

Nonlinear Resilient Behavior of Unbound Materials

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ABSTRACT: A thorough understanding of the behavior of unbound geotechnical materials is critical for the design of effective infrastructure. In particular, pavement systems are routinely designed with varying qualities of unbound materials that are used to distribute wheel loads from the pavement surface to the natural subgrade. The design of these pavement systems requires effective characterization of both the shear strength and stiffness of each of these materials. Since pavements are subjected to dynamic loading, the resilient behavior of unbound pavement layers is of particular interest. The objective of this paper is to provide an improved understanding of the nonlinear resilient behavior of unbound materials. The results of advanced resilient modulus tests were summarized to characterize the resilient response to a variety of unbound materials under isotropic and anisotropic conditions. The results of the laboratory tests were used to fit nonlinear isotropic and anisotropic material models, and the resulting models were used to evaluate general trends in the nonlinear behavior of different soil types. The results indicate that nonlinear resilient behavior models can provide more accurate representations of the actual behavior of geomaterials than the assumption of linear elastic behavior. The results of the testing and analyses also illustrate the behavioral differences between stress-hardening and stress-softening materials.

KEYWORDS: unbound materials, shear strength, stiffness, resilient modulus, nonlinear resilient behavior

1 INTRODUCTION

The design and performance of transportation infrastructure heavily rely on a comprehensive understanding of unbound geotechnical materials, which form the foundational layers in many pavement systems. These unbound materials play a crucial role in distributing traffic loads to underlying soils, directly influencing the durability and functionality of roadways (Kumar et al. 2022). Accurate characterization of their mechanical properties including shear strength, stiffness, and their resilient behavior is essential for designing pavement systems that can withstand dynamic traffic loads overtime.

The resilient modulus (M_r) is a key material property used in mechanistic pavement designs to quantify the elastic stiffness of unbound materials under repeated load conditions, directly impacting structural performance (Kaneza et al. 2020). Traditional design approaches often assume linear elastic behavior of these materials; however, the M_r of unbound materials exhibits nonlinear behavior. The resilient modulus of unbound materials at different stress states can be estimated using regression models based upon either bulk stress (θ), deviator stress (σ_d), the octahedral shear stress (τ_{oct}), or a combination of these parameters. The M_r can be estimated at different stress states using material specific constants (e.g. k_1 , k_2 , and k_3) for the modeled function of bulk stress (θ) to help describe and model the materials shear strength, stiffness, and stress sensitivity (Kumar et al. 2023).

The stress dependent models can be used to analyze how pavements react to traffic loads based on mechanical behaviors to compare the predicted performance of pavement structures using nonlinear versus traditional linear elastic material representations. Understanding the interactions between M_r , the K-value model constants, and nonlinear behavior is essential for optimizing material selection, structural design, and maintenance strategies to enhance the durability and serviceability of pavement systems. These models account for the variations of M_r specific to certain materials, enabling a more reliable evaluation of deformation potential and longevity of pavement systems (Plati et al., 2025).

This paper aims to improve the understanding of the nonlinear resilient behavior of various unbound geotechnical materials through a series of advanced M_r laboratory tests conducted under both isotropic and anisotropic stress states. The findings reveal significant distinctions in the stress-hardening and stress-softening materials and demonstrate the

improved accuracy and predictive capability of nonlinear resilient models (Kaneza et al. 2020). By integrating these advanced mechanistic material models into pavement design, this research highlights potential improvements in infrastructure performance.

2 BACKGROUND

Unbound materials such as fine-grained soils, granular materials, crushed rock, and recycled aggregates are widely used as foundation layers in pavement and railway structures due to their availability, cost-effectiveness, and satisfactory mechanical performance (Kumar et al. 2022). The resilient behavior of unbound materials refers to their ability to recover elastically after being subjected to repeated loading cycles. Linear elastic models have been routinely employed to model this behavior. However, extensive experimental evidence indicates that unbound materials display nonlinear resilient responses that are further influenced by the main isotropic or anisotropic stress condition, moisture content, and loading history (Kumar et al. 2023).

Nonlinearity arises from several factors, including particle size distribution, plastic deformation, and changes in contact mechanics under cyclic loading which cause variations in stiffness and strain properties that cannot be accurately captured by simple linear models. The accurate characterization of the nonlinear resilient behavior of unbound materials is essential for developing predictive models that can simulate pavement performance under realistic service conditions (Kumar et al. 2024). These response and associated performance models are used to estimate rutting potential, life expectancy, and structural capacity over time. By accurately characterizing the nonlinear resilient behavior of unbound materials, engineers can optimize material selection, layer thickness design, and maintenance strategies to improve infrastructure and reduce life-cycle costs (Plati et al., 2025). This research contributes to the increased understanding of the behavior of unbound materials that is necessary for advancing pavement engineering and sustainable construction practices.

3 METHODS

3.1 Resilient Modulus (M_r) Test

The resilient modulus (M_r) test is a specific application of the Repeated Load Triaxial Test (RLT) to simulate the stress

caused by traffic loads and evaluate the elastic response of unbound pavement materials (Kumar et al. 2023). The test is typically performed in accordance with AASHTO T 307. Specimens 71 or 81 mm in diameter (D) and a length of 2D are prepared and placed in the triaxial testing apparatus. The specimen is subjected to various confining pressures and cyclic deviatoric stresses that simulate 15 different stress states that represent a range of loading conditions. The loading consists of a haversine pulse, applied in predetermined sequences, and typically includes 100 repetitions for each corresponding cyclic stress. The load pulse is applied for 0.1 seconds and is followed by a relaxation period of 0.9 seconds (AASHTO 2021). From the resilient axial strain, the resilient modulus is calculated as indicated in Equation (1).

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

where σ_d is the deviator stress, defined as the difference between the axial stress (σ_1) and the confining stress (σ_3), and ε_r is the recoverable axial strain measured from the test results.

Depending on the assumed stress conditions, the RLT can be performed under different loading conditions. Typically, the triaxial testing equipment is utilized to determine the laboratory mechanical properties of a material under the assumption that the material is isotropic and that its properties are not dependent on the direction of the load application. However, the response of pavement materials presents an anisotropic behavior, meaning their material properties and response will vary depending on the direction of loading. To more accurately characterize the behavior of the unbound pavement materials, the University of Illinois at Urbana-Champaign (UIUC) developed the advanced triaxial testing system (UI-FastCell) that is capable of measuring the anisotropic behavior of unbound materials by applying the pulse load in both the vertical and horizontal directions (Tutumlur and Seyhan, 1999; Chow et al., 2014). The UI-Fast Cell utilizes two stress channels that are controlled independently, allowing the application of various stress states to the same specimen. Pulsing is individually applied either vertically or horizontally at a time, and is performed in the same fashion as in the conventional RLT, consisting of a loading period followed by a relaxation period (Tutumlur and Seyhan 1999).

