Static and dynamic analysis of stability of rocky slopes via particle methods

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ABSTRACT: This paper deals with stability analysis of rockfill slopes. A geotechnical model of rockfill behaviour is proposed. This model is based on the relationship between the particle strength and its dimension found by Marsal. It means that the rock particle crushing is modelled by a cluster of bounded circular particles. The bond is evaluated in order to fit the crushing strength of the particle. Therefore, the strength of the rockfill is calibrated by triaxial tests modelling. According to this geotechnical model, criteria to check the slope stability are proposed and applied to stability analysis of embankment. Dynamic analysis of embankment is checked with this method. Comparison with *FLAC* analysis is carried out. Elastic response are in good accordance with *FLAC*, however more unrecoverable deformations and a different mode of failure are noticed with the Particle Method. Influence of frequency is clearly identified.

1 INTRODUCTION

Rockfill dams have been designed by experience. Nevertheless, reassessment of stability of old embankment requires a physically based method of stability analysis. Finite Element Method either Finite Difference Method are well known and could be used. But, the problem is that these methods are based on continuum mechanic assumption. It means that the stress can only be defined on surface with length including a minimum of 10 particles. Considering that usual dimension of rock block is around 1 m in a lot of rockfill dams, data from these methods are not reliable for the first 10 m. Therefore, the tricky point is that sliding can occur mainly in the first 10 m at shallow depth.

A classical alternative consists in using methods of slides of limit equilibrium analysis. Here, another problem is pointed out: “What is the strength of a one meter layer or a 3 m layer ?” Laboratory tests with diameter higher than 1m don’t exist. Homogeneous tests requires a ratio diameter test on maximum particle dimension higher than 10. In consequence, Stones or boulders larger than 10 cm are very rarely tested in triaxial or oedometer apparatus. In conclusion, the other main problem is: “What is the strength of the site rockfill, based on that measured on samples limited to the finest part of the gradation?”

In order to solve these two questions, a physically based numerical approach is under development, founded by the R&D project MICROBE. Here, the first data, obtained previously under the support of EDF, are presented. These first data show that distinct element methods are well adapted to solve the both early problems and can lead to engineering approach.

This paper is dedicated to the first application of the numerical model of the rock particles, which can be used in this numerical modelling. Static stability of slope and seismic behaviour of a dyke are evaluated by this model. A comparison with elastic and elasto-plastic continuum mechanics relationships implemented in *FLAC* software is undertaken and interpreted.

The model definition is developed in a companion paper (Deluzarche, Cambou et Fry 2002): the main idea is rockfill strength is mainly dependent of the particle crushing strength, thus well calibrated crushable particles have to be introduced.

2 DEFINITION OF THE MODEL

2.1 *The basic assumptions*

Marsal (1973) carried out extensive research on rockfill. Referring to numerous large diameter tests, he demonstrated that the grain breakage is the main factor controlling the basic behaviour of the rockfill (modulus, friction angle). He developed a particle crushing test, where three particles of approximately the same dimensions were placed between two steel plate and loaded until one of them was crushed. The crushing load P_a is noted. Tests are performed on d_m= 2.5 cm, 5 cm and 10 cm particle size. The results are presented in the plot of the crushing load P_a versus the stone size d_m. The relationship between P_a and dm can be approximated by the equation (1):

$$P_a=\eta R^{\lambda} \tag{1}$$

Considering that, in any case, λ is lower than 2, and between 1.2 and 1.8, it means that the collapse stress linked to the crushing of particle as the dimension increases. On other side, it means that the friction angle decreases as the dimension of particle increases.

