# Behaviour of soils simulated by elastoplastic models

## Le comportement des sols simulés par des modèles élastoplastiques

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SYNOPSIS: Four elastoplastic models are proposed to describe the behaviour of soils. These models are considered in the framework of associated plasticity with isotropic workhardening, including the change in volume of soils. The soils parameters are obtained from a classical triaxial test. In order to simulate the observed behaviour of soils, using the models in question, a finite element programm was elaborated. The computations were made for different drained stress paths by two load conditions. A comparision between the models was investigated. Furthermore the efficiency of load conditions was analyzed. For illustration an example of simulation is presented, showing the observed behaviour of a non treated gravel which is well described by one of the proposed models.

### 1 INTRODUCTION

The design of foundations in soil mechanics is based on characteristics corresponding at failure (cohesion, angle of friction) using a safety factor. However, it is interesting to study the behaviour of soils before failure and to describe it by different models. Similar to structure analysis, the design of foundations is considered using one of these models. Nevertheless, each model needs results of laboratory testing and uses parameters, depending on the soil, which are determined by an appropriate method. The assessment of any model is based on comparision between experimental and numerical results. The latter is obtained with the values of parameters taken from experimental results. This operation is called simulation. In this way a computer programm was performed to simulate the behaviour of soils by four elastoplastic models. The simulation is made in the case of the classical triaxial test for two load conditions: fixed load, and fixed displacement.

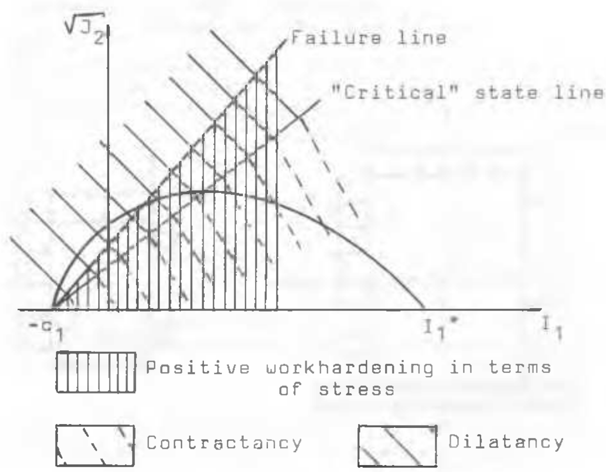



Figure 1. The yield surface, Bouassida (1988).

### 2 THE ELASTOPLASTIC MODELS

These models are based on the works of Wilde (1977), (1979) assuming an associated plasticity and taking into account the volume variation before and during failure. The yield criterion, shown in figure 1, is given by the following equation, Bouassida (1985):

$$F = \sqrt{J_2} + (I_1+c_1)[M \ln(I_1+c_1)/(I_1^*+c_1)) - K(e_1^P + 6(M^*-M)\sqrt{N})] \qquad (1)$$

where:
$I_1$ and $e_1^P$ denote the first invariants of the stress tensor and the plastic strain tensor respectively;
$\sqrt{J_2}$ and $\sqrt{N}$ are the second invariants of the stress tensor and the plastic strain tensor respectively;
$I_1^*$ is the value of $I_1$ corresponding to the initial yield surface;
$M$ is the slope of the "critical" state line in which the plastic volumic strain remains constant before failure;
$M^*$ is the slope of the flow line corresponding at failure;
$K$ is a dimensionless constant related to the workhardening;
$c_1$ is a constant related to the cohesion and the angle of friction; from Bouassida (1985) we have:

$$c_1 = (\eta c \operatorname{cotg}\varphi)/\sqrt{3}$$

here $\eta$ is the slope of the critical state line, introduced by Roscoe et al (1958), depending only on the angle of friction.
The parameters $I_1^*$, $M$, $M^*$, $K$ and $c_1$ are determined, from a classical triaxial test, using the method detailed by Bouassida (1985).
From equation (1), four elastoplastic models are founded according to the values of the parameters considered:

