Mécanismes et comportement · Méthodes de dimensionnement
Mechanisms and behaviour · Design methods

La session est ouverte sous la présidence de
J. KERISEL
Société Simecsol, France

M. KERISEL

Mesdames, Messieurs, la séance est ouverte.

Je vais avoir le plaisir de présider cette première session qui est, à mon avis, la plus importante de ce congrès. J'aurais tout d'abord le plaisir de vous présenter le rapporteur général et son adjoint, qui vont prendre la parole alternativement pendant une heure et demie. James MITCHELL est professeur de Génie Civil à l'Université de Californie à Berkeley. Il a particulièrement étudié les questions de la terre armée et à ce titre a été désigné comme rapporteur général. Il sera assisté dans sa tâche par notre ami François SCHLOSSER, directeur des études de la Terre Armée à Paris, et professeur à l'Ecole Nationale des Ponts et Chaussées, en quelque sorte actuellement le maître à penser de la terre armée en France et à l'étranger.

Notre séance d'aujourd'hui est consacrée plus spécifiquement au problème de la terre armée, à l'exclusion des problèmes spéciaux comme ceux des pieux-colonnes ballastés ou des pieux-racines. Je vous rappelle que la discussion suivant le rapport général sera partagée en trois parties principales : une première partie où seront discutés les mécanismes de base et l'interaction sur l'armature ; une deuxième partie examinera les méthodes d'analyse et les méthodes de calcul ; et la troisième traitera des problèmes spéciaux, sismique , radiers. Je donne maintenant la parole à M. le professeur MITCHELL.

Rapport général - General report

J.K. MITCHELL

University of California, Berkeley, USA

F. SCHLOSSER

Ecole Nationale des Ponts et Chaussées, France

INTRODUCTION

The use of tensile load carrying elements to increase the strength and load carrying capacity of soils dates to prehistoric times (ASCE, 1978). It is only within the last 20 years or so, however, that rigorous analytical and experimental studies, pioneered by Vidal (1966), have led to the widespread use of earth reinforcement for a wide range of earthwork construction and soil improvement applications. By now well over 2 000 Reinforced Earth structures have been completed all over the world.

Applications of tensile reinforcement have included retaining walls, tunnel stabilization in cohesionless soils, bearing capacity enhancement, slabs to bridge caverns, construction of stable embankments over loose, liquefiable sands and soft clays, strengthening of dams, strengthening of base courses materials, landslide repair, bridge abutments, containment dikes, and the construction of stable walls for coal storage slots.

It is very evident that safe, economical reinforced earth systems can be designed and constructed. In many applications, however, practice leads theory, so that optimum designs may not always result, prediction of performance may not be precise, and all aspects of behavior may not be fully understood. Furthermore, new applications are continually being conceived, and new reinforcement materials and configurations are under investigation.

The large number of papers submitted to this Conference and the high attendance from all over the world provide eloquent testimony to the strong recognition of the importance of earth reinforcement in modern construction, to the desire of engineers to understand and apply correct principles and procedures, to our desire to explain and understand all aspects of behavior, and our need to resolve differences in concepts that have emerged from the results of different studies.

Session 1 is concerned with mechanisms, behavior, and design methods. Thirty-three papers have been assigned to this session for review, and our analysis of these contributions has been organized within the following framework, which also delineates the scope of the report :

- Basic Mechanisms
- Soil-Reinforcement Interactions
- Analysis Methods
- Design Methods for Reinforced Earth Systems
- Special Problems
- Conclusions

A brief summary of the state-of-the-art prior to this Conference is presented at the beginning of each section to establish the context for the review of the new contributions. References of particular usefulness for establishing present state-of-the-art include :

(1) American Society of Civil Engineers (1978), "Soil Improvement : History, Capabilities, and Outlook", Committee on Placement and Improvement of Soils (Chapter VII and Apprendices II and III of this report, which deal with re-

inforcement using tension elements, were prepared by the late Kenneth L. Lee, who was responsible for major advances to the state-of-knowledge of the stattic and dynamic behavior of reinforced earth systems.).

(2) American Society of Civil Engineers (1978), Proceedings of the Symposium on Earth Reinforcement, Annual Convention, Pittsburgh, April 27, 1978.

(3) Mc Kittrick (1978) (References cited are listed alphabetically by author at the end of this General Report.).

(4) Schlosser (1978).

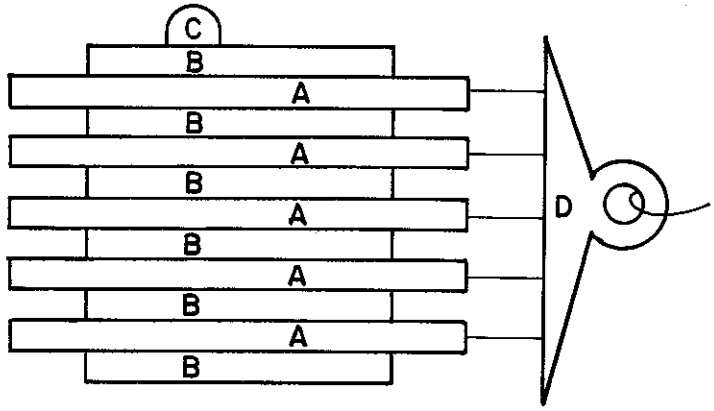


Fig. 1 - Amonton's (1699) illustration of increased frictional resistance resulting from layering

BASIC MECHANISM

Introduction

In tensional earth reinforcement the reinforcements combine with soil through frictional bonding to provide a composite material. Systems of interest are distinct from those relying on anchors and tie backs for restraint of the tension elements. Consequently the load-deformation and ultimate load capacity of the reinforced soil depends on the stress-strain properties and strength of the soil, reinforcing elements, and interactions between the elements and the soil.

There is an apparent similarity between the action of the modern Vidal Reinforced Earth and some basic physical concepts on friction enunciated by two earlier French engineers over two centuries ago. Amontons (1699) published experimental results of sliding blocks which led to the definition of the basic laws of sliding friction. Coulomb's (1773) classical essai on mechanics used Amontons concept of sliding friction to develop a mechanistic approach to the design of retaining walls.

Although Amontons specifically stated that for a constant normal force the frictional force was independant of sliding areas, he went on to give an illustration, reproduced in Fig. 1, showing that for the same normal weight C the frictional resistance D could be increased by a sandwich type of layering between resisting blocks and sliding blocks. The greater the number of A-B sandwich layers, the greater the resistance. In the same way, the strength of a reinforced earth mass (Fig. 2), is increased by increasing the concentration of frictional reinforcements and by creating tensional bonds between the active and resistant zones.

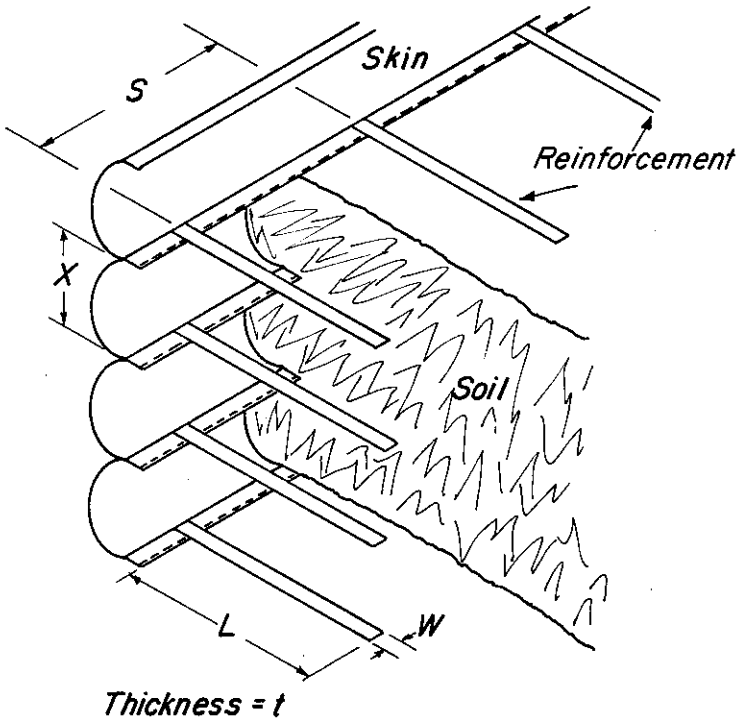


Fig. 2 - Concept of the Vidal reinforced earth wall

Additionally the lateral earth earth pressure concept, as first defined by Coulomb, has provided a first basis for analysing the lateral forces which must be resisted by the reinforcements. However, Kinematics and failure mechanisms basically different from the classical Coulomb's failure mode develop in reinforced earth structures and result in an "active" failure zone essentially different than the classical "Coulomb's failure wedge" (Juran and Schlosser, 1978).

