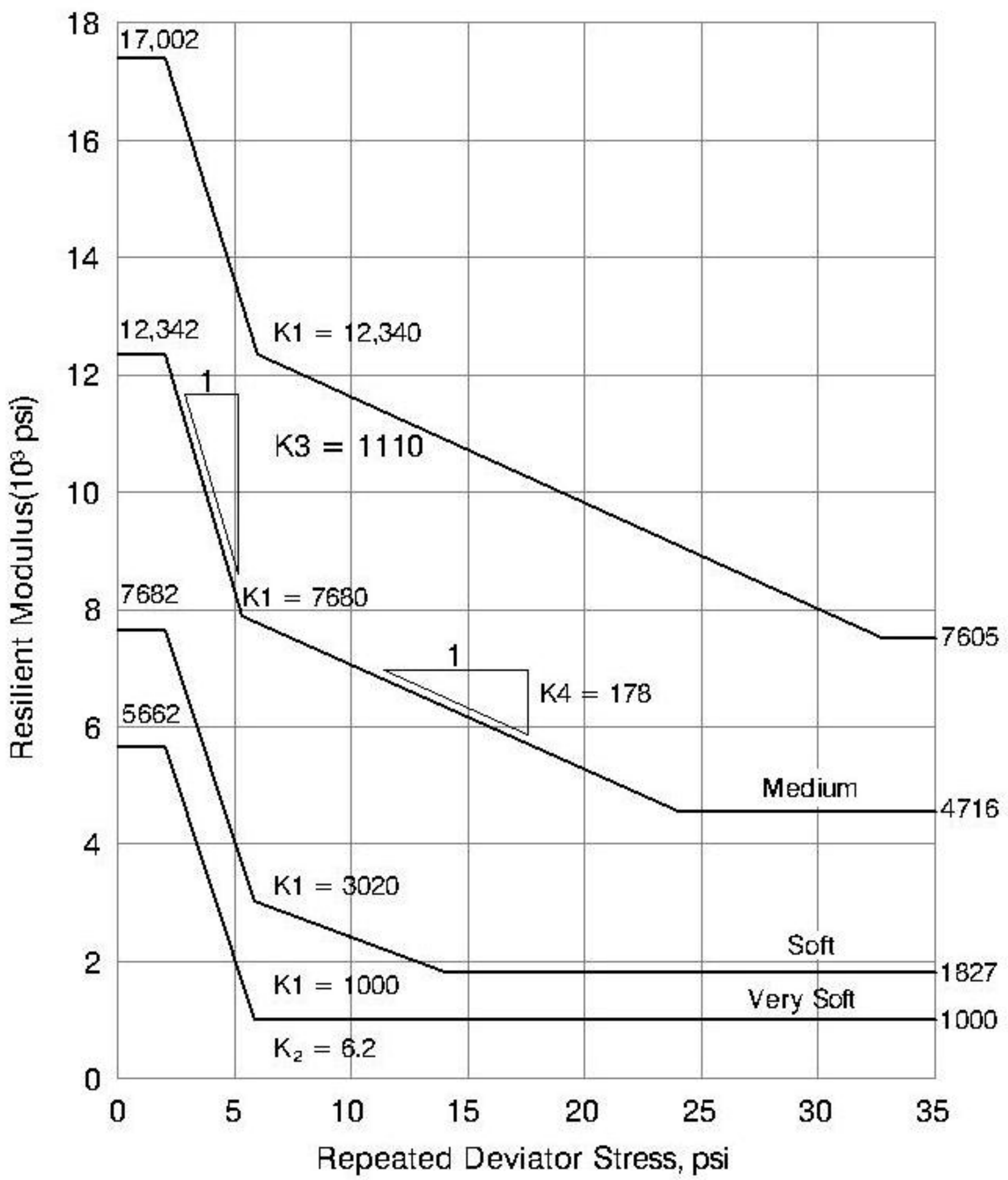
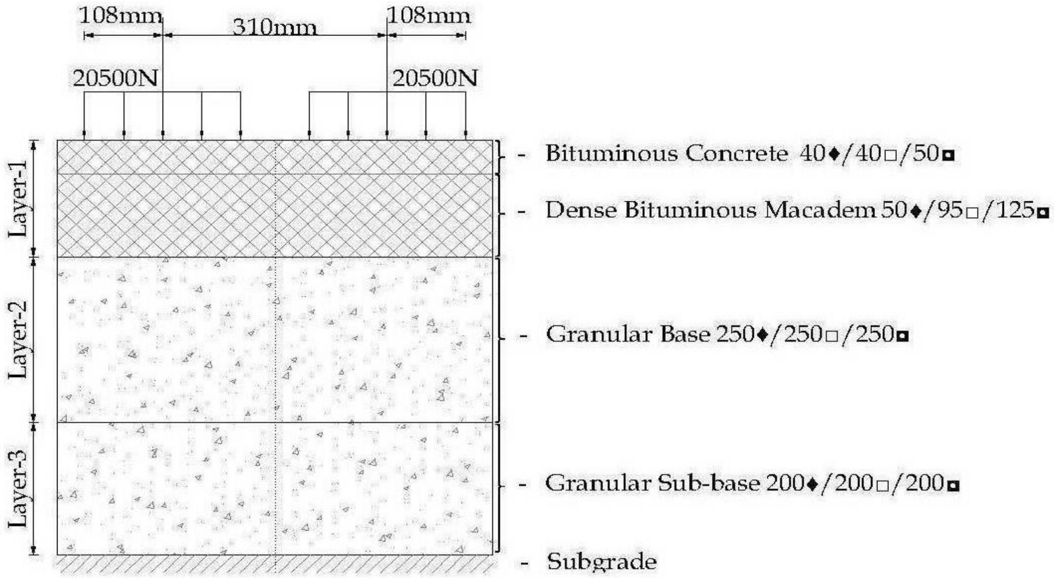


