



## Evaluation of Equations for Estimating Resilient Modulus of Fine-Grained Subgrade Soils in Nigeria

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### ABSTRACT

*This work present a resilient modulus constitutive equation adopted for fine-grained subgrade soils in Nigeria. Fine-grained subgrade materials from different locations in Nigeria were characterized for use in the Nigerian Empirical Mechanistic Pavement Analysis and Design. Three samples each from the MTSs making a total eighteen (18) samples were obtained and subjected to laboratory test to determine their basic physical properties. Testing include particle size distribution and Atterberg limits tests. The samples were classified according to American Association of State Highway and Transportation Officials (AASHTO) and the Unified Soil Classification System (USCS). The AASHTO soil classification shows that the subgrade soil samples obtained from the MTS 1 were either clayey soil (A-6 and A-7-6) or silty soils (A-4, A-5 and A-2-4). The AASHTO soil classification generally showed that the subgrade samples were "Fair to Poor" in subgrade properties for use as construction materials. The USCS soil classification indicated that most of the samples were lean clay soil with gravel (CL) except few that were either clayey gravel (GC) or clayey sand (SC). This showed that the subgrade samples were mostly clay soil. Resilient modulus constitutive equation for estimating the resilient modulus of Nigerian subgrade soils was adopted through evaluation of existing resilient modulus constitutive equations using the repeated load triaxial test result conducted on Nigerian subgrade soils. From the evaluation, the National Cooperative Highway Research Program resilient modulus constitutive equation was adopted for purposes of predicting resilient modulus of Nigerian subgrade soils.*

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### INTRODUCTION

Pavements are structures used for the purpose of carrying vehicular traffic safely and economically (Adu-Osei, 2001). A conventional flexible pavement consists of a prepared subgrade or foundation and layers of sub-base, base and surface courses (AASHTO, 1993). The pavement foundation (subgrade) represents one of the key elements in the pavement design;

its behavior will influence the overall pavement performance (Ibrahim, 2013).

With the increase in vehicular traffic (both weight and number of vehicles), more and more attention has been directed toward fundamental concepts (Southgate, et al., 1976).

Pavements fail for different reasons; poor design, poor materials and poor construction methods are the most common (Ibrahim, 2013). The assessment



of properties of soil subgrades, in terms of density, soil stiffness, strength, and other in-situ parameters is vital in the design of roads, and their performance (Rao, et al., 2008). A pavement design procedure requires characterization of the component materials of the borrowed and the supporting soil (Hossain, 2010). Subgrade characterization allows for the design of a proper foundational support for the pavement. On the other hand, base/sub-base materials provide structural capacity to the pavement. Therefore, both subgrade and base/sub-base material characterization is needed to design adequate pavement structure for expected traffic (Hossain, 2008). One of the problems faced by the pavement engineer is inadequate characterization of material properties of existing service pavements (Viswanathan, 1989). The characterization of the underlying subgrade soil allows for the design of a proper foundational support for the pavement (Hossain, 2010).

The often-used soil strength parameters in pavement design practice are California Bearing Ratio (CBR) value, Hveem *R* value, and Soil Support Value (SSV) (Huang, 1993). The CBR test is taken as direct measure of the strength of the in-situ sub grade material (Abiola, et al., 2012). All these soil parameters are based on the failures of subgrade soil specimens in the laboratory conditions (Huang, 1993). Since flexible pavements do not fail as a result of soil strength failure, the 1993 AASHTO pavement design guide recommended the use of a soil parameter known as the resilient modulus to replace strength based parameters such as CBR and SSV (Brickman, 1989; Mohammad, et al., 1994; Maher, et al., 2000; Sheng, 2010).

Resilient modulus testing, a basis for the mechanistic approach, was later incorporated into the American Association of State Highway and Transportation Officials (AASHTO) design guide for subgrade soils characterization (Hossain, 2010). The resilient modulus is

analogous to the elastic modulus used in elastic theories and is defined as a ratio of deviatoric stress to resilient or elastic strain experienced by the material under repeated loading conditions that simulate traffic loading (Uzan, 2004). Subgrade soil characterization in terms of resilient modulus has become crucial for pavement design (George, 2004). The laboratory testing procedures for determining the resilient modulus values is time consuming and needs expensive equipment and highly trained personnel (Vogrig, et al., 2003). Some agencies consider the cost, time, complication, and sampling resolution required for meaningful resilient modulus testing to be too cumbersome for its application in less critical projects (Yau & Von Quintus, 2002).

