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Resilient Modulus of Compacted Lateritic Soils from Senegal at OPM Conditions

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ABSTRACT

Repeated load triaxial tests were performed on five compacted gravel lateritic soils collected from different locations in Senegal: Sébikotane, Dougar, Pâ Lo, Mont-Rolland and Ngoundiane. The study revealed that resilient modulus decreases with the increase of the bulk and deviatoric stress in constant confining pressure. In addition, resilient modulus increases with the percentage of cement for appreciably equal contents of moisture. This effect tends to stop for higher stress. Besides, correlations were made with some models of resilient modulus such as the Uzan-Witczack model (Witczack and Uzan, 1988 [1]) and the National Highway Research Program (NCHRP) model (2004 [2]). The study confirms that both models give very good results with the best correlations being obtained with the Uzan-Witczack model.

Keywords: Gravel Lateritic Soils; Resilient Modulus; Mechanical Behaviour; Pavement; Cyclic Triaxial Test; Mechanistic Design

1. Introduction

Since several decades, gravel lateritic soils have been used in road pavements in tropical countries. In Senegal, the increased use of this material has lead to a rarefaction of careers of good quality. Therefore, the rationalization of existing resources requires a real knowledge of lateritic soils. This problem has drawn the attention of the researchers who have done a lot to understand the mechanical behaviour of lateritic gravels (Samb, 1986 [3]; Fall, 1993 [4]; Fall, Sawangsuriya, Benson, Edil and Bosscher, 2007 [5]).

Under cyclic loading, road materials are characterized by a fast increase of permanent strains from the first cycles of loading. As the number of cycles increases, these deformations stabilize and the behaviour becomes essentially reversible allowing to define a module called "resilient modulus" (Yoder and Witzack, 1975 [6]; Paute, Hornych and Benaben, 1994 [7]; Martinez, 1990 [8]). Resilient modulus represents the unloading modulus after several repeated cycles of loading, allowing to simulate road traffic (**Figure 1**):

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

$\Delta\sigma_d = \sigma_1 - \sigma_3$ = Deviatoric stress; σ_1 = Major principal stress; σ_3 = Minor principal stress et $\Delta\varepsilon_a$ = Resilient axial strain.

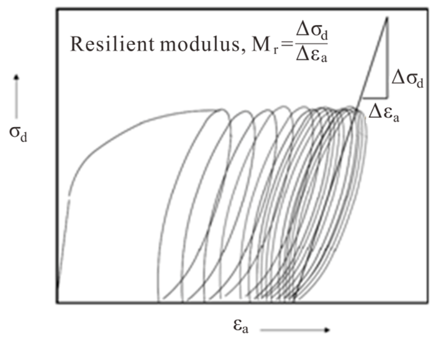

Figure 1. Definition of resilient modulus (Hopkins, Beckham and Sun, 2007 [9]).

In order to study the cyclic behaviour of gravel lateritic soils of Senegal, repeated load triaxial tests were conducted on soils collected from Sébikotane, Dougar. Ngoundiane, Pâ Lo and Mont-Rolland.

In this paper, we aim to present the experimental protocol, the determination of average resilient modulus as well as the evolution of the resilient modulus according to the level of stress and the percentage of cement. Furthermore, correlations are made depending on some generalized resilient modulus models such as Uzan-Witczack (Witczack and Uzan, 1988 [1]) and the NCHRP model (2004 [2]).

2. Material and Methods

Standard laboratory road tests were performed to classify the materials and to determine their properties. Laboratory tests consisted of particle size analysis, consistency limits. Modified Proctor Compaction test and Californian Bearing Ratio test.

The cyclic triaxial tests were then conducted to determine the resilient modulus of these soils. For this purpose, unbound gravel lateritic soils and that improved with cement (1%, 2% and 3%) were compacted in 95% of Modified Optimum Proctor (OPM) that corresponds to the value retained in road specifications of base layers. Samples are 70 mm in diameter and 180 mm height.

The **Tables 1** and **2** respectively present the summary of results of the identification test sand the nomenclature of test specimens for triaxial tests.