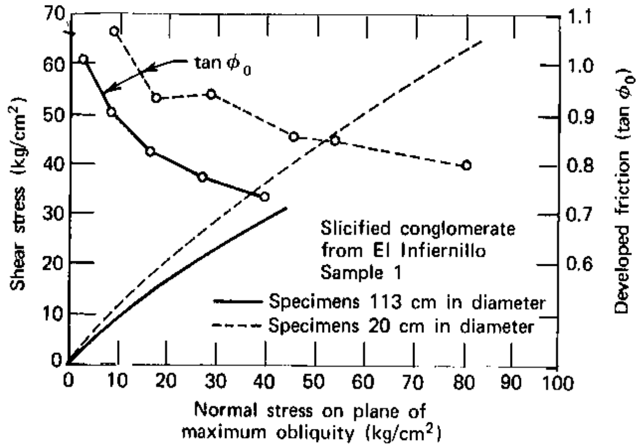

Figure 1. Friction angle versus stress for 2 triaxial diameter

Marsal measured the friction angle versus the confining stress, with two triaxial apparatus size. They are 20 and 113 cm in diameter and 50 and 250 cm high respectively. The maximum particle dimension was 20 cm and 2 cm respectively . The results clearly demonstrated that the friction angle decreases with increasing stone dimension (Figure 1) and stone breakage (Figure 2). The breakage was reported by Bg, where B_g is the percentage by weight of the particles that has undergone breakage. In Figure 2, the principal stress ratio σ_1/σ_3 at failure decreases with the grain breakage B_g increasing.

The two previous types of data, the crushing test and the triaxial test on small samples (for instance, with 100 mm diameter) are easily carried out in current laboratory. Thus, the main idea of the model is to modelling with *PFC*^{2D} crushable particles, whose the crushing loads is fitted on equation (1) from Marshall, and whose the triaxial strength of small particles is fitted on performed triaxial tests. With such a calibration, the main assumption is that the model is able to predict the true triaxial strength of the largest particles deposit. In this first approach, another important assumption is 2D model is only used to fit cylindrical triaxial test. Other details of the definition and calibration of the particle model is given in the companion paper (Deluzarche, Cambou et Fry 2002).

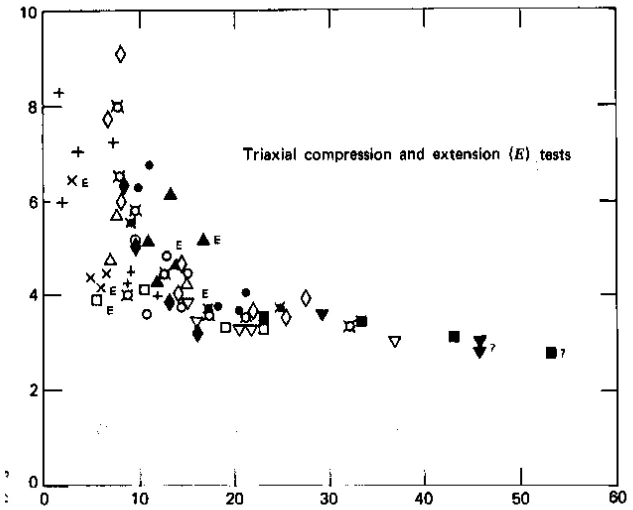

Figure 2. Principal stress ratio at failure versus grain breakage

2.2 *First results of triaxial tests modelling*

The stone particle is considered as an aggregate of several disks bound by breakable parallel bonds. At beginning (Mahboubi, Fry, Cambou, 1997), different kinds of aggregates have been tested, at two states of compaction. Figure 3 shows three of the tested shapes of particle. The gradation of the sample is the mixing of three sizes of aggregates, thus the coefficient of uniformity ($C_u = D_{60}/D_{10}$) is between 1.5 and 6.

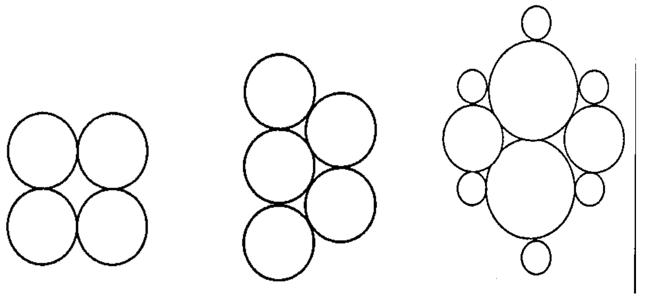

Figure 3. Representation of three shapes tested

For instance, Figure 4 presents the computed triaxial curves:

- Deviator versus vertical strain,
- Variation of volume versus vertical strain,
- And percentage of broken stones.

From these curves, it is observed that:

- A sharp decrease of the angle of friction versus confining pressure is noticed,
- The developed friction, calculated from modelling 2D compression triaxial test with shapes 1 and 2 (made with 4 and 5 disks) for the largest stress ($\tan \phi_0=0.78$) is quite similar to the measurement of Marsal (Figure 1).