With limited resources for performing laboratory and field tests to determine material properties, the need for a secondary means to obtain these material property values (i.e., through correlations or predictive models based on data from routine or less expensive tests) is obvious. Additionally, the AASHTO Mechanistic-Empirical Pavement Design Guide (MEPDG) offers users the option of using inputs obtained through correlations. Constitutive equation coefficients (*k*-values) are usually obtained from the regression analysis of resilient modulus test data for an actual soil/aggregate sample (Hossain, 2010). The proposed MEPDG procedure is an improved methodology for pavement design and evaluation of paving materials (Apeagyei & Diefenderfer, 2011). The MEPDG defines level 2 inputs as those obtained from correlations between the primary inputs (level 1 measured) and other parameters that are material-specific or are measured through simpler tests (Rao, et al., 2012). Correlation of the index properties of a soil to more mechanistic property such as resilient modulus has the potential to provide a reasonable and cost effective means of pavement design (Zeghal, 2001). The

resilient modulus can be determined from physical properties (Dione, et al., 2013).

Nigerian Empirical Mechanistic Pavement Analysis and Design System (NEMPADS) is a framework for mechanistic-empirical pavement design for tropical climate (Olowosulu, 2005; Olowosulu, et al., 2011). NEMPADS consists of two parts. Part 1 consists of the development of input values, which include traffic, climate and material. Geotechnical analysis is also performed in this part to determine the strength and stiffness of the sub-grade. Part 2 of the design process is structural response analysis. Each component of the design procedure is developed specifically for Nigeria environment. The material

characterization, load characterization and mechanistic analysis and behaviour functions developed for overlay design procedure were adopted for the new method (Claros, et al., 1986).

The soils tested in this study for the correlation of basic soil properties with the resilient modulus constitutive model were selected to provide a general representation of typical subgrade soils located in MTS 1 (Kano-Kaduna region) of Nigeria. There are six sites within MTS 1 namely MTS 1 – 1, MTS 1 – 2, MTS 1 – 3, MTS 1 – 4, MTS 1 – 5 and MTS 1 – 6 with route numbers A236, A11, A2, A235, A125 and A2 respectively (Claros *et al.*, 1986). Plate I shows the Master Test Section (MTS) 1.

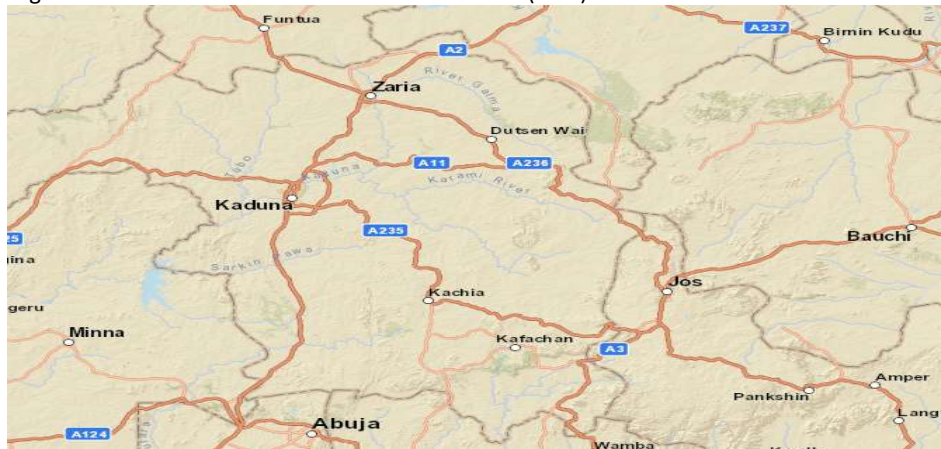


Plate I: Map Showing Roads in Master Test Section 1 of Nigeria  
Source: Google map

The MTS 1 consist of six sections as shown in Table 1. Three (3) samples each was obtained at 500mm depth from each of the six sections. A total of eighteen

(18) soil samples were obtained and tested in the laboratory to determine their basic soil index properties.

**Table 1:** Description of Master Test Section (MTS) 1

MTS	Route Number	Distance from First Node (km)	Node Number and Name
1 – 1	A236	35	241 Zaria (A126) to 394 A236/State Road
1 – 2	A11	4	239 Katabu to 353 Pambegua
1 – 3	A2	40	237 Kaduna (A235) to 711 Kaduna-Niger Border
1 – 4	A235	11	237 Kaduna (A235) to 584 Kachia
1 – 5	A125	11	238 Kaduna to 346 S.B. Gwari
1 – 6	A2	2	239 Katabu to 460 Zaria (A236)

Source: (Claros *et al.*, 1986)



### ATTERBERG LIMITS AND PARTICLE SIZE DISTRIBUTION OF THE SOIL SAMPLES

The soil index properties determined in the laboratory include Atterberg limits and soil gradation tests. Soil gradation was conducted on the soil samples in accordance with testing procedures (AASHTO T88, 2003; BS 1377, 1990; ASTM D 422, 1998). Atterberg limits were determined using the testing procedures (AASHTO T89, 2003; ASTM D 4318, 2000; BS 1377, 1990; AASHTO T90, 2003). The Liquid Limit (LL) was obtained by plotting the flow curve of the soil samples. The flow curve was made by plotting the individual moisture contents and the number of blows required to close the grooves, on a semi-logarithmic graph, for each of the points. A best-fit straight line was then drawn through the points. The moisture content corresponding to the intersection of the best-fit line with the