The experimental procedure is described by the NCHRP (2004 [2]). The triaxial apparatus in repeated loads is the experimental reference device used to characterize the mechanical behaviour of roads materials (**Figure 2**).

The study of resilient behaviour includes two phases. In the first one, the test begins with a conditioning which consists in applying a minimum of 1000 repetitions of a load equivalent to a cyclic stress of 207 kPa using a haversine-shaped 0.1-second load pulse followed by a 0.9-second rest period.

During the second phase, the test specimen is submitted to cyclic loading by applying a number of 100 repetitions of the corresponding cyclic axial stress using the same load pulse according to various loading paths given in **Table 3**.

The axial deformations are measured by two external and two internal displacements sensors called "Linear Variable Differential Transformers" (LVDT). The average deformations for each LVDT separately for the last five cycles are reported then the resilient modulus is calculated (NCHRP, 2004 [2]).

It is important to notice that the results of the resilient modulus exposed below are the ones obtained with the external deformation sensors.

3. Results and Discussion

3.1. Comparision between Static and Cyclic Modulus

Table 4 gives the values of average resilient modulus for all the materials as well as the maximal modulus found for the Unconfined Compression Test (UCT) realized on the same types of test tubes for the sites of Mont-Rolland. Dougar and Pâ Lo. In general, it is observed that resilient modulus is much more important for cyclic triaxial test than for the unconfined compression test. Indeed, the application of an increasing monotonous load is more unfavorable than the application of a cyclic load where the sample gets back part of the deformation. Furthermore, the conditioning made in the cyclic test allows increasing the stiffness of the material; which is not the case for the compression test.

On the other hand, the ratio between resilient modulus and Young modulus for the compression test seems much more high for the raw material (between 12% and 17%) and decreases with the percentage of cement. Indeed. The increase of the stiffness of the material with the percentage of cement decreases the sample deformability.

These results are very important because they show the reason why it is necessary to take into account the real stiffness of the gravel lateritic soils in mechanistic design. The use of the static modulus does not seem to be

Table 1. Identification test results of collected gravel lateritic soils.

Carrier	Sebikotane		Mont-Rolland		Ngoundiane		Pa Lô		Dougar	
	Before CBR	After CBR	Before CBR	After CBR	Before CBR	After CBR	Before CBR	After CBR	Before CBR	After CBR
% elements < 80 mm	15	17	27	30.5	20.5	24	24	31	19	27
% elements < 2 mm	6	6	7	8	9	14	12	13	10	12
D_{60} (mm)	12.00	6.00	5.20	4.80	9.20	6.00	6.00	3.00	2.20	4.00
D_{30} (mm)	3.40	0.25	0.28	0.08	1.30	0.20	3.50	0.08	0.17	0.09
D_{10} (mm)	0.04	0.05	0.02	0.008	0.04	0.02	0.03	0.02	0.02	0.02
$C_u = D_{60}/D_{10}$	300.00	120.00	325.00	600.00	255.56	300	230.77	130.43	95.65	210.53
$C_c = (D30)^2/(D10.\ D60)$	24.08	0.21	0.94	0.17	5.10	0.33	78.53	0.08	0.57	0.11
W_P	12.5	11.0	21.0	17.0	25.5	24.5	29.0	30.0	13.5	13.0
I_P	7.50	11.00	27.00	37.00	26.50	28.50	25.50	33.00	16.50	13.00
A_c	1.25	1.83	3.86	4.63	2.94	2.04	2.13	2.54	1.65	1.08
	normal		active		active		active		active	

Table 2. Nomenclature of test specimens for load triaxial tests and unconfined compression tests.

Carrier	Raw material	1% cement	2% cement	3% cement
Sébikotane	Sb_cr	Sb_cr	Sb_cr	Sb_cr
Dougar	Dg_cr	Dg_cr	Dg_cr	Dg_cr
Ngoundiane	Ng_cr	Ng_cr	Ng_cr	Ng_cr
Pâ Lo	Pa_cr	Pa_cr	Pa_cr	Pa_cr
Mont-Rolland	Mr_cr	Mr_cr	Mr_cr	Mr_cr

Table 3. Test sequence for base or subbase materials—Procedure Ia of NCHRP (2004) [2].

