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Characterization with geotechnical testings of the clay sediments of Gbédji-Kotovi in Benin

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Abstract— **Background**: The clay of Gbédji-Kotovi is a combination of smectites, and of beidélite (Etienne SAGBO, 2015; IHETA and al, 1983), which presents sometimes, heterogeneous (mixtures) and anisotropic (multilayers) states. In Africa, generally, clays are used in the homogen soil constructed core or body ofdykes, or in the construction ofzoned backfill water resistant core or body of dikes or dams and for the masqued backfilled dykes. Thus, it is importantto prove with the geotechnical testings, if the clay of Gbédji-Kotovi can be used for dykes or dams.
Objective: This paper identifies, classifies and determines the mechanical behavior of the clay sediments obtained from two wells of Gbédji-Kotovi clays deposit.
Results : The results of the testings are the follows: the clay sediments of well number 1 have a texture with 59 to 81% of fines, the water content is between 10% and 40%, the low friction angles is from 0.63 to 5.8 ° and high internal cohesions is from 13 kPa to 22 kPa. On the other hand, the sediments of well 2 have avarious textures (sandy-clay-loam, clay-loam and clays), the average friction angles is from 14°C to 24.4°, weak or medium internal cohesions is 9.59 kPaat 49 ° C, the high dry density is 1.75 t/m^3, the low water content ,13%,the low CBR, 07 and the average swelling rate is 0.2369%.
Conclusion: the use of those clays for the construction of dikes is approved by the obtained results. Some reconstitutions ofsamples of sand-clay, in the laboratory are also reproduced and the different real cases of natural masses (heterogeneous and anisotropic) observed in situ, were tested. Finally, the proportions of 21.3% of clay, 0.5% of loam, 68.2% of coarse sand and 10% of gravelimproved significantly the following parameters: the water content is 6.9%,the density is 2.06t/ m^3 , the average CBR is 20 andthe low swelling rate is 0.0790%. The clay of Gbédji-Kotovi can be used for the construction of the dykes. It is however necessary to study their permeability in situ and in the laboratory and to propose an improvement of their mechanical parameters thanks to a coarser sand.
Index Terms— Clays, activity, permeability, embankment, dike, sand, horizon.

1 Introduction

IN the composition of the clay sediments of Gbédji-Kotovi, the dominant minerals are kaolinite, smectite (montmorillonite, beidellite), attapulgite, illite, chlorite, etc., associated to quartz (SiO2), glauconite (K, Na) (Mg, Fe^{2+}, Fe^{3+}) (Fe^{3+} ,Al) (Si, Al)$_4$O$_{10}$(OH)$_2$, anatase (TiO_2), microcline ($KAlSi_3O_8$) and many other trace elements: Armel B. LAIBI et al., (2017), Etienne SAGBO (2015), N. YALO and al (2008). According to the report on the geological research of clay in the sedimentary basin area of Benin (IHETA et al., 1983), Gbedji-Kotovi deposit is estimated at 5,000,000 tons, with an average thickness of 4 m, a specific gravity of 2t/m^3 and covering an area of 644,700 m^2. This clay is a smectite, especially, the beidelite mineral (SAGBO et al., 2015). The extremely slow variation of their hydric state makes them impermeable.The assembly and the weakness of certain links between clay sheets lead to a particular "slipperiness" which strongly limits the stability of the structure in the long term (C. Hung, et al., 2006). The intermediate texture of that clay soil offers possibilities to use it in the construction of the core or body of dikes or dams. Those structures are important in integrated and sustainable water resources management (rain or surface runoff) in the rural zone and the agricultural areas. In the case where their characteristics are rather mediocre, the mechanical stabilization of clays with amended sand seems a solution to improve the geotechnical parameters of the small and medium dikes and dams, in order to avoid the relatively very high costs of physico-chemical treatment (lime, cement or binder hydrocarbons).

2 Presentation of the Site of the Study

The clay deposit of Gbedji-Kotovi is located between 2°00' and 2°02' east longitude and between 6°40' and 6°42' north latitude in the sedimentary basin of Benin. This area is characterized by a hydromorphic soils in a subequatorial climate, and a sufficient annual rainfall with an average temperature of 27.3°C. The drainage axes are streams dominated by the valley of the Couffo River.

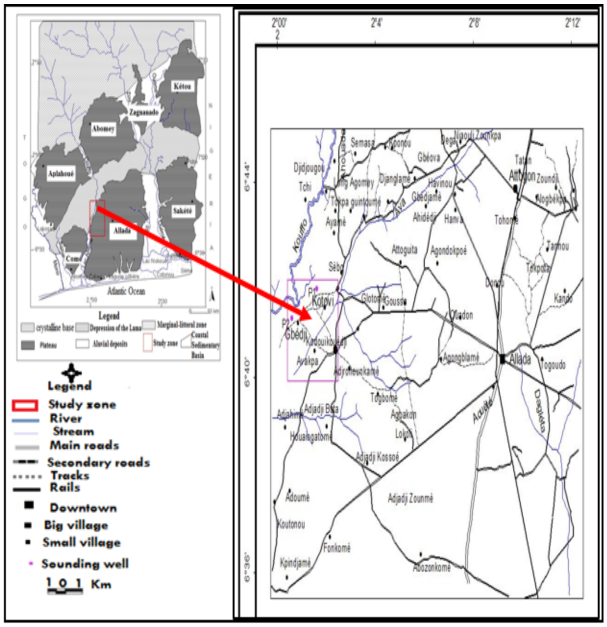

Fig. 1. Positions of the wells in the locality of Gbédji-Kotovi

The region is dominated by the mio-Pliocene deposits of the quarternary relying on the middle Eocene kaolin clays which constitute an aquiclude base level and which are exposed in the locality of Gbedji-Kotovi not far from the Coufo River. They may have to be crossed before accessing and capturing the Upper Palaeocene (variable thickness) aquifers whose recharges would come from rain, through drainage (M. Slansky, 1962).

