Earth Reinforcement, Ochiai, Yasufuku & Omine (eds) © 1996 Balkema, Rotterdam. ISBN 90 5410 833 9

Design of geocells reinforced soil structures through a homogenization method and a finite difference method: Comparison and charts

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ABSTRACT : The purpose of this paper is to compare two different methods of stability analysis developped for a new type of retaining wall reinforced with a three dimensionnal hexagonal shape geotextile. In order to explain the effect of geocells as a reinforcing member and the mechanism of confinement, a two blocks method using a theory that defines an equivalent homogeneous material is proposed.

1 INTRODUCTION

The product, trademark Armater, consists of geotextile strips linked together by stitching. Once deployed, it forms a honeycomb structure, each cell having a hexagonal shape with a side length of 25 cm and a height of 17 cm. A new type of retaining wall reinforced with this three-dimensionnal hexagonal shape geotextile has been developed. The wall is made of a continuous pile of layers linked to each other on the outer and inner face of the wall by vertical strips of non-woven geotextile stapled and other strips woven through the panels.

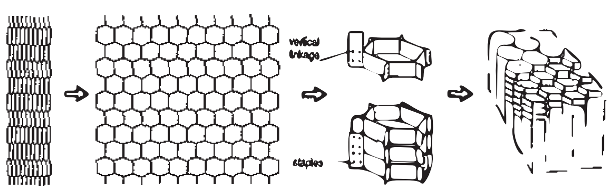


Fig.1 Reinforced geocells soil structures

One of the most advantage of overall confinement being that it makes the mass homogeneous with a material of equivalent properties. The cell and the enclosed soil behave essentially as a unit. The development of this new technique required the elaboration of a reliable as well as practical design procedure. A major objective in analyzing such structure is to determine the increase of the factor of internal stability. Knowledge of the magnitude and orientation of the confinement effect throughout the wall is of prime importance when designing the structure.

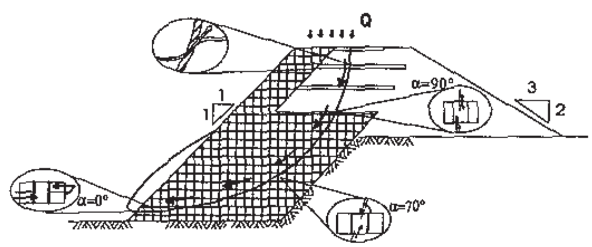


Fig.2 Failure mode

So the behaviour of the reinforced soil mass under working conditions is analyzed by applying a theory that defines the equivalent homogeneous material that can represent a sequence of alternating orthotropic layers.

2 EXPERIMENTATION

The composite structure obtained have higher mechanical properties than the content alone and it seems logical that the mobilized strength should be different for different angle of orientation. In order to investigate this aspect, tests were carried out using a simple shear box, with cells placement at 0, 30, 45, 60, 90 degrees to the plane of shearing against contact surface. The size of the box is defined by the dimension of the cells : it should be large enough to allow testing of representative honeycomb mesh. Large boxes also minimize influences associated with side wall friction (Vucetic 1982), proximity of boundaries and soil particle limitations.

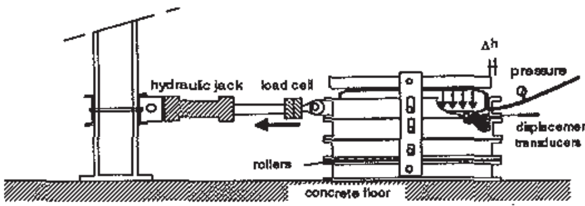


Fig.3 Plan of the simple shear box

The test apparatus used consisted of one rigid steel box as shown in figure 3. The inner dimensions of 1x1x0,5 m were choosed. The apparatus was loaded on top by a pressured air bag and on its side using an hydraulic jack. A constant vertical confining pressure of 20 kPa is maintained by decreasing the pressure in the bag to compensate for vertical strains of the soil and the apparatus. Rollers are installed between the frames to reduce the friction. Soils used for the fill in each test were a uniform sand 0/2 from the Allier river. The direct shear friction angle was obtained by a standard direct shear box and was 29° for this material. Pluviation by a hooper was employed to prepare uniform and loose sand masses. The same geotextile was used in all tests to model the geosynthetic reinforcing elements in real scale walls. The main characteristics of these materials are presented in table 1. The mesh used in these tests was a stitch material type.

