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Seismic analysis of an earth dam in a tropical geologic context

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ABSTRACT: A numerical model to investigate the seismic behavior of an earth dam currently under construction in the French West Indies is presented in this article. Tropical residual soils are used to build the dam. In a first step, the geomechanical parameters for the Modified Cam-Clay constitutive model (MCC) were deduced from oedometer and triaxial tests. The constitutive predictions are then validated using two other triaxial paths and experimental measurements. A finite difference model for the complete dam is then defined, following the actual design of the structure as well as the in-situ hydro-geotechnical conditions. The dam response to an input accelerogram that was generated from the seismic event of Les Saintes (M_w= 6.3, 2004) is applied at the base of the earth dam foundation. Large movements are observed, which are acceptable for the dam's height.

1 INTRODUCTION

The water supply for agricultural activities in Guadeloupe (French West Indies), requires the construction of several new dams located around the Island of Basse-Terre. These dams are usually constructed from local soil that covers the mountainous areas at the base of the local volcano chain and belongs to the class of tropical residual soils.

Residual tropical soils were formed by in situ weathering of the source rocks, due to a continuous change in soil particle properties resulting from chemical processes such as leaching and precipitation. This implies a progressive alteration of the soil microstructure that erases the effects of the stress history. Therefore, it is to be considered that the soils are normally consolidated. Their hydromechanical properties are also highly variable in terms of deformability, mechanical strength and permeability (Mahalinga and Williams 1995, Fookes 1997, Futai 2004, Nwaiwu 2005, Benatti 2013).

As the considered area is extremely seismic due to subsidence of the Atlantic plate beneath the Caribbean plate, dam designs should consider the seismic loading. The region is classified in zone 5 which corresponds to a high seismicity hazard in view of the European code of seismic construction (Eurocode 8). Different methods exist for this type of design, but the French authorities request that the new dam projects should be justified by a time-dependent poromechanical coupled numerical model based on a true accelerogram.

As the first stage of this study, as the knowledge of the behavior of these materials under seismic solicitations was not well understand, a research program was engaged between Antea Group and IRSTEA. This research combines laboratory characterization of the local tropical residual soils and numerical modeling under seismic solicitations.

Numerical calculations are done using a finite difference software (Flac - Itasca, 2011). As a first research step, a simplified modeling was carried out using the modified Cam-Clay constitutive model (Brenner, 1997, Nagendra Prasad, 2013). The choice of this model (MCC) is based on its simplicity in terms of the input parameters number, applicable for normally consolidated soils and which permits to consider the soil hardening.

Table 1. Hydrogeological parameters

Layers	Horizontal permeability (m/s)	Vertical permeability (m/s)	Porosity
H2	5.10^{-6}	5.10^{-7}	0.5
H4	5.10^{-5}	5.10^{-5}	0.5
H5	5.10^{-8}	5.10^{-8}	0.5
Bank	5.10^{-7}	5.10^{-7}	0.5
Wall	5.10^{-9}	5.10^{-9}	0.5
Drain	1.10^{-4}	1.10^{-4}	0.5

Calibration and validation of the modified Cam-Clay model input parameters for tropical residual soils are presented then a numerical modeling considering a study case of an embankment dam under solicitations is carried out.

2 GEOTECHNICAL CONTEXTS

2.1 *Geological context*

Two geotechnical surveys were carried out on the site with different types of boreholes and measurements: core sampling, standard penetration, pressuremeter and laboratory tests. The geological and geotechnical analysis lead to define five main strata below the ground level (Figure 2). The list of these layers is (beginning from the surface): ocher silty clay with a thickness of 10.5 m (named H2), weathered conglomerate (thickness of 7 m H4) and conglomerate (more than 40 m of thickness – named H5).

2.2 *Hydrogeological context*

The presence of groundwater was highlighted during the geotechnical investigation, with water levels encountered at a depth varying between 0.2 and 3.8 m under the ground surface. Considering the geotechnical data and the knowledge gathered from different projects carried out in Guadeloupe on similar geotechnical contexts, it was decided to consider in the geotechnical model a water level above the ground surface (50cm). In situ permeability tests were carried out using Lefranc type test (AFNOR, 2000). Table 1 presents the results of these different tests.

