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Characterization of the Ngouache Landslide by Geotechnical Survey (Bafoussam, Cameroon)

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Abstract

The heavy rainfall during the night of October 28-29, 2019, caused a landslide in the Ngouache 4 neighborhood in Bafoussam in the West Cameroon region. This landslide caused 43 deaths, several missing persons and the destruction of several houses and plantations. The present investigation attempts to find the causesof this landslide. First, the landslide area was identified and heavy dynamic penetrometer surveys were conducted. Next, intact and reworked samples were taken and, finally, slopes were measured on the slide area to determine the soil parameters. The results of the heavy dynamic penetrometer showed the presence of altered clay layers which, when saturated, exert hydrostatic pressure and unbalance the soil. The granulometric analysis noted a soil with more than 50% of grains of ϕ>80µm with liquidity limits between (50-65.3%), a consistency index between (1.01-1.24), a low cohesion between (0.22-0.30 bars) for a high internal friction angle (20.72°-23.86°), slopes between 48° and 56° and a degree of saturation Sr between (80.5° and 139.9°). These characteristics associated with the large water columns of the night of October 28 to 29, 2019 demonstrated that the water had exerted a hydrostatic pressure having made the clay material reach the limit of plasticity to the creep and raised the level of the water table. Thistherefore promoted the saturation of the substrate yet the increase in the water content reduces the cohesion of the materials and that triggered the saturation at the origin of this deadly slide. The results of the present research may help the public authorities in the decision making regarding the characterization and the securing of risk zones.

Keywords: Landslide, risk zone, survey, sampling, saturation, substrate, creep

1. Introduction

The disorders related to anarchic constructions influence the stability of certain natural factors such as the relief and the nature of the ground. Considering the lack of housing and the phenomenon of rural exodus, populations generally settle in this type of areas (lowlands, hillsides, etc.) which causes erosion as well as landslides due to high rainfall, earthquakes or other geological phenomena (Lee W. Abramson & al., 2002).

From the above, it seems urgent to delimit the causes of these multiple landslides in order to limit the loss of human life as well as the material losses that often result from these catastrophes. For this purpose, geotechnical surveys were carried out on the Ngouache site in Bafoussam, a town in West Cameroon, with the hypotheses that these natural disasters were caused by heavy rainfall, seismic activity, mountain growth or heavy urbanization.

To this end, the results of the direct shear test and those of the geotechnical characterization tests (granulometric analysis, water content, specific weight, Atterberg limits, etc.) will make it possible to evaluate the impact of rainfall on landslides. The oedometric tests and the geotechnical characterization tests will allow us to perceive the compactness of the area and to discuss the impact of the mountain ranges growth on these phenomena.The comparative studies of the various earthquakes (with the occurrence of several landslides extracted from the seismological data of Cameroon) will on their part allow us to rule on the impact of the seismic activity of the volcanic line of Cameroon on landslides. Finally, the impact of urbanization on landslides will be evaluated based on observations of their location (urban or rural) as well as their dates of occurrence.

2. Presentation of the study area

The climate of the Western region is temperate overall and it depends on a unity of factors related not only to the relief and exposure of this region to the major atmospheric currents but also to its geographical position and distance from the sea (Tsalefac 1999). Annual rainfall is abundant (1400 mm ≤ Pmoy≤ 2500 mm) with peaks between July and October.This decreases from the South to the North while increasing in the vicinity of the mountainous massifs.

The city of Bafoussam is characterized by a mountain range located between 5-7° North latitude and 9-11° East longitude. It comprises a series of plateaus that are in some places like plains or collapse basins with altitudes varying between 700 and 2740 m. These highlands are all ranged along the N30°E direction of the Cameroon volcanic line. The geomorphological disasters counted in this region are landslides and block falls while the most common meteorological disasters are storms and floods (BallaAboubakar& al. 2014).

The hydrographic network is related to the morphology of the region. The forest and tree savanna vegetation is heavily anthropized by a predominantly agricultural population. Geologically, the region is dominated by a wide variety of petrographic formations. In general, volcanic formations (rhyolitic and trachytic basaltic flows and basaltic and ignimbritic projections) of Tertiary age (Tchoua 1974) rest on the Pan-African granite-gneissic basement. All along the mountain ranges of this region, scars of ancient land movements and evidence of aborted or ongoing movements can be observed.

