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Non-linear elastic behavior of lateritic gravelly road material of Benin

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ABSTRACT
This work deals with the nonlinear elastic behavior of the lateritic gravelly. This material is used in Sub-Saharan Africa for the construction of roadway layers (foundation and base). In Benin, the materials are selected by reference to the CEBTP rules of 1972, revised in 1984. Then the sizing of pavements is done with ALIZE III whose parameters (Young's modulus E and Poisson's ratio υ) are parameters of untreated severe used in Western countries. Hence the need to conduct this study. The lateritic gravelly was modeled by Hardin and Drnevich model previously characterized in the laboratory. The essential parameters of this model are determined using the non-linear least squares method applied to the experimental data collected from the direct shear test, and statistical tests are then performed to validate the model. These essential parameters are the maximum tangential stress τ_{max} and maximum shear modulus G_{max}. The critical values obtained at 95% of the OPM are G_{max} = 10 MPa and τ_{max} = 0.323 MPa then those at 100% of the OPM are G_{max} = 11.111 MPa and τ_{max} = 0.303 MPa. These parameters associated with the oedometric modulus enabled us to determine the Young's modulus E = 32.46 MPa and the Poisson's ratio υ = 0.40 of the laterally gravely road material at 100% of the OPM. Finally, we found that the calculated Young's modulus is less than six times that of ALIZE III but no the Poisson's ratio is almost the same. We can say that the value of Young's modulus found in the library of ALIZE III does not correspond to the true value of laterite. This value of ALIZE III is at the origin of the early degradation of our roads built with laterite.

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Introduction

Gravel lateritic materials are the most used materials in road construction as a form layer, foundation or base course according to their quality in developing countries, particularly in Benin[1-2]. They are also used for the construction of dikes and access works etc.
The abundance of this material and its lower operating cost allow its use in sub-Saharan Africa despite its variable resistance to climatic zones[3].
In recent years, we see early degradations of our pavements built in gravel lateritic despite the application of sizing rules CEBTP and LCPC-SETRA [2-4]. These sizing rules are based on results obtained mainly on Western Untreated Graves. However, the characteristics of the materials used for the design of the pavement differ from one zone to another. These early deteriorations force the authorities to set up, sooner than expected, large means for road maintenance.
The use of gravel laterite in road construction must be at the center of these road projects to allow our roads to reach their life before deteriorating.
Faced with this problem, we have chosen to model the gravelly laterite by the hyperbolic model of Hardin and Drnevich (1972) [5] and then to determine the Young's modulus and the Poisson's ratio of the class 0/5 of the material. Remember that this part is more deformed and its results are representative [6].
In this work we present the geotechnical results of the lateritic gravelly, the essential parameters of the model as well as the Young's modulus and the Poisson's ratio.

Experimental

Framework of the study

The figure 1 below gives the position of the site. Djidja is a town located south of Benin in the Zou department and 100 m above sea level. It is located about 36 km north-west of the city of Abomey, in the department of Zou. Its geographical coordinates are: Latitude: 7 ° 20 '40' 'N; Longitude: 1 ° 56 '00' 'E, totaling 131 km², or 0.114% of the area of Benin. The climate is subequatorial, tending toward Sudano-Guinean in the northern parts. As a result, in these parts the two rainy seasons become practically one with average rainfall varying between 900 and 1200 mm.

Characteristics of the lateritic gravelly

A sample of lateritic gravelly material was made in the Djidja quarry in Benin for laboratory testing. The tests carried out as part of this research can be grouped in two: complete identification tests such as particle size analysis (NF EN 933-1), real density (NF EN 1097-6), Atterberg limit (NF P94-051), organic matter content (XP P 94-047), modified Proctor (NF P94-093), CBR (NF P94-78) and the mechanical tests as shear (NF P94 -71-1) of dimensions 60mmx60mmx24.5 mm and the oedometric test (NF94-090) with an oedometric cell 70 mm in diameter and 20

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mm in height. Classifications GTR (NF P 11-300) was adopted.