25-tap line was recorded as the LL. The plastic limit (PL) was determined by alternately pressing together and rolling into a 3.2-mm diameter thread a small portion of plastic soil until its water content was reduced to a point at which the thread crumbles and can no longer be pressed together and re-rolled. The water content of the soil at this point was reported as the PL. The plasticity index (PI) was calculated as the difference between the liquid limit and the plastic limit. The PL and PI are to be reported to the nearest whole number. The PI of the subgrade soil samples was calculated using Equation 1:

$$PI = LL - PL \quad (1)$$

The summary of Atterberg limits test and particle size distribution results of the soil samples are as presented in Table 2.

**Table 2:** Summary of Atterberg Limits Test and particle size distribution Results of the Soil Samples

Master Test Section	LL (%)	PL (%)	PI (%)	Passing #40 (%)	Passing #4 (%)	Passing #200 (%)	Clay (%)	Silt (%)
MTS 1-1	37	20	17	79.86	86.38	72.42	49.76	22.72
	39	24	15	94.12	99.64	77.76	46.88	30.88
	33	20	13	55.08	63.78	48.66	29.60	19.04
MTS 1-2	36	24	12	26.14	33.30	22.32	8.48	13.76
	29	21	8	96.42	99.94	73.38	37.28	36.16
	37	18	19	91.84	99.16	75.84	42.88	32.96
MTS 1-3	39	25	14	73.96	91.86	66.34	39.84	26.56
	36	26	10	50.88	73.14	43.76	27.36	16.48
	31	18	13	68.82	91.38	55.94	33.44	22.40
MTS 1-4	45	29	16	92.70	97.64	88.22	69.28	18.88
	42	25	17	95.96	99.52	92.70	48.80	48.16
	32	22	10	93.32	99.22	85.76	61.60	24.16
MTS 1-5	42	28	14	61.06	74.62	57.82	18.08	39.68
	41	31	10	74.10	87.20	69.08	42.72	26.40
	41	27	14	75.08	84.68	71.36	50.08	21.28
MTS 1-6	30	20	10	64.54	81.58	57.60	17.28	40.32
	35	21	14	59.34	73.32	50.48	18.08	32.32
	39	23	16	54.70	69.32	49.76	30.88	18.88

Source: Laboratory results

As shown in Table 2, the liquid limit of the soil samples ranged from 29 to

45 with plasticity index ranging from 8 and 19. It indicated that most of the soil



samples are materials with a high liquid limit and plasticity index. This provide an indication of the presence of clay in the soil samples and thereby will be unsuitable for many construction applications. The percent of silt content in the soil samples ranged from 13.76% to 48.16% and the percent of clay content in the soil samples ranged from 8.48% to 69.28%. This showed that most of the soil samples had more than 50% finer materials with smaller fraction of sand present and have particle sizes smaller than 75  $\mu\text{m}$ . This indicate that the soil samples contains more of clayed and silty materials than gravel and/or sand. This provides a useful information for the engineering classification of the soil. These results in higher resilient modulus values

(Thompson & Robnett, 1979; Lekarp, et al., 2000; Janoo & Bayer II, 2001).

#### CLASSIFICATION OF THE SOIL SAMPLES

The soil samples was classified using Unified Soil Classification System (USCS) and AASHTO Soil Classification. The USCS was in accordance with American Society for Testing and Materials (ASTM) standard (ASTM D 2487-98, 2000). The AASHTO Soil Classification system was in accordance with AASHTO M145 (AASHTO M145-91, 2004). Classification of the soil samples are important steps to accomplish the research objective since the resilient modulus is highly influenced by soil properties (Ping & Ling, 2008; Ping, et al., 2000). Table 3 summarized the USCS and the AASHTO Classification of the soil samples.

**Table 3:** The USCS and the AASHTO Classification Results of the Soil Samples

Master Test Section	USCS	AASHTO
MTS 1-1	CL (Lean clay with gravel)	A-6 (clayed soils)
	CL (Lean clay with gravel)	A-6 (clayed soils)
	GC (Clayed Gravel)	A-6 (clayed soils)
MTS 1-2	GC (Clayed Gravel)	A-2-4 (Silty or clayed gravel sand)
	CL (Lean clay with gravel)	A-4 (Silty soils)
	CL (Lean clay with gravel)	A-4 (Silty soils)
MTS 1-3	CL (Gravelly lean clay)	A-4 (Silty soils)
	SC (Clayed Sand)	A-4 (Silty soils)
	CL (Gravelly lean clay)	A-6 (clayed soils)
MTS 1-4	CL (Lean clay)	A-7-6 (clayed soils)
	CL (Lean clay)	A-7-6 (clayed soils)
	CL (Lean clay)	A-4 (Silty soils)
MTS 1-5	CL (Gravelly lean clay)	A-7-6 (clayed soils)
	CL (Gravelly lean clay)	A-5 (Silty soils)
	CL (Lean clay with gravel)	A-7-6 (clayed soils)
MTS 1-6	CL (Gravelly lean clay)	A-4 (Silty soils)
	CL (Gravelly lean clay)	A-6 (clayed soils)
	GC (Clayed Gravel)	A-4 (Silty soils)