3.2 Constitutive models

To characterize the behavior of unbound materials, constitutive models have been developed that predict the M_r and to describe the non-linear behavior of unbound pavement materials (Kaneza et al. 2020). Two- and three-parameter models have been developed based on regression analyses of laboratory test data. The two-parameter models include the deviatoric stress model and the bulk stress model, also known as k- θ model. The deviatoric stress model, depicted in Equation (2), is primarily used for cohesive soils, although it does not account for the effects of confining pressure on the material behavior (Kumar et al 2024).

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

Where k_1 and k_2 are deviatoric stress model constants derived from regression analysis, p_a is atmospheric pressure, and σ_d is the deviatoric stress.

The bulk stress model, also known as k- θ model, is presented in Equation (3). This model is primarily used for granular soils (Kumar et al. 2024a). When obtaining the resilient modulus of the material, the bulk stress model does not

consider the shear stresses or shear strains that are developed due to the RLT testing (Kumar et al. 2024b) but does provide a more realistic simulation of loading conditions (Plati et al. 2025).

$$M_r = k_3 p_a \left(\frac{\theta}{p_a} \right)^{k_4} \quad (3)$$

Where k_3 and k_4 are bulk stress model constants derived from regression analysis, p_a is atmospheric pressure, and θ is the bulk stress, defined as $\sigma_1 + \sigma_2 + \sigma_3$.

To overcome the constraints of the two-parameter models, various three-parameter constitutive models have been developed. A bilinear model, presented in equations (4.1) and (4.2), was proposed in 1979 by Thompson and Robnett that estimates the resilient modulus of cohesive soils based upon the applied deviator stress (Puppala et al., 2023). They determined that the resilient modulus of fine-grained soils is not dependent on the confining pressure. For the bilinear relationship, the point where the two lines intersect is referred to as the “break point”, and the M_r does not vary significantly after the break point is exceeded (Heydinger 2003).

$$M_r = k_6 + k_7(k_5 - \sigma_d); k_5 > \sigma_d \quad (4.1)$$

$$M_r = k_6 + k_8(k_5 - \sigma_d); < \sigma_d \quad (4.2)$$

Where k_6 - k_8 are the bilinear model constants derived from regression analysis, and σ_d is the deviatoric stress.

Furthermore, the bulk stress model was modified by Uzan as presented in Equation (5). For the Uzan model, the resilient modulus is a function of both the deviator stress and bulk stress.

$$M_r = K_9 p_a \left(\frac{\theta}{p_a} \right)^{K_{10}} \left(\frac{\sigma_d}{p_a} \right)^{K_{11}} \quad (5)$$

Where k_9 - k_{11} are the regression constants for the Uzan model, p_a is atmospheric pressure, θ is the bulk stress, and σ_d is the deviatoric stress

The Uzan model was further modified by Witczak and Uzan by replacing the deviator stress with the octahedral shear stress, which allows for a more accurate approximation of the actual stress state of pavement unbound materials. This model, depicted in Equation (6), represents a three-dimensional variation of the original Uzan model, and it is often referred to as the universal octahedral shear stress model (Plati et al., 2025).

$$M_r = k_{12} p_a \left(\frac{\theta}{p_a} \right)^{k_{13}} \left(\frac{\tau_{oct}}{p_a} \right)^{k_{14}} \quad (6)$$

Where k_{12} - k_{14} are the universal model regression constants, p_a is atmospheric pressure, θ is the bulk stress, and τ_{oct} is the octahedral shear stress, as defined in Equation (7).

$$\tau_{oct} = \frac{1}{3} \sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2} \quad (7)$$

The most recognized three-parameter model is the improved Witczak-Uzan model proposed through Mechanistic-Empirical Pavement Design Guide (MEPDG), known as the MEPDG model, presented in Equation (8) (Kaneza et al. 2020).

$$M_R = k_{15} p_a \left(\frac{\theta}{p_a} \right)^{k_{16}} \left(\frac{\tau_{oct}}{p_a} + 1 \right)^{k_{17}} \quad (8)$$

Where k_{15} - k_{17} are the MEPDG model regression constants, p_a is atmospheric pressure, θ is the bulk stress, and τ_{oct} is the octahedral shear stress as defined in Equation (7).

4 RESULTS

The objective of this paper is to provide an increased understanding of the nonlinear behavior of unbound materials to support improved mechanistic analyses of pavement structures. To this end, a total of 13 case studies were reviewed and summarized in the following sections. These case studies reported RLT test results for a variety of unbound pavement materials. Model parameters were generally separated based on whether the test conditions represented isotropic or anisotropic conditions. For each test condition assumption, isotropic or anisotropic, the resultant model parameters were further subdivided based upon the model reported (deviator, bulk stress, bilinear, Uzan, universal or MEPDG).

4.1 Isotropic

Table 1 summarizes the isotropic model constants for the deviatoric stress model for different fine-grained soils labeled in accordance with the Unified Soils Classification System (USCS). The maximum dry density (MDD), optimum moisture content (OMC) and associated confining pressures are also included in Table 1 for each soil type. Table 2 presents the isotropic model constants for the $k-\theta$ (bulk stress) model for different material types. Table 3 presents the isotropic model constants for the bilinear model for different soil types, while Tables 4 and 5 present the isotropic model constants for the Uzan and MEPDG models, respectively. These data can be used to estimate appropriate model constants (k_1-k_{17}) for the different nonlinear models that may be used to characterize the stress dependent behavior of unbound materials used in pavements.

The assumption of isotropy asserts that the material will behave the same when loaded in either the vertical or horizontal direction. This simplified assumption allows the use of the traditional RLT or M_r to estimate the stress dependent behavior of unbound materials based upon the applied vertical loading. Many modern mechanistic pavement design and evaluation software tools include the ability to model the nonlinear behavior of unbound materials, and Tables 1-5 can be used to estimate the model parameters for different soil types when direct measurements from laboratory tests are not available.

Table 1. Isotropic Deviatoric Stress Model.