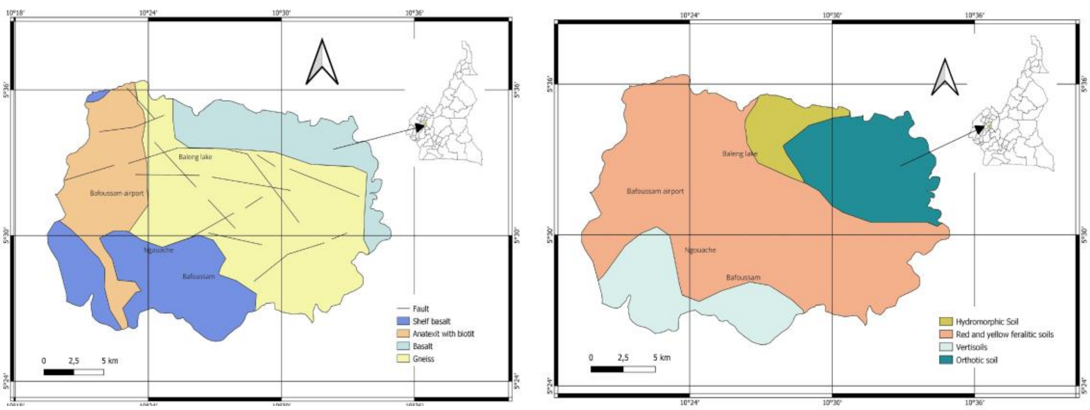

Figure 1. Reduced Geological and Pedological Maps of west Cameroon, Martin and Segalen, 1966.

3. Methods and materials
3.1 Stability of slopes by the FELLENIUS method

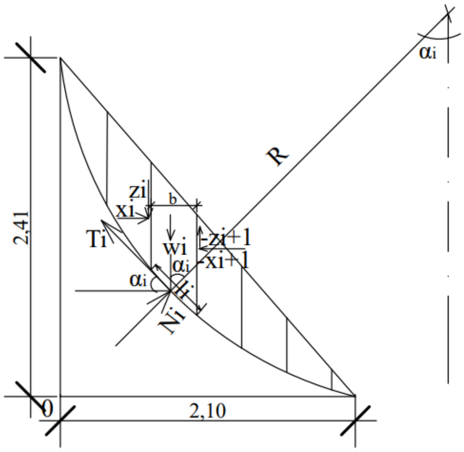

Figure 2. Study of the stability of slopes

Expression of the stress component at the base of the slice

$$\sigma i = Ni / li \quad (1)$$

$$ri = Ti / li \quad (2)$$

$$\sigma' i = N' i / li = \sigma i - ui \quad (3)$$

$$r' i = ri \quad (4)$$

r being a 1/F portion of the maximum intensity, we have :

$$ri = 1/F * (ci + \sigma' itg\phi' i) \quad (5)$$

$$Ti = 1/F * (cili + N' itg\phi' i) \quad (6)$$

$$avec N' i = Ni - uili \quad (7)$$

The balance of the slice of order "i" is written :

$$W_i + X_i - X_{i+1} + T_i + N_i = 0 \quad (8)$$

Combining the expression of Ti with the characteristics of the slice, we have:

$$\mathrm{Wi} - N\mathrm{icos}\,\alpha\,\mathrm{i} - Ti\sin\alpha\,\mathrm{i} = 0 \quad (9)$$

from which

$$Ni = \frac{\mathrm{Wi} - Ti\sin\alpha\,\mathrm{i}}{\cos\alpha\,\mathrm{i}} \quad (10)$$

Transferring in the expression of Ti we have :

$$Ti = \frac{cili + (\frac{Wi - Ti\sin\alpha i}{\cos\alpha i} - uili)tg\phi' i}{F} \quad (11)$$

With Ni and Ti respectively normal stress and tangential stress
Applying the method of FELLENIUS on the calculation of the safety coefficient of the slopes, we have:

$$F = \frac{\sum\left[cibi + (wi\cos^2\alpha i - uibi)tg\phi' i\right] / \cos\alpha i}{\sum_1^n w\sin\alpha i} \quad (12)$$

from which

$$F = \frac{\sum_1^n\left[cb + (N - uw)tg\phi\right]}{\sum_1^n w\sin\alpha i} \quad (13)$$

3.2. Characterization of geotechnical parameters

3.2.1. Sounding with heavy dynamic penetrometer

The test was conducted on the field according to the NF P 94-115 standard for type B penetrometers (P.ANTOINE and D.FABRE 1980).

$$Q_d = \frac{M^2H}{Ae(M+P)} Q_d = \frac{M^2H}{Ae(M+P)} \quad (14)$$

Table 1. GPS coordinates

DPDL1			DPDL2		
32N	0654198	GPS 3^m	32N	0654149	GPS 3^m
UTM	060549		UTM	0606580	

3.2.2. Collection of intact and reworked samples

Intact samples were carefully collected using paraffin-embedded PVC pipes ($h = 25cm, \phi = 18\,\text{cm}$), labeled and taken to the laboratory for geotechnical studies. Similarly, the reworked samples were collected in labeled nylons bags and transported to the laboratory.

