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Preliminary *k*-Values of Unbound Natural Quartzitic Gravels for Mechanistic-Empirical Pavement Design

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Abstract

The generalized constitutive model relating the resilient modulus (M_R) of flexible pavement layer materials to stress state, adopted by the Mechanistic-Empirical Pavement Design Guide (MEPDG), contains a set of constants known as *k*-values (k_1, k_2, and k_3) which are associated with the physical state of the layer materials. In Ghana, natural gravels constitute the predominant and sometimes the sole layer materials for most flexible pavements yet representative *k*-values of gravel materials, have not been determined to permit full application and implementation of the mechanistic-empirical design concept to pavements involving such materials. In this study, *k*-values characterising typical natural quartzitic gravels used for road construction in the country were derived by regression techniques from M_R values determined using laboratory repeated load triaxial test. Using multiple linear regression technique, correlation relationships were then developed between the *k*-values and the physical properties of the gravels, namely, percentages of materials passing the 9.5 mm ($P_{9.5}$) and 2.0 mm ($P_{2.0}$) sieves, liquid limit (LL), maximum dry density (ρ_{dmax}), and optimum moisture content (w_{opt}). The regression analysis returned k_1 values which ranged between 441 and 958 with a mean of 516; k_2 which varied between 0.0636 and 0.2168 with a mean value of 0.1216; and, k_3 values which ranged between 0.1257 and 3.1590 with a mean value of 1.723. Contrary to what is mostly reported in literature, the analysis returned positive k_3 values for all but one gravel material, suggesting stress hardening under octahedral shear stress for those materials. While an expanded sample base is required to fully characterize the whole gamut of natural gravels used in pavement construction in the country, this study on limited quartzitic gravel samples has given a good indication of strong linear correlations between the

k-values and the index properties of the gravels, to permit estimates of the constants for such gravels be made where capability and opportunity for conducting resilient modulus tests do not exist. However, further work is recommended to fully characterise the exact nature of k_3 values for quartzitic gravels in the country.

Keywords

Flexible Pavement, Index Properties, *k*-Values, Mechanistic-Empirical Design, Quartzitic Gravel, Resilient Modulus

1. Introduction

In the mechanistic-empirical (M-E) design/analysis of multi-layer flexible pavement structures, the resilient response of layer materials to repeated traffic loading is quantified by the resilient modulus (M_R) parameter. M_R as a material stiffness parameter was initially introduced by Seed *et al.* [1] to measure the elastic modulus of a material at a given stress state. In pavement engineering, the parameter measures the elastic property of a layer material with consideration to the material's non-linear characteristics. M_R values may be determined directly in the laboratory through repeated load triaxial testing and indirectly through correlation with empirical soil strength parameters (California Bearing Ratio, Soil Support Value, and Resistance Value) or by back-calculating from deflections measured with falling weight deflectometer. But because M_R is a basic mechanistic material property, its estimation through correlations with empirical soil strength parameters is not considered appropriate as the parameters do not reflect the conditions in a pavement under repeated traffic loading [2].

In empirical pavement design approaches, representative resilient modulus values are assigned to layer materials irrespective of position in the pavement-subgrade system. This is considered a limitation since the resilient modulus of unbound soil/aggregate layers depends on stress state and is known to increase with increasing bulk stress and confining stress [3]. Layer materials, depending on their position in the pavement-subgrade system, come under different vertical stresses from the overburden and traffic loading as well as horizontal stresses from confinement.

Figure 1 shows the state of stress in a triaxial loading and the response of the loaded material under repeated loading conditions [4]. Based on the stress conditions defined in the figure, the resilient modulus is defined as;

$$M_R = \frac{\sigma_d}{\varepsilon_r} \tag{1}$$

where,
M_R = resilient modulus;
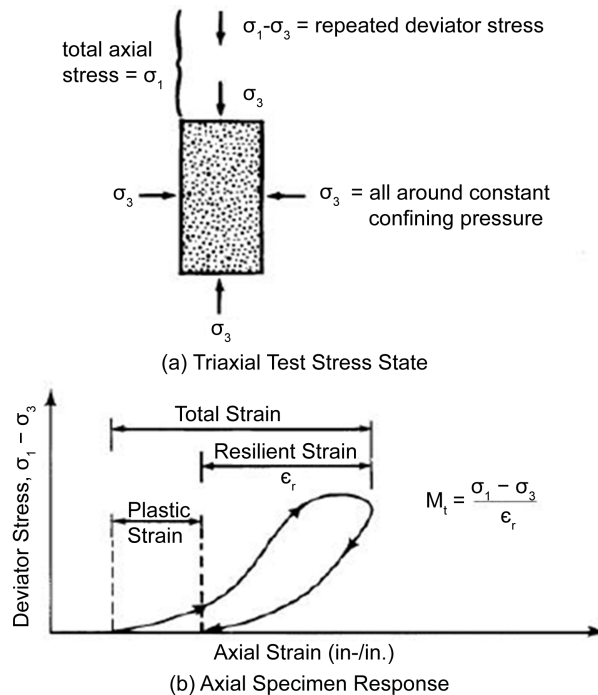σ_d = repeated deviator stress ($\sigma_1 - \sigma_3$);

Figure 1. Definition of M_R [4].

