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Influence of Anisotropic Behavior of Aggregate Base on Flexible Pavement Design Life

기층의 이방성 거동이 아스팔트 도로 설계수명에 미치는 영향

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요 지

이 논문에서는 아스팔트 도로 설계에 필요한 기층골재 재료의 비선형 이방성을 고려한 연결함수를 개발하였다. 기층이 비선형 이방성 거동으로 해석되어졌을 경우, 선형 등방성 거동으로 해석 되어질때 나타나는 기층 하부내의 인장력을 감소시켜 보다 현실적인 응력분포를 보이게 된다. 그러나 현재까지 개발된 연결함수들은 대부분 기층이 선형 등방성 거동으로 해석하여 개발된 것이므로, 비선형 이방성 거동을 근간으로 하는 연결함수의 개발이 현실적인 도로 설계를 위해 필요하다. 이 논문에서 개발된 연결함수를 이용하여 도로를 설계한 결과 AASHTO의 연결함수를 이용하여 설계 했을 경우보다, 기층 두께가 25mm 감소되는 결과를 보였으며, 이는 AASHTO 도로 설계가 보수적인 설계라는 것을 입증하였다.

핵심용어 : 기층, 이방성, 유한요소법

Abstract

This paper presents the development of transfer function accounting for cross-anisotropic behavior of aggregate base material for the pavement thickness design. The stress distributions predicted by nonlinear cross-anisotropic finite element program were realistic by eliminating excessive tensile stress at the bottom of the base layer and the critical pavement responses predicted by nonlinear cross-anisotropic model are higher than those predicted by linear or nonlinear isotropic models (Kim, 2004, Kim et al., 2005). Since the previously developed transfer functions such as Asphalt Institute and Chevron models, etc. were based on the critical responses obtained from linear isotropic model, those equations are not appropriate for the thickness design nonlinear cross-anisotropic base behavior. Therefore, the development of usable transfer functions for nonlinear cross-anisotropic model is ever more important. When the newly developed transfer functions were compared with AASHTO method for the thickness design, the newly developed transfer functions produce approximately 25mm reduced UAB thickness in AASHTO thickness design and this illustrates that linear isotropic model results in more conservative pavement design.

Keywords : unbound aggregate base, anisotropy, finite element method

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1. INTRODUCTION

A conventional flexible pavement is composed of a prepared subgrade, subbase, base, and a surface layer. The surface layer is usually a hot mix asphalt (HMA) and the base and subbase layers consist of unbound granular materials. Unbound aggregate base is a primary structural layer of a pavement. The principal functions of unbound aggregate base are to diminish the load-induced stress on the subgrade to a degree that the subgrade can sustain without significant rutting and to provide adequate support for the surfacing.

Existing flexible pavement design approaches generally assume the pavement structure as a linear isotropic layered system, which means that the properties are considered to be same in all directions. Linear elastic analysis can be used with high confidence for the full depth asphalt pavement structures, but it is not proper for unsurfaced or thinly surfaced flexible pavements because the resilient properties of unbound granular materials are nonlinear and stress dependent (Kim 2004, Kim et al., 2005).

The most serious problem of linear isotropic analysis in thinly surfaced flexible pavements is the erroneous prediction of strong tensile stresses at the bottom of the aggregate base layer. The granular materials, however, have little to no tensile strength since the load transfer is achieved through compressive and shear stress between particles. In order to correctly characterize unbound aggregate bases it is necessary to account for the directional properties or cross-anisotropy of these layers.

Recently, Kim et al. (2005) developed a methodology to characterize unbound aggregate base layers and to consider the stress-sensitivity and cross-anisotropy of these unbound aggregate layers. Since it is important to be able to consider the stress-sensitive and anisotropic nature of unbound granular materials, there is a pressing need to be able to correlate these properties with pavement design life to maintain pavements in timely manner.

The predicted anisotropic properties were used in finite element model to correctly estimate the pavement design life. To meet this requirement, specific objectives were formulated as follows:

(1) Develop a database of anisotropic resilient moduli of wide range of unbound aggregate bases with different moisture and compaction conditions,
(2) Analyze the anisotropic base behavior using a Finite Element Model (FEM) based on aggregate physical properties,
(3) Develop performance models for fatigue cracking and rutting, and
(4) Compare the predictions from newly developed transfer functions and AASHTO transfer function

2. LABORATORY TESTING

The International Center for Aggregates Research (ICAR) and the Texas Transportation Institute (TTI) developed a method to fully characterize the required gamut of stress-sensitive and cross-anisotropic properties of unbound aggregate bases (Adu-Osei et al., 2000). This method requires specialized equipment capable of applying different stress paths.