The essential phenomenon in the mechanism of reinforced earth is the friction mobilized at the soil-reinforcements interfaces. In a linear reinforcement strip, which is currently used in reinforced earth structures, the tensile force varies along the reinforcement.

The local equilibrium of a portion of the reinforcement, illustrated in Fig. 3, shows that the slope of the tensile forces distribution curve is directly proportional to the average shear stress (τ) exerted by the soil on the surfaces of the reinforcement. This gives the following expression (Schlosser and Long, 1972) :

$$\tau = \frac{1}{2b}\frac{dT}{dl} \quad \text{.....} \quad (1)$$

where :

- b is the width of the reinforcement ;
- l is the coordinate of the considered point along the reinforcement ;
- T is the tensile force at the considered point.

In reinforced earth structures the tensile stress in the reinforcements is a maximum at some distance behind the wall facing and the shear stress distribution is not generally uniform. There is an "active zone" behind the facing in which shear-stresses are directed outwards, towards the wall face, and a "resistant zone" in which the shearing

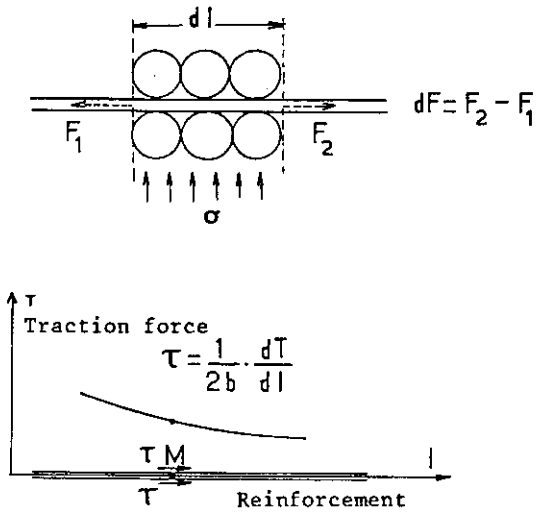


Fig. 3 - Variation of the tensile force along the reinforcement

stresses are directed towards the free ends of the reinforcements. These conditions as well as the approximate boundary between the active and the resistant zones, are shown in Fig. 4. The left part of the figure shows the locus of the maximum tensile forces deduced from observations on actual reinforced earth walls ; whereas, the right part shows the theoretical maximum tensile forces line obtained by the finite element method (Corté and al., 1974). By holding

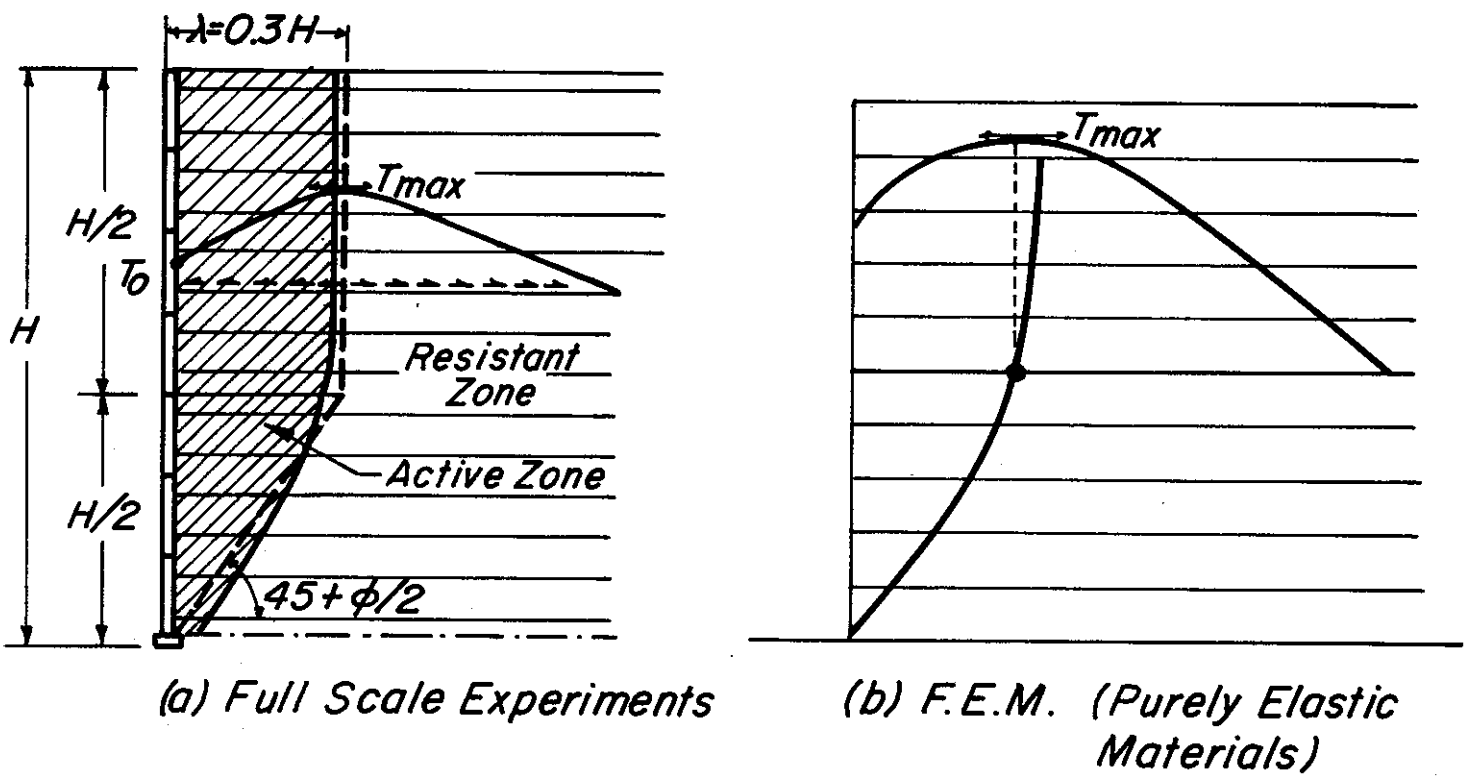


Fig. 4 - Tensile force distributions along reinforcements (Schlosser, 1978)

together the zones on opposite sides of the locus of the maximum tensile forces, the reinforcements give to the reinforced earth mass an apparent global cohesion which is directly proportional to the tensile resistance of the reinforcements and to their density.

Fig. 5 shows that the maximum tensile forces line depends on the loading conditions and on the type of the structure.

Fig. 6 shows the different zones in a reinforced earth sample under a biaxial compression and their evolutions whith the level of the applied stresses.

These theoretical results were obtained by the finite element method (L.C.P.C., 1973). They were confirmed experimentally by observations on the state of thin aluminium reinforcement discs in a triaxial specimen of reinforced sand (Schlosser and Long, 1972). Fig. 7 illustrates the cracks in the reinforcements which develop in the positions of the maximum tensile forces at the peak of the stress-deformation curve.