ε_r = recoverable axial strain

The basic definition of M_R given by Equation (1) has formed the basis of several constitutive model proposals for determining resilient modulus including those of Dunlap [5], Moossazadeh and Witczak [6], Uzan [7], Witczak and Uzan [8], and the generalized model [9] given by Equation (2).

$$M_R = k_1 P_a \left(\frac{\theta}{P_a} \right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1 \right)^{k_3} \tag{2}$$

where,

$$\tau_{oct} = \frac{1}{3} \sqrt{\left(\sigma_1 - \sigma_2 \right)^2 + \left(\sigma_2 - \sigma_3 \right)^2 + \left(\sigma_3 - \sigma_1 \right)^2} \quad \text{(octahedral stress)}$$

P_a = atmospheric pressure

σ_1 = major principal stress

σ_2 = intermediate principal stress (=σ_3 in triaxial test)

σ_3 = minor principal stress (confining pressure in triaxial test)

θ = bulk stress ($\sigma_1 + \sigma_2 + \sigma_3$)

k_1, k_2 and k_3 = constants (*k*-values) related to material's physical state

The limitation inherent in empirical designs due to failure to relate M_R to the stress state of layer materials is addressed in the Mechanistic-Empirical Pavement Design Guide (MEPDG) [9] which computes resilient modulus using Equation (2). As is clear in the model, M_R is defined by the state of stress of the layer materials in the pavement-subgrade system and the physical state of the materials through the material constants (k_1, k_2, k_3) known as *k*-values. For Level 1 analysis which has the highest design reliability in the Guide [9], it is recom-

mended to determine the *k*-values from laboratory resilient modulus test conducted under repeated loading conditions to simulate traffic loading, while for Level 2 analysis, which is of intermediate reliability, M_R may be estimated through correlations with index soil properties [10] or possibly from agency database [11].

Unlike M_R, the *k*-values are dependent only on the physical state of the material without any stress influence. The constant k_1 which can only be positive is related to material strength and tends to be higher for materials containing angular particles and less fines than those containing higher fines and rounded particles [10]; k_2 should be positive as increasing bulk stress causes stiffening effect on the material and, hence, increasing modulus while k_3 should be negative since increasing shear stress produces a softening effect on the material resulting in lower M_R [12] [13]. A number of researchers, including [14] [15] [16], have correlated *k*-values of local soils with different soil physical properties including, but not limited to, the following: dry density, moisture content, liquid limit, plastic limit, % clay and % gravel. Such correlations help evaluate the *k*-values associated with the physical state of layer materials, which in turn make it possible for the resilient modulus of the layers under any stress state to be estimated using the generalized constitutive model. While literature may be replete with some such correlations for local soils, it is important to note that they are locality-specific and do not have universal application due to the variability in the geology and mineralogy of the rock masses from which soils are derived.

For fine-grained soils, the resilient modulus decreases with increase in deviator stress at a given confining stress in a behaviour known as stress-softening whereas for granular soils, the parameter increases with increase in deviator stress (stress-hardening behaviour) [3]. Other factors which affect M_R include moisture which lowers M_R when it is above the optimum content [17], particle shape which results in higher M_R when it is angular to sub-angular but lower values when rounded to sub-rounded [18] [19] [20] and soil suction in the case of fine-grained subgrade soils [21].

The MEPDG approach to flexible pavement design has a lot of calibration protocols and performance predictions which require local input data for traffic, climate, subgrade and material properties [22]. For the most part, natural gravels have been the predominant layer materials used in Ghana for the construction of sub-base and base layers, particularly, of low- and medium-volume flexible pavements because the materials can generally be found within economic haul distance of most constructions and are relatively low in cost compared to crushed rock. Therefore, to properly implement and evaluate the benefits of the M-Edesign of flexible pavements involving such materials, the generalized resilient modulus model must be calibrated using data on local soils/gravels. Laboratory M_R tests become important in the calibration process as they generate the set of data required for the determination of the *k*-values associated with the compacted layer materials/soils. This study forms part of preliminary efforts to

establish representative *k*-values for natural gravels used in the country for flexible pavement construction.

2. Pavement-Grade Natural Gravels in Ghana

The Ministry of Roads and Highway's Standard Specification for Road and Bridge Works [23] uses the term "natural gravel" to include lateritic gravel, quartzitic gravel, calcareous gravel, soft stone, conglomerate, sand or clayey sand or a combination of any of the above materials. However, invariably, most natural gravels in Ghana are usually a mixture of quartz and laterite particles in a matrix of fines [24]. Gravels containing over 50% of quartz fractions are termed quartzitic and those with over 50% laterite particles are termed lateritic. A peculiar characteristics of lateritic gravels is their reddish-brown to ochre colour, presence of concretionary particles high in iron and silica, high clay content and high plasticity [24].

In terms of suitability as flexible pavement layer materials, natural gravels are classified into grades as G30 (CBR ≥ 30); G40 (CBR ≥ 40); G60 (CBR ≥ 60); and G80 (CBR ≥ 80) using CBR, particle gradation, Grading Modulus, Atterberg Limits, Plasticity Modulus, and 10% Fines Value. Whereas G30 and G40 grades are used for sub-base, G60 and G80 are used for base construction [23].

3. Materials and Methods

3.1. Natural Gravels for Study

Natural gravels used for road construction are mainly of two types, lateritic and quartzitic; this study focused only on the latter. The samples were collected from gravel pits distributed in five regions in the country (Figure 2). Sampling locations and sample identification are detailed in Table 1.