Sequence	Confining pressure (kPa)	Contact stress (kPa)	Cyclic stress (kPa)	Maximum stress (kPa)	Number of repetitions
0	103.5	20.7	207.0	227.7	1 000
1	20.7	4.1	10.4	14.5	100
2	41.4	8.3	20.7	29.0	100
3	69.0	13.8	34.5	48.3	100
4	103.5	20.7	51.8	72.5	100
5	138.0	27.6	69.0	96.6	100
6	20.7	4.1	20.7	24.8	100
7	41.4	8.3	41.4	49.7	100
8	69.0	13.8	69.0	82.8	100
9	103.5	20.7	103.5	124.2	100
10	138.0	27.6	138.0	165.6	100
11	20.7	4.1	41.4	45.5	100
12	41.4	8.3	82.8	91.1	100
13	69.0	13.8	138.0	151.8	100
14	103.5	20.7	207.0	227.7	100
15	138.0	27.6	276.0	303.6	100
16	20.7	4.1	62.1	66.2	100
17	41.4	8.3	124.2	132.5	100
18	69.0	13.8	207.0	220.8	100
19	103.5	20.7	310.5	331.2	100
20	138.0	27.6	414.0	441.6	100
21	20.7	4.1	103.5	107.6	100
22	41.4	8.3	207.0	215.3	100
23	69.0	13.8	345.0	358.8	100
24	103.5	20.7	517.5	538.2	100
25	138.0	27.6	690.0	717.6	100
26	20.7	4.1	144.9	149.0	100
27	41.4	8.3	289.8	298.1	100
28	69.0	13.8	483.0	496.8	100
29	103.5	20.7	724.5	745.2	100
30	138.0	27.6	966.0	993.6	100

Table 4. Comparision between static and cyclic modulus.

Material	Average resilient modulus (MPa)	Maximal Young modulus (UCT) (MPa)	Cyclic modulus/ Static modulus (MPa)
Ng_cr	903	62	14.6
Ng_1 C	35	66.75	0.5
Ng_2 C	531	137	3.9
Ng _3 C	417	218	1.9
Mr_cr	802	47	17.1
Mr _1 C	456	64.33	7.1
Mr_2 C	-	-	-
Mr_3 C	360	48	7.5
Dg_cr	122	-	-
Dg_1 C	116	-	-
Dg_2 C	168	-	-
Dg_3 C	258	-	-
Pa_cr	695	56.7	12.3
Pa_1 C	497	34.17	14.5
Pa_2 C	240	33.17	7.2
Pa_3 C	275	75.8	3.6
Sb_cr	589	-	-
Sb_1 C	890	-	-
Sb_2 C	890	-	-
Sb_3 C	691	-	-

Figure 2. Triaxial apparatus for cyclic loading—University of Madison (Bâ, 2012 [10]).

any more suitable for pavement design.

3.2. Resilient Modulus According to the Level of Stress

Previous investigations, from the earlier studies reported by Williams (1963) [11] to the recent studies by Kolisoja (1997) [12], have shown that generally stress level is the factor that has the most significant impact on resilient properties of road materials (Lekarp, Isacsson and Dawson, 2000 [13]). In this part, the effect of the bulk and deviatoric stress as well as the percentage of cement are analyzed.

3.2.1. Effect of Bulk Stress

The **Figures 3** and **4** show that resilient modulus decreases inversely with the increase of bulk stress in constant confining pressure. Indeed, gravel lateritic soils have a low void ratio after compaction because of the presence of fine grains which ensures the cohesion. After application of the load and a generation of fine particles (because of the disintegration of pisolithes), new surfaces of discontinuities are created, involving an increase of the void ratio and therefore a loss of resistance.

3.2.2. Effect of Deviatoric Stress

The **Figures 5** and **6** show that resilient modulus decreases according to deviatoric stress in constant confining pressure. These results confirm the fact, the increase of the level of stress in the material generally leads to a decrease of the rigidity of gravel lateritic soils samples.