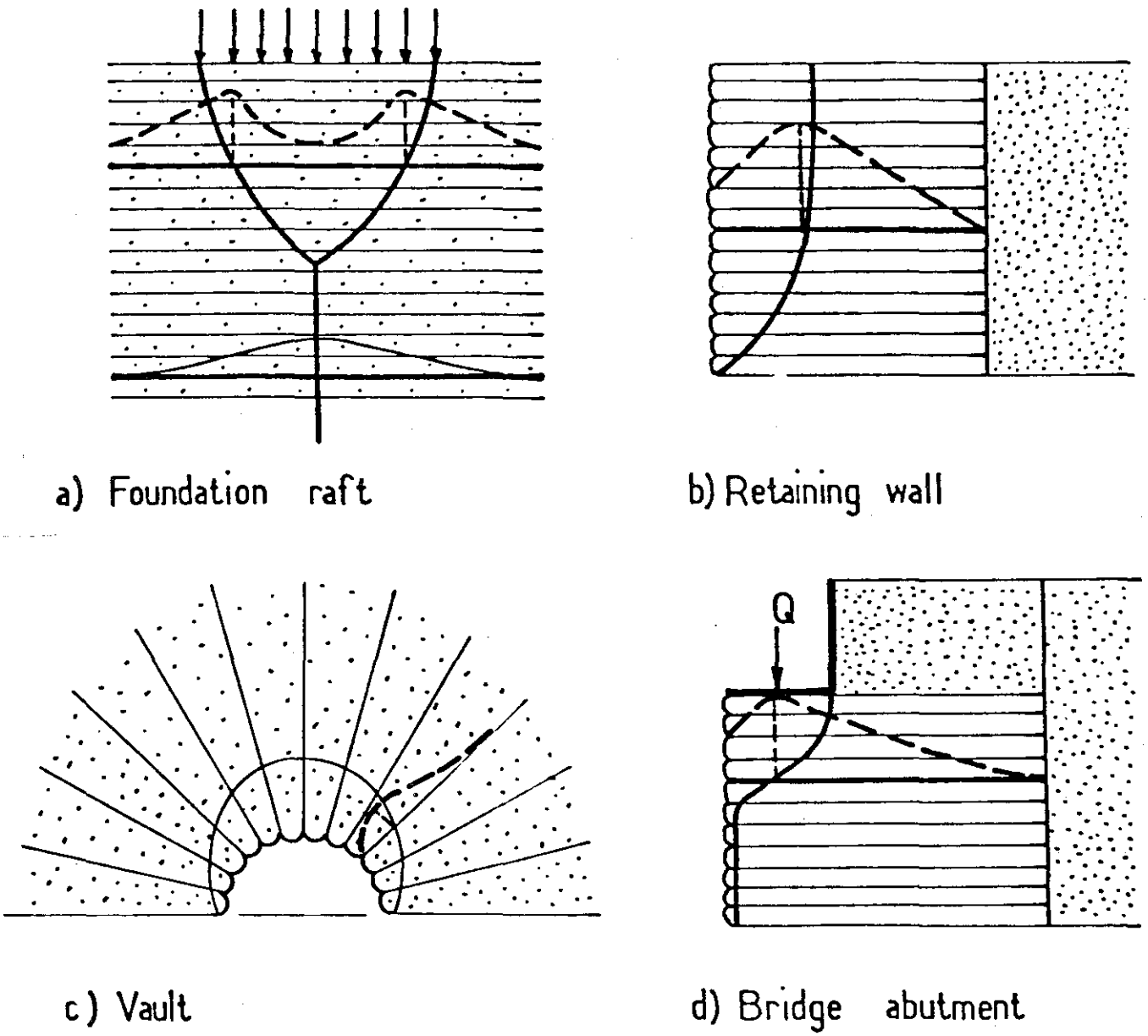


Fig. 5 - Locus of maximum tensile forces in different reinforced earth structures

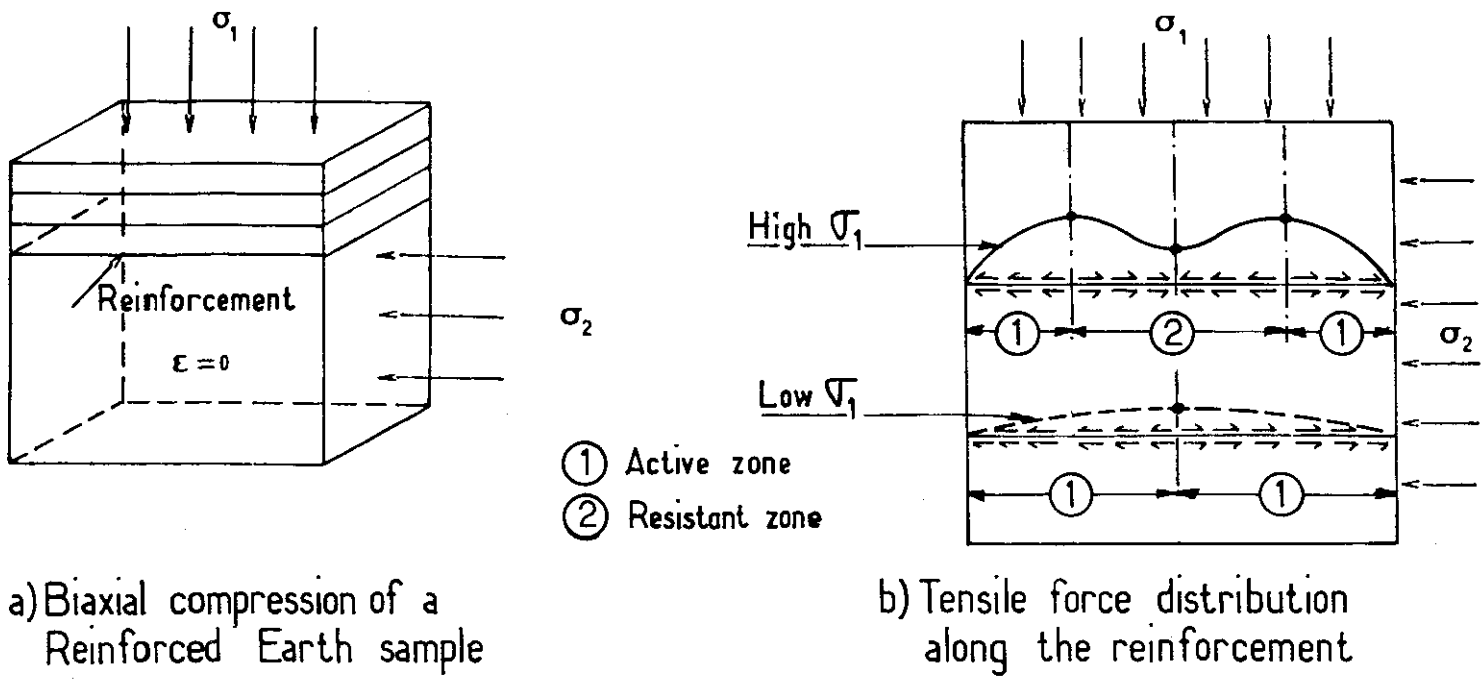


Fig. 6 - Development of tensile forces in a Reinforced Earth sample. F.E.M. (L.C.P.C., 1973)