Figure 1: Cahiers villages and city districts of the departement of Zou(CVQVDZ, 2016)

Hyperbolic model of Hardin and Drnevich (1972)

Hardin and Drnevich (1972) constructed their model by putting the hyperbolic expression proposed by Kondner (1963) in the form of a relation between shear stress τ and shear strain γ. This model is defined by:

$$\tau = \frac{\gamma}{\frac{1}{G_{max}} + \frac{\gamma}{\tau_{max}}} \quad (1)$$

where is meant by:
G_{max}, the maximum shear modulus,
τ_{max}, the maximum shear stress.

They introduced the concept of reference shear deformation γ_r below:

$$\gamma_r = \frac{\tau_{max}}{G_{max}} \quad (2)$$

The secant module G_s is defined by:

$$G_s = \frac{\tau}{\gamma} \quad (3)$$

By replacing the reference shear expressions γr and secant modulus Gs in equation (1), this equation becomes:

$$G_s = \frac{G_{max}}{1+\frac{\gamma}{\gamma_r}} \quad (4)$$

Hardin and Drnevich, (1972) have proposed a similar but more complex expression for secant shear modulus Gs:

$$G_s = \frac{G_{max}}{1+\gamma_h} \quad (5)$$

with

$$\gamma_h = \frac{\gamma}{\gamma_r}\left[1 + a\exp\left(-b\frac{\gamma}{\gamma_r}\right)\right] \quad (6)$$

where is meant by: a and b are parameters deduced from test results.

This model therefore shows five parameters namely: G_{max} ; τ_{max} ; γ_r : a and b. But according to [7-8] the hyperbolic model of Hardin and Drnevich (1972) is based on two parameters that have a real physical significance: the maximum shear modulus G_{max}, and the maximum shear stress of soils τ_{max}. In this research we have determined these two parameters by the non-linear least squares method then γ_r. The different iterations were conducted to obtain G_{max} and τ_{max}, then the validation of this model will be done by the coefficient of determination and the normality of the residuals [9].

Determination of the Poisson's ratio and Young's modulus

According to Gérard Degoutte and Paul Royet (2007) [10], the appropriate formulas for the determination of the Young's modulus and the Poisson's ratio starting from the oedometric and direct shear tests are as follows:

$$E = 2G(1 + \upsilon) \quad (7)$$

$$E = E_{oed}\frac{(1+\upsilon)(1-2\upsilon)}{1-\upsilon} \quad (8)$$

where is meant by:
G, the shear modulus,
E, the Young's modulus,
υ, the Poisson's ratio,
E_{oed}, the oedometric module.

From Equations (7) and (8) we can write

$$2G(1 + \upsilon) = E_{oed}\frac{(1+\upsilon)(1-2\upsilon)}{1-\upsilon} \quad (9)$$

After transformation, equation (9) becomes:

$$\upsilon = \frac{E_{oed}-2G}{2(E_{oed}-G)} \quad (10)$$

Results and Discussion

Geotechnical characterization of lateritic gravelly

As part of this study, tests on the lateritic gravelly were made in the laboratory and three samples each time to verify the reliability of the results. The results of the complete identification tests are shown in Table 1, and the results of the mechanical tests (sicaillement and oedometric test) that have been performed are summarized in Table 2.

The classification results according to the GTR system as a function of the plasticity index IP and the fines content C80 µm of the soil give the class B6, It is sand and serious clayey to very clayey; their behavior is similar to that of a fine soil having the same plasticity as fines, but with a greater sensitivity to water due to the presence of the fine fraction in greater quantity.