Fig 2: Resilient – modulus –deviator stress relationship for four types of subgrade (After Thompson and Elliot (1985)).

Thompson And Elliott (1985) indicated that the value of resilient modulus at the breakpoint in the bilinear curve, as indicated by K_1 in Fig 1, is a good indicator of resilient behavior, while other constant K_2, K_3 and K_4, display less variability and influence pavement to a smaller degree than K1. They classified fine grained soils into four types, viz., very soft, soft, medium, and stiff, with the resilient-modulus-deviator-stress relationship shown in Fig 2. The maximum resilient modulus is governed by a deviator stress of 13.8 kPa. The minimum resilient modulus is limited by the unconfined compressive strengths, which are assumed to be 42.8 kPa, 89.0 kPa, 157 kPa, and 226 kPa for the four soils. Equations 3.8 & 3.9 has also been incorporated in KENLAYER.

3.2.3 Method of Analysis:

3.2.3.1 Linear Analysis



♦, □ & ◘ thickness for 10 msa 50 msa & 150 msa

Fig 3: cross section of three layer system

Fig 3 show the cross section of a three-layer system subjected to a circular load with a constant radius 155mm and a constant pressure of 0.56Mpa . All three layers are linear elastic. Layer 1, has an elastic modulus of 3000Mpa and a poissons ratio of 0.35 Layer 2, has an elastic modulus of 240.163Mpa and a poissons ratio of 0.35. Layer 3, has an elastic modulus of 76.83Mpa and a poissons ratio of 0.35. Three different combination of traffic and material properties consider for this analysis is furnished in the Table 1.

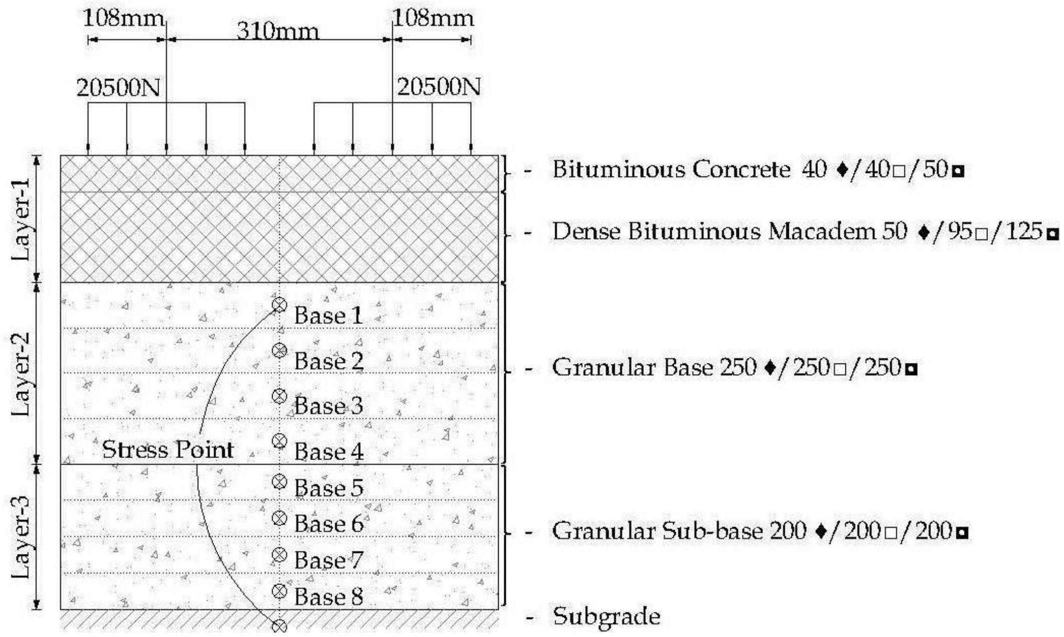
Table 1: Material Properties used in Linear Analysis