3.2. Laboratory Tests

3.2.1. Index Property Test
Grain size analysis was conducted on each of the gravel samples in accordance with Ghana Highway Authority (GHA) S7 Standard Method [25] whilst compaction was carried out in accordance with GHA Standard S1: Test Method for the Determination of Maximum Dry Density and Optimum Moisture Content of Soils and Soil-Aggregate Mixtures [25]. Atterberg Limits tests were carried out in accordance with ASTM D4318-10 [26] on the fines fraction (below 0.425 mm) of the primary samples.

3.2.2. Repeated Load Triaxial Test
The M_R test was conducted using the Industrial Process Controls' Universal Materials Testing Apparatus (Plate 1) equipped with a desktop computer, environmental chamber enclosing the load cell and the triaxial chamber, control and data acquisition system, air compressor, and air dryer. Gravel materials passing the 19 mm sieve were used for specimen preparation and compacted in a 100 mm
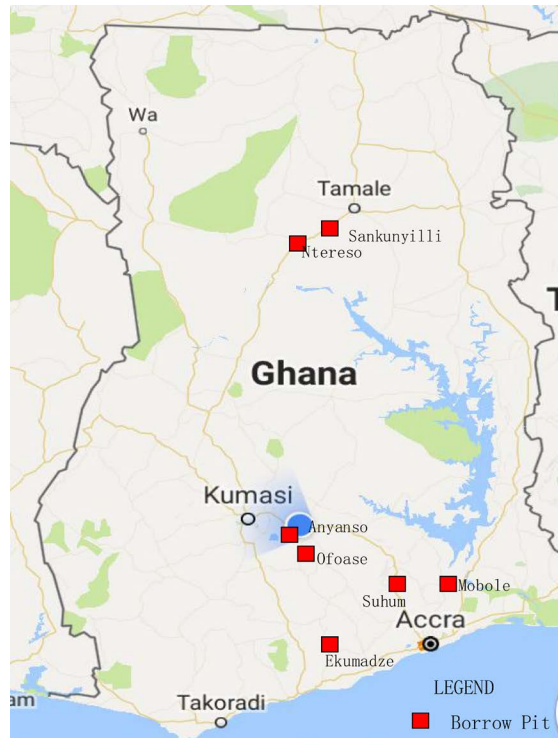
Figure 2. Map of Ghana showing locations of gravel materials used for study.

Table 1. Locations and identification of natural quartzitic gravels used for study.

Sample Id.	Location of Depoist	Region
NNR	Ntereso	Northern
SNR	Sankunyilli	Northern
OAR	Ofoase	Ashanti
AAR	Anyanso	Ashanti
SER	Suhum	Eastern
ECR	Ekumadze	Central
MGR	Mobole	Greater Accra

diameter × 200 mm high mould to the maximum dry density and optimum moisture content from the compaction characteristics of the soil. After the compaction, the specimens were removed from the mould and covered with rubber membrane for the resilient modulus test. Three specimens were prepared for each soil sample and tested for Resilient Modulus in accordance with AASHTO T307-99 test protocol [27].

3.3. Determination of *k*-Values of Gravel Materials

Equation (2) which relates M_R to the stress state and physical state of the layer material in the repeated load triaxial test was modified to the following form;

Plate 1. Resilient modulus test set-up.

$$\log\left(\frac{M_R}{P_a}\right) = \log k_1 + k_2 \log\left(\frac{\theta}{P_a}\right) + k_3\left(\frac{\tau_{oct}}{P_a} + 1\right) \tag{3}$$

Under laboratory triaxial loading conditions, τ_{oct} reduces to $\frac{\sqrt{2}}{3}(\sigma_1 - \sigma_3)$.

The logarithmic-transformed M_R values obtained from the repeated load tri-axial test were regressed on the logarithmic-transformed test stresses to obtain the k-values in Equation (3). Correlation relationships between the k-values and the physical properties of the gravels were then developed using multiple linear regression technique. Based on review of literature [13] [14] [15], and for proper material and physical state representation of the compacted gravels, the index properties selected as the independent variables for the correlation relationships were the percentages of materials passing the 9.5 mm ($P_{9.5}$) and 2.0 mm ($P_{2.0}$) sieves, the liquid limit (LL), the maximum dry density (ρ_{dmax}) and the optimum moisture content (w_{opt}).

4. Results and Discussion

4.1. Index Properties of Gravels

The natural gravels investigated were quartzitic in nature with colour ranging from light brown to yellowish brown. Visual inspection of the coarse fractions of the gravels from Ntereso (NNR), Suhum (SER), and Ekumadze (CER)revealed sub-angular particles with rough texture whereas fractions of the samples from Sankunyilli (SNR), Ofoase (OAR), Anyanso (AAR), and Mobole (MGR) were rounded in shape and smooth textured, suggesting some degree of transportation in the geologic past. The index geotechnical properties of the samples are summarised in Table 2. Because the fines portions of all the samples plotted