Cylindrical samples that are 150-mm in diameter by 150-mm high were prepared using a Texas Gyratory Compactor (TGC). The prepared specimens were tested with the Rapid Triaxial Tester (RaTT), which effectively simulates the moving wheel loads following the ICAR protocol. The basic aggregate material properties were obtained from a well graded aggregate compacted at optimum moisture content as shown in Table 1.

The ICAR protocol uses three stress regimes and ten stress states within each regime to determine stress sensitivity and cross anisotropy. The three stress regimes

Table 1. Moisture Contents and Dry Densities of Aggregate

compaction	Well		
	Dry	Optimum	Wet
wc (%)	4.6	4.8	5.2
γ_d (kg/m^3)	2148	2214	2181

are conventional triaxial compression, triaxial shear, and conventional triaxial extension. In the conventional triaxial compression mode, the confining stress at each stress state is kept constant while the axial stress is increased by $\Delta\sigma_y^c$. Thus, the sample is loaded to σ_y^c, σ_x^c, reloaded to $\sigma_y^c + \Delta\sigma_y^c$, $\Delta\sigma_x^c$, and then is unloaded to σ_y^c, $\Delta\sigma_x^c$ within each cycle. In the triaxial shear mode, the axial stress is increased slightly by $\Delta\sigma_y^s$, and the confining stress is decreased by $\Delta\sigma_x^s$ =1/2 $\Delta\sigma_y^s$. Thus, from the stress state σ_y^s, σ_x^s, the sample is loaded to σ_y^s+$\Delta\sigma_y^s$, σ_x^s - $\Delta\sigma_x^s$, and then is unloaded to σ_y^s, σ_x^s, within each cycle such that there is no change in the first stress invariant. In the conventional triaxial extension phase of the test, there is a slight decrease in the axial stress by $\Delta\sigma_y^e$, and a slight increase in the confining stress by $\Delta\sigma_x^e$. Thus, at the stress state σ_y^e, σ_x^e, the sample is loaded to σ_y^e - $\Delta\sigma_y^e$, σ_x^e+$\Delta\sigma_y^e$, and then is unloaded to σ_y^e, σ_x^e.

A loading cycle of dynamic stress consists of 1.5 seconds of loading and 1.5 seconds of unloading. Dynamic loading is applied on a sample for 25 repetitions until a stable resilient strain is achieved. The resilient axial and radial strains are determined for each stress regime and then input into the system identification scheme in order to calculate the five anisotropic elastic properties at that particular stress state. The applied static and dynamic stresses are shown in Table 2. The determined anisotropic material properties are tabulated in Table 3. This testing provides the information for an extensive database and offers the opportunity to predict k-values from basic physical properties of the aggregates, including gradation, density, and particle shape, etc.

Table 2. Static and Dynamic Stresses

Stress State	Static Stress (kPa)		Dynamic Stress (kPa)					
			Triaxial Compression		Triaxial Shear		Triaxial Extension	
	$\Delta\sigma_y$	$\Delta\sigma_x$	$\Delta\sigma_y^c$	$\Delta\sigma_x^c$	$\Delta\sigma_y^s$	$\Delta\sigma_x^s$	$\Delta\sigma_y^e$	$\Delta\sigma_x^e$
1	40	25	5	0	10	-5	-5	5
2	50	25	10	0	10	-5	-10	5
3	70	40	10	0	10	-5	-10	10
4	130	60	20	0	20	-10	-10	10
5	150	70	20	0	20	-10	-10	10
6	170	100	20	0	20	-10	-20	20
7	220	120	30	0	30	-15	-20	20
8	250	140	30	0	30	-15	-20	20
9	250	120	30	0	30	-15	-20	20
10	250	105	30	0	30	-15	-20	20