After sieve analysis, grain size curves (Figure 3) were plotted which fit entirely into the typical lateritic soil zones of the foundation and base layer CEBTP (1984) [2].We also notice that all the layers are spread out and follow the same pace. On the other hand, the organic matter content is less than 3% which indicates that our gravelly is inorganic[11]. The average specific weight of the solid grains is 2.92 g / cm³ so it is a good material that can be used as a pavement for low traffic roads. Moreover, the material has a liquid limit of 45.3% and a plasticity index 21, we can say that this material is moderately plastic and these values correspond to lateritic foundation soils defined by the CEBTP (1984) [2]. Modified Proctor tests were performed to determine the bearing capacity of the gravelly. The average values of the moisture content and the dry density are respectively 9.53% and 2.21 t / m³.

Table 1: Full Identification Test Results

N⁰	Wnat (%)	Granulometry			Atterberglimit			Organicmaterial	Real density (g/cm³)	ModifiedProctor		CBR index after 96 h of imbibition		
		Dmax (mm)	< 2mm (%)	< 0,08mm (%)	WL	WP	IP			W_{opm} (%)	γ_{opm} (t/m³)	$I_{CBR\,100}$ %	$I_{CBR\,95}$ %	$I_{CBR\,90}$ %
1	3,82	40	32,1	14,6	45	25	20	1,36	2,91	9	2,22	103	86	34
2	3,6	40	38,3	16,7	45	24	21	1,36	2,91	9,4	2,21	102	82	38
3	4,01	31,5	38,6	17,7	46	24	22	1,35	2,94	10,2	2,20	97	78	41
Average	3,81	37,2	36,3	16,3	45,3	24,3	21	1,36	2,92	9,53	2,21	101	82	38
Standard eviation	0,21	4,9	3,7	1,6	0,6	0,6	1	0,01	0,02	10,2	2,2	3	4	4

Table 2: Results of mechanical tests

	N⁰	*Shear test*			*Oedometer test*							
		Cu (kPa)	φu (⁰)	eo	σ_p' (kPa)	c_c	c_g	c_v (m²/s)	E_{oedmoy} (MPa)	$K0_o$ (m/s)	γ_d (kN/m³)	γ (kN/m³)
95% Modified Proctor optimum (OPM)	1	12,886	29,99	0,54	235	0,108	0,01	$9{,}22.10^{-7}$	69.2	$6{,}53.10^{-10}$	1,798	2,013
	2	12,524	29,98	0,46	230	0,096	0,01	$1{,}02.10^{-6}$	66.1	$6{,}68.10^{-10}$	1,902	2,13
	3	11,872	30,08	0,44	232	0,106	0,01	$0{,}984.10^{-6}$	67	$6{,}427.10^{-10}$	1,883	2,111
	Average	12,428	30,02	0,48	232,333	0,103	0,01	$0{,}984.10^{-5}$	67,433	$6{,}546.10^{-10}$	1,861	2,085
	Standardiviation	0,514	0,06	0,053	2,517	0,006	0,000	$0{,}054.10^{-6}$	1,595	$0{,}127.10^{-10}$	0,055	0,063
100% Modified Proctor optimum (OPM)	1	16,232	30,68	0,36	135	0,051	0,01	$7{,}122.10^{-7}$	70	$7{,}28.10^{-10}$	2,037	2,287
	2	14,227	28,71	0,35	138	0,051	0,01	$4{,}847.10^{-7}$	71.7	$6{,}906.10^{-10}$	2,056	2,296
	3	16,932	28,98	0,4	140	0,05	0,01	$5{,}008.10^{-7}$	67	$7{,}034.10^{-10}$	2,04	2,28
	Average	15,797	29,47	0,37	137,667	0,051	0,01	$5{,}659.10^{-7}$	69,567	$7{,}073.10^{-10}$	2,044	2,288
	Standardiviation	1,404	1,07	0,03	2,517	0,001	0,000	$1{,}269.10^{-9}$	2,380	$1{,}901.10^{-11}$	0,010	0,008