Source: Laboratory results

According to USCS and the AASHTO Soil Classification system, the soil type for each of the soil sample was identified on the basis of the results of Atterberg limit, and particle size distribution tests. For the USCS, most of the soil samples belongs to the group of CL (low plasticity clay). The USCS identifies

that the soil samples are fine-grained soils and mostly clay. The AASHTO soil classification shows that the soil samples belongs to Clayed soil (A-6 and A-7-6) or silty soils (A-4, A-5 and A-2-4) group. From Table 3, the soil classification generally showed that the soil samples were "fair to poor" in subgrade properties.

### EVALUATION OF RESILIENT MODULUS MODEL FOR FINE-GRAINED SOILS

The resilient modulus model is a general constitutive equation that was developed by researchers for implementation in the Design of New and Rehabilitated Pavement Structures. The resilient modulus model can be used for all types of subgrade materials. The resilient modulus, the bulk stress, the octahedral shear stress and the deviator stress are normalized in the resilient modulus models by the atmospheric pressure. This will result in non-dimensional model parameters.

The most general form of a three-parameter model is as shown in Equation 2 (Ooi, et al., 2006):

$$M_R = k_1 P_a [f(c)]^{k_2} [g(s)]^{k_3}$$

where

$f(c)$  is a function of confinement;  $g(s)$  is a function of shear and  $k_1$ ,  $k_2$ , and  $k_3$  are constants.

The effects of confinement in these models can be expressed in terms of the minor principal stress ( $\sigma_3$ ), bulk stress ( $\vartheta$ ), or octahedral stress ( $\sigma_{oct} = \vartheta/3$ ), while the parameter options for modelling the effects of shear include the deviatoric stress or octahedral shear stress ( $\tau_{oct}$ ) (NCHRP, 2008).

Statistical analysis based on multiple linear regression was utilized to determine the resilient modulus model

parameters  $k_1$ ,  $k_2$  and  $k_3$ . Microsoft excel was used to perform the analysis. The resilient modulus is treated as the dependent variable, while bulk, the octahedral shear and the deviator stresses are used as the independent variables for both the fine and granular soils. The analysis was carried out for each soil type to evaluate the model parameters ( $k_1$ ,  $k_2$  and  $k_3$ ) from the results of the 15 stress combinations for fine soils applied during repeated load triaxial test (15 load sequences for fine soils).

The seven resilient modulus models developed by several researchers were evaluated in this work. Based on the results of repeated triaxial loading tests performed on 56 fine-grained and 22 coarse-grained Nigeria soils by Claros *et al.*, (1986), its results were used to test the resilient modulus constitutive models proposed by Uzan (Uzan, 1985), Witczak and Uzan (Witczak & Uzan, 1988), Ni *et al.* (Ni, et al., 2002), Ooi *et al.* (Ooi, et al., 2006), and NCHRP (NCHRP, 2008). The stress ratios used in describing the soils behaviour were bulk, confining and deviator stresses log-log model. In order to determine the model parameters ( $k_1$ ,  $k_2$  and  $k_3$ ), the resilient modulus models was transformed. Table 4 showed the summary of the transformed resilient modulus constitutive models considered in the research.

**Table 4:** Summary of the Transformed Resilient Modulus Constitutive Model Equations

Model	Resilient Modulus Constitutive Equation	Transformed Resilient Modulus Constitutive Equation
(Uzan, 1985)	$M_R = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\sigma_d}{P_a}\right)^{k_3}$	$\log\left(\frac{M_R}{P_a}\right) = \log k_1 + k_2 \log\left(\frac{\theta}{P_a}\right) + k_3 \log\left(\frac{\sigma_d}{P_a}\right)$
(Witczak & Uzan, 1988)	$M_R = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\tau_{oct}}{P_a}\right)^{k_3}$	$\log\left(\frac{M_R}{P_a}\right) = \log k_1 + k_2 \log\left(\frac{\theta}{P_a}\right) + k_3 \log\left(\frac{\tau_{oct}}{P_a}\right)$
(Ni, et al., 2002)	$M_R = k_1 P_a \left(1 + \frac{\sigma_3}{P_a}\right)^{k_2} \left(1 + \frac{\sigma_d}{P_a}\right)^{k_3}$	$\log\left(\frac{M_R}{P_a}\right) = \log k_1 + k_2 \log\left(1 + \frac{\sigma_3}{P_a}\right) + k_3 \log\left(1 + \frac{\sigma_d}{P_a}\right)$