3.2.3. Effect of the Percentage of Cement

The effect of the percentage of cement was studied for gravel lateritic soils of Dougar, Sebikotane and Pâ Lo (**Figures 7-9**). The observations showed an increase of resilient modulus according to the percentage of cement for the materials of Dougar. However, this result is not confirmed for gravel lateritic soils of Sebikotane and Pâ Lo.

Besides, the observation of the moisture contents measured as in **Table 5** showed that resilient modulus decreases with the important increase of the moisture content (Sebikotane 3%. Pâ Lo 3%). Furthermore, for materials from Dougar, the resilient modulus increases with the percentage of cement, for appreciably equal moisture contents. The resilient modulus seems to be affected more by moisture content effect than that of the percentage of cement.

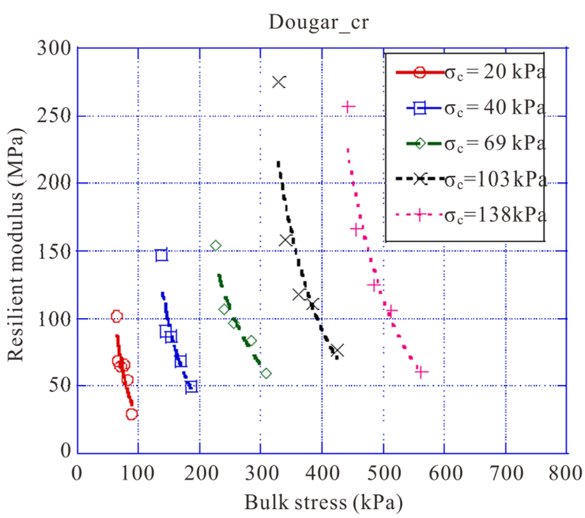

Figure 3. Variation of resilient modulus with bulk stress—Unbound gravel lateritic soil of Dougar.

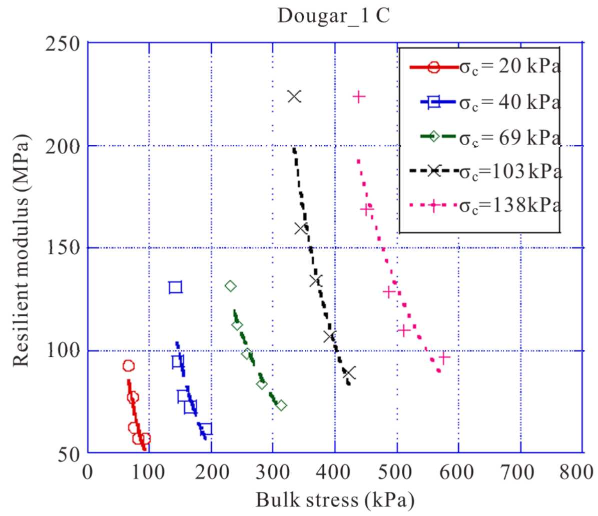

Figure 4. Variation of resilient modulus with bulk stress—Dougar 1% cement.

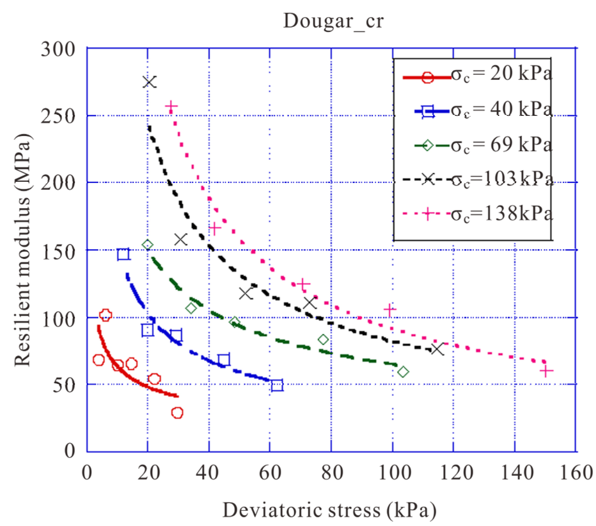

Figure 5. Variation of resilient modulus with deviatoric stress—Unbound gravel lateritic soil of Dougar.

