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Analysis of Aggregate Base Behavior Using Layered Elastic and Finite Element Methods

다층탄성해석과 유한요소법을 사용한 골재기층의 거동분석

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요 지

이 논문에서는 다층탄성해석과 유한요소법을 사용하여 도로설계를 위한 도로내 주요 변형률을 계산하여 유사한 결과치를 양산하는 경우를 비교 분석하였다. 비록 유한요소법이 보다 나은 모델이라는 것이 입증되긴 했지만, 다층탄성해석 프로그램이 간편성으로 인해 여전히 도로설계를 위해 많이 사용되어 지고 있으므로 다층탄성해석 프로그램을 사용한 주요 변형률의 예측이 시급한 실정이다. 이 연구에서는 KENLAYER 프로그램을 사용하여, 비선형 이방성 기층거동을 고려한 유한요소법을 사용했을때 얻어지는 도로내 주요 변형을 예측할 수 있는 분석기법이 소개된다.

핵심용어 : KENLAYER, 기층, 이방성, 유한요소법

Abstract

In this paper, the critical strains for pavement design were calculated from both Layered Elastic Program (LEP) and Finite Element Method (FEM) and the case studies which give similar critical responses were compared. Although FEM has been realized as a superior model, LEP is more favorable to pavement design due to its simplicity and thus, the technique to calculate the correct critical responses using LEP is significant. This study showed that KENLAYER can possibly estimate the critical responses close to ones obtained from TTIPAVE, which considers nonlinear cross-anisotropic behavior of unbound base materials, by adjusting the stress point locations.

Keywords : KENLAYER, aggregate base, anisotropy, finite element method

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1. INTRODUCTION

It has been well known that the unbound aggregate bases show nonlinear and stress dependent behavior (Tutumluer et al., 2003, Kim 2004, Kim et al. 2005, Kim et al., 2007). The stress variations along the radial and vertical directions from the surface load result in modulus variations in the radial and vertical direction. Thus, it is theoretically not correct to use a stress at a single point in the nonlinear layer to compute the modulus of the layer. Albeit the Finite Element Method (FEM) provides the best solutions for such nonlinear problems, the Layered Elastic Program (LEP) is more favorable to pavement designer due to its simplicity and short computer running time. If only the most critical strains such as the tensile strain at the bottom of asphalt layer and the compressive strain on the top of subgrade are required for design, it is possible to select a point in the base layer to compute the modulus of the layer, so that these critical responses obtained from LEP and FEM can match reasonably well. Although Huang (1993) tried to compare the results from LEP (KENLAYER) and FEM (MICH-PAVE, ILLI-PAVE), the critical responses from LEM and FEM were not matched well due to the inaccuracy of the finite element solutions. This study attempts to find the appropriate stress for computing the modulus using KENLAYER so that the reasonably same critical strains from nonlinear cross-anisotropic analysis by TTIPAVE can be obtained.

2. CRITICAL RESPONSES FOR PAVEMENT DESIGN AND MECHANISTIC COMPUTER MODEL

The first mechanistic design curves for flexible pavements, based on elastic layered theory, were developed in the early 1960s. Due to the lack of computational resources, each design curve had to be laboriously calculated by hand and thus, they could only be developed for a limited range of idealized pavement systems. The advent of innovative computational resources made it possible to calculate the load-induced pavement responses in multi-layered pavement systems. This has made it much more feasible to employ mechanistic analysis procedures in pavement design. Since 1986, the AASHTO Joint Task Force on Pavements (JTFP) has supported and prompted the development of Mechanistic-Empirical procedures for pavement thickness design. The National Cooperative Highway Research Project (NCHRP) 1-26 was the first sponsored research project for developing mechanistic empirical pavement design procedures. NCHRP 1-26 researchers proposed working versions of mechanistic empirical design processes and procedures that relate pavement response variables, such as stresses, strains and deflections ($\sigma, \varepsilon, \Delta$) due to the surface wheel loads. Since 1997, NCHRP 1-37 (Development of the 2002 Guide for the Design of New and Rehabilitated Pavement Structures) was initiated with the objective of developing mechanistic pavement analysis and design procedures suitable for use in future versions of the AASHTO guide. The general concepts of a mechanistic-empirical design procedure are illustrated in Fig. 1.