Deviatoric Stress Model											
Study	Soil Type	Study Description	MDD (kN/m ³)	OMC (%)	Confining Pressure (kPa)	k1	k2	R ²	Reference		
1	CL	Control-1	17.93	15.1	13.8 - 41.4	1528.0	-0.039	0.07	Puppala et al., 2023		
		Control-2A-7	17.89	15.0	13.8 - 41.4	1461.2	-0.083	0.24			
		Control-2A-28	17.88	15.4	13.8 - 41.4	1559.7	-0.06	0.14			
	CH	Control-1	16.87	16.5	13.8 - 41.4	2613.1	0.098	0.30			
		Control-2A-7	16.89	16.8	13.8 - 41.4	2534.3	0.083	0.22			
		Control-2A-28	16.83	16.4	13.8 - 41.4	2626.9	0.071	0.19			
	2	ML	Control-1	17.42	14.6	13.8 - 41.4	474.4	-0.19		0.37	Puppala et al., 2021
			Control-2A-7	17.39	14.8	13.8 - 41.4	482.8	-0.18		0.32	
			Control-2B	17.47	14.4	13.8 - 41.4	3973.9	0.18		0.61	
		CL	Control-1	18.22	13.1	13.8 - 41.4	3462.4	0.13		0.60	
			Control-2A-7	18.26	13.3	13.8 - 41.4	3289.2	0.1		0.55	
			Control-2A-28	18.24	12.8	13.8 - 41.4	3466.4	0.12		0.50	
CH	Control-2B	18.3	13.2	13.8 - 41.4	4317.6	0.16	0.62				
	Control-1	16.2	19.6	13.8 - 41.4	2049.8	0.08	0.61				
	Control-2A-7	16.17	19.5	13.8 - 41.4	1958.4	0.07	0.35				
9	ML	Mod. Proc.	17.52	14.5	13.8 - 41.4	474.0	20.193	0.37	Kumar et al., 2024		

Note: MDD = maximum dry density; OMC = optimum moisture content; CL = low plasticity clay; CH = high plasticity clay; ML = low plasticity silt.

Table 2. Isotropic K-Theta Model.

K-θ Model											
Study	Soil Type	Study Description	MDD (kN/m ³)	OMC (%)	Confining Pressure (kPa)	k3	k4	R ²	Reference		
1	SC	Control-1	19.92	10.6	13.8 - 41.4	1662.9	0.146	0.28	Puppala et al., 2023		
		Control-2A-7	19.97	10.8	13.8 - 41.4	1726.4	0.143	0.21			
		Control-2B	19.89	10.7	13.8 - 41.4	3976.6	0.366	0.86			
	GP-GM	Control-1	20.00	8.1	13.8 - 41.4	702.1	0.78	0.96			
		Control-2A-7	20.08	7.9	13.8 - 41.4	771.7	0.79	0.96			
		Control-2B	20.01	8.0	13.8 - 41.4	3623.0	0.46	0.95			
2	SP	Control-1	17.00	9.8	13.8 - 41.4	534.3	0.69	0.95	Puppala et al., 2021		
		Control-2A-7	17.01	9.9	13.8 - 41.4	595.4	0.71	0.92			
		Control-2B	16.98	9.6	13.8 - 41.4	2858.6	0.38	0.97			
	SM	Control-1	17.53	12.1	13.8 - 41.4	453.9	0.68	0.89			
		Control-2A-7	17.50	12.0	13.8 - 41.4	468.4	0.66	0.88			
		Control-2B	17.58	12.2	13.8 - 41.4	3868.4	0.39	0.86			
	ML	Control-1	17.42	14.6	13.8 - 41.4	563.3	0.19	0.07			
		Control-2A-7	17.39	14.8	13.8 - 41.4	565.2	0.23	0.11			
		Control-2B	17.47	14.4	13.8 - 41.4	3141.0	0.34	0.92			
	3	Quarry: Crushed Stone Materials	Arrowood	24.11	4.2	34.5	3500.0	0.586		1.00	Chow et al., 2014
			Belgrade	20.63	7.4	34.5	1950.0	0.707		0.99	
			Fountain	22.18	6.1	34.5	990.0	0.756		1.00	
Franklin			23.80	4.7	34.5	1500.0	0.734	1.00			
Goldhill			22.34	6.4	34.5	2080.0	0.648	1.00			
Hendersonsville			21.88	5.5	34.5	1590.0	0.701	1.00			
Jamestown			22.24	5.8	34.5	2410.0	0.625	1.00			
Lemon Spring			22.13	5.5	34.5	2230.0	0.637	1.00			
Moncure			23.28	5.2	34.5	2470.0	0.632	0.99			
Nash County			22.35	5.7	34.5	604.0	0.994	0.99			
N. Wilkesboro			22.39	5.0	34.5	2900.0	0.587	1.00			
Princeton			22.20	5.1	34.5	1480.0	0.737	0.99			
Raleigh			21.93	6.1	34.5	2500.0	0.628	0.99			
Rockingham			22.21	6.1	34.5	2000.0	0.659	1.00			
Rocky Point			21.16	5.9	34.5	1940.0	0.73	0.99			
Rougemont	22.64	6.1	34.5	1180.0	0.725	0.99					
8	SM	Control 1	17.50	13.5	NR	450.2	0.66	0.86	Kumar et al., 2022		
		Control 2	17.50	13.5	NR	481.7	0.65	0.85			
9	ML	MCE	17.52	14.5	NR	563.0	0.197	0.37	Kumar et al., 2024		
11	GW	Sample 1	21.19	4.0	20.7 - 137.9	2465.1	0.713	0.80	Plati et al., 2025		
		Sample 2	21.19	4.0	20.7 - 137.9	779.6	0.889	0.83			
		Sample 1	21.74	5.0	20.7 - 137.9	1590.8	0.775	0.88			
		Sample 2	21.74	5.0	20.7 - 137.9	1269.1	0.796	0.78			
		Sample 1	22.09	6.0	20.7 - 137.9	3873.4	0.646	0.84			
		Sample 2	22.09	6.0	20.7 - 137.9	6996.2	0.558	0.79			
		Sample 1	21.93	7.0	20.7 - 137.9	746.3	0.904	0.88			
		Sample 2	21.93	7.0	20.7 - 137.9	741.2	0.904	0.84			
		Sample 1	21.62	8.0	20.7 - 137.9	1941.8	0.727	0.83			
		Sample 2	21.62	8.0	20.7 - 137.9	950.7	0.858	0.83			

Note: MDD = maximum dry density; OMC = optimum moisture content; SC = sandy clay; GP-GM = poorly graded gravel and silty gravel; SP = poorly graded sand; SM = silty sand; ML = low plasticity silt; GW = well graded gravel; MCE = modified compaction effort.