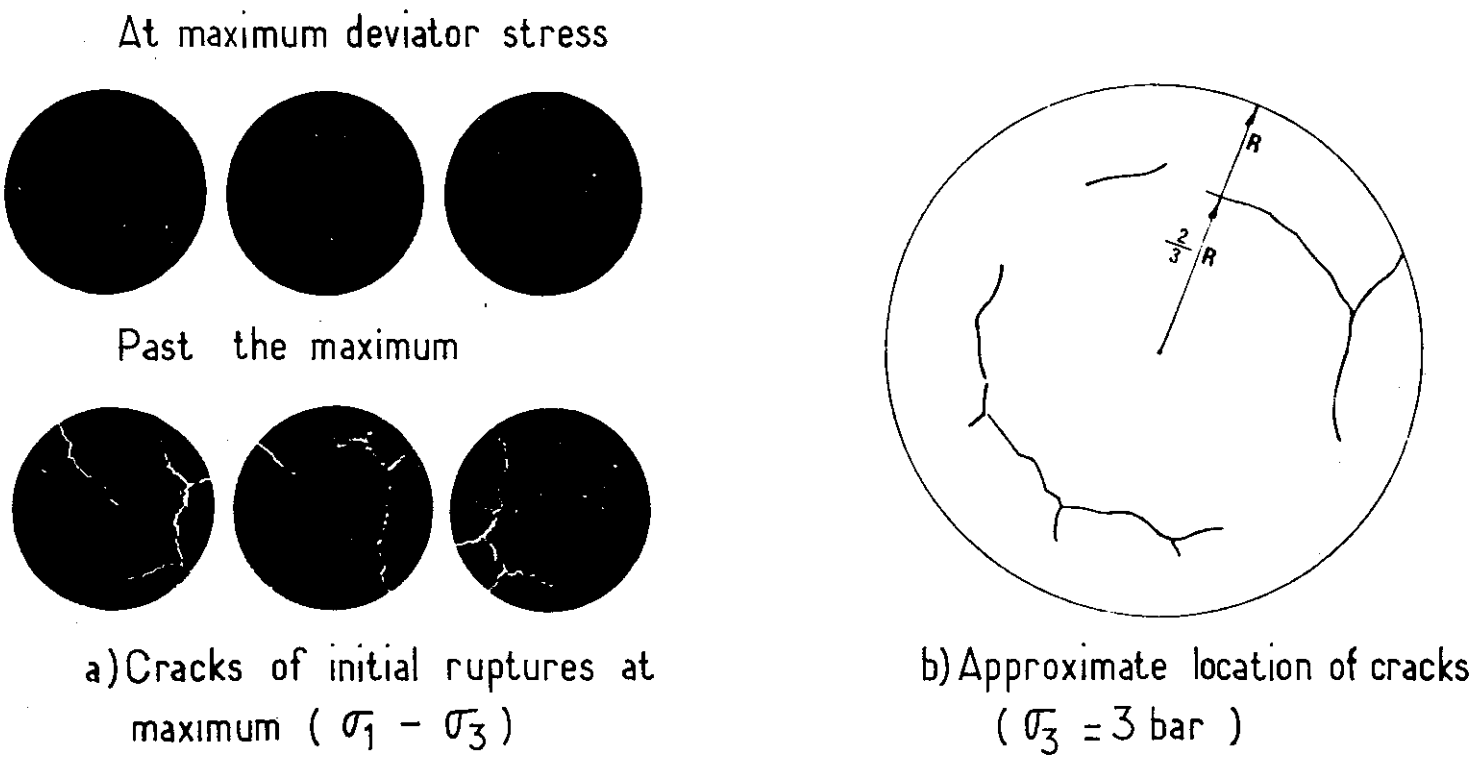
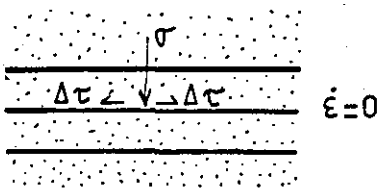


Fig. 7 - Behavior of Reinforcements in a triaxial specimen of Reinforced sand (Schlosser and Long, 1972)

Fundamental concepts of reinforcement

As illustrated in Fig. 8, apparently different concepts have been advanced to define this basic mechanism of reinforced earth. The effect of inclusion in the soil of reinforcements having an high modulus of deformation can be explained using either an induced stresses concept or an induced deformations concept.



1) **STRESSES.**

Rotation of principal directions, $\Delta\tau$ (Schlosser and Vidal - 1969)

2) **DEFORMATIONS.**

Zero extension ($\dot{\varepsilon}_1 \simeq 0$) in the direction of the reinforcements (Bassett and Last 1978)

Fig. 8 - Different concepts of the basic mechanism in reinforced earth

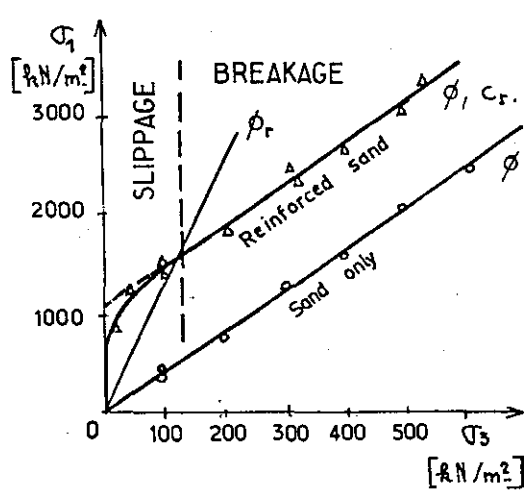
The first concept is related to an apparent cohesion. (Schlosser, Vidal, 1969). The tensile strength of the reinforcements and friction between the soil and reinforcements give an apparent cohesion to the composite material. At the same time the friction mobilized at the soil-reinforcement interfaces causes a rotation of principal stresses in the soil and modifies the initial state of stresses.

The second concept (Bassett and Last, 1978) considers that the mechanism of tensile reinforcement involves anisotropic restraint of the soil deformations in the direction of the reinforcements (undirectional, in the case of parallel reinforcements). This effect results in a rotation of the principal directions of the deformations tensor. Bassett and Last (1978) suggest that more can be learned by analysis of the modifications to strain fields caused by reinforcements than by the study of forces and stresses.

The two concepts show that in any case the reinforcements modify completely the initial state of stresses and of deformations.

The global effect of reinforcements was the object of experimental studies in the triaxial apparatus on sand samples reinforced by horizontal thin alluminium plates (Schlosser and Long, 1972), and by horizontal nets of fiber glass (Yang, 1972) uniformly spaced. Fig. 9 illustrates the failure curve of the reinforced sand in the plane of principal stresses. Failure occured either by excessive lateral deformations due to the sliding of the sand on the reinforcements (first part of the failure curve) or by the breakage of the reinforcements (second part ot the failure curve).

Schlosser and Long (1972) showed that the reinforcements give to the sand an anisotropic cohesion directly proportional to their density and to their resistance to tension. Accordingly they interpreted the strength envelope for reinforced sand at failure by breakage of the reinforcements,



Schlosser et Long (1972)

$$C_r = \frac{R_T}{\Delta H} \cdot \frac{\sqrt{K_p}}{2}$$

Yang (1972)

$$\Delta\sigma_3 = \frac{R_T}{\Delta H}$$

Hausmann (1976)

$$\phi_r = \text{Arc.sin}\left(\frac{1+F_s-K_a}{1-F_s+K_a}\right)$$

$$F_s = \Delta\sigma_3 / \sigma_1$$

$$\Delta\sigma_3 \leqslant \frac{R_T}{\Delta H}$$

Fig. 9 - Strength envelope for reinforced sand

as that of a cohesive frictional Mohr-Coulomb material with strength given by :

$$\sigma_{1f} = \sigma_{3f} \times N\phi + 2C\sqrt{N\phi} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

where :

. $C = \dfrac{R_T\sqrt{N\phi}}{2h}$; ..... (3)

. R_T : tensile resistance of the reinforced unit thick section

. h : vertical spacing between horizontal layer of reinforcing

Yang (1972) hypothesized that the tensile stresses built up in the reinforcements were transferred to the soil through sliding friction and caused an increase in the confining pressure $\Delta\sigma_3$.

The two approaches are related according to :

$$\Delta\sigma_3 = \frac{R_T}{h} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

so either can be used to analyse failure by reinforcement breakage.

Hausman (1976) interpreted the effect of reinforcing the sand on its strength characteristics considering a global apparent friction angle (ϕ_R). He assumed that when failure is caused by slip between the sand and the reinforcements, the reinforcing effect can be expressed in terms of an increased apparent friction angle (Fig. 9).

Schlosser and Long (1972) studied in the triaxial apparatus the mobilization of the strength characteristics of the reinforced earth, its behaviour and its response to a triaxial compression before failure by reinforcement breakage. They showed (Fig. 10) that under small axial deformation there is a rapid mobilization of the apparent cohesion induced by the reinforcements. On the other hand the mobilization of the internal friction angle is practically not affected by the presence of reinforcements.