**Table 2.** Index properties of natural gravels used for study.

	NNR	SNR	OAR	AAR	SER	ECR	MGR
Classification							
AASHTO(Group Index)	A-2-4(0)	A-2-4(0)	A-2-4(0)	A-2-4(0)	A-2-4(0)	A-2-4(0)	A-2-4(0)
Group Name	Clayey Sandy Gravel	Clayey Sandy Gravel	Clayey Sandy Gravel	Sandy Clayey Gravel	Clayey Sandy Gravel	Clayey Sandy Gravel	Clayey Sandy Gravel
Grain Size Distribution							
Gravel (%)	71	56	76	72	62	77	72
Sand (%)	18	26	14	13	22	15	22
Silt-Clay	11	18	10	15	16	8	6
Atterberg Limits							
Liquid Limit, LL (%)	28	22	28	25	29	28	18
Plastic Limit, PL (%)	19	15	19	16	19	19	11
Plasticity Index, PI (%)	9	7	9	9	10	9	7
Compaction Characteristics							
Maximum Dry Density, ρ_d (kg/m³)	2152	2300	2216	2211	2038	2220	2305
Optimum Moisture Content, w_{opt} (%)	4.8	4.2	5.6	9.0	9.8	4.9	5.0

above the A-Line on the Cassagrande plasticity chart, the fines were predominantly clay in content. Based on AASHTO Classification, all the samples classified as A-2-4(0) and were essentially clayey sandy gravel except that from Anyanso (AAR) which was a sandy clayey gravel. Figure 3 shows the particle size distribution curves of the gravel materials superimposed on the grading envelope for G80 and G60 base-grade gravel materials [23].

On the basis of particle gradation in terms of MRH Standards [23], almost all the gravel materials appeared to meet the requirements for G80 gravel but failed one or more other G80 requirements such as Grading Modulus, Atterberg Limits and Plasticity Modulus and hence were all clearly G60 base materials except gravel material MGR which was of G80 grade. However, according to the AASHTO T307-99 resilient modulus test protocol [27], all the materials classified as Type 1 unbound natural gravel since the proportions passing the 2 mm and 0.075 mm sieves were less than 70% and 20%, respectively, with plasticity indices being less than 10%.

The compaction characteristics of the gravels showed maximum dry densities ranging between 2038 kg/m³ and 2305 kg/m³ with an average of 2206 kg/m³. Most samples had optimum water contents ranging between 4% and 6% with an average of 4.9%; only two samples (AAR and SER) had optimum water contents about twice the average of those of the rest of the samples. Perhaps the higher than average silt-clay contents of the two samples could partly be the reason for the relatively high optimum water contents although sight is not lost of the fact that Sample SNR with similar percentage of silt-clay content, but substantially
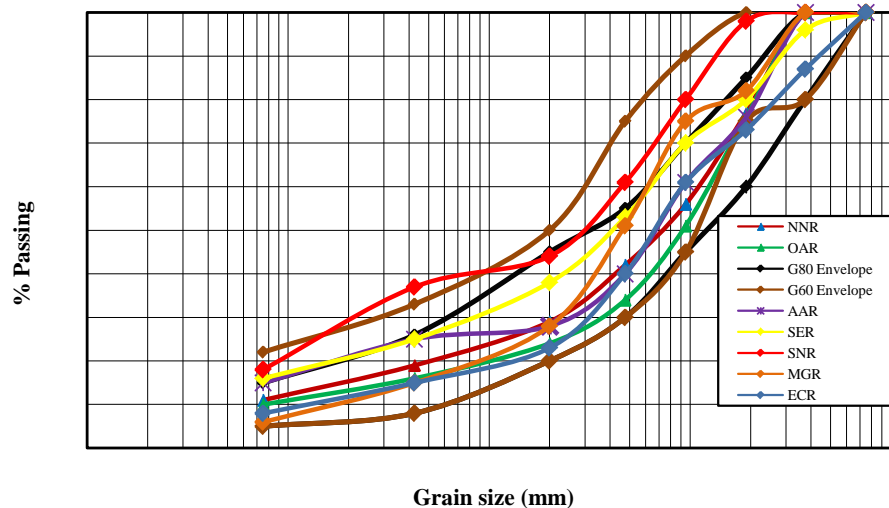
Figure 3. Particle size distribution of quartzitic gravel samples.

lower gravel fraction, belonged to the low optimum water content sample cohort.

4.2. Resilient Modulus of Study Material

Resilient modulus test results in the repeated load triaxial tests have been summarized in Table 3. As shown in Figure 4, the modulus increased with confining pressure because for granular soils, confinement is known to increase interlock between the individual aggregate particles within the compacted matrix, a behaviour referred to in literature as stress-hardening.

As expected, most of the granular materials exhibited increasing modulus with increasing bulk stress for a given confining stress (typified by Figures 5-7 for samples NNR, OAR, and SNR, respectively). The only exception to this stress-hardening behaviour was the gravel from Anyanso (Sample AAR) which exhibited both stress hardening and stress softening characteristics (Figure 8).

In the case of variation of M_R with octahedral shear stress, contrary to what is touted in literature, six out of the seven gravel samples exhibited increasing modulus with increasing octahedral shear stress at a given confining stress (typified by Figure 9 and Figure 10 for samples NNR and MGR, respectively); the only exception again being Sample AAR which generally exhibited little change in modulus initially then decreasing values with further increase in shear stress (Figure 11).

4.3. *k*-Values for Gravels

Table 4 presents the summary of the *k*-values derived from the regression analysis of the resilient modulus values for the gravel samples. As seen in the table, k_1 values ranged between 441 and 958 with a mean of 516; k_2 varied between 0.0636 and 0.2168 with a mean value of 0.1216 while k_3, which is related to the octahedral shear stress, ranged between 0.1257 and 3.1590 with a mean value of 1.723.