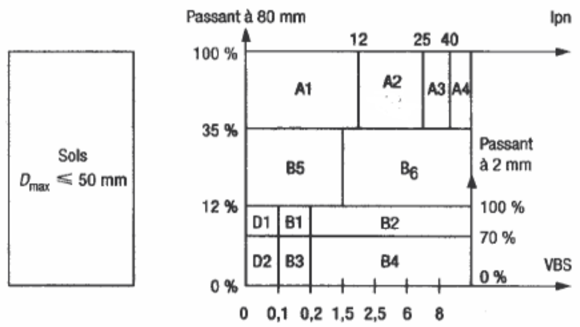

Figure 2: GTR classification of soils (LCPC SETRA, 1992)

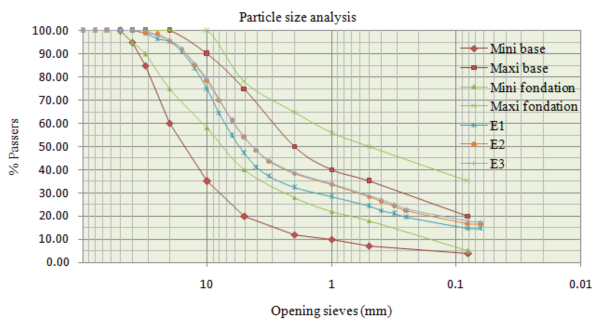

Figure 3: Granulometric curves of the three samples of the lateritic soil

According to CEBTP (1984), the minimum dry densities required for the use of a lateritic gravelly in base and base layers are respectively 1.8 and 2 t / m³, for a water content of between 5 and 12%. . The dry density of our material exceeds 2 t /mm³ and its water content is in the range, so the material is perfect for use in road construction. The average CBR index for 96 hours of 95% imbibition of the OPM is 82 greater than 80 and the 90% linear swelling of the OPM is on average 0.13% less than 1% after 96 hours. So imbibing its porosity coefficient is 24.32%.

The set of test results found in Table 1 refers to CEBTP criteria (1984). So all the parameters found at the gravelly level are above the values required for use in foundation material. By conclusion this is suitable for the foundation material of the pavement.

According to Table 2 the angles of friction are practically the same but the effective cohesion varies between 12.428 and 15.797 kPa respectively at 95% of the OPM and at 100% of the OPM. We can say that our material is a sandy clay gravel or gravel spread and also the compacting energy acts on the effective cohesion.

From Table 2, we see a decrease in the void number e_0 after 100% compaction of the OPM and an increase in the permeability coefficient from $6.546.10^{-10}$ to $7.073.10^{-10}$ m / s. On the other hand the coefficient of compressibility Cc leaves 0.103 to 0.051. This means that if the compaction is carried out at 100% of the OPM, the soil is denser and less compressible. We also note that the oedometric modulus E_{oed} at 100% of the OPM is higher than that of 95% of the OPM. It can be concluded at 100% OPM laterite is rigid.

Essential Parameters of Hardin and Drnevich's Model (1972)

Table 3: Values of the essential parameters of the model

σn (kPa)	50		100		200		400	
	95% OPM	100% OPM	95% OPM	100% OPM	95% OPM	100% OPM	95% OPM	100% OPM
Gmax (MPa)	5	3,333	5	3,333	5	10	10	11,111
Tmax (MPa)	0,041	0,591	0,099	0,078	0,136	0,153	0,323	0,303
γ_r	0.0082	0.0177	0.0198	0.0233	0.0271	0.01 532	0.0 323	0.0 273

Table 3 presents the optimal values of G_{max} and τ_{max} determined for each normal stress and then Figure 4 below represents the behavior according to the hyperbolic model of Hardin and Drnevich (1972), compared to the values observed experimentally of gravelly lateritic at 100% of the OPM and 95% of the OPM under the effect of the normal stress 400 kPa.