Table 3. Pavement Material Parameters

HMA Layer (Nonlinear Isotropic Model)			
k_1=28,000 k_2=0.100 k_3=0.001 n=1.00 m=0.38 ν_{xy}= 0.35 μ= 1.00			
Base Course			
Linear Isotropic	Non-Linear Isotropic	Linear Anisotropic	Non-Linear Cross-Anisotropic
k_1 = 4,000 k_2 =0.0, k_3=0.0 n =1.0, m=0.38 ν_{xy} =0.2, μ =1.0	k_1=4,000 k_2=0.555, k_3=0.3 n=1.0, m=0.38 ν_{xy} =0.2, μ=1.0	k_1=4,000 k_2=0.0, k_3=0.0 n=0.5, m=0.38 ν_{xy} =0.2, μ =1.0	k_1=4,000 k_2=0.555, k_3=0.3 n=0.5, m=0.38 ν_{xy} =0.2, μ=1.5
Sub-grade (Non-linear Isotropic Model)			
k_1 = 207 k_2 = 0.001 k_3 = 0.300 n = 1.00 m = 0.38 ν_{xy} = 0.35 μ = 1.00			

Adu-Osei et al. (2000) assumed that the elastic moduli in different directions obey the Uzan model and thus, can be represented as smooth functions of the stress invariants as shown in Eqs. (1), (2), and (3).

$$E_y = k_1\left(\frac{I}{Pa}\right)^{k_2}\left(\frac{\tau_{oct}}{Pa}\right)^{k_3} \tag{1}$$

$$E_x = k_4\left(\frac{I}{Pa}\right)^{k_5}\left(\frac{\tau_{oct}}{Pa}\right)^{k_6} \tag{2}$$

$$G_{xy} = k_7 \left(\frac{I}{Pa}\right)^{k_8} \left(\frac{\tau_{oct}}{Pa}\right)^{k_9} \tag{3}$$

where:

I = first stress invariant (bulk stress),

τ_{oct} = Octahedral shear stress,

P_a = atmospheric pressure, and

k_i = material coefficients.

3. EFFECT OF CONSTITUTIVE MODEL ON PAVEMENT PERFORMANCE

Fig. 1 shows the cross sections used for the pavement analysis. The thickness of HMA was 50 or 100mm and the base course had 300-mm. The thickness of subgrade was assumed as semi-infinite. To model the test sections, the wheel load was applied as a uniform pressure of 689 kPa over a circular area of radius 136-mm. A fixed boundary was assumed at the bottom of the subgrade where the concrete slab was placed.

The HMA layer and subgrade were assumed to be nonlinear isotropic and Table 3 is a summary of material input properties for finite element program. Four constitutive models were used to represent the base layer, namely linear isotropic, nonlinear isotropic, linear cross-

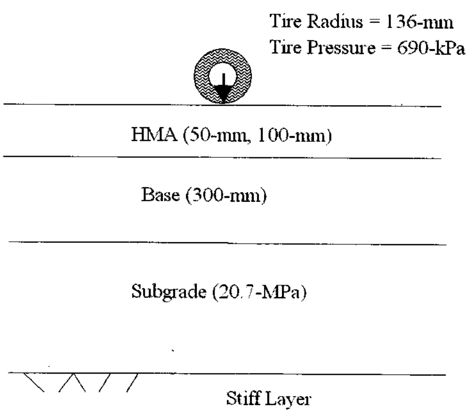

Fig. 1 Cross Section for Pavement Analysis

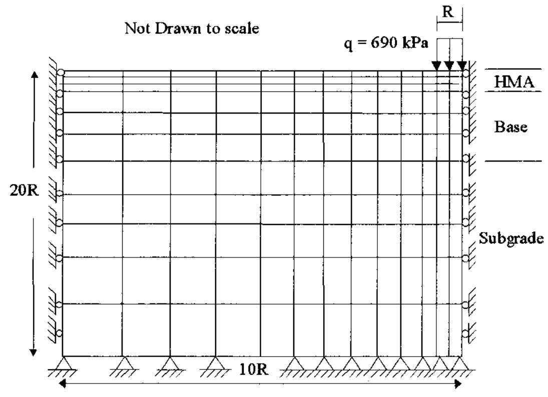

Fig. 2 Finite Element Mesh

anisotropic and non-linear cross-anistropic in the finite element program.

The Uzan's nonlinear model was implemented in TTI-PAVE, which is an axisymmetric finite element program that was developed to model a flexible pavement's response to traffic loads and to analyze an axisymmetric problem with material nonlinearity. TTI-PAVE incorporates the elastic linear, nonlinear, isotropic and cross-anisotropic model. TTI-PAVE uses an isoparametric 8-node element with 9 integration points. The material parameters needed for the finite element analysis are the nonlinear vertical resilient modulus k-values (k_1, k_2, k_3), the moduli ratios ($n = E_x/E_y$, $m = G_{xy}/E_y$), and the value of the vertical Poisson's ratio as well as the ratio of the horizontal to vertical Poisson's ratios (Kim et al. 2007).