Table 3. Isotropic Bilinear Model.

Bilinear Model														
Study	Soil Type	Study Description	MDD (kN/m ³)	OMC (%)	Confining Pressure (kPa)	k5	k6	k7	k8	R ²	Reference			
1	CH	Control-1	17.03	16.5	13.8 - 41.4	13.8	26.8	208.7	1.24	0.44	1.00	Puppala et al., 2023		
						27.6	29.4	253.0	2.27	0.08	1.00			
						41.4	29.1	264.5	2.30	0.16	1.00			
		Control-2A-7	18.03	16.8	13.8 - 41.4	13.8	29.0	205.8	1.24	0.39	1.00			
						27.6	28.9	246.4	1.84	0.12	1.00			
						41.4	27.9	260.7	1.82	0.12	1.00			
	Control-2A-28	19.03	16.4	13.8 - 41.4	13.8	25.4	213.9	0.85	0.42	1.00				
					27.6	30.1	258.4	1.45	0.09	1.00				
					41.4	28.5	270.1	1.56	0.20	1.00				
	2	CL	Control-1	20.03	13.1	13.8 - 41.4	13.8	31.8	277.5	1.70	0.79		1.00	Puppala et al., 2021
							27.6	31.0	313.6	2.24	0.63		1.00	
							41.4	31.1	326.9	2.73	0.69		1.00	
Control-2A-7			21.03	13.3	13.8 - 41.4	13.8	30.1	277.8	1.53	0.57	1.00			
						27.6	27.5	305.2	2.11	0.48	1.00			
						41.4	28.4	314.0	2.03	0.51	1.00			
Control-2A-28		22.03	12.8	13.8 - 41.4	13.8	33.5	281.6	1.70	0.66	1.00				
					27.6	31.6	320.2	2.39	0.50	1.00				
					41.4	31.6	334.0	2.27	0.61	1.00				
CH		Control-2B	23.03	13.2	13.8 - 41.4	13.8	31.4	332.9	2.38	1.09	1.00			
						27.6	29.5	375.5	3.67	1.01	1.00			
						41.4	31.0	401.8	3.76	1.07	1.00			
	Control-1	24.03	19.6	13.8 - 41.4	13.8	32.1	179.8	0.76	0.32	1.00				
					27.6	33.1	195.2	0.73	0.28	1.00				
					41.4	34.3	195.8	0.70	0.35	1.00				
Control-2A-7	25.03	19.5	13.8 - 41.4	13.8	26.9	168.2	0.58	0.29	1.00					
				27.6	28.8	189.2	0.62	0.19	1.00					
				41.4	29.9	191.3	0.77	0.35	1.00					
Control-2A-28	26.03	19.4	13.8 - 41.4	13.8	25.1	178.1	0.62	0.36	1.00					
				27.6	29.9	205.3	0.97	0.23	1.00					
				41.4	28.9	211.2	1.14	0.35	1.00					

Note: MDD = maximum dry density; OMC = optimum moisture content; CL = low plasticity clay; CH = high plasticity clay.

Table 4. Isotropic Uzan Model.

Uzan Model										
Study	Soil Type	Study Description	MDD (kN/m ³)	OMC (%)	Confining Pressure (kPa)	k9	k10	k11	R ²	Reference
11	GW	Sample 1	21.19	4.0	20.7 - 137.9	4039.6	0.164	0.572	0.99	Plati et al., 2025
		Sample 2	21.19	4.0	20.7 - 137.9	1445.5	0.306	0.591	0.99	
		Sample 1	21.74	5.0	20.7 - 137.9	2374.3	0.342	0.452	0.99	
		Sample 2	21.74	5.0	20.7 - 137.9	2296.1	0.175	0.643	0.99	
		Sample 1	22.09	6.0	20.7 - 137.9	5644.2	0.218	0.451	0.98	
		Sample 2	22.09	6.0	20.7 - 137.9	9885.9	0.112	0.479	0.98	
		Sample 1	21.93	7.0	20.7 - 137.9	1210.0	0.437	0.476	0.98	
		Sample 2	21.93	7.0	20.7 - 137.9	1368.6	0.325	0.587	0.99	
		Sample 1	21.62	8.0	20.7 - 137.9	2996.1	0.240	0.510	0.99	
		Sample 2	21.62	8.0	20.7 - 137.9	1792.4	0.268	0.597	0.99	

Note: MDD = maximum dry density; OMC = optimum moisture content; GW = well graded gravel.

Table 5. Isotropic MEPDG Model.