3.3. Granulometric analysis

Granulometric analysis were carried out by sieving according to the standard (NF P 94-056) and sedimentation (NF P 94-057) in order to obtain granulometric curves. The test was performed according to the procedure described by (Lanchon 1983).

3.3.1. Water content of the soil

According to the standard (NF P 94-050)

$$w = \frac{(W-T)-(Wd-T)}{(\mathrm{Wd}-\mathrm{T})} x100$$

(15)

3.3.2. Limits of Atterberg

According to the standard (NF P 94-051), the test was performed according to the procedure described by (Dupain et al. 1995).

3.3.3. Direct shear test (NF P 94074)

To test identical specimens in order to have different normal stresses, the values of shear stresses at failure are represented as a function of the normal stresses $\tau = f(\sigma)$ from which we have the graphical translation of the coulomb equation $\tau_{rupt} = c + \sigma_{rupt} tg\phi$

(16)

in drained consolidation and the cohesion **c** and the friction angle ϕ can be determined.

3.4. Oedometric test (XP P94-090-1)

This test makes it possible to determine the parameters of consolidation of clay soils as well as their permeability and settlement. The principle of the test consists in the uniaxial compression of a sample of soil maintained laterally by a rigid wall.

- The compression index

$$C'c = \frac{\Delta e}{\Delta(\log \sigma')};$$

(16)

- The consolidation coefficient $Cv = \frac{k}{mv * \gamma w}(m^2/s)$ (17)
- The compressibility coefficient $av = \frac{\Delta e}{\Delta \sigma'}$ (18)
- The volumetric compressibility coefficient

$$mv = \frac{\Delta e}{(1+eo)\Delta\sigma'}(\mathrm{P}a^{-1})$$

(19)

-The oedometricmodulus $E' = \frac{1}{mv}(Pa)$ (20)

Fig 3. Extraction of intact samples

4. Results and discussion

4.1. Results

The general observation of the study site allowed us to identify the pullout niches and to survey the slopes in order to study their stability by applying the FELLENIUS method.

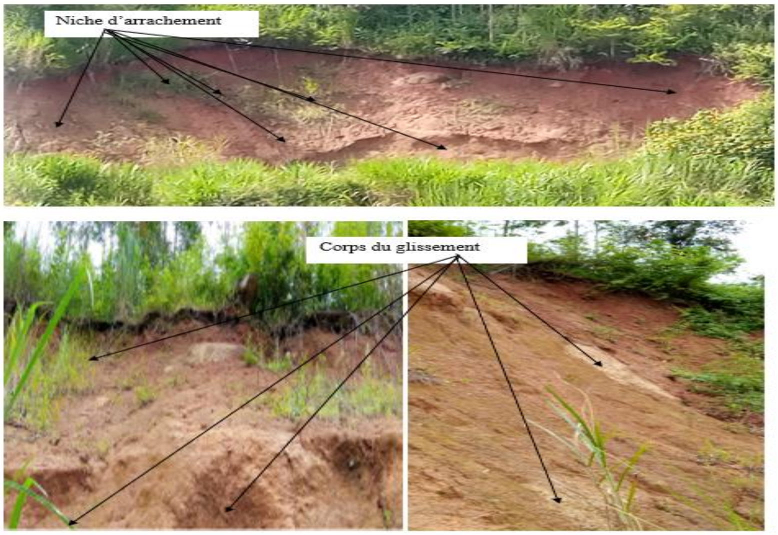

Figure 4. Location of the study area

4.1.1. Heavy dynamic penetrometer

The results of the tests on the two sounding points show refusals at relatively low depths, i.e. 7m for the first point and 7.2 for the second point, which suggests the low thickness of the soils in this mountainous area. The data sheets revealed the presence of a superficial clay layer overlying a water table at an average of 1.4 m. However, an average stress, according to the two stations of 1 Mpa, is reached at a depth of about 5m.