3 CONSTITUTIVE MODELS USED

The constitutive model which was chosen to represent the tropical residual soils behavior (layers H2, H4, H5 and embankment)is the Modified Cam-Clay (Roscoe K.H, 1968). Its features include nonlinear elasticity, and a hardening/softening behavior governed by volumetric plastic strains. The yield function for the Modified Cam-Clay model is expressed using a specific value of the mean consolidation pressure p_c which is given by the following formula:

$$q^2 + pM^2(p - p_c) = 0 \tag{1}$$

Where: M= frictional constant, q= Shear stress; p= mean principal stress

The Modified Cam Clay Model assumes an associated plastic flow, the flow rule is given by the expression:

$$\frac{d\varepsilon_d^p}{d\varepsilon_v^p} = \frac{2q/p}{M^2 - q/p} \tag{2}$$

with:
$d\varepsilon_d^p$ = Incremental plastic shear strain
$d\varepsilon_v^p$ = Incremental plastic volumetric strain

The parameters of the Modified Cam-Clay model were calibrated using drained triaxial test on the H2 layer and the results of an oedometer test. Calibration was done on the deflection (q) versus axial deformations (εa) curves and the volumetric deformation (εv) versus the axial deformation (εa) curve for the first test with σ3 = 246 kPa. The validation of the model was based on two other tests (σj = 331 and 534 kPa), keeping the same parameters.

The selected geotechnical parameters, mass density (ρ), frictional constant (M), specific volume at reference pressure (V_λ), slope of swelling line (*k*) and slope of normal consolidation line (λ) and the Mohr-Coulomb parameters are given in Table 2.

The size of the yield surface is determined by the consolidation pressure p_c, which depends on the loading history. As the weathered tropical soils are generally considered as normally consolidated, the consolidation pressure in the foundation layers is proportional to the in situ vertical stress:

$$Pc = \sigma_{v_0}(1 + 2K_0)/3 \tag{3}$$

where K_0 = the horizontal earth pressure coefficient at rest.

It can be seen in Figure 1 and Table 2 that the input parameters choice permits to obtain a good concordance with the experimental curves. It suggests that the MCC model is relevant for the mechanical modeling of the tropical residual soil considered in this study. The main drawback of this constitutive model is the fact that it is not possible to consider the soil density evolution with the cyclic loading. Since no cyclic test was performed for this first behavioral characterization analysis, the MCC is adopted for the numerical study. This is a simple model that does not require many input parameters.

Table 2. Properties of the soil materials and the wall

Layers	Model	ρ (t/m^3)	M	ν_λ	λ	*k*	Pc (kPa)	E (MPa)	ν	C' (kPa)	Ø'(°)
Bank	MCC	1.7	1.28	4.9	0.009	0.006	20	-	-	-	-
H2	MCC	1.6	1.28	4.9	0.009	0.006	73	-	-	-	-
H4	MCC	1.7	1.28	4.9	0.009	0.006	220	-	-	-	-
H5	MCC	1.7	1.28	4.9	0.009	0.006	400	-	-	-	-
			1.28	4.9	0.009	0.006					
Drain	MC	1.7	-	-	-	-	-	200	0.33	5	33
Cutoff Wall	MC	2.3	-	-	-	-	-	1 000	0.2	5000	30