Study	Soil Type	Study Description	MEPDG Model						Reference			
			MDD (kN/m ³)	OMC (%)	Confining Pressure (kPa)			R ²				
			k15	k16	k17							
1	SC	Control-1	19.92	10.6	13.8	-4.4	1965.8	0.264	-1.082	0.88	Puppah et al., 2023	
		Control-2A-7	19.97	10.8	13.8	-4.4	2134.1	0.294	-1.376	0.97		
		Control-2B	19.89	10.7	13.8	-4.4	3589.3	0.293	0.65	0.97		
		Control-1	17.92	15.1	13.8	-4.4	1780.6	0.262	-0.889	0.96		
		Control-2A-7	17.89	15.0	13.8	-4.4	1896.7	0.27	-1.257	0.96		
	CL	Control-2A-28	17.88	15.4	13.8	-4.4	1911.6	0.272	-1.069	0.96		
		Control-1	16.87	16.5	13.8	-4.4	2235.1	0.272	0.068	0.89		
		Control-2A-7	16.89	16.8	13.8	-4.4	2249.7	0.299	-0.087	0.93		
		Control-2A-28	16.82	16.4	13.8	-4.4	2380.9	0.276	-0.11	0.93		
		Control-1	20.00	8.1	13.8	-4.4	843.0	0.91	-1.17	0.99		
GP-GM	Control-2A-7	20.08	7.9	13.8	-4.4	928.3	0.92	-1.18	0.99			
	Control-2B	20.01	8.0	13.8	-4.4	3277.3	0.39	0.64	0.97			
SP	Control-1	17.00	9.8	13.8	-4.4	632.7	0.81	-1.08	0.99			
	Control-2A-7	17.01	9.9	13.8	-4.4	694.6	0.82	-0.98	0.99			
2	SM	Control-2B	16.98	9.6	13.8	-4.4	2704.2	0.34	0.35	0.98	Puppah et al., 2021	
		Control-1	17.53	12.1	13.8	-4.4	546.3	0.81	-1.18	0.98		
		Control-2A-7	17.50	12.0	13.8	-4.4	566.0	0.8	-1.21	0.98		
		Control-2B	17.58	12.2	13.8	-4.4	3465.4	0.31	0.7	0.97		
		Control-1	17.42	14.6	13.8	-4.4	829.8	0.47	-2.47	0.95		
	ML	Control-2A-7	17.39	14.8	13.8	-4.4	830.7	0.5	-2.46	0.94		
		Control-2B	17.47	14.4	13.8	-4.4	2821.6	0.27	0.68	0.96		
		Control-1	18.22	13.1	13.8	-4.4	2678.9	0.2	0.5	0.93		
		Control-2A-7	18.25	13.3	13.8	-4.4	2730.4	0.17	0.33	0.93		
		Control-2A-28	18.24	12.8	13.8	-4.4	2768.4	0.23	0.36	0.95		
3	Crushed Stone Materials	Control-2B	18.20	13.2	13.8	-4.4	3167.5	0.23	0.62	0.94	Chow et al., 2014	
		Control-1	16.20	19.6	13.8	-4.4	1740.6	0.12	0.34	0.93		
		Control-2A-7	16.16	19.5	13.8	-4.4	1741.7	0.17	0.12	0.91		
		Control-2A-28	16.15	19.4	13.8	-4.4	1876.9	0.2	0.12	0.93		
		Arrowood	24.11	4.2	34.5		1160.0	0.611	-0.027	1.00		
	Quarry:	Belgrade	20.63	7.4	34.5		857.0	0.611	0.105	1.00		
		Fountain	22.18	6.1	34.5		511.0	0.74	0.018	1.00		
		Franklin	23.80	4.7	34.5		723.0	0.672	0.068	1.00		
		Gokhill	22.34	6.4	34.5		814.0	0.652	-0.004	1.00		
		Hendersonville	21.88	5.5	34.5		716.0	0.711	-0.011	1.00		
5	Granite Agg.	Jamestown	22.24	5.8	34.5		875.0	0.597	0.031	1.00	Fares et al., 2025	
		Lemon Spring	22.13	5.5	34.5		829.0	0.589	0.053	1.00		
		Monroe	23.28	5.2	34.5		888.0	0.542	0.091	1.00		
		Nash County	22.35	5.7	34.5		610.0	1.018	-0.027	0.99		
		N. Wilkesboro	22.39	5.0	34.5		968.0	0.609	-0.024	1.00		
	Dolomite Agg.	Princeton	22.20	5.1	34.5		725.0	0.681	0.062	0.99		
		Raleigh	21.93	6.1	34.5		952.0	0.713	-0.093	1.00		
		Rockingham	22.21	6.1	34.5		820.0	0.721	-0.067	1.00		
		Rocky Point	21.16	5.9	34.5		941.0	0.703	0.029	0.99		
		Rougemont	22.64	6.1	34.5		571.0	0.73	-0.006	0.99		
6	Quartzite Agg.	QBA	21.38	5.2	NR		6.4	0.62	-0.47	0.93	Kaneza et al., 2020	
		Granite Agg.	GBA	21.65	6.7	NR		6.5	0.61	-0.38		0.95
		Limestone Agg.	LBA	22.45	6.9	NR		6.6	0.62	-0.28		0.92
		Dolomite Agg.	DBA	22.20	7.1	NR		6.7	0.59	-0.22		0.90
		CH	Dallas Soil	13.59	33.0	13.8	-4.4	469.1	0.1278	-0.9712		0.92
	7	CL	Carrollton Soil	15.14	24.0	13.8	-4.4	285.4	-0.0204	3.654		0.97
			SCE	16.31	19.0	13.8	-4.4	1010.0	0.1	-1.05		0.96
	8	SM	MCE	18.30	13.0	13.8	-4.4	2679.0	0.2	0.5		0.93
			Control 1	17.50	13.5	13.8	-4.4	544.9	0.8	21.22		0.98
	9	ML	Control 2	17.50	13.5	13.8	-4.4	587.7	0.79	21.27		0.99
MCE			17.52	14.5	13.8	-4.4	830.0	0.466	22.473	0.95		
10	SP	MCE	17.03	10.0	13.8	-4.4	632.7	0.811	-1.079	0.99		
		MCE	17.56	12.0	13.8	-4.4	553.7	0.805	-1.210	0.98		
11	GW	Sample 1	21.19	4.0	20.7	-137.9	621.9	0.298	1.23	0.95	Plati et al., 2025	
		Sample 2	21.19	4.0	20.7	-137.9	464.7	0.417	1.29	0.97		
		Sample 1	21.74	5.0	20.7	-137.9	546.7	0.434	0.996	0.97		
		Sample 2	21.74	5.0	20.7	-137.9	477.1	0.297	1.429	0.96		
		Sample 1	22.09	6.0	20.7	-137.9	716.5	0.326	0.988	0.96		
		Sample 2	22.09	6.0	20.7	-137.9	839.8	0.223	1.069	0.95		
		Sample 1	21.93	7.0	20.7	-137.9	476.3	0.534	1.024	0.97		
		Sample 2	21.93	7.0	20.7	-137.9	472.4	0.444	1.265	0.96		
		Sample 1	21.62	8.0	20.7	-137.9	524.9	0.351	1.118	0.96		
		Sample 2	21.62	8.0	20.7	-137.9	489.2	0.387	1.306	0.96		

Note: MDD = maximum dry density; OMC = optimum moisture content; Agg. = aggregate; SC = sandy clay; CL = low plasticity clay; CH = high plasticity clay; GP-GM = poorly graded gravel and silt/gravel; SP = poorly graded sand; SM = silty sand; ML = low plasticity silt; GW = well graded gravel; SCE = standard compaction effort; MCE = modified compaction effort.

4.2 Anisotropic

Since it is generally recognized that the unbound materials are not isotropic, it may be useful to employ anisotropic models to improve the characterization of these materials. However, there is a significant added level of complexity, and few laboratories possess the sophisticated equipment necessary to load specimens in both the vertical and horizontal directions. Furthermore, the available laboratory apparatus can utilize either cylindrical or prismatic specimens. Tables 6-8 present model constants for anisotropic testing for the bulk stress, Uzan, and MEPDG models, respectively. Study 12 includes prismatic specimen testing therefore, horizontal x and y components were averaged for this summary to provide a reasonable comparison to cylindrical specimens utilized in the Studies 4, 5, and 13.

Table 6. Anisotropic k-Theta Model.