Section	**Plate -7 (CBR 10%) as per IRC:37-2012.**					
	Layer	**Thickness (mm)**			**E (Mpa)**	**ν**
		10msa	**50msa**	**150msa**		
BC & DBM	**1**	90	135	175	3000.00	0.35
G.Base	**2**	250	250	250	240.163	0.35
GSB	**3**	200	200	200	240.163	0.35
Subgrade	**-**			-	76.83	0.35

3.2.3.2 Nonlinear Analysis

The same pavement section shown in Fig 3 for nonlinear analysis was used for nonlinear analyses by considering layer 1 to be linear with the same elastic modulus and poisson ratio used in the linear analysis, layer 2 to be no linear granular base with K1 =62.10 MPa and K2 = 0.33, and layer 3 to be nonlinear soft grade with K1 =20838 MPa. A Bilinear relationship between resilient modulus and deviator stress for fine – grained soils, similar to Fig 2, with no maximum and minimum limits a very large EMAX and a zero EMIN were used in KENLAYER. Other properties are shown in the Table 2.

In applying KENLAYER granular layer can be divided into eight layers as per the Fig 4.



♦, □ & ◘ thickness for 10 msa 50 msa & 150 msa

Fig 4: Division of Granular Base into Eight Nonlinear layers

Table 2: Pavement Layer thickness & Material properties used in Nonlinear Analysis.

Section	Layer	Thickness (mm)	E (Mpa)	ν	Material Properties				
BC & DBM	1	175	3000	0.35	Isotropic and Linear Elastic				
G.Base	4 layer	62.50mm for each Layer (250mm)	240.163	0.35	Nonlinear: K- Ø Model				
					K_1(Kpa)	K_2	K_0		
					62100	0.33	0.6		
GSB	4 layer	50mm for each Layer (200mm)	240.163	0.35	Nonlinear: K- Ø Model				
					K_1(Kpa)	K_2	K_0		
					62100	0.33	0.6		
Subgrade	-	-	76.83	0.35	Nonlinear : Thompson and Elliott (1985)				
					K_1(Kpa)	K_2	K_3	K_4	K_0
					20838	42.78	7659	1228.2	0.8

Table 3: Predicated Pavement responses from Linear & Nonlinear analysis

Pavement Reponses point	Plate -7 (CBR 10%) - 10msa			Plate -7 (CBR 10%) - 50msa			Plate -7 (CBR 10%) - 150msa		
	90mm - BC& DBM, 250mm - G.Base & 200mm - GBS = 540mm			135mm - BC& DBM, 250mm - G.Base & 200mm - GBS = 585mm			175mm - BC& DBM, 250mm - G.Base & 200mm - GBS = 625mm		
	Linear	**Nonlinear**	**% of + or -**	**Linear**	**Nonlinear**	**% of + or -**	**Linear**	**Nonlinear**	**% of + or -**
Vertical Deflection (Disp Z) (mm)	0.457	0.521	14.08 %	0.386	0.465	20.49 %	0.337	0.419	24.29 %
Horizontal Tensile strain at the bottom of bituminous layer (epT) x10^{-6}	243.10	201.40	-17.15 %	190.00	180.20	-5.16 %	150.60	151.90	0.86 %
Vertical Compressiive strain at the top of subgrade (epZ) x10^{-6}	381.7	442	15.80 %	300.9	363	20.57 %	245.8	302	22.99 %