(i) by setting: $c_1=0$, $M=M^*\neq 0$, equation (1) is reduced to the initial Cam-clay model proposed by Roscoe et al (1958). To identify the values of the parameters introduced by the authors, the following relationships are obtained:

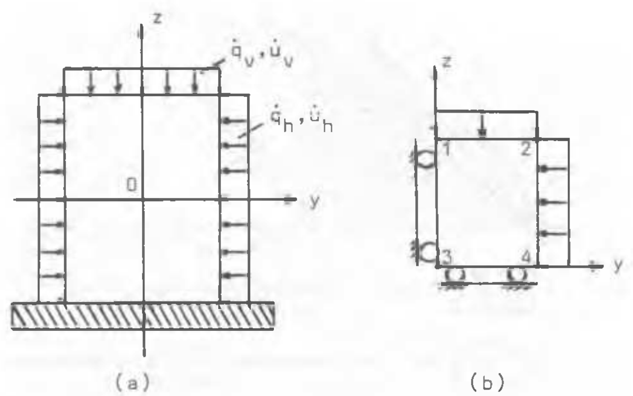
$$I_1^* = 3p^*;$$

$$p^* = 2.72\,\Gamma/(\lambda-\kappa);$$

$$M = \eta/3\sqrt{3};$$

$$K = \eta(\lambda-\kappa);$$

where $\lambda$ and $\kappa$ are the slopes of the normal consolidation line during loading and unloading respectively. $\Gamma$ is the value of the specific volume corresponding to the critical state.
(ii) By setting: $c_1\neq 0$, $M=M^*\neq 0$, the workhardening density model proposed by Wilde (1977) is obtained.
(iii) By setting: $c_1=0$, $M\neq M^*\neq 0$, we obtain the model proposed by Wilde (1979) which uses two invariants of the plastic strain tensor.
(iiii) By setting: $c_1\neq 0$, $M\neq M^*\neq 0$, the model recently proposed by Bouassida (1988) is obtained as a combination of the last two models.




$\dot q$ and $\dot u$ are the increments of fixed load and fixed displacement respectively;
the indices v and h correspond to the vertical and the horizontal direction respectively.

Figure 2. Boundary conditions of a specimen in the triaxial test.

## 3 THE SIMULATION METHOD

The simulation is considered for a cylindrical specimen loaded as indicated in figure 2a. Due to the geometrical and mechanical symmetry, only a quarter of the specimen is considered which is represented by a four nodes quadrilateral finite element (figure 2b). The incremental load is applied at the condition of fixed load or fixed displacement. For each increment the plastic loading conditions are verified, Bouassida (1985). By considering the hypothesis indicated above, a simulation programm was elaborated. The following drained stress paths are tested (see figure 3):
(i) isotropic and anisotropic consolidation: path O'I and path O'A respectively;
(ii) compression: path O'C;

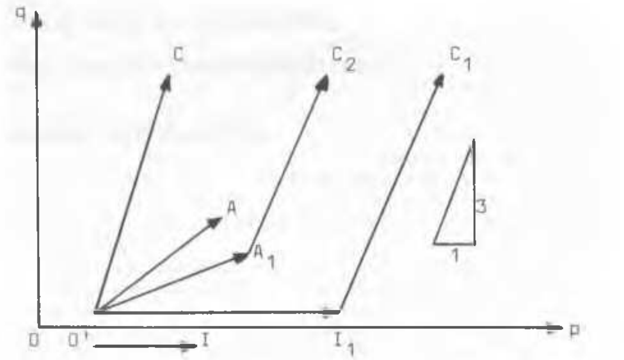



Figure 3. The drained stress paths considered by the simulation programm.

(iii) isotropic and anisotropic consolidation: path $O'I_1$ and path $O'A_1$ respectively, followed by a shearing test with constant confining pressure: path $I_1C_1$ and path $A_1C_2$ respectively.
All these paths are precedeted by an isotropic consolidation (path OO') corresponding to the elastic phase.