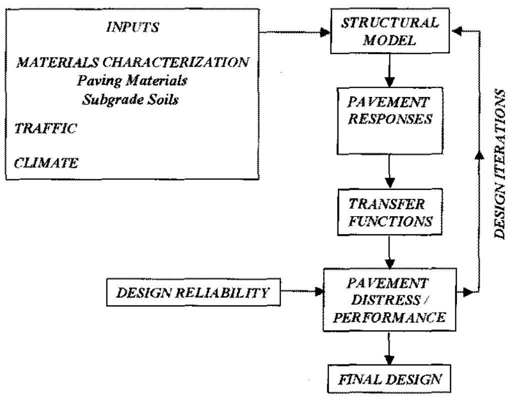

Fig. 1 Components of Mechanistic-Empirical Pavement Design

The two major components are: (1) a pavement structural model to calculate as accurately as possible the critical pavement responses ($\sigma,\varepsilon,\Delta$) and (2) transfer functions to translate those responses into measures of pavement performance. The design process entails iteratively adjusting the pavement structure until the desired level of performance and reliability are achieved.

Recommendations developed in NCHRP 1-26 Phase I indicated that the elastic layer programs (ELPs), such as BISAR, WESLEA, CHEVRON, JULEA, ELSYM5, and KENLAYER, and finite element programs (FEP), like ILLI-PAVE and MICH-PAVE are adequate to support the development of mechanistic-empirical pavement thickness design procedures. In the ELPs, which are computationally much simpler than the finite-element models (FEMs), pavement materials are assumed to be linearly elastic, isotropic, and homogeneous within well-defined horizontal layers. The stress- and direction-dependent (anisotropic) mechanical properties of the unbound granular materials and subgrade soils naturally conflict with the previous assumptions. The limitation of the ELPs is that moduli are kept constant within each horizontal layer and thus, the material non-linearity which exists in unbound granular material is not considered and the variation of horizontal stress distributions along depth are not effectively taken into account. The FEMs, on the other hand, easily accommodate irregular geometries and anisotropic and stress-dependent material properties and provide the most modern technology and the state-of-the-art sophisticated characterization of the pavement materials. Such realistic characterizations of the UAB accomplished through the use of finite element solutions significantly improve the ability to reliably predict pavement responses, which leads to a better design methodology. The consideration of the nonlinear cross-anisotropic behavior of unbound aggregate base is still in its early stage in the ELPs and FEMs.

There are currently only few ELPs and FEM programs existing that possibly take into account the cross-anisotropic analysis, which are CIRCLY and TTI-PAVE. CIRCLY is a layered elastic program which has special ability to consider material anisotropy of each layer and TTI-PAVE is a finite element program which accounts for linear, nonlinear, isotropy, and anisotropic model in the unbound granular layer.

The finite element program has capability to consider the material non-linearity, different types of loading conditions, and interface conditions. Since unbound aggregate materials are known to show nonlinear behavior, many researchers have preferred to use the finite element method for analysis of the unbound granular base in a flexible pavement. However, significant problem that tends to predict the horizontal tensile stress at the bottom of unbound granular layer was encountered. To make up the defects of predicting the tensile stress at the bottom of base layer, efforts to incorporate the cross-anisotropic model in the finite element program such as GT-PAVE and TTI-PAVE has been made by several researchers (Adu-Osei 2000, Kim 2004, Tutumluer 2003).

TTI-PAVE is an axisymmetric finite element program using elasto-plastic theory and has been developed to model a flexible pavement's response to traffic loads. The finite element code originally developed by Owen and Hinton (1993) was modified to analyze an axisymmetric problem with material non-linearity. Adu-Osei (2000) made efforts to modify the code to incorporate a cross-anisotropic model.