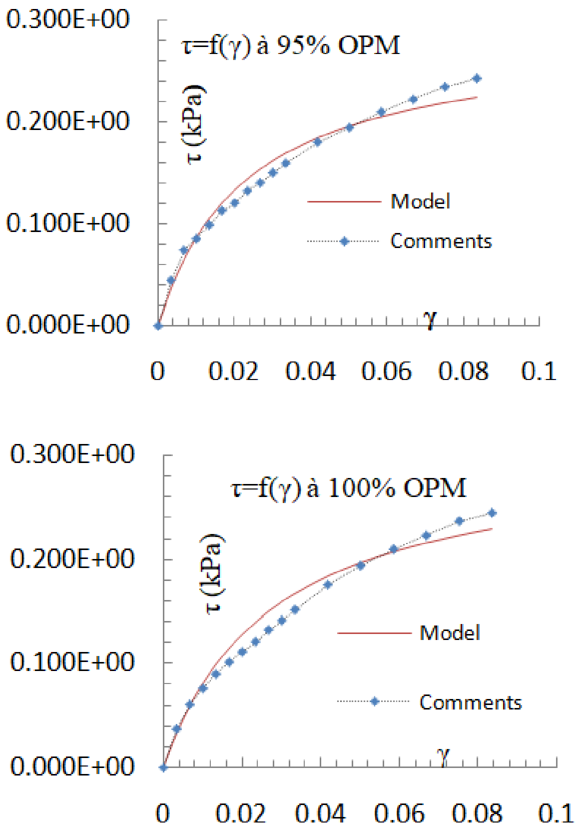

Figure 4: shear stress as a function of the deformation according to the hyperbolic model of Hardin and Drnevich of the gravelly

From Fig. 4, we notice that the stress-strain curve of each specimen (100% of the OPM and 95% of the OPM) from the results of the model is very close to the experimental one: we deduce that the model of Hardin and Drnevich fits well the observations.

Validation of the model

All the calculation details for model validation are shown in Tables 5 and 6.

Table 4: Values of the calculated coefficient of determination

σn (kPa)	50		100		200		400	
OPM (%)	95	100	95	100	95	100	95	100
R^2	98.86	97.97	98.88	94.64	98.29	95.29	98.99	98.33

where we sign by:

$R^2 = 1 - \frac{SS_E}{SS_R}$; $SS_R = \sum_{i=1}^{n}(y_i - \bar{y}_i)^2$ et $SS_E = \sum_{i=1}^{n}(y_i - \hat{y}_i)^2$;

y_i ,the experimental value,

$\bar{y}_i$,the average of the experimental value,

$\hat{y}_i$,the value predicted by the model.

Suitability test of each model

Table 4 shows that the coefficient of determination of each compaction energy is close to 100% (varies from 95.29 to 98.99%). It appears that at least 95% of the experimental values are explained by the model of Hardin and Drneviche (1972).

In Table 5, we find that 100% of the standard residuals di belong to the set [-2, + 2]. This proportion is well above the recommended value of 95% [9], which shows that residues are normally distributed.

Determination of the Poisson's ratio and Young's modulus

Table 6 below gives the Poisson and Young's modulus coefficients from Equations (8) and (10).

Table 6: Table of values of Poisson's ratio and Young's modulus of laterite

	υ	*E (Mpa)*
95% OPM	0,40	29,00
100% OPM	0,40	32,46
ALYZE III (laterite material used as a foundation layer)	0,35	200-600

According to Table 6, the Poisson coefficients obtained and that of ALIZE III are practically the same, moreover the Young's modulus at 100% of the OPM is less than ten times that of ALIZE III. There is not much difference with that found from the formula of C. Regis (1972). This difference is due to the fact that we did not use all the material (class 0/5). We can conclude that the value of Young's modulus found in the library of ALIZE III does not correspond to the true value of laterite. This value of ALIZE III could be at the origin of the early degradation of our roads built with laterite, in addition for all uses of laterite in road construction it is necessary that compaction exceeds 95% of the OPM.

Conclusions

From this work we can say that the lateritic gravelly is class B6 according to GTR. Referring to the CEBTP criteria, gravelly is suitable for the shape and foundation layer material of the pavement.