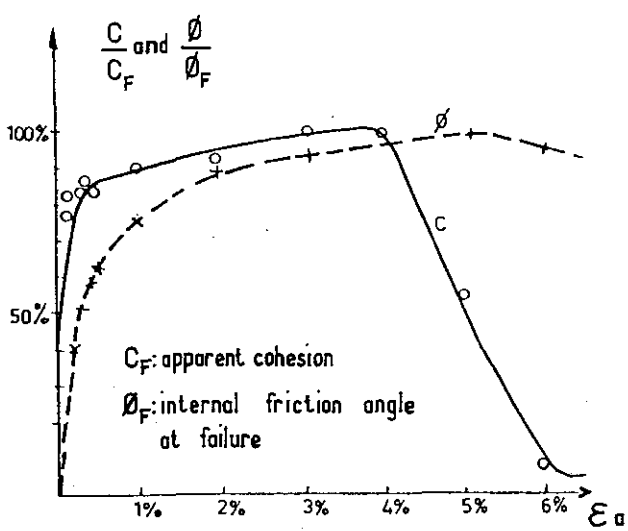


Fig. 10 - Mobilization of strength characteristics C and ϕ

The global effect of the reinforcements is to restrain the lateral deformations of the soil (Bassett and Last, 1978 ; McKittrick, 1978 ; Swiger, 1978). An axial load on a sample of dense granular material will cause lateral expansion as shown in Fig. 11. For a dilatant material the lateral strain would be greater than 0.5 times the axial strain. If, however, inextensible horizontal elements are placed within the soil mass as shown for the reinforced case in Fig. 11, then the behaviour will be as if a lateral restraint had been applied.

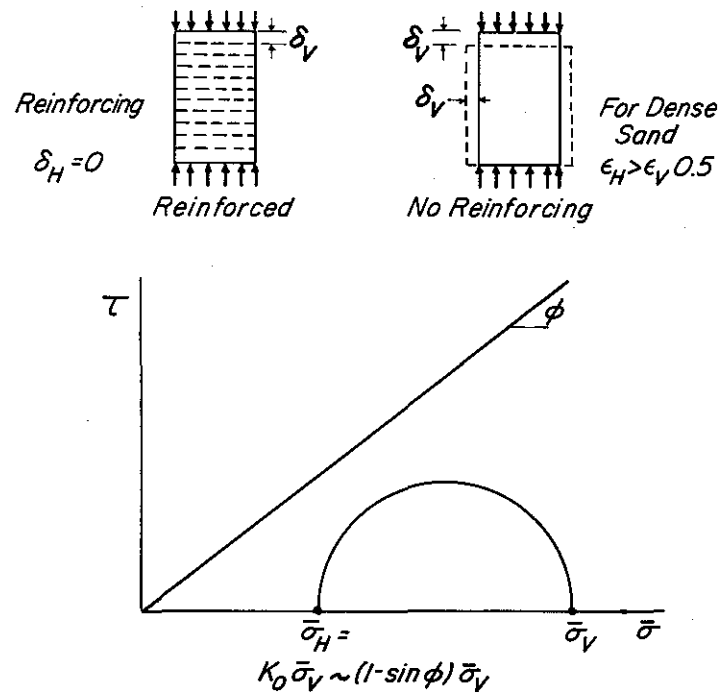


Fig. 11 - State of stress in earth reinforced with tension elements

For complete prevention of lateral strain this equivalent lateral load would equal the at rest earth pressure. Thus the equivalent effective confinement σ'_H would be :

$$\sigma'_H = \sigma'_V \cdot K_o = \sigma'_V\,(1 - \sin\phi) \quad \ldots\ldots\ldots \quad (5)$$

where : K_o is the at rest earth pressure coefficient, equal to $(1 - \sin\phi)$ for normally consolidated sands, and σ'_V is the vertical effective stress. Thus as the vertical stresses increase, the horizontal stresses increase in direct proportion, and the stress circle lies as well below the rupture curve at all points.

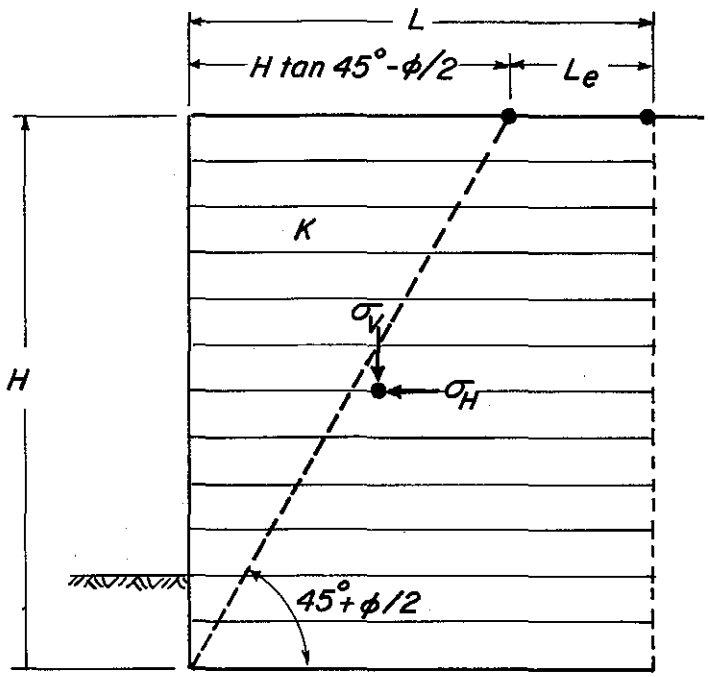
Schlosser and Long (1972) have demonstrated that, under small axial deformations, the reinforced sand is actually at a K_o state of stresses corresponding to relation (5).

However, as deformations increase this equivalent state of stresses changes progressively to the failure state, and the frictional resistance of the sand becomes fully mobilized.

Thus the reinforced earth provides the economical advantage of being a composite material with an equivalent resistance which results from the entire mobilization of the resistances of the constitutive elements - the sand and the reinforcements.

Behavior of Reinforced Earth Structures

The basic mechanism of reinforced earth structures has been the object of many full scale experiments, laboratory model studies and theoretical analyses. Many investigators have used a "tie-back structure" hypothesis for analysis of reinforcement forces, required reinforcement lengths, failure surface geometry, etc. ; as shown in Fig. 12. A "coherent gravity structure hypothesis", as shown in Fig. 13, however, appears to more adequately represent the failure surface, reinforcement forces, and reinforcement length requirements.



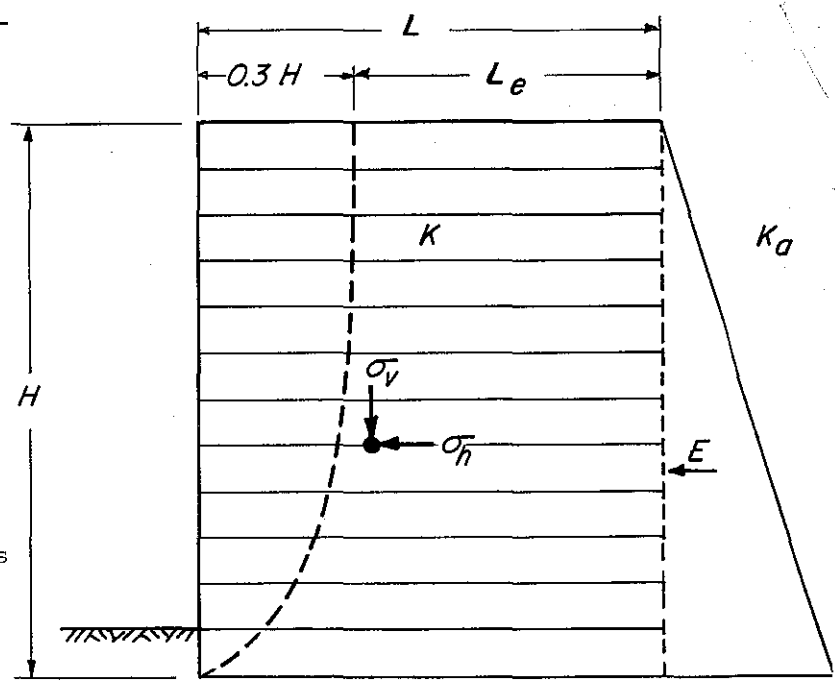
$$\sigma_h = K \cdot \gamma H$$

$$L \geq L_e \geq L - H \tan(45° - \phi/2)$$

for $\phi = 30°$:

$$L \geq L_e \geq L - 0.58H$$

Fig. 12 - Tie-back structure hypothesis for reinforced earth wall



$$\sigma_h = K(1 + K_a)\gamma H$$

$$L \geq L_e \geq L - 0.3H$$

for $\phi = 30°$:

$$\sigma_h = 1.33K \cdot \gamma H$$

Fig. 13 - Coherent gravity structure hypothesis for reinforced earth wall

Investigators at L.C.P.C. (e.g. Schlosser and Long, 1972) showed that the locus of the maximum tensile forces in the strips of a reinforced earth wall, representing a potential failure surface of reinforcement breakage, is essentially different from the classical Coulomb's failure plane. The presence of the reinforcements modifies the overall displacements pattern of the reinforced earth mass and the shape of the corresponding "active zone". The maximum tensile force line is practically vertical at the upper part of the wall and is relatively closer to the facing than Coulomb's failure plane (Fig. 4). However, investigators at UCLA (Lee et al., 1973) and other researchers disagreed with these results and assumed that the presence of the reinforcements has no effect on the shape of the failure surface. This divergence has been the object of many discussions reported in the literature.

In 1978, Bassett and Last used Roscoe's failure criteria for sands, based on the zero extension concept, to demonstrate that the presence of the reinforcements leads to a rotation of the principal direction of the deformations tensor. They showed

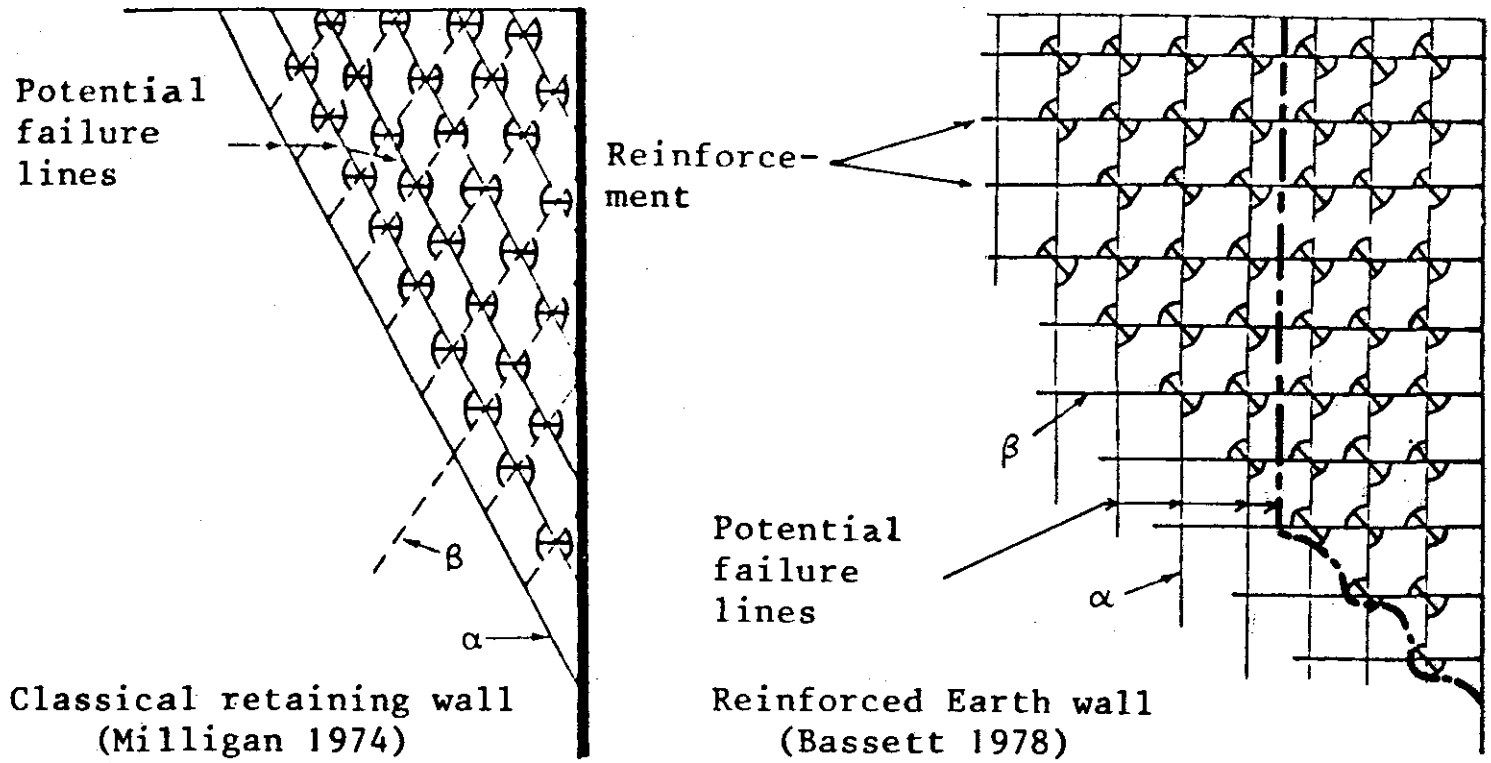


Fig. 14 - Zero extension characteristics (α and β) and potential failure lines for sand and for reinforced sand structures

(Fig. 14) that in a reinforced earth wall, as the direction of the reinforcement must be aligned with, or very close to, that of zero extension, the failure surface must be vertical to comply with the assumption of suppressed dilation rate. Bolton et al. (1978), demonstrated by a centrifugal model study of reinforced earth walls that the locus of the maximum tensile forces in the strips remains vertical during the construction of the wall. The construction was simulated by gravitational acceleration (Fig. 15).

The results of these recent studies were confirmed by observations on actual structures reported by Schlosser and Elias (1978). Fig. 16 illustrates the locus of the maximum tensile forces in the strips of 4 reinforced earth walls of different heights. The results of several model test studies reported in this Conference add further support to the basic soundness of these previous findings

Bacot and Laréal present the results of experiments on Reinforced Earth walls with a constant reinforcement density and with reinforcement lengths that are significantly less than the wall height. The failure surface was observed to start at the base of the wall and progress as shown in Fig. 17. Along the inclined lower portion there was reinforcement breakage. The upper portion is a vertical plane passing along the back of the reinforcements, where there is a

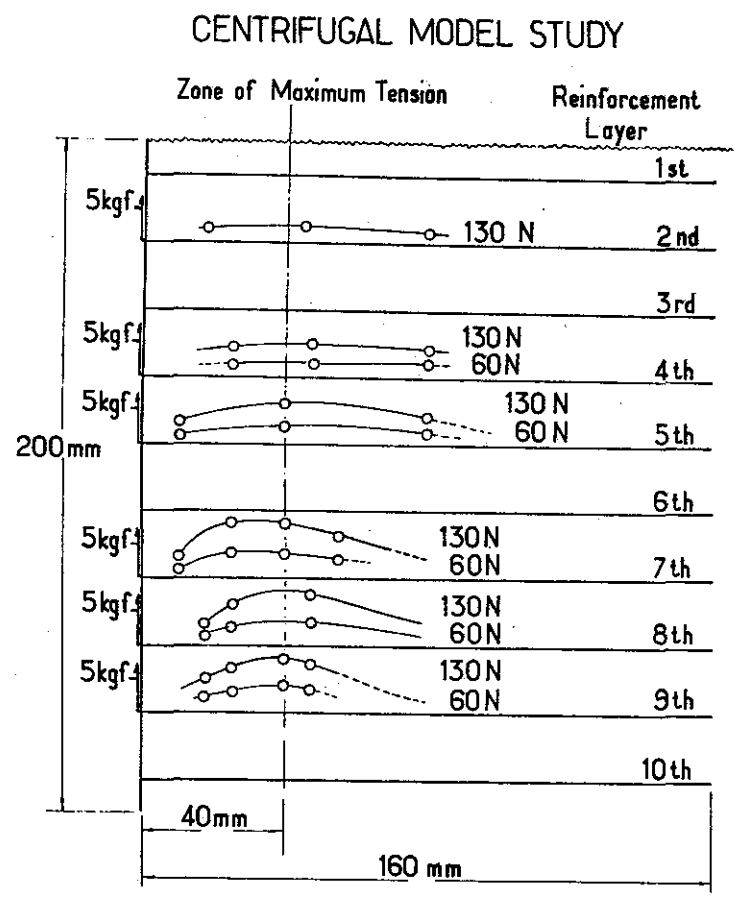


Fig. 15 - Tension Distribution in Models of Reinforced Earth walls at N Gravities up to N = 130 (Bolton and al., 1978)