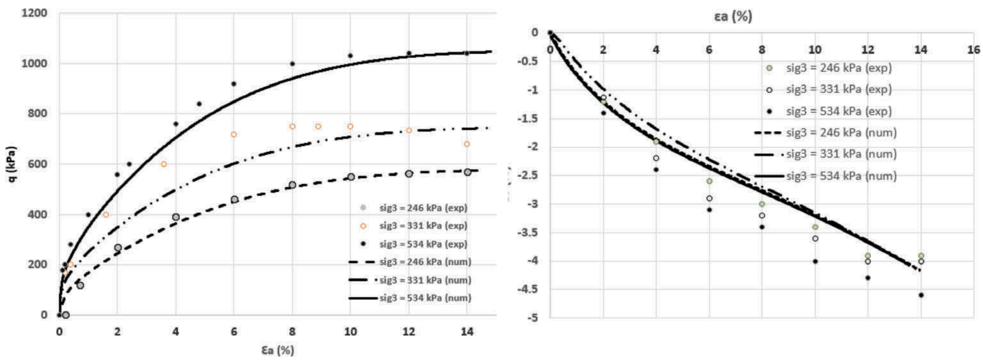

Figure 1. Deflection curves (q) as a function of the axial deformations εa (left) and volumetric deformation εv as a function of the axial deformation εa (right)

For the sake of simplicity, the cutoff upstream wall and the drain are modeled using a linear elastic perfectly plastic constitutive model with a shear failure criterion of Mohr Coulomb type. This model is named MC. This model necessitates 5 input parameters:Young modulus (E), Poisson's ratio (ν), cohesion (c'), friction (Ø'), and dilation angle (ψ).

4 NUMERICAL MODEL AND INPUT DATA

4.1 *Dam geometry*

The geometry of the dam in represented in Figure 2. The altimetric levels are expressed above the reference of the mean sea level in Cayenne (NGG). The length of the dam is assumed to be sufficiently large (413.5 m) to use the plan strain condition in the numerical modelling. Computations are made using a finite difference software FLAC (Itasca, 2011).

Considering the natural topography of the valley, the dam is 20m high and the upstream and downstream slopes are inclined of 19 degrees (slope 3H/1V). The crest width is 5m. The lack of material with low hydraulic conductivity ($k_h \leq 1.10^{-9}$ m/s) leads to a specific design with an upstream sealing using a geomembrane and a 19m depth cutoff upstream wall installed to decrease the risk of piping. A 0.80m height and 61.5m width drain is setup at the downstream foundation-embankment. A 5m-wide berm is considered halfway up the downstream slope of the dam. The operating water level in the dam is equal to 17.5m.

4.2 *Boundary conditions*

The boundary conditions under static loading (construction and filling) are the following ones:the horizontal displacements are fixed on the edge of the model, and the horizontal and vertical displacements are fixed on the bottom. Under dynamic loading, quiet and free field boundaries were introduced in order to prevent wave reflection of propagating waves and to avoid seismic wave trapping.

The signal is imposed at the model base using applied stresses (Itasca, 2011). The seismic action of the accelerogram v_x is introduced as a shear stress τ_{xy} and can be calculated using the following formula:

$$\tau_{xy}(t) = -\rho V s v_x(t) \quad (4)$$

where V_s is the velocity of the shear wave; ρ the density.

The mesh size is chosen to guaranty that the seismic signal can be properly described for frequencies (f) lower than 20Hz. Therefore, the maximum size of the elements was estimated considering the following condition:

$$\Delta l \leq \frac{v}{10f} \quad (5)$$

with v the displacement speed.