TTI-PAVE uses axisymmetric, isoparametric 8-node elements and a 3rd order quadrature with 9 integration points. The material parameters needed for the finite element analysis are the non-linear vertical resilient modulus coefficients, the moduli ratios $(n = Ex / Ey,$

$m = Gxy / Ey$) and the value of the vertical Poisson's ratio as well as the ratio of the horizontal to vertical Poisson's ratios. Since the moduli ratios were observed as constant for a particular material at all stress states, horizontal and shear moduli ratios were used as an input instead of resilient modulus coefficients.

The vertical Poisson's ratio was assumed to be stress-dependent and parameters, k_1, k_2, and k_3 are used to predict the Poisson's ratio as expressed by Eq. (1):

$$\frac{2}{3}\frac{\partial v_{xy}}{\partial J_2'} + \frac{1}{I_1}\frac{\partial v_{xy}}{\partial J_1} = v_{xy}\left[\frac{1}{3}\frac{k_3}{J_2'} + \frac{k_2}{I_1^2}\right] + \left[-\frac{1}{6}\frac{k_3}{J_2'} + \frac{k_2}{I_1^2}\right] \quad (1)$$

where,

v = Poisson's ratio,

k_1, k_2, k_3 = material coefficients,

I_1 = normalized first stress invariant, and

J_2 = normalized second invariant of the deviatoric stress tensor.

A numerical solution to Eq. 1 based on the backward difference method was included in the finite element code. To ensure convergence, two convergence criteria were included in the finite element program as expressed by Eq. (2). The equilibrium criteria are based on residual force values such that:

$$\frac{\sqrt{\sum_{i=1}^{N}(\Psi_i^r)^2}}{\sqrt{\sum_{i=1}^{N}(f_i)^2}} \times 100 \le TOLLER \quad (2)$$

where,

N = the total number of nodal points,

r = the iteration number,

ψ = the total applied force,

f = the applied nodal force, and

TOLLER = tolerance in convergence (percent).

To avoid unreasonable moduli predictions at low stress levels, cutoff values for both the first stress invariant and octahedral shear stress are used. The shortcoming of TTI-PAVE is that the maximum input for Poisson's ratio value is confined as 0.48 albeit Poisson's ratios above 0.5 are frequently observed in the laboratory. This shortcoming would be covered by the field conditions, which has residual and confining stress.

The NCHRP research team for project 1-37A has selected a layered elastic model of the pavement to be used in the proposed 2002 AASHTO Pavement Design Guide. This fact alone emphasizes the importance of being able to assess nonlinear behavior of aggregate base using elastic layered systems instead of using solution methodologies based on finite element analysis. KENLAYER provides the flexible pavement analysis of the multilayer system under single, dual, dual-tandem, or dual-tridem wheels with each layer behaving differently, either linear elastic, nonlinear elastic, or viscoelastic (Huang, 1993).

3. ANALYSIS AND DISCUSSION

Three different methods were performed for nonlinear analysis. In method 1 as shown in Fig. 2, the unbound aggregate base is subdivided into several sublayers and