Model	Resilient Modulus Constitutive Equation	Transformed Resilient Modulus Constitutive Equation
(Ooi, et al., 2004)	$M_R = k_1 P_a \left(1 + \frac{\theta}{P_a}\right)^{k_2} \left(1 + \frac{\sigma_d}{P_a}\right)^{k_3}$	$\log\left(\frac{M_R}{P_a}\right) = \log k_1 + k_2 \log\left(1 + \frac{\theta}{P_a}\right) + k_3 \log\left(1 + \frac{\sigma_d}{P_a}\right)$
(Ooi, et al., 2004)	$M_R = k_1 P_a \left(1 + \frac{\theta}{P_a}\right)^{k_2} \left(1 + \frac{\tau_{oct}}{P_a}\right)^{k_3}$	$\log\left(\frac{M_R}{P_a}\right) = \log k_1 + k_2 \log\left(1 + \frac{\theta}{P_a}\right) + k_3 \log\left(1 + \frac{\tau_{oct}}{P_a}\right)$
MPEDG (Titi, et al., 2006)	$M_R = k_1 P_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(1 + \frac{\tau_{oct}}{P_a}\right)^{k_3}$	$\log\left(\frac{M_R}{P_a}\right) = \log k_1 + k_2 \log\left(\frac{\theta}{P_a}\right) + k_3 \log\left(1 + \frac{\tau_{oct}}{P_a}\right)$

### STATISTICAL ANALYSIS OF RESILIENT MODULUS OF FINE-GRAINED SOILS

The statistical summary of the resilient modulus of fine-grained soils (Claros, et al., 1986) are shown in Tables 6 – 8. Table 6 presents the mean resilient modulus values, standard deviation, and COV of the resilient modulus of the fine-grained soils obtained from MTS 1 and 2. From Table 6, the COV ranges between

0.32 and 0.50, and from 0.28 to 0.30 for fine-grained soils obtained from MTS 1 and MTS 2 respectively. Table 6 indicates that the resilient modulus of the soil samples increases with the increase in the deviator stress ( $\sigma_d$ ) under constant confining pressure ( $\sigma_c$ ). It also showed that the resilient modulus increases with the increase in confining pressure under constant deviator stress.

**Table 6:** Statistical Summary of Resilient Modulus of Fine-Grained Soils obtained from MTS 1 and 2

Test Sequence	Confining Stress (kPa)	Deviator stress (kPa)	Resilient Modulus (kN/m <sup>2</sup> ) for MTS 1			Resilient Modulus (kN/m <sup>2</sup> ) for MTS 2		
			Mean	SD	COV	Mean	SD	COV
1	0	7.38	103663	48751.7	0.47	62634	18566.9	0.30
2	0	14.09	106338	38562.2	0.36	72285	20568.5	0.28
3	0	29.08	111466	35739.9	0.32	85211	24163.1	0.28
4	0	57.25	118357	41619.4	0.35	99771	29349.6	0.29
5	0	71.64	121103	44967.1	0.37	105205	31552.4	0.30
6	20.68	7.14	103574	49434.6	0.48	62180	18487.1	0.30
7	20.68	14.09	106338	38562.2	0.36	72285	20568.5	0.28
8	20.68	28.48	111287	35681	0.32	84804	24034.4	0.28
9	20.68	56.78	118259	41506.2	0.35	99575	29272.7	0.29
10	20.68	71.40	121060	44912.6	0.37	105121	31517.2	0.30
11	41.37	6.42	103323	51700.9	0.50	60761	18245.3	0.30
12	41.37	13.49	106102	39041.9	0.37	71584	20403.4	0.29
13	41.37	29.68	111644	35805.5	0.32	85612	24290.7	0.28
14	41.37	57.25	118357	41619.4	0.35	99771	29349.6	0.29
15	41.37	68.77	120583	44313.3	0.37	104188	31129.9	0.30

Table 7 presents the mean resilient modulus values, standard deviation, and COV of the resilient

modulus of the fine-grained soils obtained from MTS 3 and 4. From Table 7, the COV ranges between 0.22 and 0.45, and from

0.18 to 0.41 for fine-grained soils obtained from MTS 3 and MTS 4 respectively. Table 7 indicates that the resilient modulus of the samples decreases with increase in the deviator stress ( $\sigma_d$ ) under constant confining pressure ( $\sigma_c$ ). This is in consistent

with other researchers (Sandefur, 2003; Titi, et al., 2006). Also, Table 7 showed that the confining stress has a less significant effect than deviator stress for the soil samples.