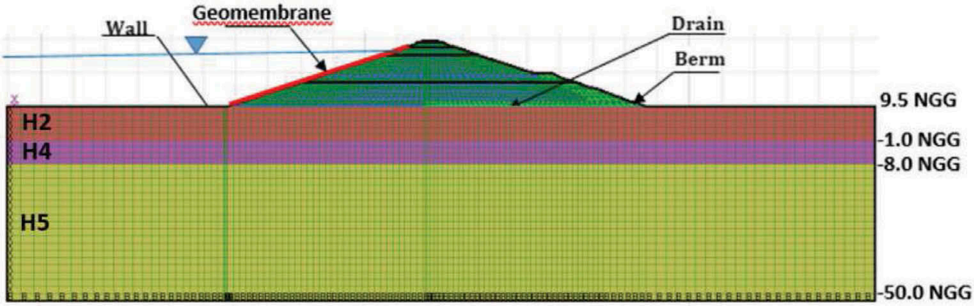

Figure 2. Dam design considered in the calculations

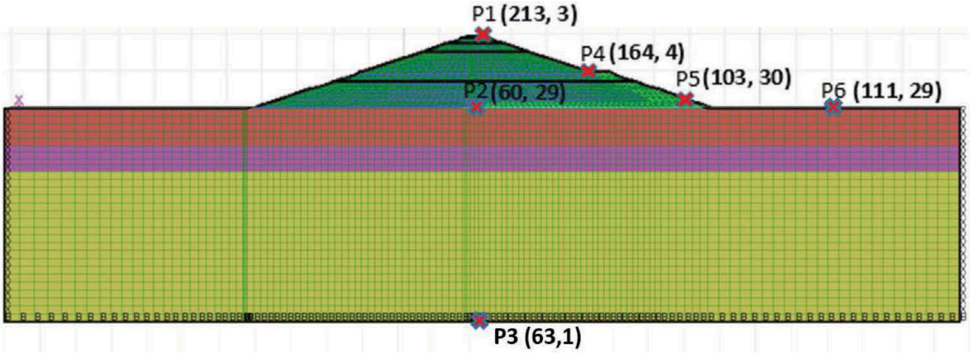

Figure 3. Horizontal component of the acceleration record from Les Sainte earthquake.

4.3 *Calculation assumptions*

The numerical analysis is done in two different stages: simulation of the construction layer by layer follows a quasi-static regime and simulation of the earthquake is in a dynamic regime.

The calculations were carried out during the construction and reservoir filling step under drained conditions and undrained conditions where used during the earthquake loading step.

4.4 *Input ground motion*

An input accelerogram was generated from the seismic event of Les Saintes (M_w= 6.3, 2004). The accelerogram (horizontal component) was adjusted to match the target response spectrum using the wavelets algorithm proposed by Abrahamson, (1992) and Hancock (2006). The target spectrum was provided by Eurocode 8 (EC8) for soil type B for an earthquake return period of 3000 years. The adjusted accelerograms are characterized by a wide frequency content, peak ground acceleration a_{max} = 4.2 m/s^2 and a duration of the event equal to 50 s (Figure 3).

5 RESULTS AND DISCUSSIONS

Figure 4 shows the reference points where mechanical quantities (displacement, acceleration, pore pressure, etc.) are monitored.

Figure *5* presents the shear deformations within the model. The higher value (20%) can be identified as the dam base and represents an initiation slip zone. The foot of the embankment is impacted by the shear deformations. A shear surface is developed mainly in the downstream part of the embankment.

At the end of the earthquake (after 50s), the maximum horizontal displacement is equal to 0.6m, at the foot of the downstream slope. The maximum vertical displacement (settlement) is equal to 0.2m at the dam ridge (Figure 6).

Figure 7 shows the vertical displacement evolution of points P1, P4, P5 and P6 with time. The two-dimensional modeling carried out with FLAC software shows that the settlements are lower than 3% of the dam height. The dam crest is not subjected to strong shear deformations unlike the shear deformations at the downstream foot of the dam.

The numerical results permit also to validate the fact that there is no significative pore pressure increase in the dam after the earthquake. This conclusion is important and permits to confirm the dam functioning mode.

Figure 8 presents the evolution of the horizontal acceleration of the model in three controlled points:top of the dam, base of the dam, and base of the numerical model. It can be seen that the peak of acceleration is located at the base of the dam.

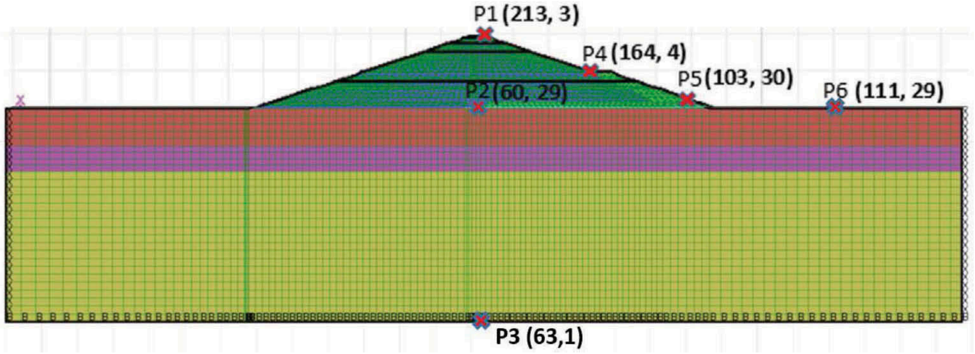

Figure 4. Positions of the histories.