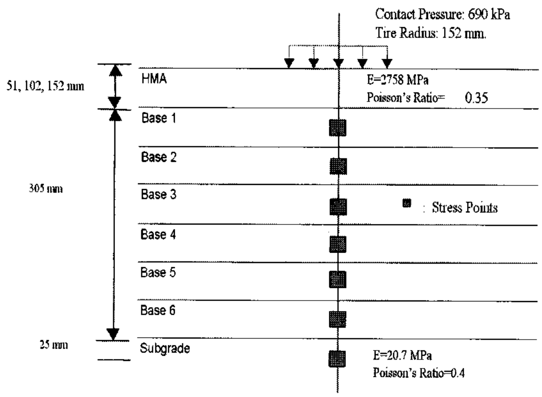

Fig 2. Method 1 in KENLAYER

the stresses at the middepth of each sublayer are used as stress points that calculate the modulus. If the horizontal stress is tension (negative), KENLAYER sets the horizontal tensile stress to zero. Thus, this method avoids the unrealistic negative first stress invariant and modulus calculations. In method 2, the unbound aggregate base is regarded as a single layer and appropriate stress point (the upper quarter, upper third, and upper half of the layer) is selected to compute the modulus. Since the selected stress points are within upper part of the base layer (compression zone), the negative first stress invariant cannot be calculated in method 2. The method 1 gives more accurate results but it requires more computing time. By selecting an appropriate point for computing the modulus, method 2 yields comparable results. Huang (1993) performed the results between method 1 & 2 and observed that the results from method 1 lie between those obtained by method 2, with one stress point at the upper quarter and the other at the upper third. Huang (1993) compared the nonlinear solutions of KENLAYER and MICH-PAVE and found that the selection of stress point at upper half with a internal friction angle of 40 degrees gives the best fit in HMA tensile strains, but the match in the subgrade compressive strain is poor when asphalt layer is thin.

Pavement analysis was performed in method 1 and method 2 using KENLAYER by varying the thickness of HMA and base layer. As shown in Figs. 2 and 3, the HMA thicknesses vary with 51, 102, and 152mm and the base thickness is 305mm. KENLAYER incorporates the $K-\Theta$ model for granular base as shown in Eq. (3). Resilient modulus and the first stress invariant can be related as following:

$$E = K_1 \Theta^{K_2} \tag{3}$$

where,

E = the resilient modulus (MPa),

Θ = the first stress invariant, and

K_1 and K_2 = the resilient modulus coefficients.

Table 1 shows the typical ranges of k_1 and k_2 for unbound aggregate materials and the values of 62 MPa and 0.33 were inputted as k_1 and k_2, respectively (Huang, 1993). Following four cases were considered in KENLAYER and compared with TTI-PAVE.

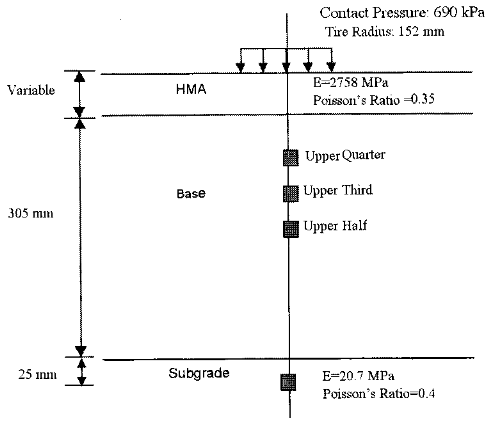

Fig 3. Method 2 and 3 in KENLAYER

Table 1. Range of k_1 and k_2 for Untreated Granular Materials

Reference	Material	K_1 (MPa)	k_2
Hicks	Partially crushed gravel, crushed rock	11 - 34	0.57-0.73
Hicks and Finn	Untreated base at San Diego Test Road	14.5 - 37	0.61
Allen	Gravel, Crushed stone	12.4 - 55	0.32-0.70
Kalcheff and Hicks	Crushed stone	27 - 62	0.46-0.64
Boyce et al.	Well-graded crushed limestone	55	0.67
Monismith and Witczak	In service base and subbase materials	20 - 53	0.46-0.65

Case 1: The unbound aggregate base is subdivided into six layers with 50mm thickness for each sub-layer. The vertical coordinates of the stress points are located at mid-depth of each layer and at the 25mm below the top of subgrade.

Case 2: The unbound aggregate base is regarded as single layer with the stress points at the upper quarter in the layer and at the 25mm below the top of subgrade.

Case 3: This case is same as case 2 except that the stress point is located at the upper third instead of at the upper quarter.

Case 4: This case is same as case 2 except that the stress point is located at the upper half instead of at the upper quarter.