Study	Soil Type	MDD (kN/m ³)	OMC (%)	k-Theta Model						Reference	
				Horizontal			Vertical				
				k3-h	k4-h	R ²	k3-v	k4-v	R ²		
12	MA1	21.7	7.5	17.68	0.43	0.65	0.96	0.63	0.97	0.17	Cetin, 2025
		MA2	22.3	7	11.52	0.63	0.70	0.70	0.62	0.96	
	#610 Limestone	23.1	4.1	9.64	0.67	0.76	1.89	0.35	0.72	0.06	
	Gravel Clay	21.9	5.8	25.09	0.25	0.46	20.22	0.35	0.91	0.09	
	Quartzite Agg.	21.4	5.2	10.57	0.57	0.70	114.8	0.46	0.95	0.13	
	Granite Agg.	21.7	6.7	16.73	0.39	0.62	116.7	0.53	0.96	0.17	
	Dolomite Agg.	22.2	7.1	14.63	0.39	0.49	125.7	0.58	0.98	0.13	
	Limestone Agg.	22.5	6.9	16.66	0.38	0.48	90.1	0.72	0.95	0.20	
	CL	18.2	13.5	17.22	0.19	0.06	129.5	0.29	0.39	0.13	
	CH	17.4	16.1	12.45	0.13	0.04	180.0	0.00	0.05		

Note: MDD = maximum dry density; OMC = optimum moisture content; MA = Michigan based aggregate; Agg. = aggregate; CL = low plasticity clay; CH = high plasticity clay.

Table 7. Anisotropic Uzan Model.

Study	Soil Type	MDD (kN/m ³)	OMC (%)	Uzan Model				Reference						
				Horizontal		Vertical								
				k9-h	k10-h	k11-h	R ²	k9-v	k10-v	k11-v	R ²	Modulus Ratio (Mrh/Mrv)		
4	Crushed Stone (5% Fines)	21.71	9.2	235.5	1.314	-0.56	NR	1963	0.827	-0.209	NR	0.44	Tutunbner et al., 2016	
		IL-DOT (10% Fines)	22.13	6.2	113.2	1.604	-0.72	NR	2023	0.979	-0.21	NR		0.47
		IL-DOT (2% Fines)	22.42	6.8	1067.4	0.996	-0.41	NR	1034	0.861	-0.26	NR		0.72
		IL-DOT (5% Fines)	22.56	7.3	716.7	1.179	-0.53	NR	2816	0.782	-0.24	NR		0.69
		IL-DOT (7% Fines)	22.86	6.3	692.8	1.066	-0.47	NR	2412	0.831	-0.238	NR		0.5
		IL-DOT (8% Fines)	22.95	6.1	754.8	1.083	-0.43	NR	1164	0.996	-0.233	NR		0.7
		IL-DOT (12% Fines)	23.28	5.7	754.6	1.264	-0.71	NR	880	1.067	-0.269	NR		0.96
		IL-DOT (16% Fines)	23.50	6.2	436.8	1.521	-0.95	NR	596	1.135	-0.290	NR		0.95
		IL-DOT (20% Fines)	23.80	8.0	456.0	1.400	-0.77	NR	422	1.228	-0.417	NR		0.53
		VA-DOT	21.52	5.5	432.1	1.351	-0.63	NR	1495	0.877	-0.228	NR		0.9
		PA-DOT	22.46	5.5	334.9	1.223	-0.45	NR	1174	0.864	-0.220	NR		0.57
		PA-DOT	21.68	6.8	648.7	0.970	-0.62	NR	674	0.965	-0.235	NR		0.44
		PA-DOT	20.74	8.6	60.5	1.790	-0.85	NR	672	0.982	-0.160	NR		0.48
		OR-DOT	22.07	7.6	205.1	1.370	-0.59	NR	912	0.928	-0.173	NR		0.54
		NN-DOT (Fountain Quarry)	22.35	6.1	460.9	1.232	-0.56	NR	1700	0.916	-0.230	NR		0.51
		NN-DOT (Eik River)	21.57	6.5	808.8	1.139	-0.55	NR	921	0.952	-0.250	NR		1.1
		IN-DOT	20.34	11.0	962.8	1.130	-0.57	NR	2407	0.739	-0.163	NR		0.82
		IN-DOT	21.52	9.5	95.9	1.628	-0.66	NR	1084	1.014	-0.203	NR		0.42
		CA-DOT	22.31	7.5	351.0	1.521	-0.93	NR	935	0.933	-0.195	NR		0.66
		TX-DOT	21.21	6.2	316.4	1.740	-1.24	NR	369	1.372	-0.581	NR		0.88
		TX-DOT Sandy Gravel	19.89	8.5	711.8	1.270	-0.58	NR	1067	0.999	-0.310	NR		1.33
		MN-DOT-S (Fountain Quarry)	22.35	6.9	2806.5	1.553	-0.93	NR	360	1.238	-0.307	NR		0.62
		VA-DOT-S	22.46	7.3	785.2	1.065	-0.49	NR	1905	0.814	-0.192	NR		0.63
		PA-DOT-S	21.68	8.4	995.5	1.101	-0.23	NR	1992	0.41	-0.001	NR</		

For Study 12, two different Modulus Ratios were originally reported for the x and y directions of the prismatic specimens. For this paper, the two different Modulus Ratios were averaged to provide comparative information for the cylindrical specimens used in the other studies. For Study 13, the Modulus Ratio was computed as k_1/k_4 .

4.3 Interpretation of Stress-Hardening vs. Stress-Softening Responses Under Traffic Loading

The compiled model constants and laboratory trends across the 13 case studies illustrate two dominant behavioral mechanisms in unbound geomaterials: 1) stress hardening and 2) stress softening. Granular materials, particularly well-graded aggregates, GP-GM gravels, and crushed rock generally exhibited stress hardening behavior, where the resilient modulus increases with increasing bulk stress. This response reflects improved particle interlock and load distribution efficiency under higher confining pressures; conditions commonly induced under heavier traffic or deeper pavement layers. Under repeated wheel loads, these materials tend to stiffen as the stress state increases, which reduces permanent deformation and enhances load-carrying capacity.

Conversely, fine-grained soils such as CL, CH, and ML classifications consistently showed stress-softening behavior where the resilient modulus decreases with increasing deviator stress. This behavior indicates a progressive breakdown of soil structure and increased susceptibility to shear deformation under repeated loading. Under traffic conditions, stress-softening soils tend to accumulate plastic stress more rapidly, particularly in the upper subgrade layers where confining pressures are low, and the alternating stress path is more severe.