Table 3. Summary of test results in repeated load triaxial tests on gravel materials.

Test Sequence	Conf. Pressure (kPa)	NNR Dev. Stress (kPa)	NNR M_R (MPa)	SNR Dev. Stress (kPa)	SNR M_R (MPa)	OAR Dev. Stress (kPa)	OAR M_R (MPa)	AAR Dev. Stress (kPa)	AAR M_R (MPa)	SER Dev. Stress (kPa)	SER M_R (MPa)	ECR Dev. Stress (kPa)	ECR M_R (MPa)	MGR Dev. Stress (kPa)	MGR M_R (MPa)
1	20.7	10.2	54.3	10.5	49.6	11.0	51.8	9.3	62.3	9.6	84.4	9.8	47.1	8.0	56.8
2	20.7	21.1	67.9	21.4	57.0	22.0	58.5	20.2	66.4	20.3	96.0	20.4	58.6	16.8	60.8
3	20.7	32.3	78.3	32.5	66.2	33.4	67.8	31.1	66.4	31.1	106.0	31.0	71.3	25.9	71.7
4	34.5	17.3	62.1	17.8	58.7	18.4	60.6	16.4	70.0	16.6	99.9	16.6	58.3	13.2	60.2
4	34.5	35.5	84.1	36.1	71.3	36.9	73.8	34.5	70.2	34.3	109.7	34.2	76.2	28.6	69.7
6	34.5	52.4	97.8	53.8	86.1	55.5	84.3	53.2	72.0	52.0	118.8	52.9	89.1	44.4	84.5
7	68.9	34.6	81.4	35.7	76.3	36.8	77.8	34.6	78.6	33.4	112.6	34.5	77.4	27.8	70.9
8	68.9	69.0	114.0	71.1	102.8	74.2	99.2	71.9	78.9	69.3	123.8	73.5	99.6	59.9	91.6
9	68.9	101.2	148.1	107.0	129.1	111.5	118.7	111.2	69.2	105.9	140.2	110.6	97.8	93.2	110.9
10	103.4	33.6	78.1	35.6	81.9	36.7	82.3	36.4	72.1	31.4	132.9	36.7	78.6	27.0	72.4
11	103.4	51.0	101.1	53.8	95.9	55.4	93.5	54.8	73.1	49.1	138.4	55.0	83.6	41.8	82.5
12	103.4	99.7	153.1	106.9	133.6	110.9	125.9	111.2	106.7	100.2	148.0	111.2	135.8	90.2	111.3
13	137.9	48.9	102.6	53.1	99.6	55.4	98.7	54.5	111.8	44.7	145.6	55.4	103.2	40.9	82.9
14	137.9	65.7	121.3	71.3	111.7	74.0	110.7	73.4	111.3	62.5	147.6	74.2	117.8	56.7	91.4
15	137.9	133.2	177.8	144.3	155.0	148.0	139.1	148.2	83.2	138.0	153.2	-	-	132.0	114.3
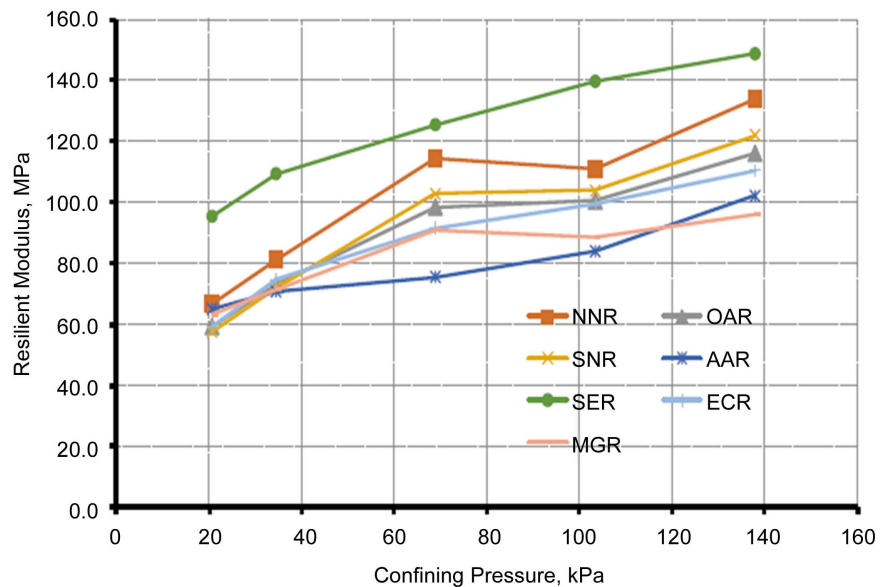
Figure 4. Laboratory M_R for gravel samples at different confining pressures.
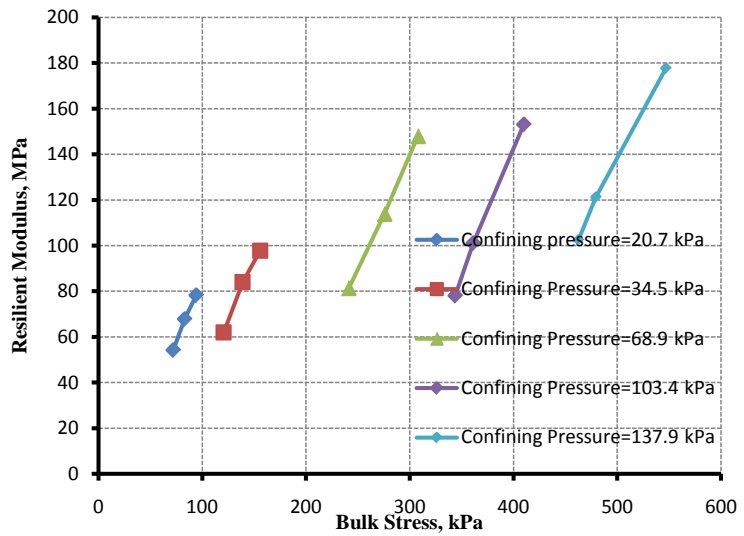
Figure 5. Laboratory M_R for gravel sample NNR at different confining and bulk stresses.
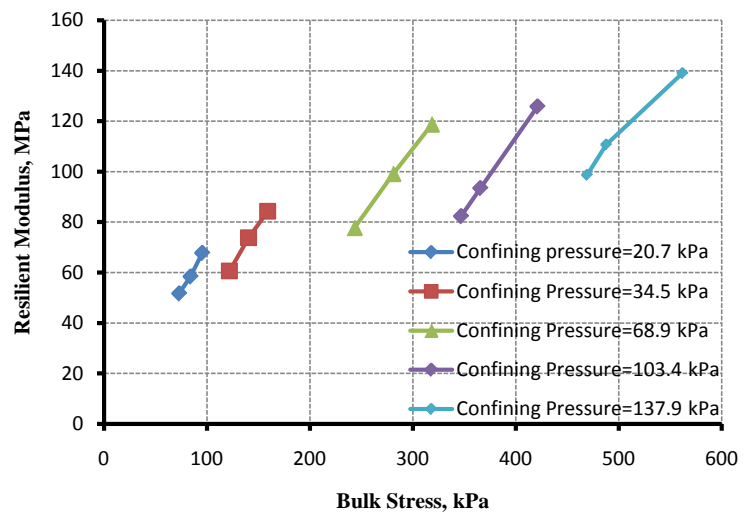