Table 1: Features of the honeycomb structure textile

characteristic	mass per unit area	thickness	tensile strenght	elongation at failure
notation	ρ	e	Rt	ε
unit	g/m²	mm	kN/m	%
non-woven	350	1,9	20,2	25

The load was applied at a constant horizontal displacement rate of 2mm/min. Load and displacement transducers were employed for force and displacement measurements. Displacement transducers were installed along the height and width of the box. Readings from instrumentation were taken for each loading stage applied to the box.

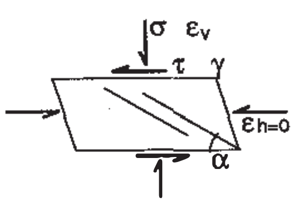


Fig-4 Notations

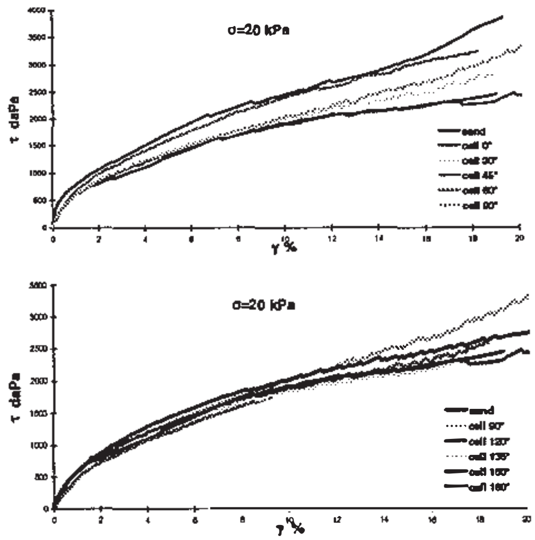


Fig.5 Results of shearing tests

The tests presented above point out the anisotropic behaviour of shearing resistance. Assuming that there is no change in the friction angle ϕ, we say that the cohesion parameter varies with α the orientation of the reinforcement (Jewell 1987) (Kuwano 1994).This variation can be shown using a polar diagram figure 6.

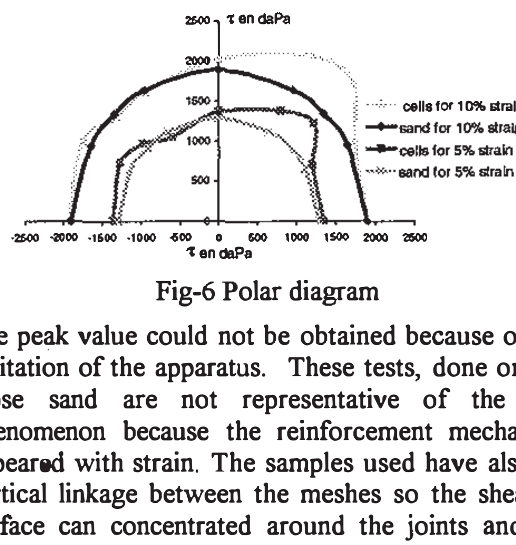


Fig-6 Polar diagram

The peak value could not be obtained because of the limitation of the apparatus. These tests, done on dry loose sand are not representative of the real phenomenon because the reinforcement mechanism appeared with strain. The samples used have also no vertical linkage between the meshes so the shearing surface can concentrated around the joints and the result show the behavior of the interface. Nevertheless these tests can give a realistic approch of the honeycomb structure behavior. The maximum of allowed shearing resistance is for α between 45° and 60° which is in good agreement with Coulomb's failure wedge.

We use the expression developped by Jewell

τ_{max}=N.Ar.σ_α.sinϕ.(sinα.tanϕ +cosα) (1.1) with N = number of reinforcement per square meter of shearing surface, Ar = surface of the reinforcement and

$$\sigma_\alpha = \sigma_{yy}.\frac{1+\sin\phi.\sin(\phi+\pi-2\alpha)}{\cos^2\phi} \qquad (1.2)$$

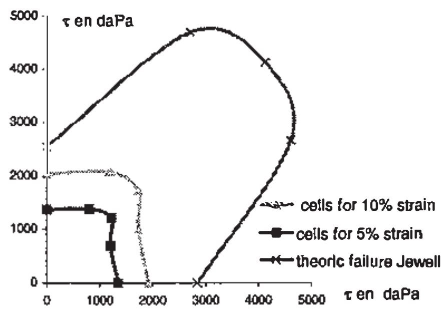


Fig.7 Comparison of experiments and theory

This expression seems to overestimate the shearing resistance and the shape don't fit well with tests.
In order to isolate the performance benefit confer by the confinement we also use compression tests (Bathurst 92).