When considered in the context of traffic loading, these behaviors directly influence rutting performance and pavement life. Stress-hardening granular bases help mitigate load-induced strains by mobilizing additional stiffness as traffic stresses increase. In contrast, softening fine-grained subgrades require more conservative design inputs, stabilization, or increased layer thicknesses to prevent excessive deformation. The nonlinear model constants summarized in this study reinforce these mechanistic trends, demonstrating that accurate representation of stress-dependent behavior in pavement analysis tools is essential for predicting rutting, optimizing layer configurations, and evaluating long-term pavement performance.

Table 8. Anisotropic MEPDG Model.

Study	Soil Type	MDD (kN/m ³)	OMC (%)	MEPDG Model								Reference	
				Horizontal				Vertical					Modulus Ratio (Mrh/Mrv)
				k15-h	k16-h	k17-h	R ²	k15-v	k16-v	k17-v	R ²		
5	Quartzite Agg.	21.38	5.2	4.96	0.510	0	0.46	7.0	0.500	-0.13	0.96	0.13	Fares et al., 2025
	Granite Agg.	21.65	6.7	5.41	0.330	0	0.36	7.1	0.540	-0.03	0.96	0.17	
	Limestone Agg.	22.45	6.9	5.51	0.280	0	0.27	6.8	0.760	-0.11	0.95	0.20	
	Dolomite Agg.	22.20	7.1	5.18	0.480	0	0.48	7.1	0.610	-0.11	0.98	0.13	
12*	MA1	21.7	7.5	5.09	0.435	-0.04	0.65	6.9	0.750	-0.36	0.99	0.17	Cetin, 2025
	MA2	22.3	7.0	4.61	0.630	0	0.70	6.8	0.740	-0.36	0.98	0.13	
	#610 Limestone	23.1	4.1	4.55	0.665	0	0.76	7.6	0.430	-0.28	0.74	0.06	
	Gravel Clay	21.9	5.8	5.49	0.250	0	0.46	7.6	0.460	-0.40	0.97	0.09	
	CL	18.2	13.5	5.37	0.185	-1.44	0.30	7.4	0.460	-1.13	0.74	0.13	
	CH	17.4	16.1	5.025	0.155	-1.26	0.30	7.8	0.120	-1.73	0.88	0.05	
13	CD	NR	NR	679.8	1.127	-1.54	NR	1465.3	0.603	-0.43	NR	0.64	Martinez et al., 2025
	L	NR	NR	648.4	1.171	-1.82	NR	1357.2	0.653	-0.49	NR	0.46	
	L	NR	NR	496.7	1.329	-1.89	NR	1240.3	0.626	-0.38	NR	0.48	
	L	NR	NR	567.2	1.319	-1.9	NR	1158.1	0.638	-0.34	NR	0.40	
	CD	NR	NR	640.1	0.801	-0.78	NR	1026.2	0.781	-0.54	NR	0.49	
	Agg.	NR	NR	436.4	1.064	-1.35	NR	939.5	0.704	-0.41	NR	0.62	
	Agg.	NR	NR	829	1.015	-1.69	NR	893.2	0.702	-0.41	NR	0.46	
	Agg.	NR	NR	397.1	1.075	-1.33	NR	874.9	0.693	-0.41	NR	0.93	
	Agg.	NR	NR	631.3	0.860	-1.09	NR	853.4	0.648	-0.32	NR	0.45	

Note: NR = not reported; MDD = maximum dry density; OMC = optimum moisture content; * = Average of X and Y horizontal components; CL = high plasticity clay; CH = high plasticity clay; MA = Michigan Aggregate; CD = Crushed Dolomite; L = Crushed Limestone; Agg. = aggregate.

4.4 Statistic Summary

A statistical summary of the reported model constants based upon the assumed condition (isotropic/anisotropic) and the soil type (e.g. fines, sand, aggregate) were performed for each nonlinear model. The statistic summary consists of the average parameter associated with the model type, standard deviation (std. dev.), the median parameter, and the range. The results of the statistic summaries are provided in Table 9 for the isotropic model constants and Table 10 for the anisotropic model constants. Note that Table 10 includes k-h and k-v; these parameters represent the associate k statistic in both the horizontal (k-h) and vertical (k-v) direction. These statistical summaries can be used to assess whether laboratory experimental results fall within the expected ranges for individual materials. To further assist with soil type selection, the average and standard deviation for MDD and OMC were included in the statistical summary. Since these characterization tests are relatively sophisticated and the model constants have little physical meaning to the average model user, the summary of the expected mean and range of values for individual soil types can be extremely helpful in assessing the relative reasonableness of laboratory results. In addition, the statistical range of the values of the model constants can be used by designers to perform sensitivity analyses based upon the high and low values that can provide insight into the behavior of unbound materials. This insight can be used to address the risk and uncertainty associated with the use of different materials.

Table 9. Isotropic Model Parameters Statistic Summary.

Model	Soil Type	MDD (kN/m ³) Mean ±	OMC (%) Mean ±	Parameter	Mean	Std. Dev.	Median	Range	
								Low	High
Deviatoric Stress Model	Fines	17.4 ± 0.8	15.6 ± 2.2	k1	2259.8	1194.1	2144.9	474	3973.9
				k2	1.22	4.89	0.08	-0.19	20.2
K-θ Model	Fines	17.5 ± 0.1	14.6 ± 0.2	R ²	0.38	0.18	0.35	0.07	0.62
				k3	1208.1	1288.6	564.3	563.0	3141.0
				k4	0.239	0.069	0.214	0.190	0.340
				R ²	0.37	0.39	0.24	0.07	0.92
	Sand	18.0 ± 1.2	11.3 ± 1.4	k3	1552.4	1403.0	595.4	450.2	3976.6
				k4	0.498	0.220	0.650	0.143	0.710
				R ²	0.78	0.27	0.86	0.21	0.97
				k3	1992.2	1323.7	1940.0	604.0	6996.2
Agg.	21.9 ± 1.0	6.0 ± 1.2	k4	0.720	0.117	0.725	0.460	0.994	
			R ²	0.93	0.08	0.99	0.78	1.00	
			k5	29.8	2.2	29.7	25.1	34.3	
			k6	258.6	62.8	259.6	168.2	401.8	
Bilinear Model	Fines	21.5 ± 2.9	16.1 ± 2.7	k7	1.67	0.85	1.63	0.58	3.76
				k8	0.445	0.279	0.375	0.080	1.090
				R ²	1.00	0.00	1.00	1.00	1.00
				k9	3305.3	2691.2	2335.2	1210.0	9885.9
Uzan Model	Agg.	21.7 ± 0.3	6.0 ± 1.5	k10	0.259	0.097	0.254	0.112	0.437
				k11	0.536	0.070	0.541	0.451	0.643
				R ²	0.99	0.00	0.99	0.98	0.99
				k15	1941.8	869.7	1938.7	285.4	3589.3
MEPDG Model	Fines	17.4 ± 1.3	17.0 ± 4.3	k16	0.247	0.117	0.263	-0.020	0.500
				k17	0.73	4.79	0.09	-2.47	22.47
				R ²	0.94	0.02	0.94	0.88	0.97
	Sand	17.3 ± 0.3	11.5 ± 1.5	k15	1092.8	1066.1	610.2	544.9	3465.4
				k16	0.710	0.203	0.803	0.310	0.820
				k17	4.89	10.12	-0.32	-1.21	21.27
Agg.	21.9 ± 0.9	6.1 ± 1.2	k15	717.0	546.6	716.5	6.4	3277.3	
			k16	0.584	0.186	0.610	0.223	1.018	
			k17	0.268	0.691	0.031	-1.180	1.429	
			R ²	0.98	0.03	0.99	0.90	1.00	