Figure 6. Laboratory M_R for gravel sample OAR at different confining and bulk stresses.
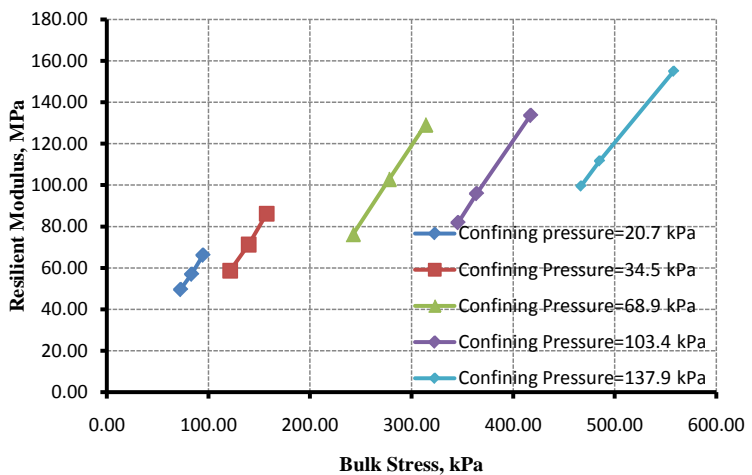

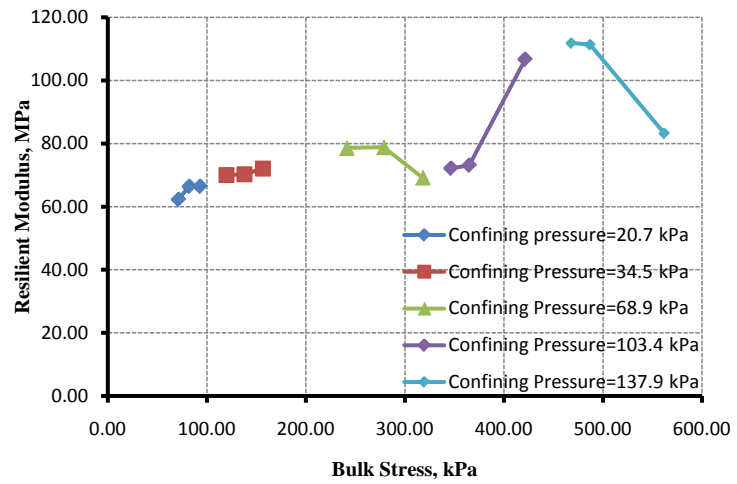**Figure 7.** Laboratory M_R for gravel sample SNR at different confining and bulk stresses.