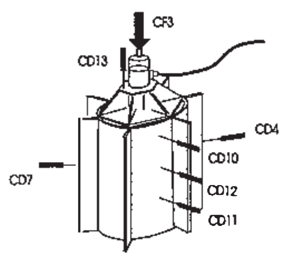
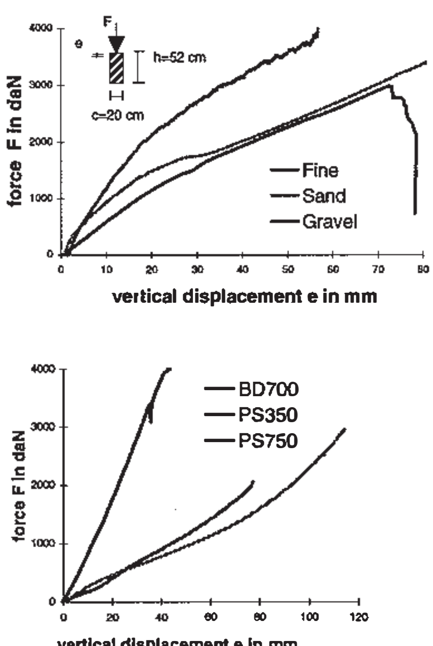


Fig-8 Compression test



a.

b.

Fig.9 Results of compression tests

The effects of the reinforcement properties (stiffness and strenght figure 9a. and 9b.) and geometry (study of the influence of cells size) upon the deformation behaviour have been investigated. Two of the most obvious factors are the reinforcement properties and interface properties. Therefore attention should be paid to the determination of elastic parameters as Young modulus and bearing capacity of the interface.
The compaction increase the lateral tension in cells. In this way, the required tensile force needed to have the effect of confinement is obtained and the vertical resistance is increased. In first approximation, we obtained $Cr=\sigma lr/2.\tan(\pi/4-\phi/2)=36$ kPa at $\gamma=10\%$ for loose dry sand and for the non-woven described in table 1. The observed shear strenght increase, due to reinforcement, can be attribute to the developpement of an effective lateral equivalent confining pressure $\Delta\sigma3$ experienced by the soil.

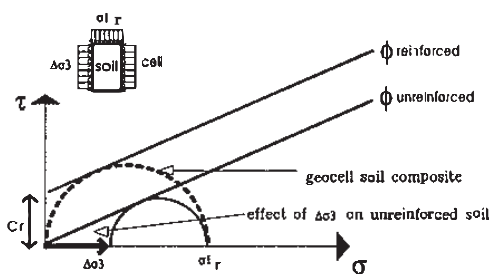


Fig.10 Definition of parameters

The effective confining stress can be compute using relations between the cohesion and the caracteristics (Bathurst 92)(Jewell 87)(Boyle 94). For reinforced cohesionless soil or drained cohesive soil the increase in shear strenght is given by :

$$C_r = \frac{\Delta\sigma_3}{2}.\tan(\frac{\pi}{4}+\frac{\phi}{2}) \qquad (1.3)$$

where

$$\Delta\sigma_3 = \frac{2.J}{D}.\frac{1-\sqrt{1-\varepsilon_a}}{1-\varepsilon_a} \text{ for } \alpha = 90°$$

where ε_a = vertical strain, J= textile modulus and D=diameter of the cell.

$$\Delta\sigma_3 = \frac{Tension}{D.h} \text{ for } \alpha = 0°$$

where *Tension*=tension in the textile, h= cell's height we use (1.1) to determine $\Delta\sigma3$ for $\alpha = 45°$

Bishop proposed a simple expression where $C(\alpha)$ is given as a function of α which denote the inclination of the major principal stress with respect to horizontal axis, $Cv=C(0)$, $Ch= C(\pi/2)$ and $C(\pi/4)$ (Bishop 1966):

$$C(\alpha)=Cv.(1-a.\sin^2\alpha)(1-b.\sin^2 2\alpha) \quad (1.4)$$

with $a=1-Ch/Cv$ and $b= 1-(Ch/Cv)/(1-a/2)$

The limit equilibrium analysis presented above is used for calculating the shear strength for the three angle 0, $\pi/2$ and $\pi/4$.