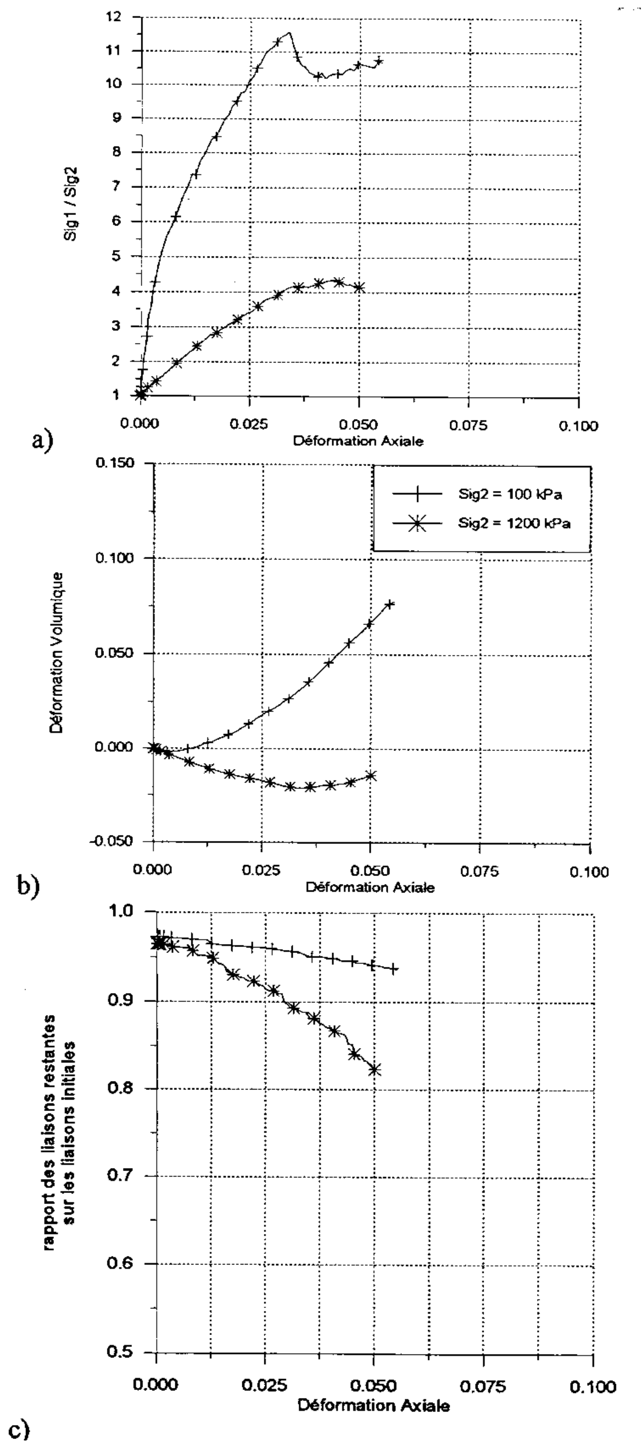

Figure 4. Friction angle versus confining stress for two models of four and five disks

More generally, the shape 1, with the more cubical shape, gave the strongest sample, the shape 3 gave the softest. Maximum difference of 10° in friction angle was associated to the shape. No large difference was noticed between shape 1 and 2. The calibration of the numerical model on measured triaxial is reported in the companion paper.

3 STATIC ANALYSIS OF STABILITY

3.1 *Purpose of the tests*

The purpose of this step is to select clear criteria for the stability of the embankment. The particles of the fill are placed with a certain level of compaction confined between rigid walls. The removal of the walls changes a lot the forces equilibrium. Convergence of numerical process can be time-consuming, so the question is risen: At what convergence cycle the stability is ensured?

3.2 *Tested criteria*

In a view to answering the problem, different criteria were tested on a simulation of a steep embankment. Two kinds of criteria are assessed: local and general criteria.

- The local criteria analyses the equilibrium of any particle. Unstable particles are easily viewed by such criteria.
- The global criteria analyses the equilibrium of the whole structure.

Three local criteria for one particle were tested:

- Force criteria: The sum of the applied forces on any particle has to be less than a little part of the weight of the particle.
- Momentum criteria: The momentum of any particle has to be less than a little part of the momentum caused by a free fall d_{ref}, which is 10% of the size of the particle.
- Energy criteria: The variation of potential energy of one particule between two convergence cycles has to be a little part of the potential energy from a free fall d_{ref}, which is 10% of the size of the particle.