An axisymmetric finite element mesh is shown in Fig. 2. It was assumed that the nodal radial strains were negligible at approximately 10 times R (radius of loaded area) from the area of applied wheel load and the nodal stresses and displacements were assumed to be negligible at 20 times R below the pavement surface.

Fig. 3 through Fig. 6 show the vertical and horizontal stress distribution in unbound aggregate base and significant differences occur among the constitutive models (i.e. tension is positive and compression is negative). The vertical stress distributions within the base

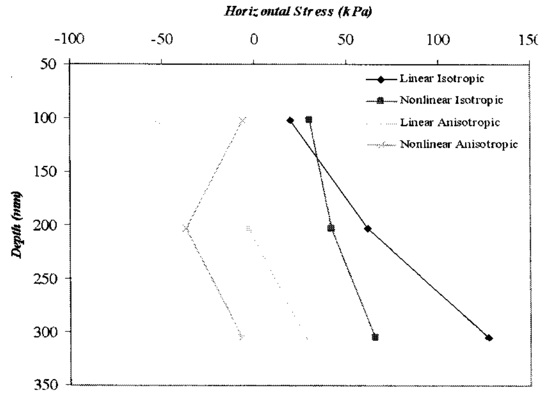

Fig. 3. Horizontal Stress for 50mm HMA, 300mm Base and 20.7 MPa subgrade

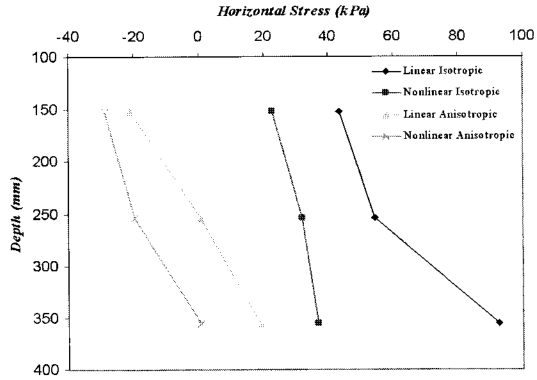

Fig. 4 Horizontal Stress for 100mm HMA, 300mm Base and 20.7 MPa subgrade

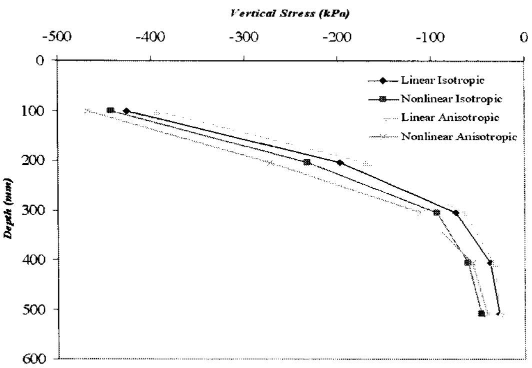

Fig. 5 Vertical Stress for 50mm HMA, 300mm Base and 20.7 MPa subgrade

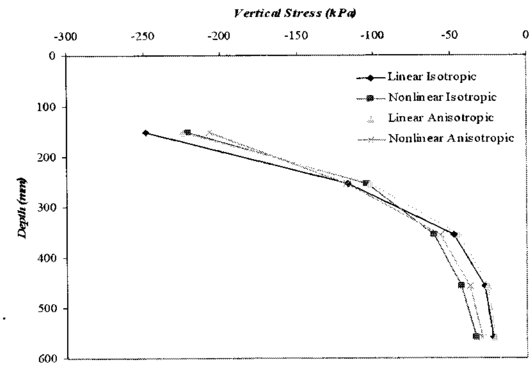

Fig. 6 Vertical Stress for 100mm HMA, 300mm Base and 20.7 MPa subgrade

layer do not have specific trend with respect to the constitutive models except the observation that linear anisotropic model generally gives lower vertical compressive stress. Pavement profiles and subgrade moduli rather than constitutive models have a significant effect on the vertical stress distributions. Modeling the unbound granular base layer as linear isotropic, nonlinear isotropic, linear anisotropic and nonlinear anisotropic in that order, gradually shifts the horizontal stresses from a tension to a compression. Also, increased HMA thickness for a given base layer thickness and subgrade modulus gives less magnitude of stresses in both horizontal and vertical directions.