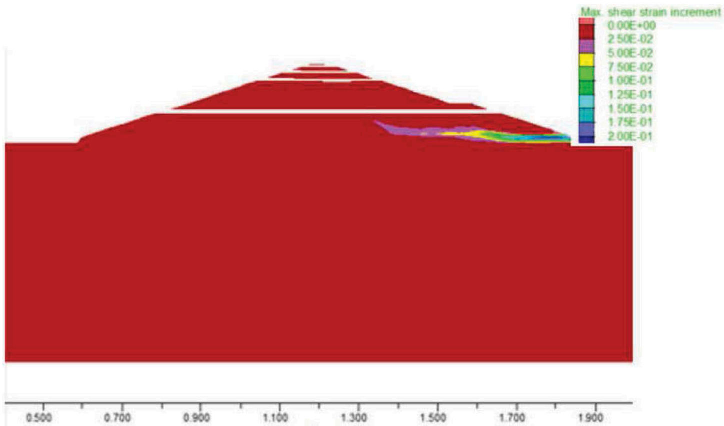

Figure 5. Shear deformations within the earth damat the end of the earthquake

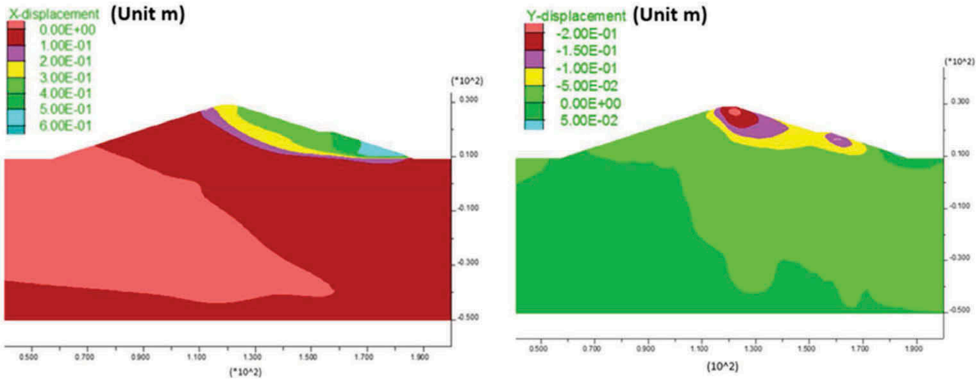

Figure 6. Horizontal displacement at the end of the calculation (left) and Vertical displacement at the end of the calculation (right)