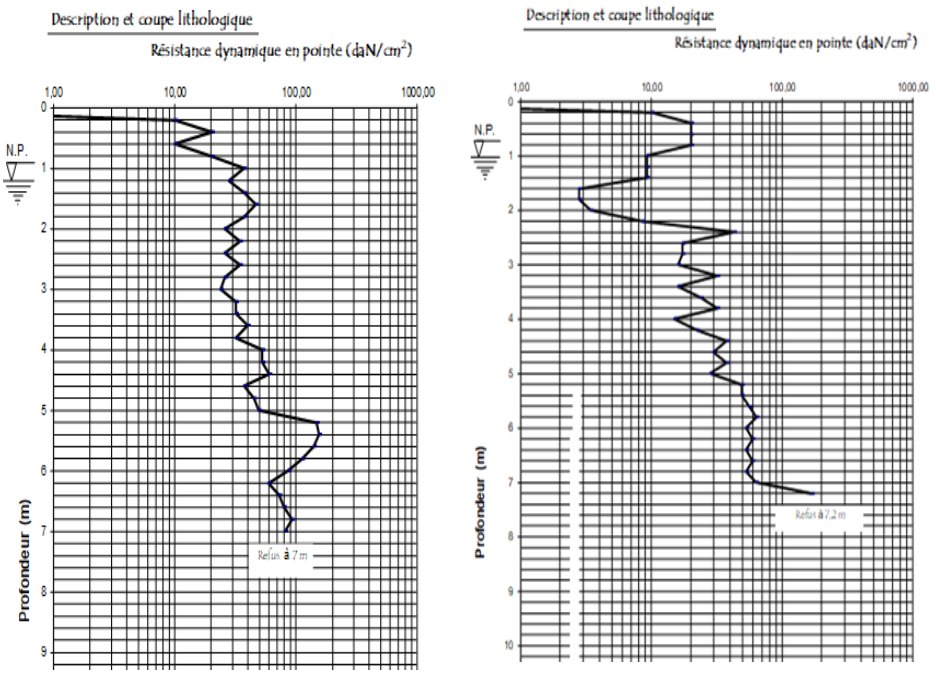

Figure 5. Penetrogram

4.1.2. Identification tests

The geotechnical characterization tests yielded the results summarized in Table 2. For reasons of convenience, we opted for a case by case analysis of the different samples. Thus, for the samples taken on the site of NGOUACHE, the following results wereobtained:

Table 2. Results of particle size analysis

	Prof (cm)		Wnat	A.G % passing through the sieve (mm)								
	De	A	(%)	31,5	25	20	16	10	5	2	0,5	0,08
1	0	100	30,4	100	100	100	100	100	100	99	85	62
2	0	100	29,7	100	100	100	100	100	100	80	65	45
3	0	100	29,4	100	100	100	100	100	100	99	91	68
4	0	100	33,4	100	100	100	100	100	100	98	93	69

- For well P1: a liquidity limit of about 50.3%; a plasticity limit of 30.6%, a plasticity index of 19.7% for a consistency index of 1.01%. We note that it is composed of 14% of fine particles and has a density of grains of 16.95KN/m3
- For well P2: a liquidity limit of 59.8%; a plasticity limit of 38.1%, a plasticity index of 21.8% for a consistency index of 1.24. Note that it is composed of 8% of fine particles and has a density of grains of 16.95KN/m3.
- For well P3: a liquidity limit of 54.5%; a plasticity limit of 34.4%, a plasticity index of 20.1% for a consistency index of 1.24. Note that it is

composed of 10% of fine particles and has a density of grains of 16.95KN/m3.
-For well P4: a liquidity limit of 65.3%; a plasticity limit of 36.2%, a plasticity index of 29.1% for a consistency index of 1.09. We note that it is composed of 20% of fine particles and has a density of grains of 16.95KN/m3.

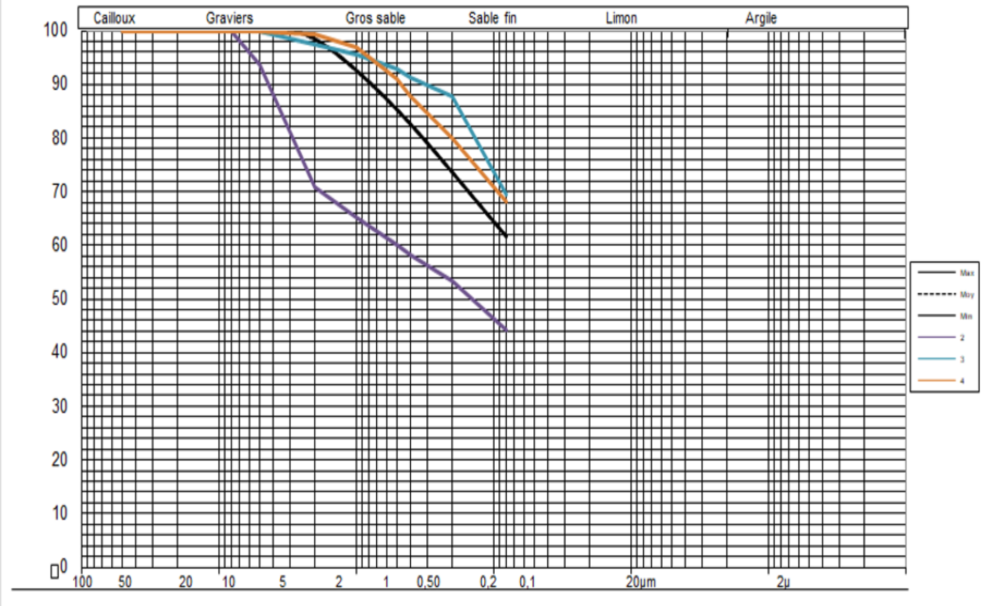

Fig 6. Size curves of materials taken from the soil P1, P2, P3 and P4

4.1.3. Oedometric test

In this part, the settlement of the soil samples and the consolidation ratio will be evaluated to determine the nature or behavior of the soil.