Figs. 4 and 5 show comparisons of four cases of nonlinear isotropic solutions from KENLAYER with the nonlinear cross-anisotropy solutions from TTI-PAVE. Solutions from four cases are getting close together as the HMA thickness increases. The solutions by case 1 were close to those by case 3 and case 4. Especially, the HMA tensile strains by case 1 shows good agreement with case 4 in TTI-PAVE analysis, which has the stress point at the upper half in the layer. It is observed that nonlinear cross-anisotropic solutions by TTI-PAVE show higher critical responses than nonlinear isotropic solutions in KENLAYER and TTI-PAVE solutions gives best fit in case 4. This is because the computed modulus of the granular base decreases and it results in the increase of vertical compressive strain at the top of subgrade when the stress point moves down. It is noticed that the case 4 is suitable to obtain comparable critical responses with nonlinear cross-anisotropic solutions. The rut depth has been calculated based on the Tseng and Lytton model (1989) and Fig. 6 shows a comparison of permanent deformation of each case by KENLAYER and TTI-PAVE. The dotted line, which is the calculated permanent deformation by case 1 lie between case 3 and case 4. The permanent deformation by TTI-PAVE is higher than that of case 1 and fit well with the results of case 4. Therefore, it could be mentioned that the KENLAYER solutions by case 4 are reasonably similar to those by nonlinear cross-anisotropic TTI-PAVE solutions.

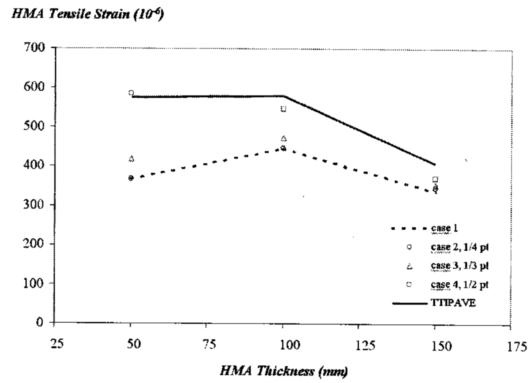

Fig. 4. Nonlinear Solutions of HMA Tensile Strain between KENLAYER and TTI-PAVE

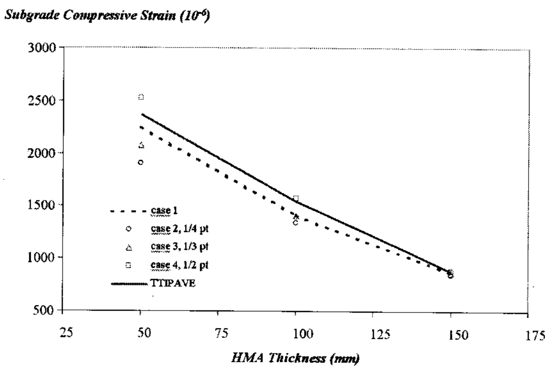

Fig. 5. Nonlinear Solutions of Subgrade Compressive Strain between KENLAYER and TTI-PAVE

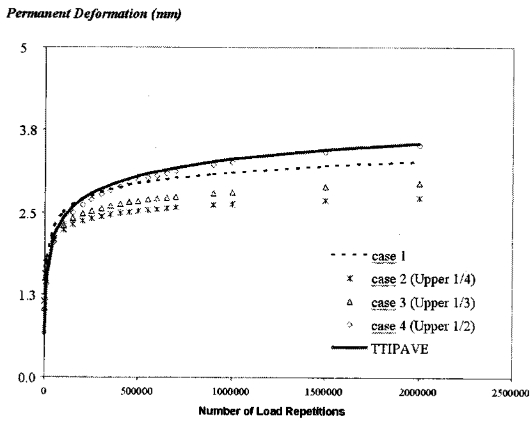

Fig. 6. Comparison of Permanent Deformations between KENLAYER and TTI-PAVE

4. CONCLUSION

In the paper, KENLAYER and TTI-PAVE were used as LEP and FEM, respectively, and it was observed that unbound aggregate base needs to be considered as single layer with the stress points at the upper half in KENLAYER to produce similar critical responses from TTI-PAVE. Nonlinear cross-anisotropic solutions of unbound aggregate base can be simulated by TTI-PAVE and the critical responses from TTIPAVE were generally higher than ones from nonlinear isotropic solutions in KENLAYER. It is noticed that the analysis technique to consider aggregate base as single layer with the stress points at the upper half in the KENLAYER is suitable to obtain comparable critical responses with TTI-PAVE, which is nonlinear cross-anisotropic solutions. This is because the computed modulus of the granular base decreases and it results in the increase of vertical compressive strain at the top of subgrade when the stress point moves down.

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