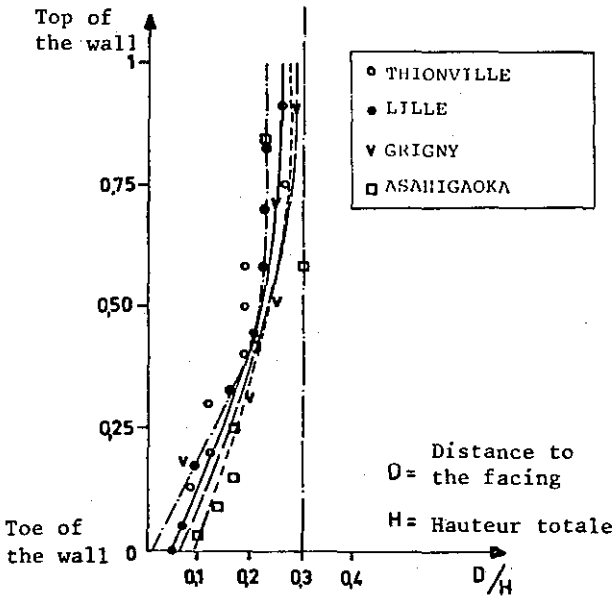


Fig. 16 - Maximum tensile forces lines in experimented Reinforced Earth walls (Schlosser - 1978)

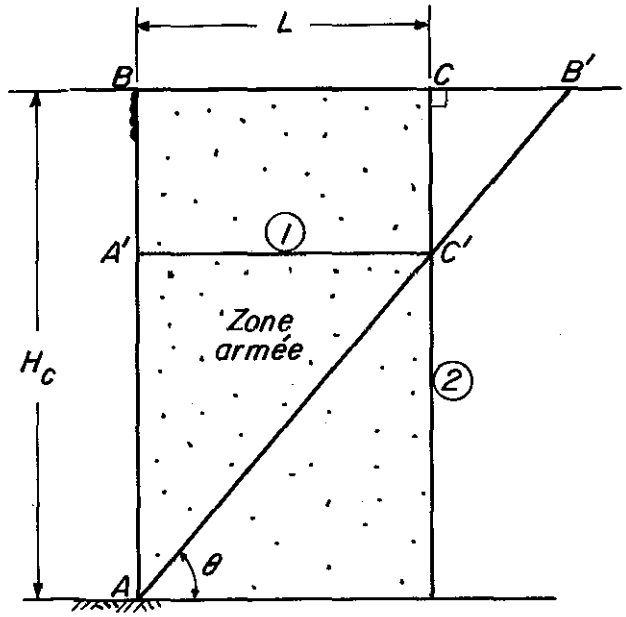


Fig. 17 - Trapezoïdal failure zone for wall with short reinforcements (Bacot and Lareal)

mobilization of soil-soil friction. Failure zone equilibrium determines the value of the inclination angle O.

The failure mode comprises simultaneously a failure caused by lack of adherence between soil and reinforcement in the upper part and reinforcement breakage at the lower part of the wall. Bacot and Laréal present an analysis to show that for a wall of this type with a constant reinforcement density, the height at failure permits consideration of the global apparent cohesion concept discussed above. This apparent cohesion C is a function of the two failure modes. For large values of L/H there is no failure by reinforcement slippage, and the global apparent cohesion is a maximum. It can be compared with the value obtained in a triaxial test given by equation (3). For the case of short strips where failure is due to reinforcement slippage, the apparent global cohesion depends on both wall geometry and the value of L/H.

Two papers in the conference deal with photoelastic studies of the displacements and of the corresponding shape of the failure surface in bidimensional laboratory models of reinforced earth walls.

Pietrzyrk shows that the classical Coulomb's plane failure surface is inadequate to represent reinforced earth structure behavior. His results are basically consistent with failure plane observations on laboratory models (L.C.P.C., 1974) and on actual structures (Schlosser, 1978). However, neither theoretical interpretation nor specific conclusions are given by the author.

Reinforcement stresses and displacement vectors in reinforced earth models were determined by Petrik. Strain fields in the form of principal strain magnitudes and directions were plotted. The pattern obtained was similar in some respects to that given by Basset and Last (1978) for the idealization of no extension in the direction of the reinforcements. Maximum homogeneity of the backfill strain field was observed for reinforcement length slightly greater than the wall height. Failures developed along a vertical line at a distance of 0.3 to 0.5 times the wall height from the face, a finding consistent with the coherent gravity structure concept illustrated in Fig. 13.

Fiber Inclusions

The possibility that fiber inclusions might improve the strength and deformation resistance of soils has been considered previously by Hausmann (1976). Hausmann studied in the triaxial apparatus the effect of the diameter of inclusions on the failure by lack of adherence. He showed that the apparent friction angle increases with the dimension of the inclusion (Fig. 18).

Two papers in this conference deal with this topic :

Various proportions of randomly oriented fibers were mixed with a 5.0 - 0.6 mm dry crushed sandy gravel with angular particles by Hoare. Two fiber types were used :

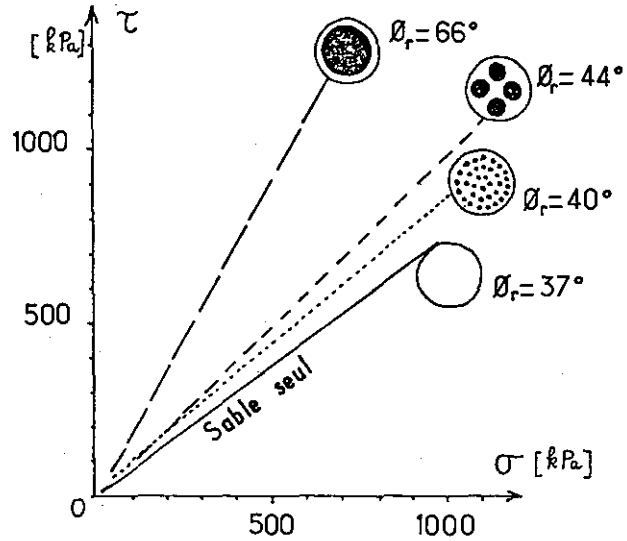


Fig. 18 - Variations of ϕ_r with the dimension of the inclusion (Hausmann, 1976)

7 mm x 66 mm Terram 140 strips and polypropylene fibers in the form of twisted 2 in (50 mm) chopped staple fiber. Triaxial specimens were compacted using seven different methods.

The presence of the fibers gave increased resistance to compaction so that for a given compactive effort the density decreased. Because the density decreased as the reinforcement content was increased, the friction angle was found to decrease also. Increased fiber concentrations resulted in increased strain at failure. Hoare's data suggest that some increase in ø' may result with fiber addition provided that the compactive effort is increased to maintain a constant density.

Both drained and undrained triaxial tests were done on mixtures of kaolinite and pulp (almost pure cellulose) fibers by Andersland and Khattak. The fibers had a weighted average length of 1.6 mm and a typical diameter of 0.02 mm. Fiber contents of 16 and 40 percent by dry weight were used, and test specimens were consolidated from a slurry.