Note: MDD = maximum dry density; OMC = optimum moisture content; Agg. = aggregate.

Table 10. Anisotropic Model Parameters Statistic Summary.

Model	Soil Type	MDD (kN/m ³) Mean ±	OMC (%) Mean ±	Paramet er	Range				
					Mean	Std. Dev.	Median	Low	High
K-θ Model	Agg.	22.1 ± 0.5	6.3 ± 1.2	k3-h	15.31	4.99	15.64	9.64	25.09
				k4-h	0.461	0.144	0.408	0.250	0.665
				R ²	0.605	0.115	0.633	0.460	0.760
				k3-v	127.8	44.3	115.8	87.0	202.2
				k4-v	0.530	0.134	0.555	0.350	0.720
	Fines	17.8 ± 0.6	14.8 ± 1.8	R ²	0.93	0.09	0.96	0.72	0.98
				Mth/Mrv	0.14	0.04	0.13	0.06	0.20
				k3-h	14.83	3.38	14.83	12.45	17.22
				k4-h	0.155	0.042	0.155	0.125	0.185
				R ²	0.045	0.014	0.045	0.035	0.055
Uzan Model	Agg.	21.7 ± 0.8	7.7 ± 1.6	k3-v	154.7	35.7	154.7	129.5	180.0
				k4-v	0.145	0.205	0.145	0.000	0.290
				R ²	0.20	0.28	0.20	0.00	0.39
				Mth/Mrv	0.09	0.05	0.09	0.05	0.13
				k9-h	564.7	352.2	480.6	13.7	2067.0
	Fines	17.8 ± 0.6	14.8 ± 1.8	k10-h	1.413	0.250	1.425	0.111	1.946
				k11-h	-0.757	0.219	-0.770	-1.237	0.220
				R ²	NR	NR	NR	NR	NR
				k9-v	2478.7	1471.7	2341.2	359.9	10422.7
				k10-v	0.785	0.164	0.739	0.410	1.372
MEPDG Model	Agg.	22.1 ± 0.6	6.3 ± 1.2	k11-v	-0.174	0.072	-0.169	-0.581	0.106
				R ²	NR	NR	NR	NR	NR
				Mth/Mrv	0.72	0.19	0.74	0.08	1.33
				k15-h	5.20	0.24	5.20	5.03	5.37
				k16-h	0.170	0.021	0.170	0.155	0.185
	Fines	17.8 ± 0.6	14.8 ± 1.8	k17-h	-1.348	0.131	-1.348	-1.440	-1.255
				R ²	0.30	0.00	0.30	0.30	0.30
				k15-v	7.58	0.33	7.58	7.35	7.81
				k16-v	0.29	0.24	0.29	0.12	0.46
				k17-v	-1.430	0.424	-1.430	-1.130	-1.730
Agg.	22.1 ± 0.6	6.3 ± 1.2	R ²	0.81	0.10	0.81	0.74	0.88	
			Mth/Mrv	0.090	0.050	0.090	0.054	0.125	
			k15-h	315.7	316.3	397.1	4.6	829.0	
			k16-h	0.785	0.366	0.801	0.250	1.329	
			k17-h	-0.79	0.81	-0.78	-1.90	0.00	
			R ²	0.52	0.17	0.47	0.27	0.76	
			k15-v	580.3	579.4	853.4	6.8	1465.3	
			k16-v	0.638	0.105	0.648	0.430	0.781	
			k17-v	-0.324	0.145	-0.360	-0.537	-0.030	
			R ²	0.94	0.08	0.97	0.74	0.99	
Mth/Mrv	0.35	0.24	0.40	0.06	0.93				

Note: MDD = maximum dry density; OMC = optimum moisture content; Agg. = aggregate

5 CONCLUSIONS

This paper provides a synthesis of nonlinear resilient modulus model constants reported for unbound materials under both isotropic and anisotropic conditions, along with a statistical summary to assist pavement engineers in evaluating parameter validity and material sensitivity. The compiled data confirm that the nonlinear behavior of unbound materials is highly dependent on soil type. Fine-grained soils typically exhibit stress-softening behavior and limited dependence on confining stress, whereas granular materials demonstrate pronounced stress-hardening controlled by bulk stress. These distinctions highlight the importance of selecting material models that accurately reflect the governing stress mechanisms when conducting mechanistic pavement analyses.

The synthesized findings also support practical model-selection guidelines for engineering applications. Stress-softening fine-grained soils are best represented by deviator stress or bilinear models, while stress-hardening granular bases and aggregates are more accurately captured using k-θ or Uzan formulations. For moisture-sensitive, stabilized, or mechanically complex geomaterials, the universal or MEPDG models provide improved predictions due to their incorporation of both mean and shear stress effects. In climates susceptible to seasonal saturation or freeze-thaw cycles, models that account for shear-induced stiffness changes (Uzan or MEPDG) are recommended. Where directional loading or material anisotropy is important, such as in heavy-duty pavements or high-confidence design applications, advanced anisotropic models should be used to improve the reliability of performance predictions.

Overall, the accurate characterization and appropriate selection of nonlinear constitutive models enable more realistic simulation of unbound material behavior under traffic loading, supporting improved mechanistic pavement performance

predictions and the potential for less conservative, more efficient pavement designs.

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