In the sample PEI 01, the void index at discharge is $e_o = 0,815$ for a displacement $\Delta h = 143x10^{-2}mm$; the vertical stress $\sigma_{vo} = 0,27bar$; a pre-consolidation stress $\sigma_{p'} = 0,70bar$; a compression index $Cc = 0,322$; a recompression index $Cs = 0,087$ which gives an over-consolidation ratio $Roc = 2,59$

In the sample PEI 02, the vacuum index at discharge is $e_o = 0,621$ for a displacement $\Delta h = 356x10^{-2}mm$; the vertical constraint $\sigma_{vo} = 0,09bar$;a pre-consolidation constraint $\sigma_{p'} = 0,90bar$;a compression index $Cc = 0,320$; a recompression index $Cs = 0,1$ giving an over-consolidation ratio $Roc = 10$

In the sample PEI 03, the vacuum index at discharge is $e_o = 1,334$ for a displacement $\Delta h = 49x10^{-2}mm$;the vertical constraint $\sigma_{vo} = 0,263bar$; a pre-consolidation constraint $\sigma_{p'} = 0,85bar$;a compression index $Cc = 0,444$;a recompression index $Cs = 0,024$ which gives an over-consolidation ratio $Roc = 3,23$

4.1.4 Direct shear test

In this section, the criterion for soil failure under loading will be evaluated.

In well PEI 01, there is a friction angleφ_r(°) = 22,9°,cohesion C_{CD} =0,22bar,for a displacement δI= 1,76 mm. The soil at this level has a collapsible elasto-plastic behavior (see Fig. 9). This shows that the soil has a low cohesion and can slip under the effect of heavy rainfall or constant loading.

Table 3. Characteristics of PEI 01 soil specimens

Name of the sample : PEI 01 ; Test carried out in accordance with standard NFP94-071-1									
CHARACTERISTICS OF THE SAMPLE							Calibration coefficient (daN/mm)		
Approximate nature of the soil									
Reddish clay							**0.56**		
CHARACTERISTICS OF THE SOIL SAMPLES									
After shearing									
N°1	**Parameters**								
	gd(t/m^3)	W(%)	σ_n(bars)	t $_{f,rés}$ (bars)	t $_{f,pic}$ (bars)	σ_n (bars)	Data atpeak		
							φ_p(°)	**C_{CD}(bars)**	**δI (mm)**
1	1,363	31,8	0,50	0,396	0,396	1,363	22,94	0,22	1,76
2	1,368	30,5	1,00	0,689	0,689	1,368	**Residual data**		
3	1,361	31,8	2,00	1,047	1,047	1,361	**φ_r(°)**	**C_{CD} (bars)**	**δI (mm)**
Average							22,94	0,22	1,76

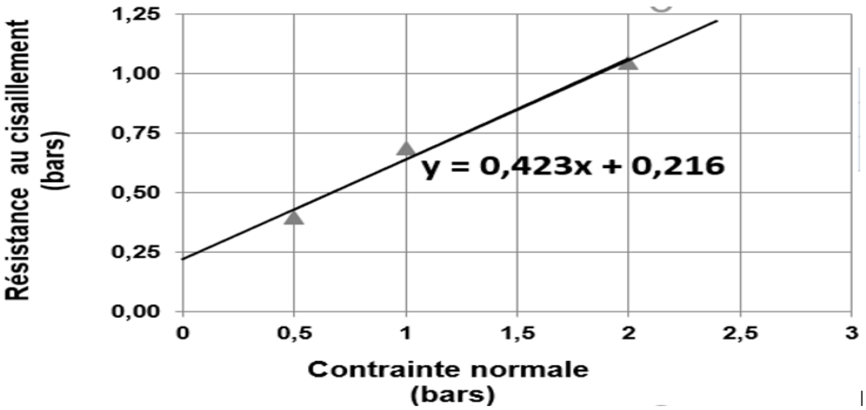

Fig 7. Cohesion and friction angle of PEI 01

In the well PEI 02, there is a friction angle $\varphi_r(°) = 20,72°$,cohesion $C_{CD} = 0,27 bars$, for a displacement δI= 1,65 mm. The soil at this level has a collapsible elasto-plastic behavior (see Fig10). This shows that the soil has a low cohesion and can slip under the effect of heavy rainfall or constant loading.