The modeling of the class 0/5 mm of the lateritic gravelly by the hyperbolic model, allowed us to have the essential parameters the maximum shear modulus and the maximum tangential stress by the least square method. linear and validated by the coefficient of determination and the normality of the residues. This modeling was done on the thin part of the material because it is in this part that we observe more deformation.

It should also be noted that the Poisson's ratio and the Young's modulus were determined by combining the results of the model and that of the oedometric test. So we recommend to do this, the design of the data library of our materials and also the design of software for structural calculations

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Table 5: Residue Analysis Statistics

95 % OPM							100 % OPM						
y_{exp}	y_{cal}	$e_i = y_{exp} - y_{cal}$	e_i^2	$d = y_{exp} - \overline{y_{exp}}$	d^2	d_i	y_{exp}	y_{cal}	$e_i = y_{exp} - y_{cal}$	e_i^2	$d = y_{exp} - \overline{y_{exp}}$	d^2	d_i
0,000	0,000	0,000	0,000	-77,867	6063,314	0,000	0,000	0,000	0,000	0,000	-76,750	5890,563	0,000
37,037	30,215	-6,822	46,534	-40,830	1667,109	-0,681	37,315	33,001	-4,314	18,610	-39,435	1555,134	-0,346
57,778	55,262	-2,516	6,330	-20,090	403,588	-0,251	60,833	59,516	-1,317	1,735	-15,917	253,340	-0,106
71,481	76,361	4,880	23,812	-6,386	40,778	0,487	76,204	81,286	5,083	25,834	-0,546	0,298	0,407
85,370	94,378	9,008	81,144	7,503	56,296	0,900	90,093	99,481	9,388	88,140	13,343	178,025	0,752
98,704	109,943	11,239	126,315	20,836	434,156	1,123	101,481	114,914	13,432	180,424	24,731	611,646	1,076
110,370	123,523	13,153	172,995	32,503	1056,451	1,314	111,389	128,169	16,780	281,581	34,639	1199,853	1,345
123,426	135,476	12,050	145,212	45,559	2075,590	1,204	121,111	139,678	18,567	344,728	44,361	1967,908	1,488
136,667	146,078	9,412	88,577	58,799	3457,367	0,940	132,500	149,764	17,264	298,037	55,750	3108,063	1,383
147,500	155,546	8,046	64,732	69,633	4848,715	0,804	141,389	158,675	17,286	298,816	64,639	4178,186	1,385
157,500	164,051	6,551	42,921	79,633	6341,369	0,654	151,204	166,606	15,402	237,232	74,454	5543,354	1,234
178,889	181,962	3,073	9,445	101,022	10205,365	0,307	175,000	183,077	8,077	65,238	98,250	9653,063	0,647
196,667	196,246	-0,421	0,177	118,799	14113,293	-0,042	193,333	195,995	2,661	7,082	116,583	13591,674	0,213
214,444	207,903	-6,541	42,790	136,577	18653,321	-0,653	208,981	206,397	-2,585	6,681	132,231	17485,165	-0,207
230,093	217,597	-12,495	156,137	152,225	23172,545	-1,248	222,037	214,953	-7,084	50,186	145,287	21108,323	-0,568
241,667	225,786	-15,881	252,211	163,799	26830,238	-1,586	235,833	222,114	-13,719	188,207	159,083	25307,507	-1,099
248,426	232,794	-15,632	244,367	170,559	29090,250	-1,561	243,796	228,197	-15,600	243,346	167,046	27904,465	-1,250

With

$\mathbf{d_i} = \frac{\hat{\varepsilon}_i}{\sqrt{\hat{\sigma}^2}}$: standardized residue; $\hat{\sigma}^2 = \frac{\sum_{i=1}^{n}(y_i-\hat{y}_i)^2}{n-p}$: statistical variance; $\hat{\varepsilon}_i = y_i - \hat{y}_i$: estimated residue

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