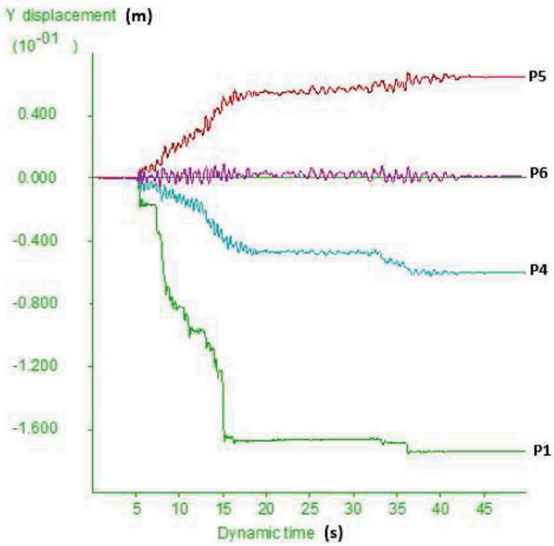

Figure 7. Vertical displacement of the points P1, P4, P5, P6

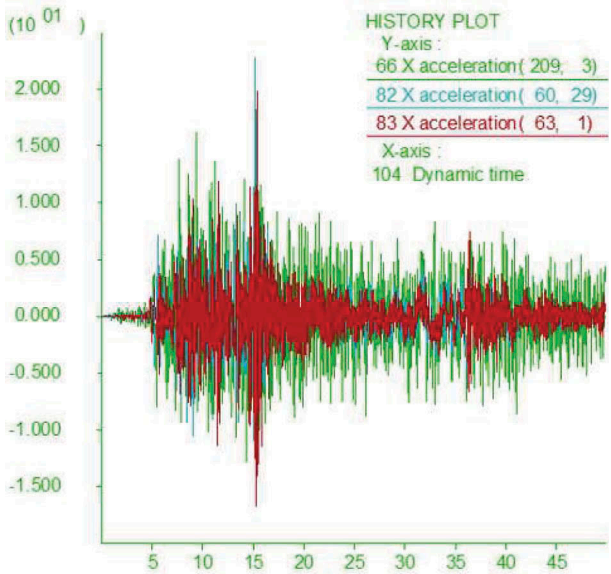

Figure 8. Horizontal acceleration of points P1, P2, P3

6 CONCLUSIONS

A seismic analysis of an earth dam in a tropical geologic context is presented using a 2D numerical model. The modified Cam-Clay constitutive model is a simple model, which does not require many input parameters. It is well adapted to model the static behavior of this type of soil by the fact that it permits to simulate experimental results obtained by various tests, namely, the oedometric test and the triaxial revolution test.

With this numerical analysis, it can be shown that the settlement value is acceptable in relation to the height of the dam. Shear deformations are initiated at the downstream foot of the dam and are relatively important (20%).

This first modeling is considered as a preliminary study. Complementary laboratory tests (dynamic triaxial and resonant column tests) and calculations will be carried out during the next stages of the detailed embankment dam design. They will permit to consider during the next stages of this research work a more complex behaviour of the considered tropical soils.

REFERENCES

Abrahamson N.A. 1992. Non-stationary spectral matching. Seismol res let.

AFNOR. 2000. NF P 94-132 Sols : Reconnaissance et Essais - Essai d'eau Lefranc.

Benatti, J. and Miguel, M. (2013). A proposal of structural models for colluvial and lateritic soil profile from southwestern brazil based on their collapsible behavior. Engineering Geology, 153, 1-11.

Brenner R.P, Grava V.K. and Blight. 1997. shear strength behavior and measurement of shear strength in residual soils; In mechanics of residual soils G.E Blight Editor, A.A Balkema; pp 155-219

Comité Français des Barrages et Réservoirs - Recommandations pour la justification de la stabilité des barrages et des digues en remblai –Octobre 2015

Fookes, P. (1997). Tropical Residual soils. Geological Society of London.

Futai, M., Almeida, S., and Lacerda, W. (2004). \Yield, strength, and critical state behavior of a tropical saturated soil." Journal of Geotechnical and Geoenvironmental Engineering, 130, 1169-1179.

Hancock, J., Watson-Lamprey, J., Abrahamson, N.A., Bommer, J.J., Markatis, A., McCoy, E. and Mendis, E. 2006. An improved method of matching response spectra of recorded earthquake ground motion using wavelets. Journal of Earthquake Engineering. PP 67-89.

Itasca Consulting Group. 2011. Fast Lagrangian Analysis of Continua. Minneapolis: Itasca Consulting Group Inc.

K. Nagendra Prasad • N. Sulochana U. Venkata Ramana. 2013. Applicability of Cam-Clay Models for Tropical Residual Soils. The Institution of Engineers (India).

Mahalinga, U. and Williams, D. (1995). Unsaturated strength behaviour of compacted lateritic soils. Géotechnique, 45(2), 317-320.

Nwaiwu, C. Osinubi, K., and Afolayan, J. (2005). Statistical evaluation of the hydraulic conductivity of compacted lateritic soil." Geotechnical Testing Journal, 28(6), 586-595.

Report of a working group from the French Ministry Risque Sismique et Sécurité des Ouvrages Hydrauliques, October 2014.

Roscoe K.H. and Burland J.B, 1968. On the generalised stress-strain behaviour of wet clay, Eng. plasticity, Cambridge Univ; pp. 535-609.