**Table 7:** Statistical Summary of Resilient Modulus of Fine-Grained Soils obtained from MTS 3 and 4

Test Sequence	Confining Stress (kPa)	Deviator stress (kPa)	Resilient Modulus (kN/m <sup>2</sup> ) for MTS 3			Resilient Modulus (kN/m <sup>2</sup> ) for MTS 4		
			Mean	SD	COV	Mean	SD	COV
1	0	7.38	194320	84496	0.43	118241	46355.2	0.39
2	0	14.09	175790	61931.7	0.35	115092	32483.3	0.28
3	0	29.08	159158	43590.6	0.27	113899	20437.3	0.18
4	0	57.25	146925	33307.0	0.23	115267	23831.8	0.21
5	0	71.64	143501	31601.1	0.22	116305	28838.6	0.25
6	20.68	7.14	195378	85826.8	0.44	118452	47083.7	0.40
7	20.68	14.09	175790	61931.7	0.35	115092	32483.3	0.28
8	20.68	28.48	159583	44017.8	0.28	113897	20636.4	0.18
9	20.68	56.78	147058	33387.9	0.23	115234	23674.6	0.21
10	20.68	71.40	143550	31620.0	0.22	116288	28753.2	0.25
11	41.37	6.42	198820	90184.7	0.45	119159	49407.7	0.41
12	41.37	13.49	176919	63259.5	0.36	115244	33381.8	0.29
13	41.37	29.68	158746	43178.2	0.27	113904	20256.6	0.18
14	41.37	57.25	146925	33307.0	0.23	115267	23831.8	0.21
15	41.37	68.77	144105	31844.9	0.22	116093	27821.3	0.24

Table 8 presents the mean resilient modulus values, standard deviation, and COV of the resilient modulus of fine-grained soils obtained from MTS 5 and 6. From Table 8, the COV ranges between 0.45 and 0.49, and from 0.29 to 0.40 for fine-grained soils obtained from MTS 5 and MTS 6 respectively. Table

8 indicates that the resilient modulus of the samples increases with the increase of the deviator stress ( $\sigma_d$ ) under constant confining pressure ( $\sigma_c$ ). Moreover, the resilient modulus increases with increase in confining pressure under constant deviator stress.

**Table 8:** Statistical Summary of Resilient Modulus of Fine-Grained Soils obtained from MTS 5 and 6

Test Sequence No.	Confining Stress, (kPa)	Deviator stress, (kPa)	Resilient Modulus (kN/m <sup>2</sup> ) for MTS 5			Resilient Modulus (kN/m <sup>2</sup> ) for MTS 6		
			Mean	SD	COV	Mean	SD	COV
1	0	7.38	66108	32240.4	0.49	101840	38260.3	0.38
2	0	14.09	76419	36278.3	0.47	105981	31818.8	0.30
3	0	29.08	90102	41710.7	0.46	113746	32515.1	0.29
4	0	57.25	105366	47989.8	0.46	124059	38769.6	0.31
5	0	71.64	111029	50399.9	0.45	128158	41733.4	0.33

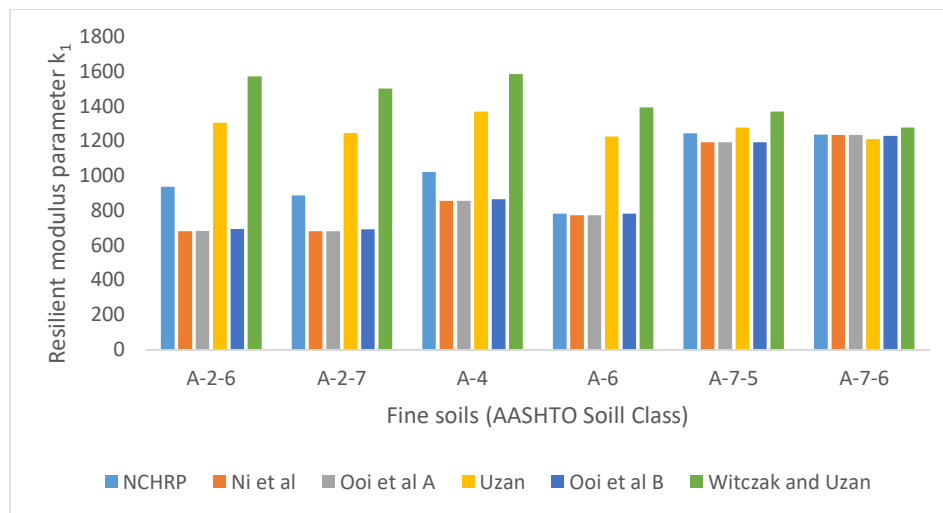


6	20.68	7.14	65621	32049.8	0.49	101700	38783.9	0.38
7	20.68	14.09	76419	36278.3	0.47	105981	31818.8	0.30
8	20.68	28.48	89673	41537.9	0.46	113476	32396.8	0.29
9	20.68	56.78	105161	47903.7	0.46	123913	38667.2	0.31
10	20.68	71.40	110942	50362.2	0.45	128093	41685.8	0.33
11	41.37	6.42	64096	31452.8	0.49	101302	40575.5	0.40
12	41.37	13.49	75674	35985.5	0.48	105619	32029.5	0.30
13	41.37	29.68	90525	41881.0	0.46	114013	32635.8	0.29
14	41.37	57.25	105366	47989.8	0.46	124059	38769.6	0.31
15	41.37	68.77	10997	49945.4	0.45	127382	41161.6	0.32