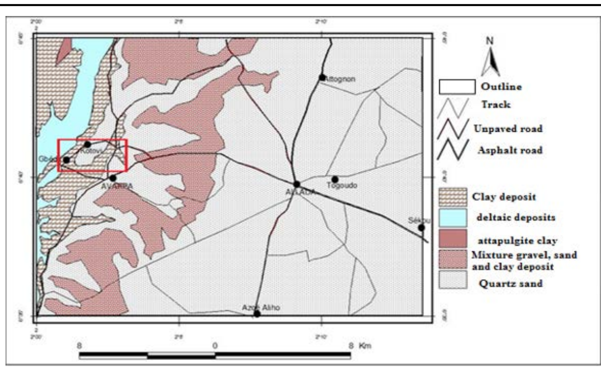

Fig. 2. Geomorphological map of the site of Gbédji-Kotovi **[5]**

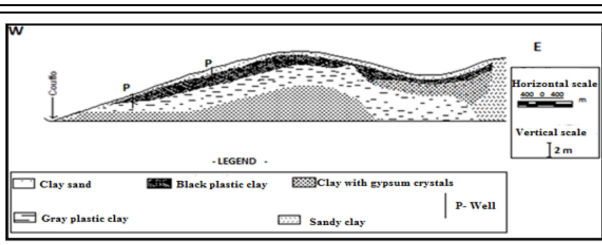

Fig. 3. West-East geological section of the Gbédji-Kotovi clay site **[4]**

3 Materials And Methods

3.1 The Granulometry Testing

The results of the granulometric analysis are obtained according to the French standard (NFEN 933-1). The particles size analysis consists by determining the dimensional distribution of the grains of the aggregate.This analysis makes it possible to determine the parameters which are used to classify the sand and to foresee the appropriate domains for its use. The determined parameters are the coefficient of uniformity (C_u), the coefficientof curvature (C_c), and of the fineness module (M_f), which formulas are the following:

The coefficient of uniformity (C_u) can be calculated as:

$$C_U = \frac{D_{60}}{D_{10}} \quad (1)$$

The coefficient of curvature (C_c) can be calculated as:

$$C_C = \frac{(D_{30})2}{D_{60}.D_{10}} \quad (2)$$

and of the fineness module (M_f)

Where D_{60}, D_{30} and D_{10} are the sieve diameters (mm) corresponding to 60%, 30% and 10% of passers, respectively.

$$M_f = \frac{1}{10}\Sigma \, \text{Refus}(0{,}16 + 0{,}315 + 0{,}63 + 1{,}25 + 2{,}5) \quad (3)$$

According to the standard NFP94-057 (1992) **[9]**, the sedimentation is concerned by the soil sample obtained after wet sieving through the 63 µm opening sieve. This measure is based on the Stocks law which expresses the equilibrium relationship between the decantation velocity (assumed constant) and the diameter of a supposedly spherical particle:

$$V = \frac{(\rho_s - \rho_w).g.D^2}{18\eta} \quad (4)$$

Where V is the settling velocity of the particles (cm/s); ρs is the density of solid soil grains; ρwis the density of water (g/cm^3); g is the acceleration of gravity (cm/s^2); D is the diameter of the particle (cm) and η is the viscosity of the fluid (poise). This formula is used to estimate the equivalent diameter D of the clayey particles (which have irregular shapes) greater than 1 µm (MAGNAN, 2011).The density of the suspension at time t is deduced from the measurements made at the densimeter at different times after the start of sedimentation:

$$\rho_t = (R + C_t + C_m + C_d)\rho_d \quad (5)$$

Where R is the corrected reading of the densitometer, C_t, C_m, and ; C_D, are determined by calibration of the apparatus. Knowing the velocity (V), and the density of the suspension, the percentage C_D of elements that dimension is smaller than or equal to D in the suspension at time t is calculated by the formula:

$$C_D = \frac{V}{m_s} \cdot \frac{\rho_s}{(\rho_s - \rho_w)} \rho_w \left(\frac{\rho_t}{\rho_w}\right) \quad (6)$$

Where C_D is the percentage of the total mass of the particles at the beginning test (m_s); ρ_s and ρ_w are respectively the mass of particles and the mass ofwater.The dimension D of the particles that have not yet sedimented in the solution is determined by the formula:

$$d= \sqrt{\frac{1}{g}\frac{18\eta}{(\rho_s-\rho_w)}}\sqrt{\frac{H_t}{t}} \quad (7)$$

Where H_t is the depth of the center of thrust of the densimeter at time t which is deduced from the geometry of the densimeter and from the R value read on the graduation of the densimeter.

3.2 The Atterberg Limits

The limits of Atterberg permitto determinate the differents states of the soil in function of differents water contents. Their determination contributes to recognize the composition of given soil. The Atterberg limits are determined according to the standard NF P 94-051, using the Casagrande cup to find the soil water content for which the notch in sample closes by 10 mm, under 25 moves applied at a standardized speed, according to the formula:

$$W_L = \left(\frac{N}{25}\right)^{0,121} \quad (8)$$

Where N is the number of applied moves. The limit of plasticity WP was considered to be the water content for which approximately 3 mm x 10 cm cuts break while being rolled by hand. The plasticity index is the difference between the liquidity limit wL and the plasticity one, wP. The plasticity index is calculated by the so-called Skempton activity formula (Mitchell 1993):

$$A_C = \frac{\text{Ip}}{(\text{\% of particles lower than } 2\mu\text{m})} \quad (9)$$

The Limits of Aterberg aim to determine many other identification parameters. Among them, we have the consistency index:

$$\text{IC} = (w_L - w_P) / \text{IP} \quad (10)$$

The highly clayey soils have a relatively high plasticity limit. Their index of plasticity makes it possible to say whether they are organic soils or not clayey.

3.3 Value of methylene blue (VBS)

The term "blue value" refers to fines, the quantity expressed in grams of methylene blue adsorbed per 100 g of fines. This capacity globally accounts for several characteristics of fine particles:

i) The Coefficient of argilosity, A, (Magnan and Youssefian 1989):

$$A = \text{VBS} / \% < 2\mu\text{m} \quad (11)$$

ii) Cationic exchange capacity (CEC):

$$CEC\ (meq/g) = \frac{\text{VBS (g/100 g)}}{374} \times 1000 \quad (12)$$

iii) S_s Total specific surface area (Santamarina et al., 2002):

$$S_s(m^2/g) = \text{VBS (g/100g)}\frac{A_V}{A_{MB}} \quad (13)$$

Where: A_V is the Avogadro number (6.02 x 10^{23} atoms /mol); A_{MB}: area covered by a molecule of methylene blue (130 $Å^2$); and IP is the plasticity index.