Five global criteria for the structure were tested:

- Number criteria: Ratio between the number of unbalanced particles on the total number of particles.
- Mass criteria: Ratio between the sum of masse of unbalanced particles on the total mass of the fill.
- Threshold of unbalance criteria: Maximum ratio between the unbalance of one particle on the threshold of unbalance.
- Momentum criteria: Ratio between momentum of the fill and reference momentum. The reference momentum is the product of the fill mass by reference the velocity V d_{ref}, which is 10% of the size of the particle.
- Energy criteria: Ratio between potential energy of the fill and reference potential energy. The reference momentum is the product of the fill weight by a reference vertical fall d d_{ref}, which is 10% of the size of the particle.

3.3 *Result of the test : adopted criteria*

The studied structure was half a fill, 12 m high with very steep slopes 60° to 66°. Figure 5 shows a stable 62° slope with the detected unstable particles.

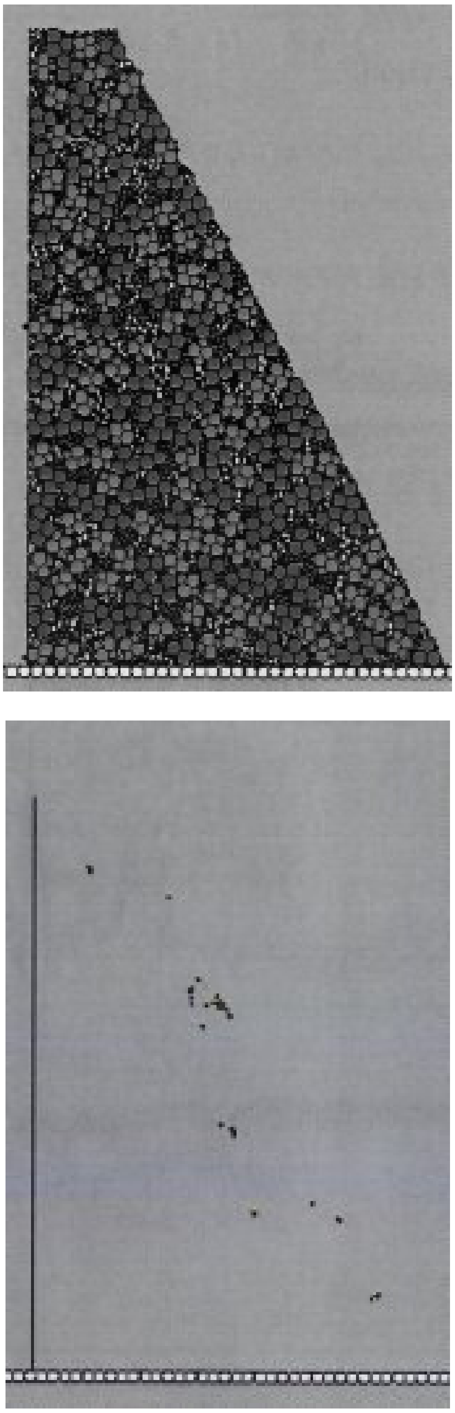

Figure 5. View of the studied fill and of the unbalanced particles.

From the series of tests, the most reliable local tests are:

- Force criteria

$$\left\|\sum \vec{F}_{contact} + m.\vec{g}\right\| \le 0.015\left\|m.\vec{g}\right\| \qquad (2)$$

- Momentum criteria

$$m\left\|\vec{V}\right\| < 0.0005.m.\sqrt{2.g.d}_{\ ref} \qquad (3)$$

The most reliable global tests are:

- Masse criteria: The mass of unbalanced particles has to be lower than 3% of the mass of the shallow layer whose the thickness is around five large particles.
- Momentum criteria: The variation of the ratio between momentum of the fill and reference momentum between 50 000 cycles has to be lower than 4.10^{-4}. This criterion is used to compare the stability of a 60° to 66° slope (Figure 6).

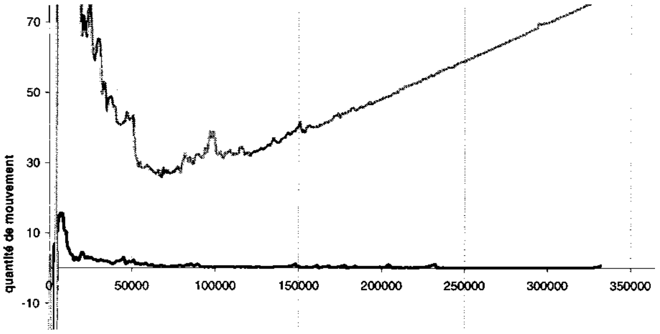

Figure 6. Variation of momentum for two fills with 60 and 66° slope

At least, another criterion, the ratio between the momentum of the fill and the reference momentum, is recommended like a threshold for instability.