## 4 RESULTS AND DISCUSSIONS

The difference between the two load conditions is shown in figure 4; with fixed displacement condition, which corresponds to the operating system in triaxial tests,the plastic level of the stress-strain curve is well described. In the case of fixed load condition, computations could not be carried out when the plastic loading conditions are not satisfied anymore. In figure 5 the influence of cohesion (the constant $c_1$) on the deviatoric stress is shown. The shearing resistance is well improved, mainly for large strains, by an increase of cohesion.

The main hypothesis of the critical state theory, considered by Wroth et al (1968) and Wilde (1977), is well verified as shown in figure 6: in fact the volume variation of soil is constant during the phase of failure.

Some results of simulation are given in figures 7, 8 and 9. The soil in question is a non treated gravel, its behaviour is predicted by the model proposed by Bouassida (1988). For small strains, numerical results agree with the experimental ones. For large strains this agreement is less pronounced. The discrepency could be attributed to the associated plasticity theory which remains questionable for soils, as mentionned by several authors, Mrŏz (1963), etc. Following these remarks, the elastoplastic model retained for gravel could be applied in the field of small deformations; it has been used for a flexible pavement design, Bouassida (1985). This model permitted to obtain more realistic stress distributions, than those predicted by the linear elasticity assumption. In particular, this latter predicts radial tensile stresses which are inadmissible for gravel.

## 5 CONCLUSION

A simulation programm of soils behaviour using

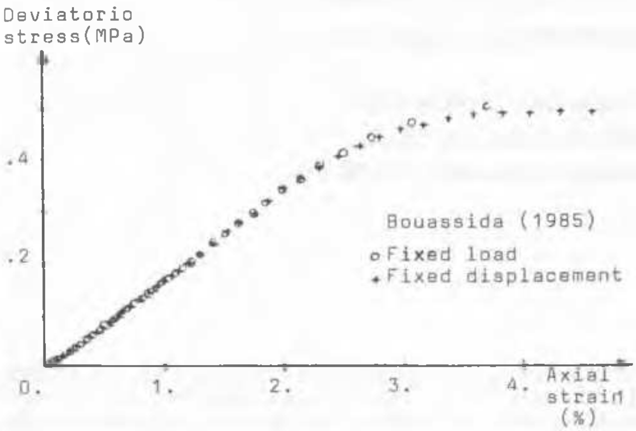



Figure 4. Comparision between the two load conditions.

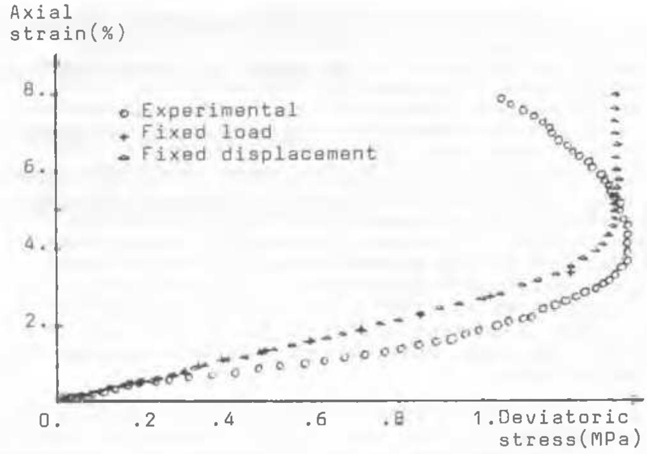



Figure 7. The stress-strain curve of a non treated gravel in triaxial shear.

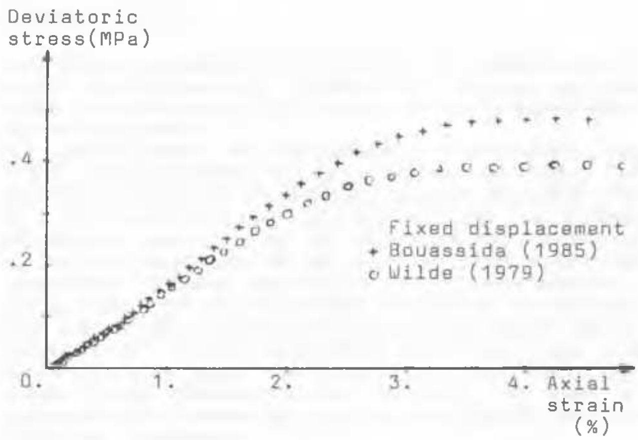



Figure 5. The influence of cohesion on the shearing resistance.