3.4 The unconsolidated and undrained direct shear test

This test gives the possibility to know the state of the consolidation of a soil and its characteristics of rupture (cohesion of the grains and their angle of friction). It ensures the stability of the structures just after their construction (in initial phase or construction site) endersollicitications without drainage.For each specimen, the actual normal stress σ'is calculated according to the expression:

$$\sigma' = \frac{N}{A} \quad (14)$$

Where N is the normal force expressed in Newton and Ai the area of the cross-section of the specimen before shear in cm^2. The shear stress (τ') is equal to:

$$\tau' = \frac{T}{A} \quad (15)$$

Where T is the horizontal shear force (Newton). For overconsolidated clays, the limit state curve is affine.
The calculation at break of an embankment consists in considering all the forces which ensure the balance of the volume of soil situated above the surface of rupture. As long as the shearing force along the surface remains below the maximum resistance that the soil can mobilize, the slope is stable; it is unstable in the opposite case. The criterion of rupture retained is that of Mohr-Coulomb which is translated by the equilibrium relation limit:

$$\tau_{max} = c + (\sigma - u)\tan\varphi \quad (16)$$

Where τ_{max}: the maximum shear resistance that the soil can mobilize, σ the normal stress, u the interstitial pressure, c the cohesion and φ, the internal friction angle of the soil.
For normally consolidated clays, the linear rupture envelope has zero cohesion and an angle φ'which increases as a function of the Plasticity Index (Bjerrum and Simons, 1960).
The safety factor F, assumed to be constant along the failure surface, is defined as the ratio of maximum shear strength tmax to the shear stress t along the failure surface:

$$F = \tau_{max}/\tau \quad (17)$$

3.4 The Proctor Testing

For eachconsidered water content value, the dry density of the material is determined and the PROCTOR curve representing the dry density as a function of the water content is plotted. The maximum dry density (ρ_d) obtained for a particular value of the water content is:

$$\rho_{dopt} = \frac{m_s}{V} = \frac{m}{(1+w)V} \quad (18)$$

Where ρ_d is the dry density of the sample, msis the mass of the solid grains; V is the volume of the PROCTOR mold; and w is the water content of the compacted sample.

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3.5 The VBR Testing

The CBR (Californian Bearing Ratio) laboratory test is intended to determine by punching the lift of a compacted soil under the conditions of the Proctor test. The charts are then used to calculate the thickness of the foundation layers necessary for the construction of a road surface (eg pavement), depending on the underlying subsoil, the traffic (expected axle loads) and the conditions. A future water supply that such embankments will undergo.The equipment includes a Proctor mold, a punching piston, a device for applying a constant overload and force and displacement sensors. The principle of the test is to measure the forces to be applied on a cylindrical punch to make it penetrate at a constant speed (1.27mm / min) into a test piece of material. During the test, the effeort-strain curve established on the dynamometer ring the force required for 1.25mm recesses; 2mm; 2.5mm; 5mm; 7,5mm and 10mm. The following characteristic values are defined:

$$I_1=\frac{m_s}{V}=\frac{\text{Effort de pénétration à 2,5mm (kN)}}{13{,}35\ \text{kN}}\ \text{x}\ 100. \quad (19)$$

$$I_2=\frac{m_s}{V}\_\frac{\text{Effort de pénétration à 5mm (kN)}}{19{,}93\ \text{kN}}\ \text{x}\ 100. \quad (20)$$

The desired CBR index is, by convention, the greater of the two values: max (I_1, I_2). This index characterizes the evolution of the lift of a compacted soil and/or subjected to variations of hydric regime. Indeed, the sample is optionally subjected to partial or total imbibition (4 days) before punching.
In the calculation of the minimum thickness of earth pavements, the CBR is calculated for a depression of 2.5 mm from the pressure equal to half of the breaking stress obtained on the stress-strain curve. This gives the corrected CBR which is used for the determination of the thickness e of the roadway. According to R. PELTIER (1958) **[17]**, this thickness can be obtained by the formula:

$$e=\frac{100+(75+50\mathrm{Log}\frac{N}{10})\sqrt{P}}{I+5} \quad (21)$$

Where e is the thickness in cm; N the number of heavy goods vehicles of more than 3 tonnes per day; P the weight of the maximum wheel in tonnes (or two twin wheels), I the C.B.R index

3.6 The compressibility test

The compressibility or the decrease in volume of the granular skeleton of clays depends on four main factors: 1) temperature, 2) strain rate, 3) strain path, 4) structure and reworking (Leroueil, 1996). The clay soil deformation curve is divided into three sections: recompression, virgin compression and rebound. The vertical stress that marks the transition between recompression and virgin compression defines the preconsolidation pressure (σ'p): it is the maximum stress that has fallen on the ground during its geological history. For the three segments, we observe an approximately linear relationship between the decimal logarithm of the vertical stress σ'z and the void index e. The slopes of these straight lines (Δe /Log [σ'v]) are: the initial recompression index, the blank compression index C_c and the rebound or swelling index I_g: These different slopes can to be determined by the expression

$$I=\frac{-\Delta e}{\Delta(lg\sigma')}. \quad (22)$$

According to MOUNOU S. K. **[5]**, the clay sediments of Gébdji-Kotovi (more than 75% of fines) are very compressible (Cc = 0.4) and swelling (Ig = 0.08), but, this study did not calculate the compressibility of sediments in the case of the percentage is less than 50% fines.