4 DYNAMIC STABILITY

4.1 *Case study.*

The dynamic stability of a rockfill embankment 18 m long and 5 m high with 50° slope was studied. It is made with 7986 particles, including 23 833 disks (Figure 7). This embankment is built on a one row of fixed disks, simulating a hard rock.

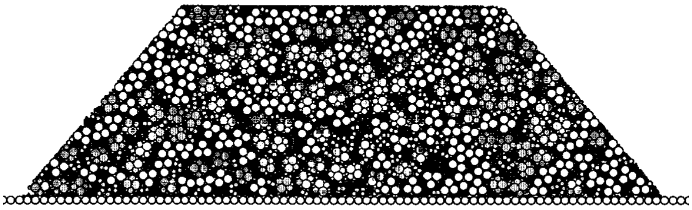

Figure 7. Embankment of the case study

His properties were calibrated by modelling very large triaxial tests (H =4 m, L =2.5 m) at 1 Hz frequency. The response of the sample was considered acceptable (Figure 8).

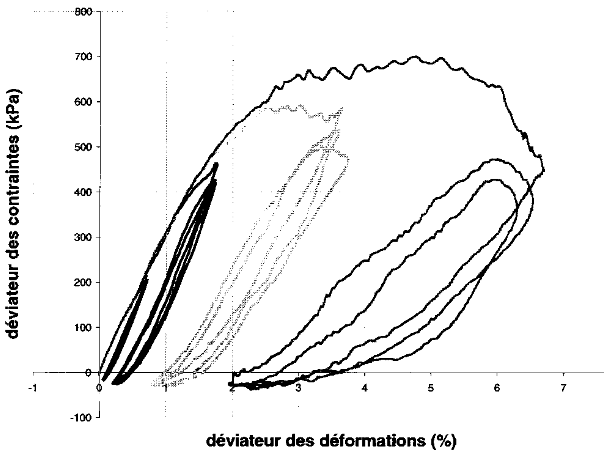

Figure 8. Cyclic triaxial test at 1 Hz

The solicitation is a five cycles of sinusoid with Peak ground Acceleration of 0.2g at three different frequencies 3 Hz, 1 Hz and 0.3 Hz. The solicitation is applied by an homogeneous velocity of the first fixed row of disks.

4.2 *Result from PFC^{2D}*

The behaviour of the fill was completely different depending upon the frequency. Although the first natural frequency of vibration of the fill is higher than 3 Hz, the most destructive frequency was the lowest one. Figure 9 clearly show that:

- No large irrecoverable deformation exist after the cycling at 3 Hz, the imposed displacement at base was 1 cm and the largest displacement of the particles was 15 cm.
- Shallow sliding appears at 1 Hz. With an imposed displacement of 10 cm at the base, the maximum observed displacement was 80 cm after 5 cycles.
- Deep failure appears at 0.3 Hz. Unfortunately the calculation lasted 15 days for one cycle, and the rest of the solicitation was abandoned. The maximum imposed displacement at the base was 80 cm, and the maximum displacement of the particles reaches 1.5 m. It is noteworthy to quote that the failure mechanism is not sliding but extension along a traction line 2 m inside the slope and parallel to the face.