Figure 8. Laboratory M_R for gravel sample AAR at different confining and bulk stresses.
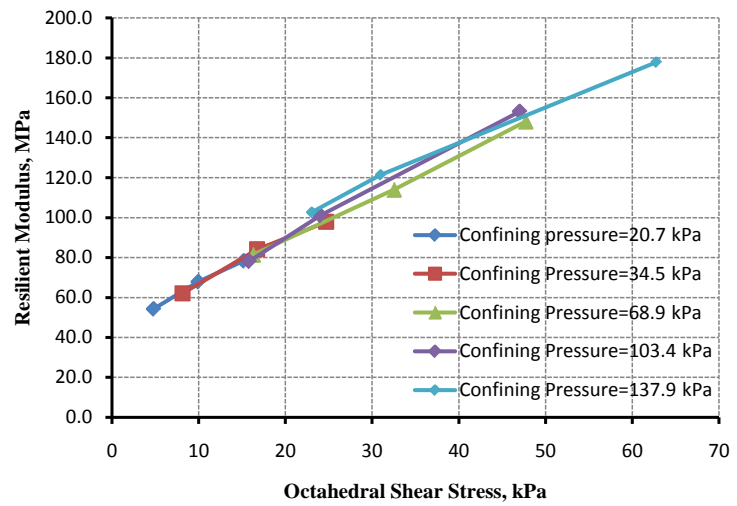

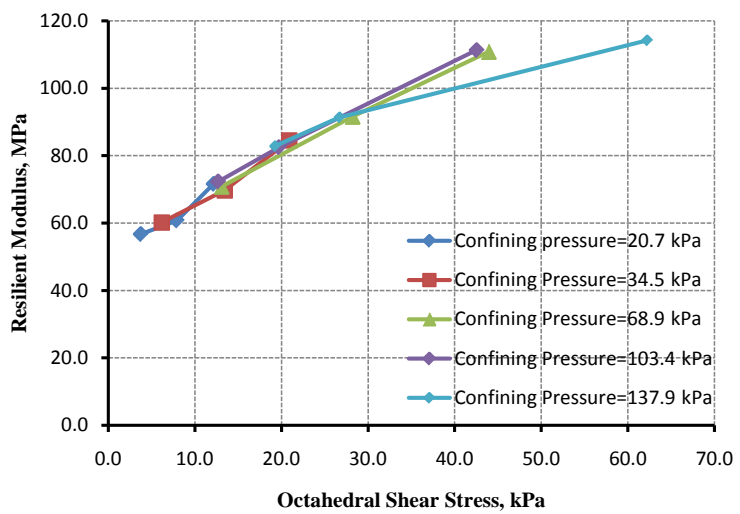**Figure 9.** Laboratory M_R for gravel sample NNR at different confining and shear stresses.
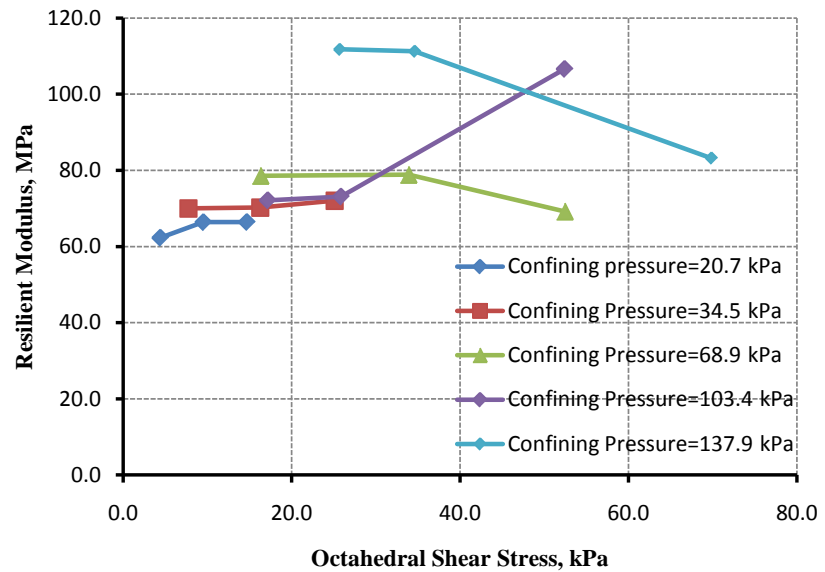

Figure 10. Laboratory M_R for gravel sample MGR at different confining and shear stresses.

Figure 11. Laboratory M_R for gravel sample AAR at different confining and shear stresses.

Table 4. Average *k*-values of quartzitic gravels.

Sample ID	Average *k*-values			R^2
	k_1	k_2	k_3	
NNR	531	0.0785	2.3883	0.99
SNR	503	0.1820	1.7182	0.99
OAR	535	0.1843	1.3425	0.98
AAR	676	0.1016	-0.1257	0.56
SER	958	0.2168	0.2772	0.94
ECR	441	0.0998	3.1590	0.97
MGR	568	0.0636	1.4810	0.94

Contrary to what have been reported in literature, the regression analysis returned positive k_3 values instead of negative values for six out of the seven samples. The resilience response of a material to increase in octahedral shear stress determines the positivity or negativity of its k_3 value. As increasing shear stress is expected to lead to the softening of materials, and hence a decreasing resilient modulus, k_3 should be negative, but this phenomenon was not exhibited by the quartzitic gravels as they showed stress-hardening behaviour with increase in octahedral shear stress (see for example **Figure 9** and **Figure 10**). The only exception to this behaviour was sample AAR from Anyanso which initially hardened at low shear stress levels but then softened at increased shear stress levels (**Figure 11**). It is not exactly clear what accounted for this unexpected behaviour but the coefficient of multiple determination (R^2) associated with the regressions analysis exceeded 0.90 for the investigated materials except for the sample from

Anyanso (AAR) for which the value was a low 0.56 (see Table 4).