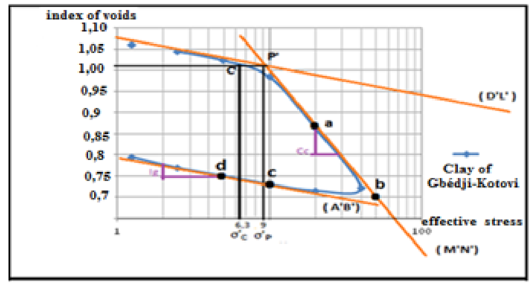

Fig. 4. Oedometric curve of a Gbédji-Kotovi clay sediment **[5]**

4 Results and Discussions

4.1 The Stratigraphy and Particle Size Analysis

Due to the geological and topographical environment, two wells are involved in our experimentation. Indeed, the site is characterized by two types of deposits: low-lying clay deposits and sand-covered clay deposits up the slopes, with medium and high altitudes. The well 1 consists of an entirely clayey profile up to the depth of 3.5m. This profile ranges from black plastic clay, from 0 m to 1 m deep, to gray plastic clay between 1 m and 2 m deep and to black plastic thick clay from 2 m to 3,5 m (Figure 5 a). The well 2 consists of varying horizons with different textures. It shows very organic humus of 0 to 0.5m, medium loamy sand between 0.5 m and 1 m, a loosely compacted loam with ferruginous concretions between 1 m and 2.7 m and compact silt between 2.7 m and 4.2 m (Figure 5 b). These two wells show the variability of the deposit granulometry. These different textures which determine the geotechnical parameters of a soil led us to carry out a study by layer or facies in each well, and to make mixtures of different granulometries, to obtain the ideal and varied conditions of exploitation of this soil.

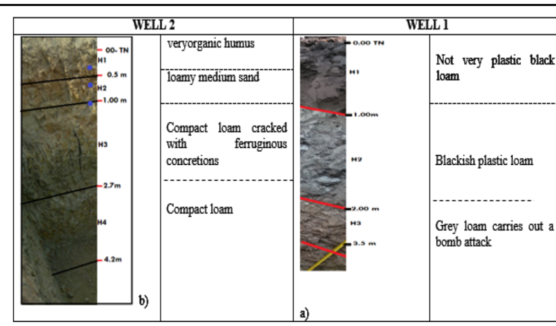

Fig. 5. Stratigraphy of the wells

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The observation of the stratigraphic logs above shows that the Gbédji-Kotovi clay site consists of:

- ✓ clays from the natural soil (a flush deposit): Figure 4 a;
- ✓ Alternating sandy, sandy-clay, sandy-clay or clayey deposits, depending on the conditions that prevailed in the depositing environment (Figure b). These differences in mechanism and deposit facies have led us, in the rest of our study, to make a geotechnical characterization that takes into account the change in facies at the level of the two wells.

4.2 The Granulometry testing

In the well 1, the clayey particules (lower than 2 μ m) is 59 % at 1.5 m of deph, 74 % at 0.5m of depth and 81 % at 3.5m of depth. But the proportion of particles passing 0.063 mm sieve at each deph is superior to 50 % in the three horizons. The sedimentometric curves of well 1 have parabolic forms (Figure 6). They are clays and fine vases whose median grain is less than 2 micronmeters. The more or less rectilinear parts of these curves show an extreme evolution of the current transport and the segments with a concavity turned downwards indicate a deposition by sedimentation in calm water after a long path (loss of less fine elements).

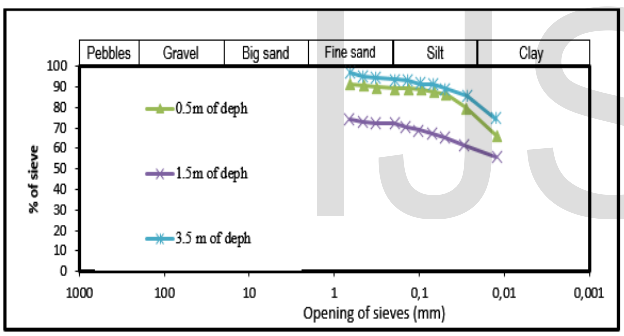

Fig. 6. Granulometric curves from the well 1

The analysis of the sediment grain size curves of well 2 (Figure 7) shows:

- Between 0 and 0.5 m deep, a parabolic facies (sand and silt deposited by charriaget, during the reduction of the speed and the turbulence of the conveyor current). This sand has a coefficient of uniformity C_u= 3,575 a curvature coefficient C_c = 1.12 and a fineness modulus M_f= 3.07: it is very fine sand. In fact, the sieving granulometric curve shows that this sediment contains 3% coarse sand, more than 77% fine sand, 20% silt and 0% clay.

- From 1m depth, the sediments are intermediate (mixture of sands and clay loams). The percentage of fines is 42% at 2.7m depth, 34% at 1.5m depth and 32% at 1m depth. Thus, all the sediments of well 2 contain at least 32% clay (<2μm) and more than 70% fine particles with a diameter of less than 80 μm.

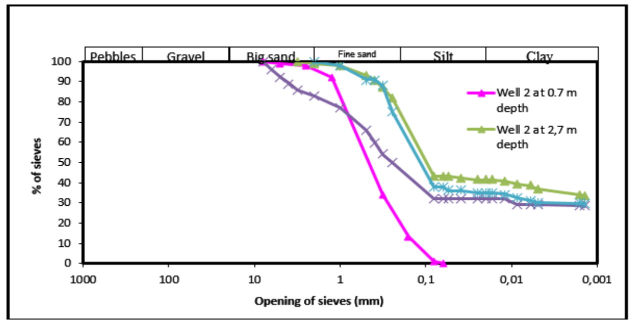

Fig. 7. Granulometric curves from the well 2

The analysis in Figure 7 shows:

- ✓ Between 0 and 0.5 m deep, a parabolic facies (sand and silt deposited by charriaget, during the reduction of the speed and the turbulence of the conveyor current). This sand has a coefficient of uniformity C_u = 3.575 a curvature coefficient C_c= 1.12 and a fineness modulus M_f = 3.07: it is about a rather coarse sand which will give concrete resistant but less manageable;
- ✓ From 1m deep, a soil is intermediate (mixture of sands and clay loams). The percentage of fines is 42% at 2.7m depth, 34% at 1.5m depth and 32% at 1m depth. With regard to particle size, the sealing criterion (in the case of a dam, for example) can be reached when the clay particles content (<2%) exceeds 15% and the percentage of particles fines exceed 30%.