The nonlinear cross-anisotropic model gave much more realistic stress distribution as shown in the compressive horizontal stresses at the bottom of the base layer while the significant tensile stresses were computed using the other models. Thus, it would not be correct or desirable way to design pavement thicknesses by increasing the thickness of unbound aggregate base until the tensile stress that is obtained by linear isotropic model are diminished.

4. DISTRESS MODELS FOR ANISOTROPIC RESPONSES AND PAVEMENT STRUCTURAL LIFE ANALYSIS

Design criteria and distress models were developed in terms of maximum allowable load repetitions and used as the indicator of selecting the thickness of unbound aggregate base. The design charts are based on multi-layered elastic anisotropic system in cylindrical coordinates under axial symmetry. The materials in each layer are characterized by a resilient modulus and Poisson's ratio and traffic is expressed as repetitions of 80 kN single axial load. The single tire is approximated by one circular plates with radius = 152mm corresponding to an 80 kN axle load and 550 KPa contact pressure. The multi-layered elastic system assumes that the surface traffic loading produces two critical pavement responses which are horizontal tensile strain, ε_t, at the bottom of the HMA layer, and vertical compressive strain, ε_c, at the top of the subgrade layer as shown in Fig. 7. Excessive horizontal strains at the bottom of the HMA layer result in the fatigue cracking while excess compressive strains at the top of subgrade layer result in the permanent deformation.

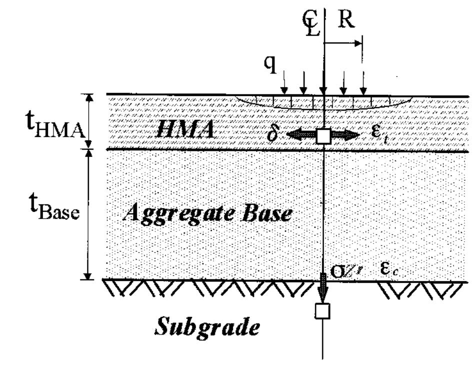

Fig. 7 Critical Pavement Responses of Conventional Flexible Pavement

The empirical part of Mechanistic-Empirical (M-E) design is the pavement life equations, which is transfer function. The transfer functions (or distress models) relate the computed pavement responses to pavement performance as measured by the type, severity, and extent of distress (e.g., rutting, cracking, etc). The most commonly used transfer functions relate pavement life to asphalt flexural strains (fatigue cracking) and to subgrade stresses and deflections (rutting).

The transfer function determines the allowable number of load applications before the pavement failure and the amount of damage done to the pavement can be expressed as the ratio of applied (n_{ij}) to allowable (N_{ij}) loads. To match predicted and observed pavement distress and performance, extensive field calibration and verification are often required to establish reliable distress prediction models (Kim et al., 2007). In this paper, AASHO road test data were used for the transfer function development.

The stress distributions predicted by nonlinear cross-anisotropic finite element program are realistic by eliminating excessive tensile stress at the bottom of the base layer and critical pavement responses predicted by nonlinear cross-anisotropic model are higher than those predicted by linear or nonlinear isotropic models. Fatigue and rutting characteristics of the flexible pavement are represented by the equation:

$$N_f = 8.148 \times 10^{-6} \varepsilon_t - 3.376 E_{mix} - 0.199$$

(R-square: 72 %) (4)

$$N_d = 1.5 \times 10^{-8} \varepsilon_c - 4.35$$

(R-square: 84 %) (5)

Where,

N_f = allowable number of 80 kN equivalent single axle load (ESAL) applications based on fatigue cracking,

N_d = allowable number of 80 kN equivalent single axle load (ESAL) applications based on rutting,

ε_t = predicted tensile strain at the bottom of the asphalt surface layer, and

ε_c = predicted compressive strain at the top of the subgrade

Eqs. (4) and (5) show that reduced strain corresponds to increased pavement life. Once the allowable and applied numbers of load applications are obtained, Miner's hypothesis is used to obtain the accumulating damage as shown in Eq. (6). When damage exceeds 1, the pavement thicknesses need to be increased while the pavement thicknesses need to be decreased when damage is much less than 1. When damage is near, but not exceeding 1, a desirable pavement design can be obtained.

$$D=\sum_{j=1}^{n}\sum_{i=1}^{m}\frac{n_{ij}}{N_{ij}}=1 \tag{6}$$

where,

n_{ij} = expected number of load applications

N_{ij} = predicted number of load applications

m = number of axle load intervals, and

n = number of seasons or time periods.