Table 4. Characteristics of PEI 02 soil specimens

Name of the sample : PEI 02 ; Test carried out in accordance with standard NFP94-071-1								
CHARACTERISTICS OF THE SAMPLE						Calibration coefficient (daN/mm)		
Approximate nature of the soil								
Reddishclay						**0.56**		
CHARACTERISTICS OF THE SOIL SAMPLES								
Aftershearing								
N°1	**Parameters**							
	g_d (t/m^3)	w (%)	σ_n (bars)	t $_{f,rés}$ (bars)	t $_{f,pic}$ (bars)	Data atpeak		
						φ_p(°)	**C_{CD} (bars)**	**δI (mm)**
1	1,446	33,2	0,50	0,407	0,407	20,72	0,27	1,65
2	1,409	34,3	1,00	0,764	0,764	**Residual data**		
3	1,404	34,0	2,00	1,031	1,031	**φ_r(°)**	**C_{CD} (bars)**	
Average						20,72	0,27	1,65

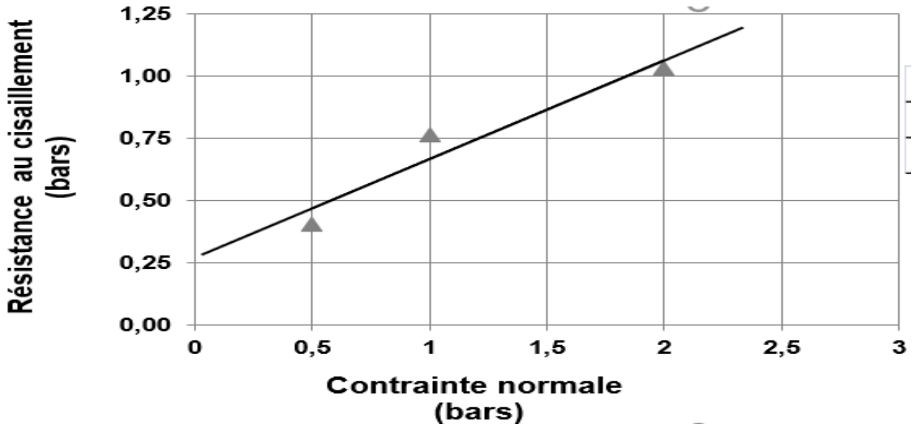

Fig 8. Cohesion and friction angle of PEI 02

In well PEI 03, there is a friction angle $\varphi_r(°) = 23,86°$, cohesion $C_{CD=} = 0,30 bars$, for a displacement δI = 2,26 mm. The soil at this level has a collapsible elasto-plastic behavior (See Fig 11). This shows that the soil has a low cohesion and can slip under the effect of heavy rainfall or constant loading.

Table 5. Characteristics of PEI 03 soil specimens

Name of the sample : PEI 03 ; Test carried out in accordance with standard NFP94-071-1								
CHARACTERISTICS OF THE SAMPLE						Calibration coefficient (daN/mm)		
Approximate nature of the soil								
Reddishclay						**0.56**		
CHARACTERISTICS OF THE SOIL SAMPLES								
Aftershearing								
N°1	**Parameters**							
	g_d (t/m^3)	w (%)	σ_n (bars)	t $_{f,rés}$ (bars)	t $_{f,pic}$ (bars)	Data atpeak		
						φ_p(°)	**C_{CD} (bars)**	**δI (mm)**
1	1,335	35,1	1,01	0,761	0,761	23,86	0,30	2,26
2	1,348	38,3	2,10	1,144	1,144	**Residual data**		
3	1,360	33,0	4,20	2,094	2,094	**φ_r(°)**	**C_{CD} (bars)**	**δI (mm)**
Average						23,86	0,30	2,26

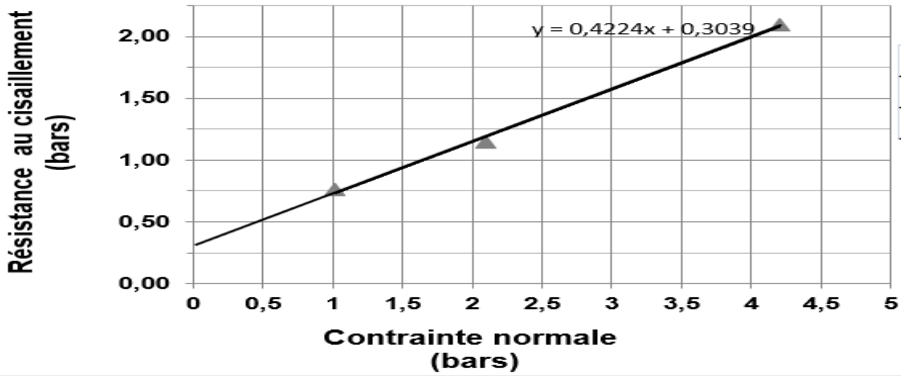

Fig 9. Cohesion and friction angle of PEI 03

In well PEI 04, there is a friction angle $\varphi_r(°) = 22,30°$, cohesion $C_{CD=} = 0,27 bars$,for a displacement δI= 1,74 mm. The soil at this level has a collapsible elasto-plastic behavior (See Fig 12).This shows that the soil has low cohesion and can slip under heavy rainfall or constant loading.