4.3 The Atterberg Limits

The table 1 summarizes Atterberg Limits obtained through the two wells. The high liquidity and plasticity limits correspond, on the ground, to highly cohesive clay loams and clays which lose their moisture.

TABLE 1
PERCENTAGES OF SAND, LOAM AND CLAY IN WELL 1 AND WELL 2 SEDIMENTS

Parameter	Well 1			Well 2				A + 0%S	A + 5% S	A + 15% S	A + 25% S	A + 35% S	A + 45% S	A + 55% S	A + 65% S
Deph (m)	0.5	1.5	3.5	0.7	1	1.5	2.7	-	-	-		-	-	-	-
Sand (%)	2	2	3	80	53	32	25	40,7	43.61	46.64	53.64	61.73	70.67	71.18	78.34
Loam(%)	16	12	15	19	15	34	32	0.82	2.19	3.43	2,37	1.6	3.03	2.52	0.36
Clay (%)	74	60	80	1	32	34	43	58.5	51. 3	49.93	44	36.67	26.3	26.3	21.3

The Atterberg limits have been projected onto the Casagrande plasticity abacus. All the points are located between the line U and the line A which limits the ranges of montmorillonites and illites-kaolinites. These sediments contain both clay minerals of the class montmorillonites, illites and kaolinites: Gbedji-Kotovi clay is not monomeral, but it is intermediate. So, the results confirm those of previous research (**[4]**, **[2]**). Those clays are normal or inactive, clay activity less than 1.25 (Robert Medjo Eko, 1999). The Atterberg limits and other physical parameters are summarized in Table 2. The Atterberg limits of the Well 1 sediments correspond to very plastic and very wet compact clays. These parameters have decreased significantly for Well 2, indicating

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moderately moist, low plastic sediments.

Fine materials with a plasticity index greater than 35 (sediments from well 1) pose not only problems of stability but also of settlement, swelling and implementation **[14].** In this case, the sediments of Well 1 are less suitable for the construction of dikes and other water retaining structures. On the other hand, sediments from well 2 are suitable forthis type of work (plastcity index less than 20).

TABLE 2

IDENTIFICATION PARAMETERS OF SEDIMENTS FROM THE WELL 1 AND THE WELL 2

Deph (m)	LL	PL	IP	OM %	% of fine	Ac	SW (%)	WC %	VBS g /100 g	CEC *meq/100g*	S_S m^2/g
Well 1											
O,5	105	31	74	1,05	74	1	2,75	38,07	19,26	51,49	68,60
1,5	81	29	52	0,97	59	0,88	2,71	30,92	20,34	54,38	193,70
3,5	88	31	57	0,91	81	0,70	2,73	31,62	21,32	57	75,94
Well 2											
0,7	-	-	-	1	1	-	2,65	10	-	-	-
1,00	44	26	18	0,85	32	0,56	2,47	18,87	8,3	22,19	29,56
1,5	45	26	19	0,67	38	0,50	2,49	15,42	9,9	26,47	35,26
2,7	47	27	20	0,044	43	0,47	2,39	14,78	12	32,08	42,74

LL= Liquidity Limit; LP= Plasticity Limit; IP= Plasticity index; A= Activity; (SE) = sand equivalent;
SW= Specific weight; OM= Organic material; WC= Water content.

The Atterberg limits have been projected on the Casagrande plasticity diagram. All the points are located between the line U and the line A which limits the ranges of montmorillonites and illites-kaolinites. These sediments thus comprise the clay minerals of the class of montmorillonites, illites and kaolinites: The clay of Gbedji-Kotovi is not therefore monomineral, but it is intermediate. These results confirm those of previous research (IHETA et al., 1982, E. Sagbo et al., 2015). Gbédji-Kotovi clays are normal or inactive (clay activity varies between 0.47 and 1). Their cation exchange capacities and their specific surfaces increase with the percentage of clay particles. These values are lower than those of smectites (respectively 80 to 150 meq/100g and 700 to 840 m²/g). These values show that the Gbédji-Kotovi clay sediments consist of a mixture of smectites and other clay minerals. This is verified by the projection of these clays on the Casagrande plasticity abacusor on Holtz and Kovacs diagram. On these diagrams, it is noticed, for sediments of the well 1and the well 2, a non-monomiral clays that have respectively high and medium plasticities.

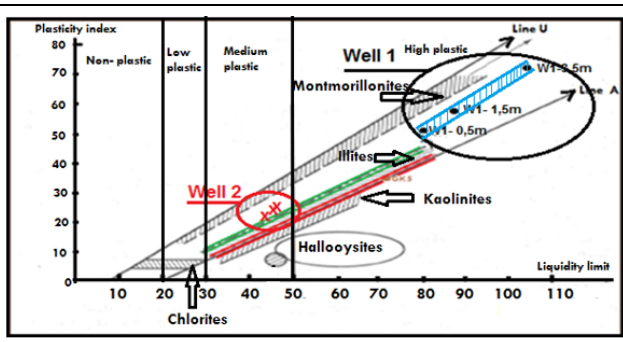

Fig. 8. Caracterisation of sandy clay samples according to the Holtz and Kovacs diagram

According to the table 2 showing the different proportions of sand, silt and clay, samples was placed on the textureal triangle. All the sediments in Well 1 are clays. On the other hand, the sediments of Well 2 are silty sands (0.5 m deep), clay-silty sands (1m deep), clay loams (1.5m deep) and clays (2m deep). All sands + clay mixes have a balanced texture that corresponds to the optimum, as it has most of the qualities of the three previous types (sand, clay and silt). Mixtures A + 0% S, A + 5% S and A + 15% S have a sandy-clay texture while mixtures A + 25% S, A + 35, A + 45% S, A + 55% Sand A + 65% S have a sandy-clay-loam texture. The textural classes of the ground shaded in red in the triangle are the best for the construction of dikes and fish ponds.