The design case studies for 20.7MPa subgrade are developed with ICAR design method and compared with AASHTO and AI methods in Table 4.

Given subgrade modulus of 20.7MPa, ESAL = 2×10^6, and an unbound aggregate base of 345MPa, AASHTO design guide determines the thicknesses of HMA and UAB as 127 and 798mm, respectively. Asphalt Institute method is not applicable because the maximum base thickness is 457mm. In ICAR design method, several combinations of thickness designs are determined as following: (102mm HMA, 851mm UAB) / (152mm HMA, 699mm UAB). In ICAR method, 51mm thinner HMA layer makes average 152mm thicker UAB. Compared with the thickness design by AASHTO method, ICAR method produces 127mm in HMA and 775mm in UAB thickness, which is approximately 25mm reduced UAB thickness in AASHTO thickness design.

The high quality unbound granular material, which has less elongation, more angularity, and rough texture has better ability to spread a surface loading and higher resilient modulus and the increase of resilient modulus of granular material affects the pavement thickness design (Kim et al., 2005). Table 4 shows the comparison of thickness design when the resilient modulus of unbound granular base is varied in ICAR method. It is observed that the 60% increase of UAB resilient modulus generates the 45% decrease of unbound aggregate base thickness.

Table 4. Comparison of Thickness Design of Various Design Methods

Design Method	UAB Resilient Modulus (MPa)	HMA thickness (mm)	UAB thickness (mm)
AASHTO	20.7	127	798
AI	Not Applicable		
ICAR	20.7	127	775
	345	127	559
	551	127	305

5. CONCLUSION

The studies have indicated that the unbound aggregate base material should be modeled as nonlinear and cross-anisotropic to account for stress sensitivity and the significant differences between vertical and horizontal moduli and Poisson's ratios. The advantage of the use of cross-anisotropy for the analysis of unbound granular bases is the drastic reduction of bottom tensile strain predicted by linear elastic analysis based on the assumptions of isotropy. When the newly developed transfer functions were compared with AASHTO method for the thickness design, the newly developed transfer functions produce approximately 25mm reduced UAB thickness in AASHTO thickness design. This illustrates that the incorporation of cross-anisotropic model for aggregate base behavior results in the accurate pavement thickness design and reduction of the pavement construction, rehabilitation, and life cycle costs of the pavements.

References

Adu-Osei, A., Little, D. N., and Lytton, R. L. (2000). "Structural Characteristics of Unbound Aggregate Bases to Meet AASHTO 2002 Design Requirements." *Interim Report, International Center for Aggregate Research (ICAR) Project 502*, Texas Transportation Institute, Texas A&M University, College Station, TX, July.

Kim, S.H. (2004). "Determination of Aggregate Physical Properties and Its Effect on Cross-Anisotropic Behavior of Unbound Aggregate Materials." *Ph.D. Dissertation, Texas A&M University*, College Station, TX.

Kim, S. H., Little, D. N., and Masad, E. (2005). "Simple Methods to Estimate Inherent and Stress-Induced Anisotropic Level of Aggregate Base." *Transportation Research Record 1913*, TRB, National Research Council, Washington D.C, 24-31.

Kim, S. H., Little, D.N., Masad, E., and Lyton, R. L. (2005). "Estimation of Level of Anisotropy of Unbound Granular Layer Considering Aggregate Physical Properties." *International Journal of Pavement Engineering*, Volume 6, Number 4, 217-227.

Kim, S. H., Tutumluer, E., Little, D. N., Kim, N. (2007), "Effect of Gradation on Nonlinear Stress-Dependent Behavior of a Sandy Flexible Pavement Subgrade" *Journal of Transportation Engineering*, American Society of Civil Engineering, Vol. 133, No. 10, Oct 1, 2007.

Kim, S. H., Kim, N. (2007), "Micromechanics Analysis of Granular Soils to Estimate Inherent Anisotropy" *Journal of Highway Engineering*, Korean Society of Civil Engineers, Vol. 11, No. 3. May 2007, pp.145-149.

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