4. Review of Resilient Modulus Models

Prediction of roads behaviour requires stress-strain relationships modelling by constitutive laws. Several models were proposed in the literature. The *K-θ* model (Seed, Mitry, Monismith and Chan, 1967 [14]; Brown and Pell, 1967 [15]; Hicks and Monismith, 1971 [16]) which is one of the most popular models expresses the resilient modulus according to the bulk stress:

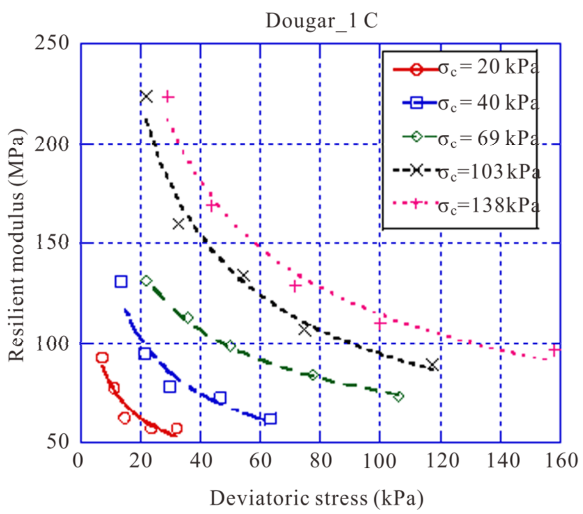

Figure 6. Variation of resilient modulus with deviatoric stress—Dougar 1% cement.

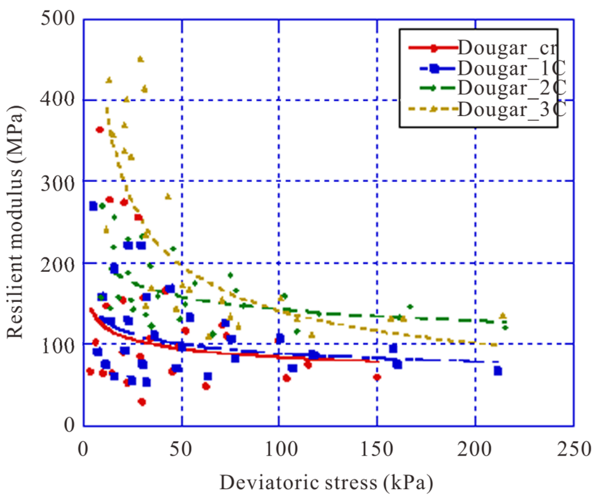

Figure 7. Effect of the percentage of cement on resilient modulus of gravels lateritics soils of Dougar.

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

$\theta = (\sigma_1 + 2\sigma_3) = \text{Bulk stress}$; *$k_1$ and k_2 are the regression constants*.

It is widely used to model the resilient modulus as a function of the level of stress applicable to the granular materials. However, the *K-θ* model presents some disavantages. Uzan (1985 [17]) introduces the deviatoric stress as the additional component according to the effect of shearing behaviour and obtains better correlations with the trial results:

$$M_r = k_1 \left(\frac{\theta}{p_a} \right)^{k_2} \left(\frac{\sigma_d}{p_a} \right)^{k_3} \tag{3}$$

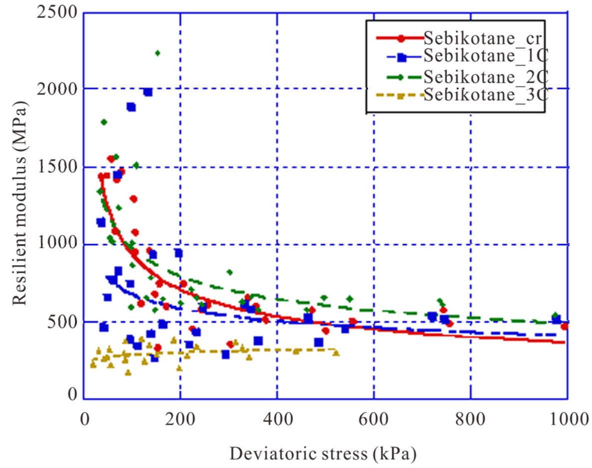

Figure 8. Effect of the percentage of cement on resilient modulus of gravels lateritics soils of Sebikotane.