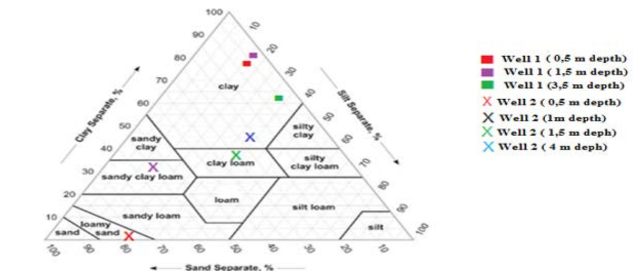

Fig. 9. Soil textural triangle, Duchaufour, 1997

4.4 The shear testing

Figure 10 shows the sediment shear parameters of the two wells in the unconsolidated and undrained state (short teme).The elements of well 1 have low friction angles (0.63 to 5.8 °) and high internal cohesions (13 kPa to 22 kPa). Affine lines characterize overconsolidated clays. On the other hand, for the clay sediments of well 2, there are significant friction angles, ranging from 14 ° C to 16.3 ° C and internal cohesions ranging between 3.51 kPa and 49.58 kPa. It is noted for the sand, a very weak cohesion (less than 10 kPa) and a high angle of internal friction (24.4 ° C).

In foundation, if the poorest soft layer (generally saturated or nearly undrained cohesion value of 20, 40, 60, 80, 100 kPa (values greater than 100 kPa are uncommon), we can respectively build a dam of small heights (5, 10, 15, 20, 25 meters) without having to widen substantially its base, compared to the same structure that would be based on rock **[14]**.

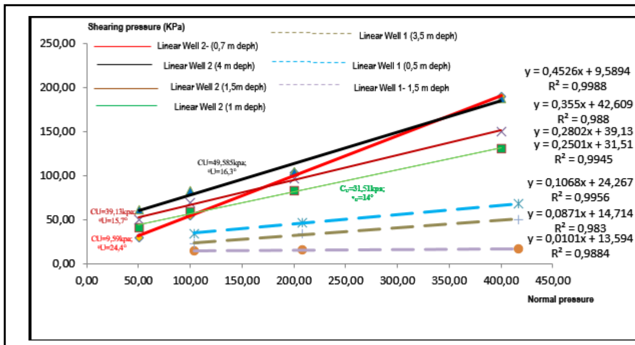

Fig. 10. Shear test parameters of well 1 and well 2

A comparison of grain size and mechanical parameters indi-

ISSN 2229-5518

cates that sediments from well 2 are the most likely to be permeable to water,while the sediments of well 1 which have a very high percentage of clay represent impermeable soils.

4.5 The Proctor testing

Figure 11 shows that Proctor Optimum values for Well 1 at 0.5 m and 2.7 m depth are 1.49 t/ m^3 and 1.67 t / m^3. The water contents are respectively 23.7% and 22%. The degree of optimal Proctor saturation is greater than 80%. This is a relatively high value that could be explained by the presence of organic matter in the soil. Regarding the Proctor curves; for five different compaction energies at 15; 21; 26; 31; 36 strokes per layer, the more or less straight shape of the curve is due to the heterogeneity of the material and the disparity of the applied energy. This texture has a dispersion in the points measured, which gives the spread form. It is possible that the shape of the curve is horizontal when the sample is very dry, because the influence of the compaction becomes weaker on the dry density.

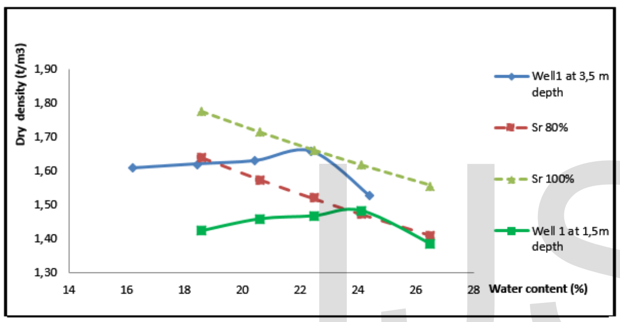

Fig. 11. Proctor test of intact sediments of the well 1

4.6 The oedometric testing

The results of the oedometric test (Figure 12) show that the sediments of well 2 (mixture of sand, clay and silt) are both less compressible (C_c between 0.123 and 0.150) and less swollen ($I_g$between 0.0198 and 0.0267) than the predominantly clayey sediments of the same site **[5]** which have a pressure index of 0.40 and a swelling index of 0.08. It can be said that this increase in swelling and compressibility is due to the presence of a smaller amount of sand in the sediment of the clay well which is located 5 m from the well 1. Moreover, it can be noted that these two parameters vary within the well 2. This variation is the result of the variation of both the granulometric parameters and the applied stresses.

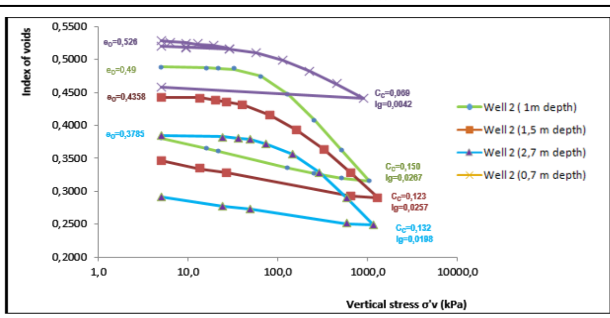

Fig. 12. Oedometric test of the sediments of well 2

5. Improvement

5.1 The samples preparation

Samples of sand + clay reconstituted in the laboratory so as to reproduce the different real cases of natural mass (heterogeneous and anisotropic) observed on the site were analyzed in the laboratory, following the French standard of standardization (AFNOR). Among the different homogenization methods proposed (Howell et al, 1997), we chose to mix sand and clay dry, then moisten and homogenize the mixture (Tanaka et al., 2001), which allows to overcome the difficulties of homogenization encountered by the other methods **[19]**. Thus, the rates of 5%, 15%, 25%, 35%, 45%, 55%, and 65% of sand were added respectively to 95%, 85%, 75%, 65%, 55%, 45% , and 35% clay sediment from well 2 (homogeneous mixture of the three clay facies). These rates were chosen to ensure that the mixture will be able to move from clay-dominated soil to sand-dominated soil. The granulometric synthesis of these mixtures is illustrated in Figure 14. The particles size analysis presents the following situation:

The grain size curves of the different proportions of sand and clay mixtures indicate that:

- The mixture without addition of sand (A + 0% S) contains 58.51% of particles smaller than 2 µm. This rate of fines characterizes a clay sediment;
- The additions of fine sand (5%, 15%, 25%, 35%, 45%, 55%, and 65%) decreased in a decreasing and linear way the rate of fines in the mixtures. These percentages of fines in the additions are respectively 51.3%, 49.93%, 44%, 36.67%, 26.3% and 21.3%. It can be said that the additions have made it possible to pass from a sediment dominated by fines (clay) to sediments dominated by sand;

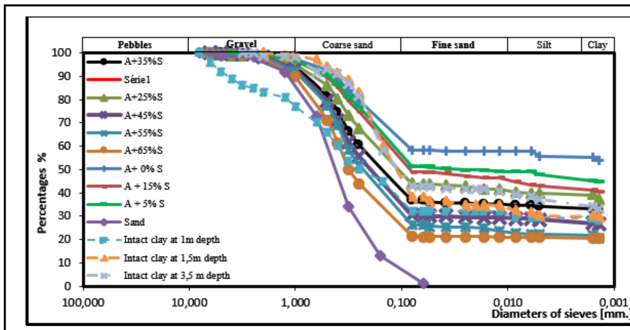

Fig. 13. Granulometric curves of the different Clay + Sand mixtures

5.2 The Proctor Testing

The characteristics of the Proctor curve (concavity, moisture content and dry density) were corrected by adding the amounts of sand to the clay portion of the sediments (Clay + sand). Figure 15 shows that the mixture of intact clay facies without addition of sand (A + 0% S) and the intact and non mixture clayey samples have the highest water contents and the lowest dry densities. Overall the Proctor curves of the mixtures give a good concavity except for the mixture A + 55% S. This particularity could be due to errors during compaction which was carried out manually. The dry density increases with the decrease of the water content, to reach, with a mixture A + 65% S, a value of 2.06 t/ m^3 corresponding to 6.8% water content. Furthermore, the compaction parameters for A + 15% S (10%, 1.9t/ m3) are better than for A + 25% S (9%, 1.871.9t/ m3) and almost the same as for A + 35% S (8.96%, 1.91 t/m3). So the rate of fines alone is not sufficient to predict the compaction parameters. Other parameters such as the presence of organic matter, compaction conditions, percentage of swelling clay minerals, etc. could participate in obtaining effective compaction or not.

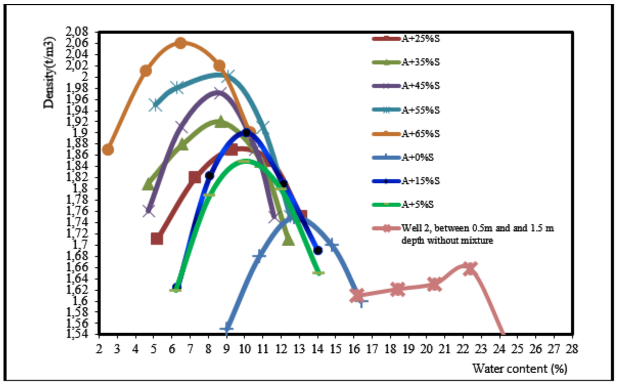

Fig. 14. Proctor curve of clay + sand

5.3 The CBR Testing

The CBR curves (figure15) show that the dry density increases with the increase of the water content at the level of all the mixture samples. A + 55% S with the highest CBR index (67%) did not show the highest maximum dry density (2.01t / m³). The highest maximum dry density (2.06 t / m³) was obtained for A + 65% S, corresponding to a mean CBR index (20%).

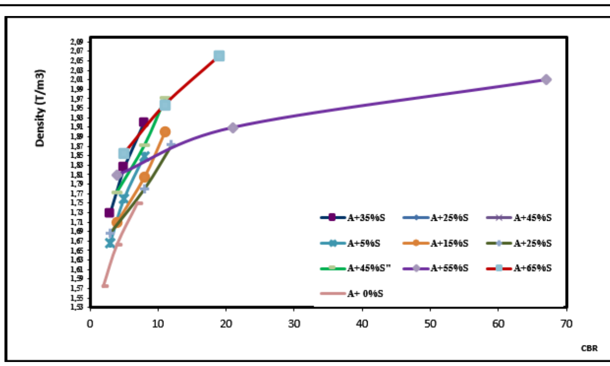

Fig. 15. CBR test curves of clay+sand

The following swelling rates were obtained after immersion of the specimens (Table 3).

TABLE 3

PARAMETERS OF POCTOR AND CBR TEST ON THE DIFFERENT MIXTURES (CLAY + SAND MIXTURES.)

Water content after immersion (%)			CBR index (90%)	CBR index (95%)	CBR index (100%)	swelling (%)	% of fines	Proctor Water content optimum	Proctor Weight density optimum	Class
A + 0 % S										
15.1	17.3	19.5	2	4	7	2.369	58.51	13	1.75	A4
A+5% S										
13.3	16.4	19.2	3	5	8	1.124	51.3	10.2	1.85	A7-6
A+15% S										
13.7	17.2	20.7	4	8	11	1.816	49.93	10	1.9	A6
A+25% S										
13.2	16.7	20.2	3	8	12	0.529	44	10	1.87	A4
A+35% S										
12	15.5	18.4	3	5	8	0.513	36.67	8.9	1.92	A4
A+45% S										
12.3	16.2	20.4	4	8	11	0.458	32	8.5	1.97	A3
A+55% S										
10.2	12.3	14.6	4	21	67	0.118	26.30	8.3	2.01	A3
A+65% S										
8.3	10.4	12.6	5	11	20	0.079	21.3	6.9	2.06	A1-b

A+ 65% S = Clay + 65% of sand

The results in this table show that the swelling rate of the samples immersed in the water decreases with the decrease in the percentage of fines. It can be said that the presence of sand does not promote swelling and that the rate of fines amplifies the swelling of compacted sediments. So, it can be concluded that A + 65% S and A + 55% S mixtures can significantly reduce (up to 30 times) the swelling and obtain better compaction (the lowest water content, and the highest high dry density).