In summary, observing Tables 6 through 8 indicates that each soil specimen showed consistent behaviour during each test sequence.

### EVALUATION OF THE RESILIENT MODULUS MODEL PARAMETERS FOR THE FINE-GRAINED SOILS

Histograms showing the distribution of resilient modulus model parameters ( $k_1$ ,  $k_2$ , and  $k_3$ ) values obtained for fine-grained soils by soil class using the six resilient modulus equations are presented in Figures 1 through 3.

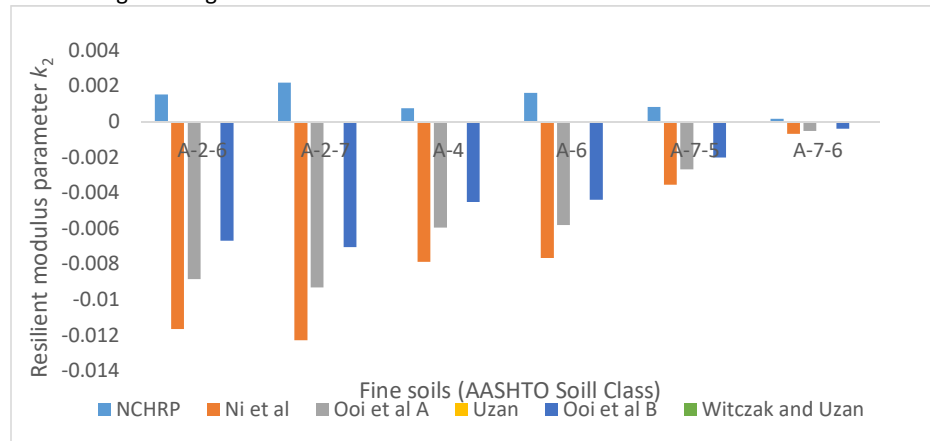


**Figure 1:** Resilient Modulus Parameters  $k_1$  for fine-grained subgrade soils in Nigeria

Figure 1 showed the relationship between resilient modulus parameter  $k_1$  obtained for fine-grained soils and the resilient modulus equations evaluated. Figure 1 showed that the magnitude of  $k_1$  was always greater than zero. This is in line with other researchers (Titi, et al., 2006; Santha, 1994; Rao, et al., 2012; Yau & Von

Quintus, 2002). This also agrees with the statement that the coefficient  $k_1$  is proportional to Young's modulus, it should always be positive as resilient modulus can never be negative (George, 2004). In conclusion, all the resilient modulus equations evaluated in this research can

be adopted for fine-grained soils considering the magnitude of  $k_1$ .

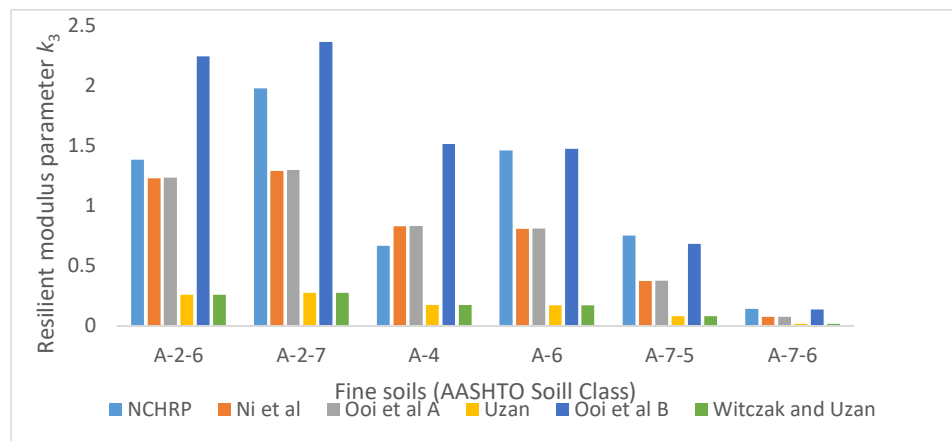


**Figure 2:** Resilient Modulus Parameter  $k_2$  for fine-grained subgrade soils in Nigeria

Figure 2 showed the relationship between resilient modulus parameter  $k_2$  obtained for fine-grained soils through the evaluation of the six resilient modulus equations evaluated. From Figure 2, it is observed that the parameter  $k_2$  was generally less than zero for all the models evaluated except that proposed NCHRP. As it was reported by several researchers (Santha, 1994; Rao, et al., 2012; Yau & Von Quintus, 2002; George, 2004; Titi, et al., 2006), the coefficient  $k_2$  should be positive, because increase in volumetric stress produces stiffening or hardening of the material, thereby yielding higher modulus and that the resilient modulus decreases with the increase in the deviator