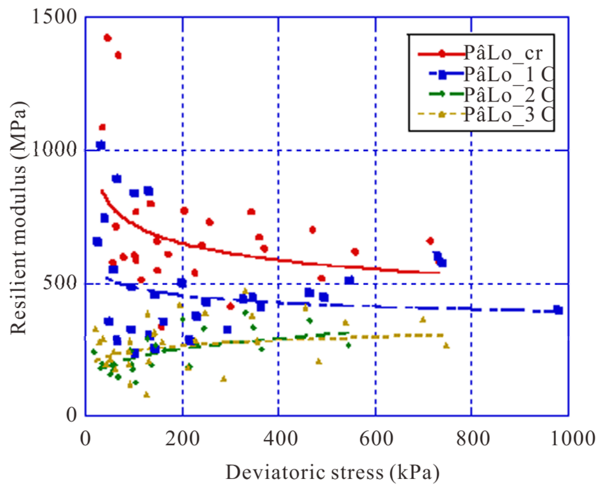

Figure 9. Effect of the percentage of cement on resilient modulus of gravels lateritics soils of Pâ Lo.

Table 5. Water content of soils samples after tests.

Material	Dougar	Sebikotane	Pâ Lo
Raw	7.40	11.91	9.45
1%	9.05	10.11	8.15
2%	9.58	12.66	11.85
3%	9.65	14.40	13.48

$\sigma_d = \sigma_1 - \sigma_3 = \text{deviatoric stress}$; k_1, k_2 and k_3 regression constants.

Witczack and Uzan (1988 [1]) proposes an improvement of the model of Uzan (1985 [17]) the by replacing the octahedral shear stress:

$$M_r = k_1 p_a \left(\frac{\theta}{p_a} \right)^{k_2} \left(\frac{\tau_{oct}}{p_a} \right)^{k_3} \tag{4}$$

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

= Octahedral shear stress;

k_1, k_2 and k_3 regression constants.

A general form for these proposed models is the Andrei model (1999 [18]):

$$M_r = k_1 p_a \left(\frac{\theta - 3k_6}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + k_7\right)^{k_3} \quad (5)$$

$k_1 - k_7$ = regression constants.

The Andrei model (1999 [18]) was then adopted by the National Highway Research Program (NCHRP, 2004) [2] in its simplified version ($k_6 = 0$ and $k_7 = 1$).

$$M_r = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + 1\right)^{k_3} \quad (6)$$

k_1, k_2 and k_3, regression constants.

It is worth noting that gravel lateritic soils consisted of soft and hard concretions as well as quartzites in a matrix consisting of fines particles composed of a mixture of clays and graves. Therefore, both the Uzan-Witczack model (1988 [1]) and the NCHRP one (2004 [2]) which can be adapted to the fine soils as well as to the granular soils seem to well fit with gravel lateritic soils that are located between these two types of material. These two models were so retained for correlation studies.

Correlations with Uzan-Witczack (1988) [17] and NCHRP (2004) Models [2]

The results of the correlations of the resilient modulus for Uzan-Witczack (1988 [1]) and NCHRP (2004 [2]) models are presented below (**Tables 6** and **7**). It is observed that the found coefficients of regression are very close to 1 (between 0.902 and 0.999). This observation means that both models give very good correlations of the resilient modulus and can be used to model the resilient behaviour of gravel lateritic soils. However, the comparison by box-diagram of the values of regression coefficients of both models (**Figure 10**) showed that the Uzan-Witczack model (1988 [1]) give better correlations than the NCHRP model (2004 [2]).

5. Conclusions

Cyclic triaxial tests made on gravels lateritics soils allowed to have a number of very important results.

The study of the cyclic behavior of gravel lateritic soils confirms the importance of the effect of stress level on resilient modulus. Indeed, resilient modulus decreases inversely with the increase of bulk and deviatoric stress. However, the effect of cement percentage on the increase

Table 6. Coefficients k_i and r^2 obtained with Uzan-Witczack model (1988) [1].