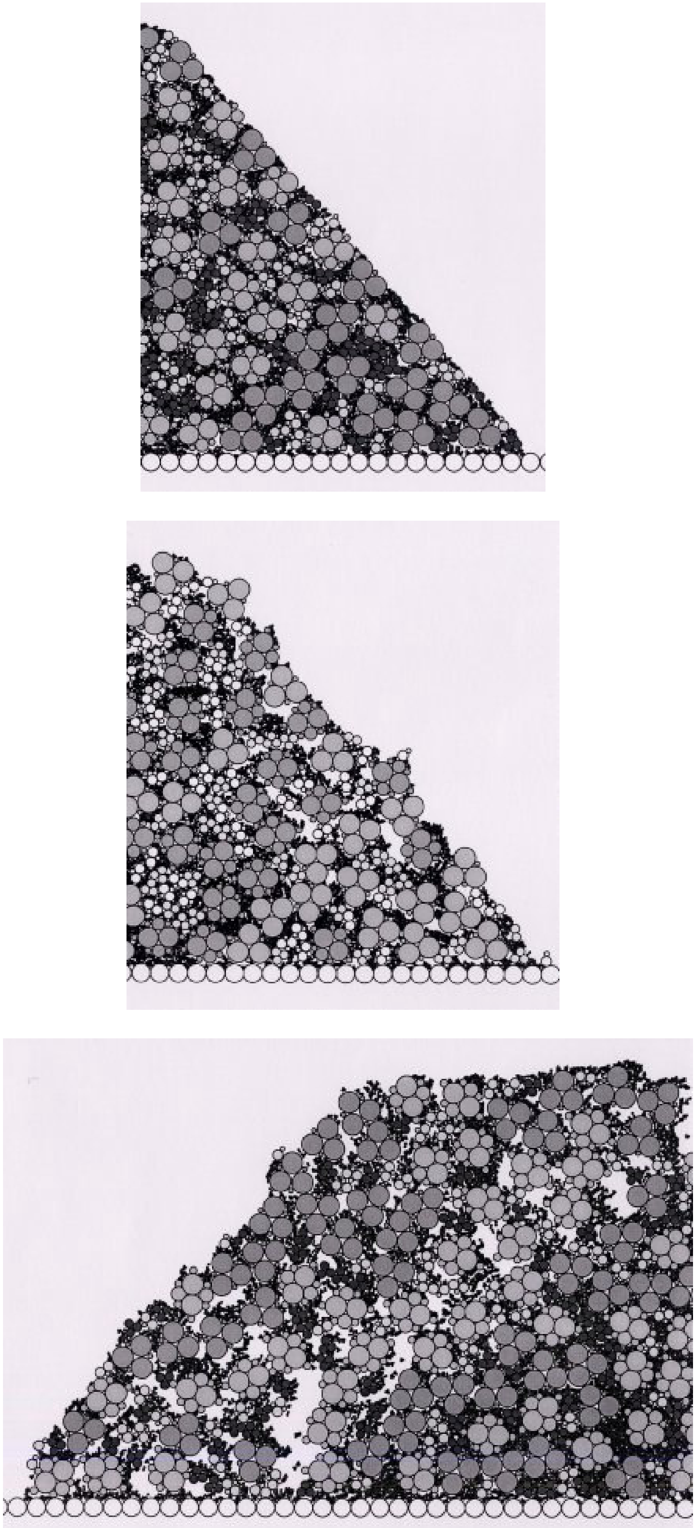

Figure 9. Behaviour of the fill according to frequency (at the top: 3 Hz, in the middle: 1 Hz, at the bottom:0.3 Hz)

4.3 *Comparison with FLAC*

In a view to validating these first results, the embankment was modelled by *FLAC*. The mesh is presented on the Figure 10.

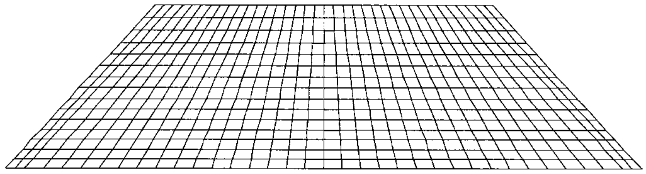

Figure 10. Mesh of the embankment for FLAC2D analysis

Two constitutive equations were used to analyse the dynamic behaviour: A Mohr-Coulomb model and a Ramberg-Osgood model coupled with a Mohr-Coulomb failure criterion. The first one is purely elastic-plastic and the second one has the modulus and the plastic damping dependent of the strain.

The parameters of the Mohr-Coulomb model were:

– K, bulk modulus:	13.6 MPa
– G, shear modulus:	16.1 MPa
– C, cohesion:	0 MPa
– Phi, friction angle:	61°
– Psi, angle of dilatancy:	20°

The Ramberg-Osgood model is defined by the equation 3.

$$\gamma - \gamma_c = \frac{1}{G\max}\frac{1}{1-H(t)}(\tau - \tau_c) \qquad (3)$$

Where γ_c et τ_c are the shear strain and stress at the last peak, while H(t) is given by:

$$H(\tau) = \left[\frac{\left(\dfrac{|\tau - \tau_c|}{2\tau_y}\right)^{2B}}{1-\left(\dfrac{|\tau - \tau_c|}{2\tau_y}\right)^{2B}}\right]^{A} \qquad (4)$$