stress. In conclusion, the resilient modulus equation proposed by NCHRP will be adopted for fine-grained soils considering the magnitude of  $k_2$ .

Figure 3 showed the relationship between resilient modulus parameters  $k_3$  obtained for fine-grained soils and the resilient modulus models evaluated. Figure 3 showed that the parameter  $k_3$  was also greater than zero since the resilient modulus increases with increase in the confining stress. In conclusion, all the resilient modulus equations evaluated in this research can be adopted for fine-grained soils considering the magnitude of  $k_3$ .



**Figure 3:** Resilient Modulus Parameters  $k_3$  for fine-grained subgrade soils in Nigeria

Based on the evaluation of the resilient modulus equations for fine-grained subgrade soils in Nigeria, it was observed that the resilient modulus equation adopted by NCHRP was the best in determining resilient modulus for fine-grained soils. This expression has gained considerable acceptance over time. The strength of this equation is its ability to handle two of the resilient modulus dependencies, stress-state and material type (Rao, et al., 2012).

Table 9 presents the statistical summary of the resilient modulus parameters of fine-grained soils obtained

from the resilient modulus equations evaluated. Table 9 showed that the magnitude of  $k_1$  was always greater than zero for all the models evaluated. This implies that resilient modulus should always be greater than zero. Also, the parameter  $k_2$  which, is related to the confining stress, were always less than zero for all the models evaluated except for that of NCHRP model. The parameter  $k_3$  was always greater than zero for all the models evaluated. This showed that the resilient modulus increases with increase in deviator stress at constant confining stress.

**Table 9:** Statistical data of  $k_i$  obtained from the test results of the investigated fine-grained soils using resilient modulus models.

Model	Parameter	Mean	Median	Minimum	Maximum	Standard Dev	Standard Error
Uzan	$k_1$	1320.92	1230.78	21.9186	2107.274	422.720	1.0000
	$k_2$	-5E-17	-4E-17	-3.1E-16	1.3E-16	9E-17	3.6E-15
	$k_3$	0.2045	0.1931	0.0042	0.4042	0.1040	3.6E-15
Witczak and Uzan	$k_1$	1548.03	1463.34	27.9011	2576.806	527.610	1.0000
	$k_2$	-9E-18	0.0000	-2.7E-16	1.88E-16	8.5E-17	3.6E-15
	$k_3$	0.2045	0.1931	0.0042	0.4042	0.1040	3.8E-15
Ooi et al A	$k_1$	781.924	750.168	9.2038	1430.843	296.985	3.2722
	$k_2$	-0.0070	-0.0066	-0.0138	-0.0001	0.0036	1.7799
	$k_3$	0.9744	0.9198	0.0202	1.9259	0.4955	2.9995
Ooi et al B	$k_1$	791.406	763.520	9.4025	1435.499	297.729	3.6492
	$k_2$	-0.0053	-0.0050	-0.0105	-0.0001	0.0027	1.9633
	$k_3$	1.7720	1.6728	0.0367	3.5025	0.9012	6.0592
Ni et al	$k_1$	781.001	748.621	9.1845	1430.387	296.915	3.0067
	$k_2$	-0.0092	-0.0087	-0.0182	-0.0002	0.0047	3.4579
	$k_3$	0.9693	0.9150	0.0200	1.9158	0.4929	2.7137
NCHRP	$k_1$	857.780	803.924	9.3952	1435.330	291.160	2.7412
	$k_2$	0.0018	0.0018	4E-05	0.0038	0.0010	0.5701
	$k_3$	1.6647	1.5941	0.0363	3.4729	0.8818	5.3643

## CONCLUSION

Based on the results of this study, the following conclusions was reached:

1. The AASHTO soil classification generally showed that the subgrade soil samples were "Fair to Poor" in subgrade properties.
2. The USCS soil classification indicated that the samples were mostly clay soil.
3. The equations that correlate resilient modulus model parameters ( $k_1$ ,  $k_2$ , and  $k_3$ ) to basic soil properties for fine-grained soil can be utilized to estimate level 2 resilient



modulus input for the NEMPADS.

4. Based on the results of this research, the Equation of resilient modulus adopted by NCHRP should be adopted for determining resilient modulus of fine-grained subgrade soils in Nigeria.

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