Table 6. Characteristics of PEI 04 soil specimens

Name of the sample : PEI 04 ; Test carried out in accordance with standard NFP94-071-1								
CHARACTERISTICS OF THE SAMPLE						Calibration coefficient (daN/mm)		
Approximate nature of the soil								
Reddishclay						**0.56**		
CHARACTERISTICS OF THE SOIL SAMPLES								
Aftershearing								
N°1	**Parameters**							
						Data atpeak		
	g_d (t/m^3)	w (%)	σ_n (bars)	t $_{f,rés}$ (bars)	t $_{f,pic}$ (bars)	**φ_p(°)**	**C_{CD} (bars)**	**δI (mm)**
1	1,318	37,7	0,50	0,667	0,667	22,30	0,27	1,74
2	1,337	36,3	1,00	1,050	1,050	**Residual data**		
3	1,320	34,3	2,00	1,836	1,836	**φ_r(°)**	**C_{CD} (bars)**	**δI (mm)**
Average						22,30	0,27	1,74

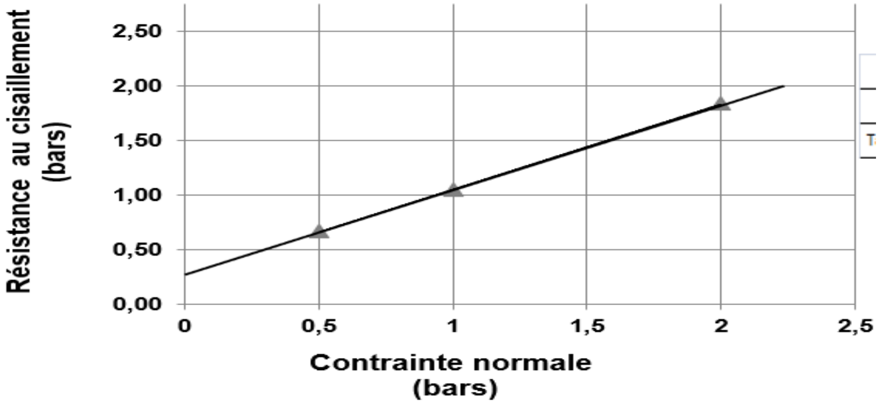

Fig 10. Cohesion and friction angle of PEI 04

4.2. Discussion

4.2.1. Geotechnical classification

Based on the values of plasticity indices, percentages passing 80µm and group indices, these soils were identified according to the H.RB classification. This is how the sampled materials were identified. A low percentage rate of fine particles could be found and this allowed us to classify this soil as a gritty soil according to the classification of LPC (Road and Bridge Laboratory) because more than 50% of the solid elements have a ϕ >80µm with a consistency index >1, which characterizes a high permeability and the flow of water is therefore very fast.

Table 7. Values of the consistency limits HRB classification

Well 1	wL(%)	50,3			Content in MO	VBS	Class.HRB
	Wp(%)	30,6	$\gamma_s(0/0,4)$				
	Ip(%)	19,7	w%nat(%)	30,4		0,3	A-7-5(6)
Well 2	wL(%)	59,8	$\gamma_s(0/20)$		Content inMO	VBS	Class. HRB
	Wp(%)	38,1	$\gamma_s(0/0,4)$				
	Ip(%)	21,8	w%nat(%)	29,7		0,3	A-7-5(6)
Well 3	wL(%)	54,5	$\gamma_s(0/20)$		Content inMO	VBS	Class.HRB
	Wp(%)	34,4	$\gamma_s(0/0,4)$				
	Ip(%)	20,1	w%nat(%)	29,4		0,5	A-7-5(6)
Well 4	wL(%)	65,3	$\gamma_s(0/20)$		Content inMO	VBS	Class.HRB
	Wp(%)	36,2	$\gamma_s(0/0,4)$				
	Ip(%)	29,1	w%nat(%)	33,4		0,7	A-7-5(6)

4.2.2 Impact of volcanic activity on the Ngouaché slide

It can be notedthat the over consolidation ratio is greater than 1 in all samples, which allows one to deduce that this soil is over consolidated. Secondly, the particle size analysis shows that the percentage of fine particles in these soils is less than 30% and therefore not assimilable to their fine particles; these soils are therefore assimilable to their coarse particles resulting from the metamorphism of the thin and consolidated volcanic structure.