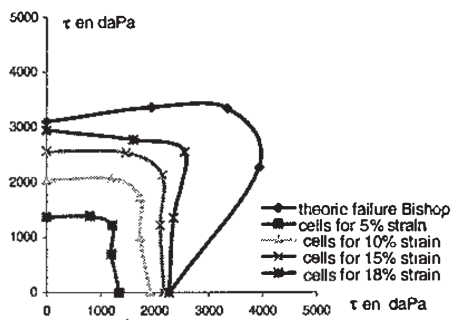


Fig. 11 Comparison of experiments and theory

Therefore a less important disagreement between the shape of the predicted and observed behaviour is found. The results illustrate that the orientation and magnitude of the stiffening effect and strength increase imparted to the soil by the enhanced confinement effect are well reproduced by the modelling. The strenght criterion of such an equivalent homogeneous material is then determined starting from the strenght characteristics of the reinforced soil components.

$f(\sigma)=(\sigma1-\sigma3)/2 -C(\alpha)= 0$

We use Mohr Coulomb criterion even if the value of $\sigma2$, the intermediate effective principal stress has no influence on the strength, experimental results support it as a simple criterion of reasonable generality.

3 MODEL DEVELOPPED

Limit equilibrium methods have been widely adopted for slope stability analysis mainly due to the simplicity that the method offer but suffer from a lack of displacement considerations. Classical method of stability analysis like calculation of shear along the slip line can be applied to this system.We use a two blocks method i.e. a bilinear line of slippage. This method does not allow the three equation of equilibrium to be satisfied (equilibrium of moments). Our software used the "displacement method" (Gourc 1994) inducing a displacement at the crest and along the line of slippage, allowed to determine the tensile forces even if we do not take in account the densification of the soil.

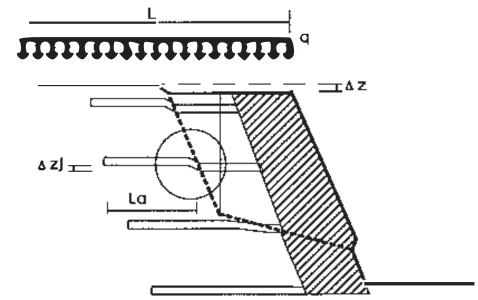


Fig.12 Two blocks method

The calculation method consist in increasing Δz by small amount until overall equilibrium of the active wedges is obtained. To permit an approach of the deformation behaviour taking into account the deformation of the geotextile and the confinement at the edge of the wall, we introduce for each Δz the stress distribution calculated using the properties of the equivalent material. During the calculation of the factor of safety, we use the macroscopic failure criterion presented above.

The idea here is to obtain the stress of the reinforced soil along the slip surface due to Δz and to compare it to the failure criterion. Then the available shear resistance of the wall is determined. A limit equilibrium analysis is then performed to determine the stability of the soil mass above the probable slip surface. The final determination of the overall stability is achieved by iterating the computations for the least factor of safety. It is obvious that the calculated factor of safety of the slope is only the average value along an assumed sliding surface.

The other method is a numerical modelling conducted in plane strain using the computer program FLAC (distributed by Itasca Inc.)(Billaux 1992). The Finite Difference method used here allows an approach of the behaviour during the deformation by taking into account the geotextiles distorsion capacity The space discretization of the soil consists of 900 rectangular elements, the one of the fill consisting of 200 cables elements. The mesh and displacement boundary are shown in figure 13. Our model are based on the exact geometry of tests with their different zones.The model is blocked on its lateral boundaries considering that we are far from disturbed zones so that we can neglect their effects (displacements, stresses ...).

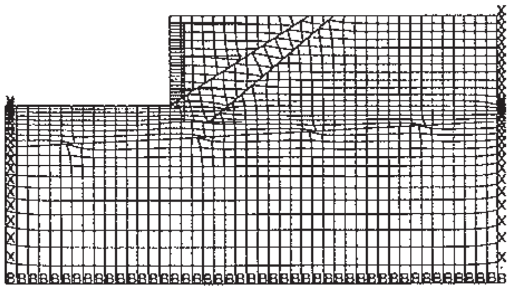


Fig.13 Geometry and boundary conditions

The soil was modelled using elastic-perfectly plastic material with a Mohr Coulomb yield criterion. The reinforcement was represented by means of line elements with no bending stiffness and a linear axial force-extension relationship. The requested parameters are then the Young modulus, the Poisson ratio, the internal friction angle, the cohesion and the density. Interface elements were provided between the reinforcement and the soil. These permitted slip according to a Mohr Coulomb criterion.