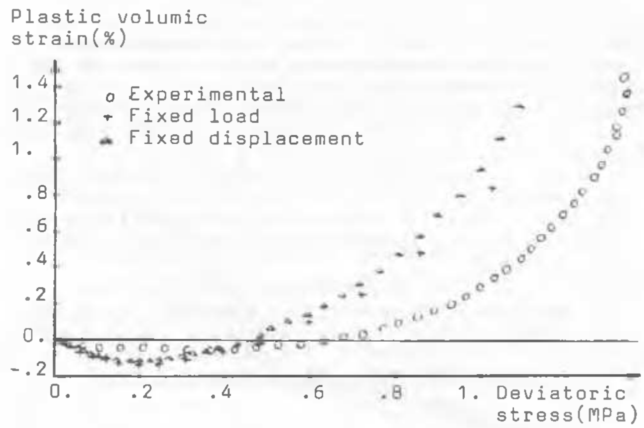



Figure 8. Evolution of the plastic volumic strain of a non treated gravel in triaxial shear.

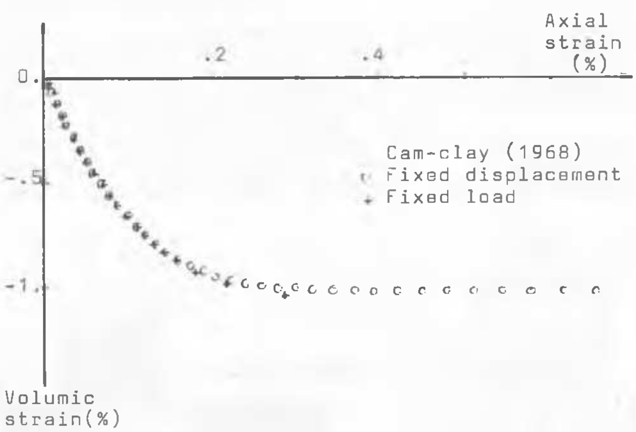



Figure 6. The evolution of the volumic strain as predicted by the Cam-clay models.

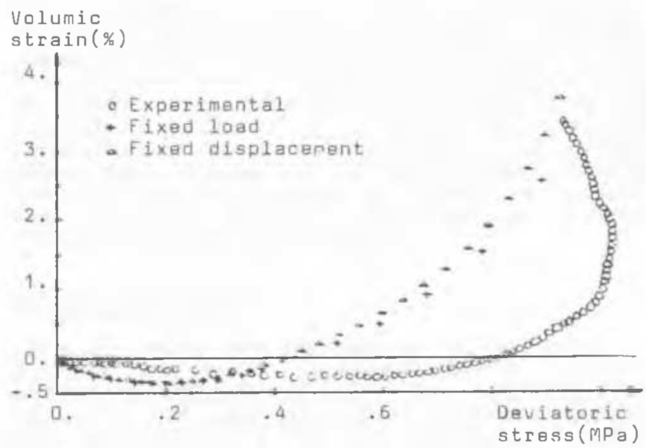



Figure 9. Evolution of the volumic strain of a non treated gravel in triaxial shear.

four elastoplastic models was elaborated. The case of a cylindrical specimen, for triaxial tests, was studied on the basis of simplified assumptions. The results obtained allow us to verify the efficiency of the fixed displacement condition, to simulate a triaxial test, compared to the fixed load condition. Furthermore, the role of cohesion on shearing resistance was analyzed. Based on experimental results, the models presented in this paper give a more realistic description of the soil behaviour. Using these models, a finite element code, for two-dimensionnal multilayer structures, was performed; Bouassida et al (1987). Such codes are useful for soil mechanics analysis, as in the case of soil-structure interaction problems, pavements design, etc...

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