5.4 The Suitability of a Soil for the Construction of Dikes

The dikes have a cross section similar to that of small dams (Degoutte et al., 1997). The dike structure is usually composed of an embankment with the presence, sometimes, of a waterproof core composed of a clay backfill **[20]**.Generally speaking, a soil can be considered suitable for the construction of a dam, a dike or a pond, which can ensure good water retention. This ability varies with changing textures (including increasing or decreasing the percentage of fine particles). We have clay sands, clay-silt sands, clay loams or silty clay silts (Duchaufour, 1997). Projected on the textural triangle, textures of soils suitable for the construction of dikes and dams are the textural spindle in red color (Figure 16). Indeed, a good ground of dike must be with variable granulometry (or spread) and contain a roughly equivalent percentage of gravel, sand, silt and clays. The almost pure clay is not suitable because it dries quickly and crackles, which induces leaks. The best soils are those where clays and silts are exactly enough to fill the voids of the skeleton of gravelly sands. We can mention the embankments sand-clayey (Pierrefitte-sur-Loire, Vallenay and Nozières, Cher) or sandy-clay fill (Coulanges): **[15]**. The essential element of the stability of an earth embankment resides in the homogeneity of these embankments and in the constancy of their processes of implementation, a necessary condition of the uniformity of their internal cohesion. As regards the particle size, the sealing criterion (in the case of a dike, for example) can be reached when the clay particles content (<2% µm) exceeds 15% and the percentage of particles fine particles (<80 µm) exceed 30%: this type of soil is likely to constitute a tight core for the dike. However, fine materials with a plasticity index greater than 35 pose not only problems of stability but also of settlement, swelling and processing (ALONSO et al., 2002).

In foundation, if the poorest (usually saturated or nearly all) soft layer has an undrained cohesion value of 20, 40, 60, 80, 100 kPa (values greater than 100 kPa are uncommon), construct a dam 5, 10, 15, 20, 25 meters in height without having to substantially widen its base, compared to the same structure that would be based on rock (ALONSO et al., 2002)

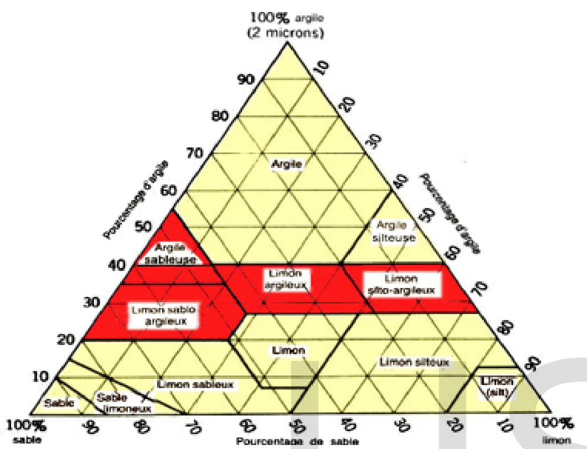

Fig. 16. Triangle textural, Duchaufour, 1997

During dike construction, the undrained unconsolidated test (corresponding to short-term behavior) indicates that interstitial pressures are not dissipating. In foundation, if the weakest (usually saturated or nearly all) soft layer has an undrained cohesion value of 20, 40, 60, 80, 100 kPa (values greater than 100 kPa are uncommon), construct a dam 5, 10, 15, 20, 25 meters in height without having to significantly widen its base, compared to the same structure that would be based on rock (ALONSO et al., 2002).

6. Conclusion

Following their geotechnical characterization, it is found that Gbédji-Kotovi's clay sediments are very sensitive to water. Their activities vary between 0.47 and 1, indicating normal and inactive clays. They comprise a large clay fraction and spreading plasticity characteristics (low, medium and high). Proctor optima of intact clay sediments have mean dry densities (1.49t / m^3 and 1.67t / m^3) corresponding to high water contents (23.7% and 22% respectively). This type of soil could be appropriate for making the tight zone of a dike or a dam. Their plasticity index is high (> 35), showing that it is probable that these clays cause not only problems of stability but also of settlement, swelling and implementation (ALONSO et al., 2002). Some values of the internal cohesions of the ediments of the two wells are greater than 20 (22kPa for the well 1 and 49.58 kPa for the well 2). So these parameters are in agreement with the granulometric data mentioned above. On the other hand, the sediments of well 2 (consisting of mixtures of clay and sand) have a varied grain size (between 21.3% and 58.51% fines). These sediments give better geotechnical characteristics, because of the increase of the rate of sand. To test this hypothesis, mixtures of sand (5%, 15%, 25%, 35%, 45%, 55% and 65%) and clay were made. The mechanical characteristics of this experiment are gradually improved with the reduction of fines. The best mixtures are obtained with respective proportions of fines of 21.3% clay, 0.5% of loam, 68.2% of coarse sand and 10% of gravel provides significantly improved parameters: 6.9% of water content and 21.3% , 26.3%. Among these two mixtures, that composed of 21.3% of clay and 78.7% of sand gives the mechanical characteristics favorable to a sediment use of this texture, for the realization of small dikes and bararages. With regard to particle size, this criterion can be achieved when the clay-size particle content (<2%) exceeds 15% and the percentage of fine particles exceeds 30%. Indeed, Daniel (1987) and Kenney et al. (1992) determine that the hydraulic conductivity decreases as the clay-sized particle amount increases in a material, up to 12% and remains constant for larger values. According to Appolonia (1980), the hydraulic conductivity is inversely proportional to the percentage of fine particles. Shakoor and Cook (1990) and Shelley and Daniel (1993) determine above 50% the percentage of large grains (gravel) contributes to increase the hydraulic conductivity of the soil. For the Atterberg limits, the watertightness is reached when the liquidity limit and the plasticity index exceed respectively 20 and 7, which shows that the Gbédji-Kotovi clay sediments can constitute a watertight dike barrier. As regards the conductivity, it decreases with the increase of the activity, therefore with the increase of the increase of the percentage of fines. In view of all the foregoing, it is important that the following studies be done to deepen the susceptibility of Gbédji-Kotovi clays to be used in the construction of dikes:

i) The permeability test on site and in the laboratory, to assess the operating condition of the dike;

ii) Consolidated shear test and drains, to assess long-term soil behavior;

iii) The compressibility test at the oedometer to assess the swelling of said soil.

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