4 VALIDATION BY CASE STUDIES

In order to collect datas to improve its calculation method, Armater realised in cooperation with Blaise Pascal University a real scale experimental wall and several experimentation on scaled models. The main objectives of these experiences were to evaluate the wall face deformation using the proposed method and to see how well these predictions compare with the measured values. Both parts of the full scale wall have been instrumented (figure 14). Measurements of external displacement have been carried out on several sections by surveying. The experimentations on scale models reported here were intended to complement the previous investigations and to study factors likely to influence the deformation behaviour. We built '1g' models, 10 times smaller but geometrically identical to Charade wall. We cannot ensure similarity between the response in a model and the response in the corresponding full-scale case. The geometry of these wall are presented in figure 14.

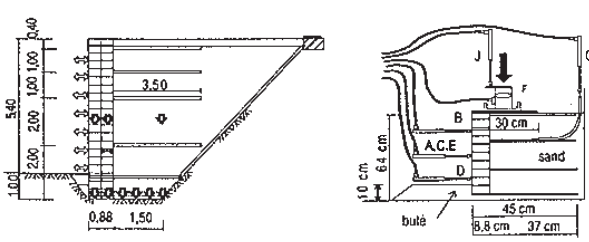


Fig.14 Geometry

Displacement measurements on real scale trial wall an on medium scale models were taken during the test and failure mechanisms were identified. Other field measurements such as soil stresses and strains in the walls are not discussed in this paper (LRPC 1994).

Excellent comparisons were obtained for the front face displacement at various stages. A good agreement between observed and predicted failure surface using FLAC was noted as shown in figure 16.

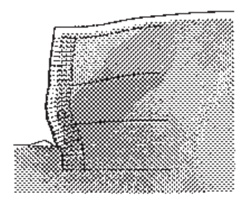


Fig.15 Failure zone predicted by finite difference

A good correlation can be noticed between the maximal tensions existing in the cable elements and the seam breakings.

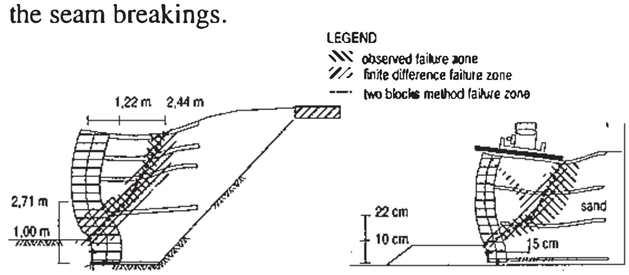


Fig.16 Comparison of the different line of failure

Figure 16 shows a comparison between the predict failure surface and the observed zones of soil mass movement for full scale wall and scale models in both case the double wedge surface closely approximate the soil mass failure zone. This results confirms that deformation response of reinforced soil wall and the influence on the behaviour due to confinement can be predicted accurately by these models.

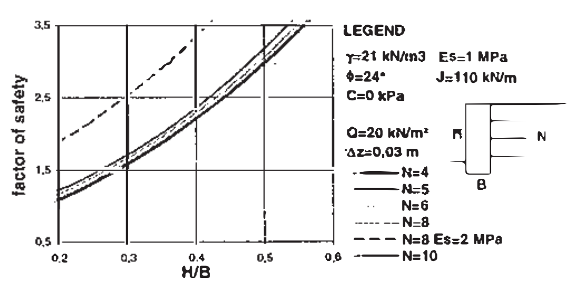


Fig.17 Example of chart

The magnitude of displacement is very sensitive to the modulus values for both soil and geotextile.

CONCLUSIONS

The methods presented here have proved successful in explaining the action of the reinforcement and quantifying the potential improvement in stability. Their main interest is that it take into account the deformability of the whole structure. Therefore allowing us to use this method of design for all this new type of wall structures.

ACKNOWLEDGEMENT

The General Council of Puy de Dôme, the Regional Council of Auvergne, the French transport research laboratories and the LERMES/CUST of the Blaise Pascal University in Clermont Ferrand provided founding for this study. Their support is greatly appreciated.

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