Material	Regression constants			
	k_1	k_2	k_3	r^2
Ng_cr	837275.78	0.13	−0.33	0.981
Ng_1 C	66127.31	0.00	−0.06	−0.963
Ng_2 C	279074.39	0.65	−0.50	0.972
Ng _3 C	170562.25	0.88	−0.56	0.988
Mr_cr	697580.89	0.36	−0.72	0.984
Mr _1 C	281407.48	0.50	−0.33	0.970
Mr_2 C	66126.88	0.05	0.00	0.902
Mr_3 C	197787.45	0.52	−0.28	0.970
Dg_cr	16540.50	1.14	−0.85	0.981
Dg_1 C	24539.62	0.95	−0.66	0.991
Dg_2 C	80614.98	0.42	−0.38	0.967
Dg_3 C	70174.97	0.37	−0.87	0.971
Pa_cr	402316.60	0.48	−0.33	0.961
Pa_1 C	131998.56	1.21	−0.88	0.976
Pa_2 C	131730.92	0.48	0.00	0.968
Pa_3 C	77074.25	1.22	−0.67	0.967
Sb_cr	320926.05	0.78	−1.16	0.979
Sb_1 C	150787.25	1.52	−1.27	0.976
Sb_2 C	1143330.27	0.00	−0.42	0.944
Sb_3 C	150919.73	0.63	−0.37	0.986

Table 7. Coefficients k_i and r^2 obtained with the NCHRP (2004) [2] model.

Materials	Model parameters			
	k_1	k_2	k_3	r^2
Ng_cr	11280.77	0.07	−0.62	0.941
Ng_1 C	91.98	0.90	0.00	0.981
Ng_2 C	3800.59	0.59	−0.85	0.948
Ng _3 C	3185.14	0.66	−0.95	0.939
Mr_cr	16538.34	0.04	−1.28	0.936
Mr _1 C	4031.72	0.39	−0.59	0.944
Mr_2 C	140.44	0.84	0.00	0.963
Mr_3 C	2552.40	0.46	−0.51	0.960
Dg_cr	890.25	0.79	−3.17	0.983
Dg_1 C	892.94	0.79	−2.94	0.994
Dg_2 C	1724.44	0.36	−1.79	0.970
Dg_3 C	5536.06	0.19	−5.84	0.974
Pa_cr	4791.10	0.49	−0.61	0.968
Pa_1 C	2404.21	0.57	−0.54	0.918
Pa_2 C	1126.03	0.64	−0.22	0.968
Pa_3 C	1108.12	1.03	−0.84	0.976
Sb_cr	15591.42	0.59	−3.12	0.959
Sb_1 C	4421.89	1.84	−3.42	0.968
Sb_2 C	12033.82	0.00	−0.57	0.912
Sb_3 C	2269.47	0.52	−0.70	0.980

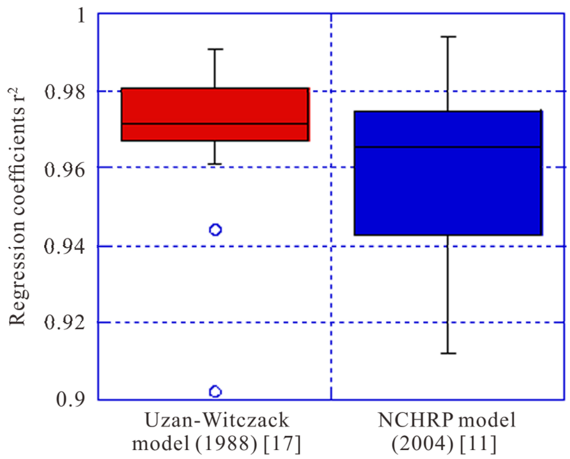

Figure 10. Comparison of the models coefficients of correlation.

of the stiffness of the material is not confirmed for all the materials and differs from one gravel to another. On the other hand, the effect of the moisture content seems to affect the values of the resilient modulus in a sensitive way.

These results play a key important role in the prediction of the behaviour model of resilient modulus of gravel lateritic soils. Indeed, the Uzan-Witczack (1988) [1] and the NCHRP model (2004) [2] had allowed very good correlations with better results with the first one. These models can be used for finite element modelling of pave--ments.

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