The MEPDG recommends that test results and equipment should be checked for possible errors and/or test specimen disruption if R^2 for a particular test specimen is less than 0.90 [9]. Because this condition did not prevail for practically all the soils except one, it is difficult to ascribe the sample behaviour under increasing shear stress to test/equipment error. And what was even more surprising is the fact that the sample which probably could be associated with test error because of a low R^2 value was rather the gravel sample for which the analysis returned negative k_3 value in accordance with expectation based on literature. For the other samples, the higher R^2 values from the regression analysis, therefore, appeared to rule out any test errors during the triaxial test in spite of their k_3 values being positive.

Given the stress state in the repeated load triaxial test for which the intermediate principal stress (σ_2) is equal to the minor principal stress or confining pressure (σ_3), the generalized M_R constitutive model (Equation (2)) may be re-written solely as a function of deviator stress (σ_d) and confining pressure as follows:

$$M_R = k_1 P_a \left(\frac{\sigma_d + 3\sigma_3}{P_a} \right)^{k_2} \left(\frac{\sqrt{2}\sigma_d}{3P_a} + 1 \right)^{k_3} \tag{4}$$

It is reported that for granular materials, the resilient modulus increases with increasing confining pressure [28] and deviator stress [3] [29]. On the strength of those observations, and with reference to Equation (3), two possibilities could exist regarding the nature of the k_3 parameter: 1) if k_3 were to be positive, the observations above per references [3] [28] [29] would still hold but perhaps up to some stress thresholds as stress-hardening could not be perpetuated *ad infinitum* with increasing deviator stress and confining pressure; 2) if k_3 were to be negative, the observation would also hold if and only if the increase in the value of M_R due to the k_2 term (bulk stress term) more than offsets the decrease caused by the k_3 term (octahedral shear stress term). It stands to reason, therefore, that in this case, M_R would only begin to exhibit decreasing values beyond the point where the decrease in its value caused by the k_3 term overrode the increase caused by the k_2 term. In either of the two possibilities considered above, stress-hardening behaviour of granular materials should still be anticipated only up to some threshold stress levels as confining and deviator stresses increased during the triaxial testing. This probably could be the behaviour being exhibited by Sample AAR (Figure 11). At this stage, however, no definite conclusion regarding the gravels' k_3 values can be made other than a recommendation to carry out more extensive studies and expand the gravel sample base.

4.4. Correlation of *k*-Values with Gravel Properties

The following correlation equations were the outcome of the regression analysis involving *k*-values from the resilient modulus test and the index physical prop-

erties, namely, percent finer than the 9.5 mm and 2.0 mm sieves, liquid limit, maximum dry density, and optimum moisture content of the samples.

$$k_1 = 2531.94 - 3.11P_{9.5} + 5.50P_{2.0} - 21.09LL + 0.81\rho_{dmax} + 71.30w_{opt}, \quad R^2 = 0.95 \quad (5)$$

$$k_2 = -3.54 - 0.005P_{9.5} + 0.008P_{2.0} + 0.014LL + 0.0014\rho_{dmax} + 0.05w_{opt}, \quad R^2 = 0.95 \quad (6)$$

$$k_3 = 8.10 + 0.12P_{9.5} - 0.12P_{2.0} + 0.227LL - 0.0055\rho_{dmax} - 0.72w_{opt}, \quad R^2 = 0.95 \quad (7)$$

where,

k_1, k_2 and k_3 = k-values of gravel

$P_{9.5}$ = percentage of gravel material passing 9.5 mm sieve

$P_{2.0}$ = percentage of gravel material passing 2.0 mm sieve

LL = liquid limit

ρ_{dmax} = maximum dry density

w_{opt} = optimum moisture content (%)

The high R^2 values obtained for the regression suggest a strong correlation between the k-values and the index properties of the gravel materials. Even though the recommendation is to evaluate k-values from M_R tests, the above correlations become especially useful when capability and opportunity for repeated load tests do not exist.

5. Conclusion

This study set out to determine the resilient modulus of quartzitic natural gravels using repeated load triaxial test and to evaluate the k-values (k_1, k_2, k_3) of the materials thereof. The quartzitic gravels used in the study came from limited borrow pit sources and did not cover the whole range of natural gravel types generally used for flexible pavement construction in the country which also include lateritic gravels. The k-values from the resilient modulus tests were used to develop correlation equations involving the index physical properties of the gravels to help predict resilient modulus when the materials are used as pavement layer materials. The regression analysis returned k_1 values which ranged between 441 and 958 with a mean of 516, k_2 values which ranged between 0.0636 and 0.2168 with a mean of 0.1216 and k_3 values which ranged between 0.1257 and 3.1590 with a mean of 1.723. The positive nature of the constant k_3 is in contrast to the mostly negative values reported in literature on the parameter. Nonetheless, all the k-values correlated highly with the following index properties of the gravels: the percentages of gravel material passing the 9.5 mm and 2.0 mm sieves, liquid limit, maximum dry density and optimum moisture content. The gravels exhibited stress hardening behaviour for the most part since the resilient moduli from the triaxial tests increased with increasing confining pressure, bulk stress, and octahedral shear stress. Even though the recommendation is to evaluate k-values from resilient modulus tests, the correlations developed in this study, despite the limited sample base, may permit useful estimates of the constants to be made for such gravels under any stress state and physical condition, especially

when capability and opportunity for conducting resilient modulus tests do not exist. Further work, however, is recommended to cover the entire range of natural gravels deposits in the country and investigate into detail, the nature of the constant k_3 with increasing octahedral shear stress.

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