The parameters of the model are:

– K_{max}, bulk modulus:	13.6 MPa
– G_{max}, shear modulus:	16.1 MPa
– C, cohesion:	0 MPa
– Phi, friction angle:	61°
– τ_y:	60 kPa
– A:	2.1
– B:	6.7

The most comparison of maximum acceleration and maximum displacement between the three models is full of interest. According to the first natural frequency of the structure which is higher than 3 Hz, it higher is the frequency, more amplified is the acceleration (Figure 11). It is satisfactory to notice no large discrepancy between the two kinds of approach Distinct Elements Method and Finite Difference Method. The Rambert-Osgood model gives the highest amplification related to the decrease of a fundamental frequency with the decrease of the modulus. Nevertheless, the distinct element model gives higher amplification at low frequency.

It is very impressive to notice no large difference between the both approaches in the comparison between the maximum displacement of the fill after the first sinusoidal peak. Nevertheless, the difference increases with the number of cycles for the lowest frequencies (Figure 12). The maximum plastic deformation are quite larger with *PFC^{2D}* than with *FLAC*.

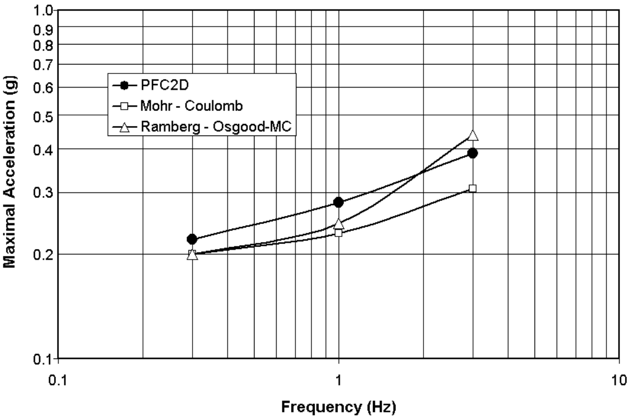

Figure 11. Comparison of maximum accelerations

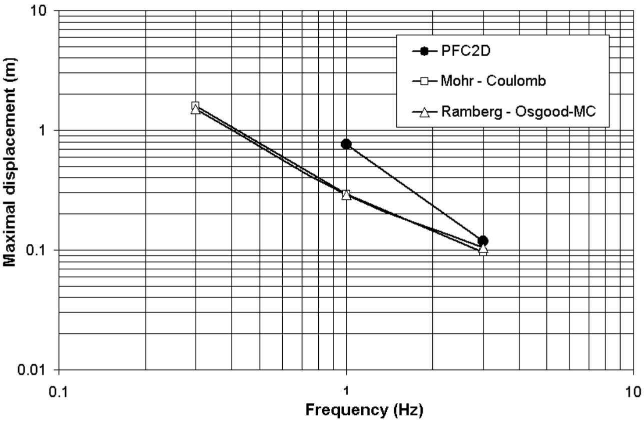

Figure 12. Comparison of maximal displacement

5 CONCLUSION

The DEM approach is very promising for civil engineering. Here application of this approach to modelling rockfill embankment is full of interest. The main features of the rockfill can be analysed by such an approach. Comparison with Continuum Mechanic approach implemented in *FLAC* showed different modes of failure and larger irrecoverable deformation for *PFC^{2D}*. This last point is very important for engineering and has to be confirmed by other detailed surveys. That conclusion was the justification of the launch of the MICROBE project.

6 ACKNOWLEDGEMENT

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REFERENCES

Barton, N. & Kjærnsli, B. 1981. Shear Strength of rockfill. *Journal of the Geotechnical Engineering Division*, 107(7): 873-891.

Deluzarche R., Cambou B., Fry. J-J. 2002. Modelling of Rockfill behaviour with crushable particles. First International PFC Symposium *Gelsenkirchen Germany.* Balkema.

Mahboubi A, Fry J-J, Cambou , 1997. Numerical modelling of the mechanical behavior of non spherical, crushable particles. *Powder and Grains*, : 139-144.

Marsal, R.J. 1973. Mechanical properties of rockfill. In Hirschfeld & Poulos (eds), *Embankment-dam Engineering - Casagrande volume*: 109-200. J. Wiley & Sons.