Besides, there is no coincidence between the different landslides. Moreover, the presence of swelling clays during heavy dynamic penetrometer drilling has repercussions on the stability of the slopes because they are very sensitive to humidity variations; this analysis thus agrees with that of V. ROBITAILLE and D. REMBLAY (1997). The Casagrande diagram shows that the materials have a high liquidity limit (50-65.3%). These materials are very fluid and likely to play the role of "soap layer", as envisaged by BallaAboubakar& al (2014).

4.2.3 Impact of urbanization on the Ngouaché landslide

Table 8. Values of safety factors as a function of pore pressure

u	**0,5**	**0,55**	**0,6**	**0,65**	**0,7**
Fs	**1**	**0,82**	**0,61**	**0,39**	**0,18**

The low cohesion (0.22-0.30 bar), high internal friction angle (20.72°-23.86°), and pore pressure greater than 0.5 bar indicate an unstable balance during water saturation to join the research of (R.E MAYER 2003).

The slope is very steep (48-56°) in this site, high enough to create the gravity contrast and induce the mass movement to join the work of (TANGMOUO TSOATA & Al, 2020).

Triggering factors

Water is the main trigger for the Ngouaché landslide. This locality receives a lot of rainfall (>1500 mm per year) (Olivry 1986) with peak rainfall in August (501.5 mm), September (480 mm) and October (344 mm). The region is endowed with superficial aquifers (Aboubakar 2010) that are rapidly recharged, given the fissured structure of the formations that overlie it. Fluctuations in water level related to weather conditions contribute to the increase in water content of soil and subsoil materials. Some authors (Costet&Sanglerat 1975) agree on the fact that as soon as the water content of the clays exceeds the bound water content and free water appears, the resistance of the materials drops. The latter thus pass from the solid state to the plastic state and then to the liquid state. At Ngouaché, the water has more or less saturated the soils by a degree of leachability of the materials from which there areAtterberg limits (Wp=30.6-38.1%), WL = (50.3-65.3%). All this has weakened the stability of the soil and triggered the instantaneous and brutal displacement of a large quantity of material on the Ngouaché site.

Anthropogenic factors

The human action that contributed to the weakening of the stability of the Ngouaché slope can be summarized in two factors:

Deforestation and the cultivation of this steep slope have contributed, through regular plowing, to loosening the surface levels and increasing the infiltration rate;

The exploitation of the gneiss for the foundations of the houses: the populations dig to have healthier blocks, these hollows fill with rainwaters and thus contribute to feed the deep layers.

Conclusion

The study of the Ngouache landslide aimed to assess the degree of risk and the triggering phenomenon of the landslide that occurred during the night of 28 to 29 October 2019 with a toll of 43 deaths and numerous material damages. To achieve this, the present work was based on the mechanical characterization of materials and heavy dynamic penetrometer surveys to know the strength of the soil layers. The following results were obtained:

(1) The heavy dynamic penetrometer soundings showed a variation of the stresses from which, for the second sounding point, we crossed a water table between 1.50m and 2.00m. The clay in this area is not visible enough but it is felt by the effects.

(2) From the Atterberg limits and particle size analysis, we obtained a very high percentage of coarse elements of ϕ >80µm with a percentage ranging from 50% to 85% and a low rate of fine particles (14%;8%;10%;20%) depending on the sampling wells. The plasticity limits give percentages (30.6% ;38.1% ;34.4% ;36.2%) depending on the sample wells while the liquidity limits have a very high percentage (50.3% ;59.8% ;54.5% ;65.3%).

(3) For the special tests, analyses to the oedometric compressibility of the sampled wells were made and they gave indices of voids respectively with a ratio of over consolidation; as for the direct shear of the sampled wells, there are high angles of internal friction between 20.72° and 23.86° with a low cohesion between 0.22 bar and 0.30bar with a degree of saturation Sr (80.5°-139.9°).

From all the above, it can be said that the soil studied is powdery with a low rate of fine particles that are affirmed by an over consolidationratio. It can therefore be saidthat the heavy rainfall of the night of 28 to 29 October

2019 is the cause of the slide because the materials reached a very high liquidity limit which participated in the leachability of fine particles that played a role of cohesion between solid particles. This is justified by the low cohesion and a high angle of friction, hence the collapse of the boulders that made this slide deadly. With the presence of a perched water table and laboratory results, it can be concluded that saturation is the cause of this deadly landslide.

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