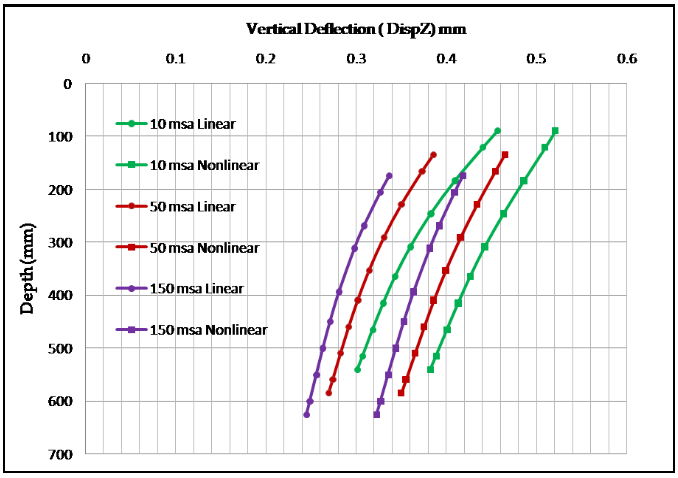


Fig 5: Comparison of linear and nonlinear horizontal tensile strains

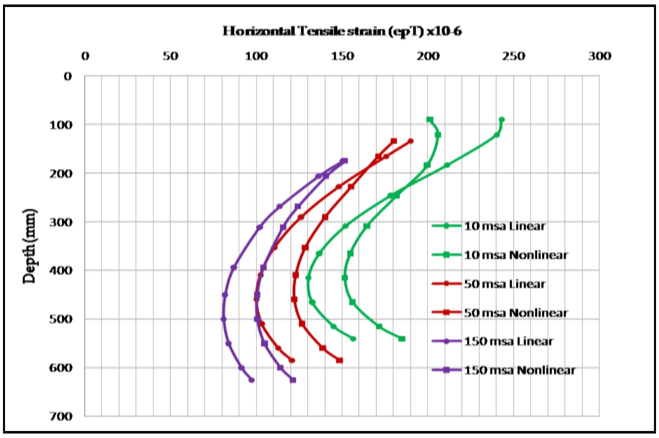


Fig 6: Comparison of linear and nonlinear vertical compressive strain

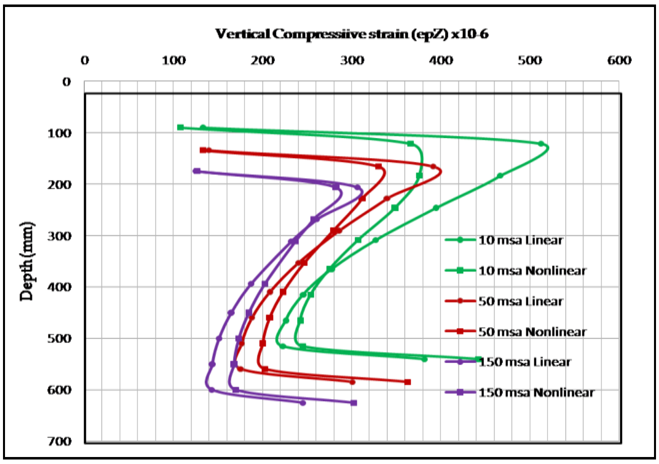


Fig 7: Comparison of linear and nonlinear vertical deflection

4.0 Analysis of Results:

Nonlinear analysis results were next compared to linear elastic analysis solution to draw conclusions and emphasize the important of proper nonlinear stress dependent geometrical characteristics.

The comparison of various layers of pavement with linear nonlinear layers are discussed below:

i) The **horizontal tensile strain** is show in Fig.5. The results indicate that the consideration of nonlinearity yields 17.15% decrease in horizontal tensile strain at the bottom of bituminous layer for 10 msa, it is at 5.16% decrease for 50msa and close the result (+0.86) of the linear case for 150 msa.

ii) The **vertical compressive strain** is show in Fig.6. The results indicate that the consideration of nonlinearity yields 15.80% increase in vertical compressive strain at the top of subgrade for 10 msa, it is at 20.57% increase for 50msa and it is at 22.99% increase for 150 msa.

iii) **Vertical deflection** distribution at the centre of loading is shown in Fig.7. The results indicate that the consideration of nonlinearity yields 14.08% increase in vertical deflection in the surface for 10 msa, it is at 20.49% increase for 50msa and it is at 24.29% increase for 150 msa.

5.0 CONCLUSION

The results show that the nonlinear base has a considerable influence on the pavement responses. The case of only nonlinear base material characterizations has a remarkable effect on critical pavement responses, especially, tensile strain at the bottom of the BC and vertical strain on the top of subgrade. Nonlinear characterization of the base material caused a maximum decreases of 17.15% in horizontal tensile strain at the bottom of the BC&DBM, 22.99% increase in vertical compressive strain on the top of subgrade and 24.29% increase in surfaces deflection. The nonlinearity of subgrade also affects the critical pavement responses. Table 3 shows the predicted critical pavement responses in each case. For the combined nonlinear base and subgrade characterizations, the most accurate pavement responses, still considerably different from the linear elastic solutions, were predicted especially for the tensile strain at the bottom of asphalt concrete and the vertical strain on the top of subgrade. Note that these differences in pavement responses, in these cases specific to the pavement geometries, layer material properties and the loading condition considered, were contrasted to demonstrate the important effects nonlinear pavement foundation modeling.

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