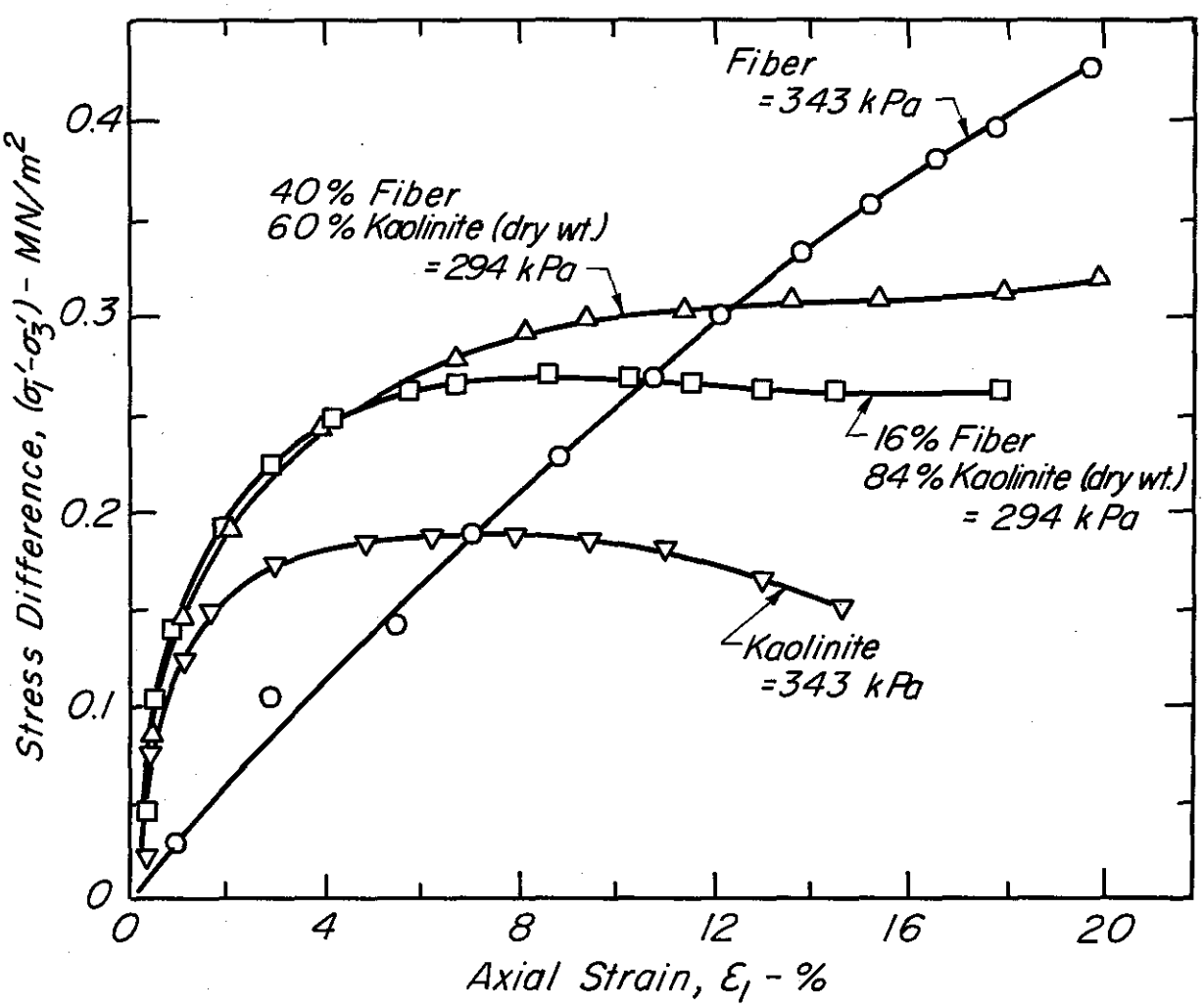


Fig. 19 - Stress-strain curves for undrained conditions and samples with different kaolinite/fiber combinations

The addition of fiber significantly increased the strength of kaolinite for undrained loading conditions, as may be seen in Fig. 19, which illustrates also the influence of the fibers on the stress-strain behavior. The reinforcing effect of the fiber is further illustrated by an increase in effective stress friction angle from 20° for pure kaolinite to 39° for 100 percent fiber samples.

In the authors' opinion these observations agree, at least qualitatively, with the results previously reported by Hausmann. They demonstrate that as long as the failure mechanism is rather the result of a lack of adherence between the soil and the small fibers the reinforcing effect results in increasing the apparent internal friction angle with no evidence of an apparent cohesion. Actually, as indicated by Andersland and Khattak, the small fibers did not break during the tests.

Fig. 20a illustrates a surprising result obtained by Andersland and Khattak. The effective internal friction angle ϕ' determined by undrained tests is much greater than the one determined by drained tests. In the authors' opinion this divergence can be partially explained by the reinforcing effect on the effective stress paths in these two tests. According to Yang's concept of reinforced sand this effect can be interpreted as an increase of the effective lateral confining stress $\Delta\sigma_3'$, which modifies the real effective stress path.

Fig. 20b illustrates the apparent and real effective stress paths in a drained test.

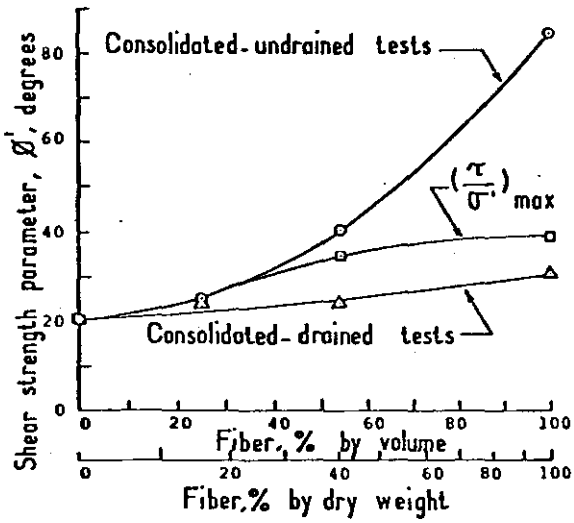


Fig. 20a - Fiber content versus shear strength parameters ϕ' for kaolinite/fiber mixtures

The apparent path corresponds to the classical one ($\theta = 45°$). The real path is obtained by adding to each point on the apparent path the increment of $\Delta p' = \Delta\sigma_3'/2$ due to the reinforcement effect.

It is clearly demonstrated that at failure, the apparent friction angle of the reinforced Kaolinite ϕ_a' can be much greater than the internal friction angle of the unreinforced Kaolinite ϕ'.

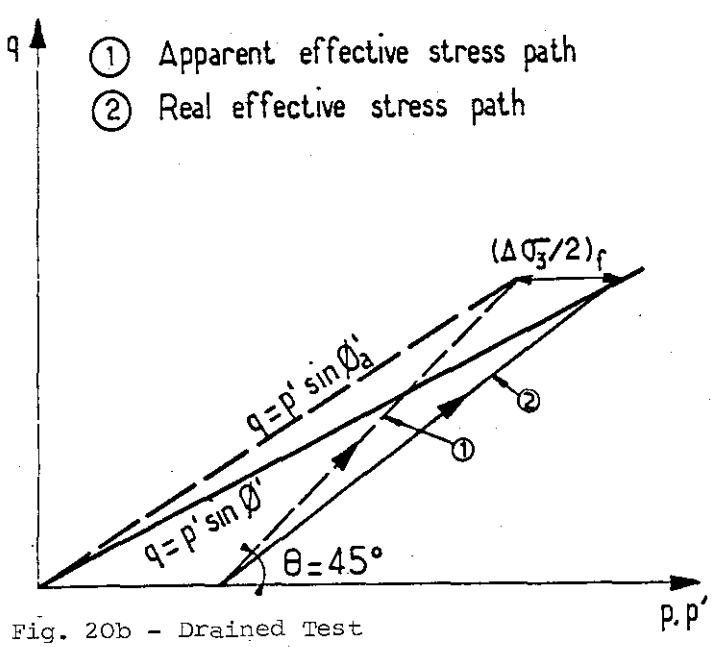


Fig. 20b - Drained Test

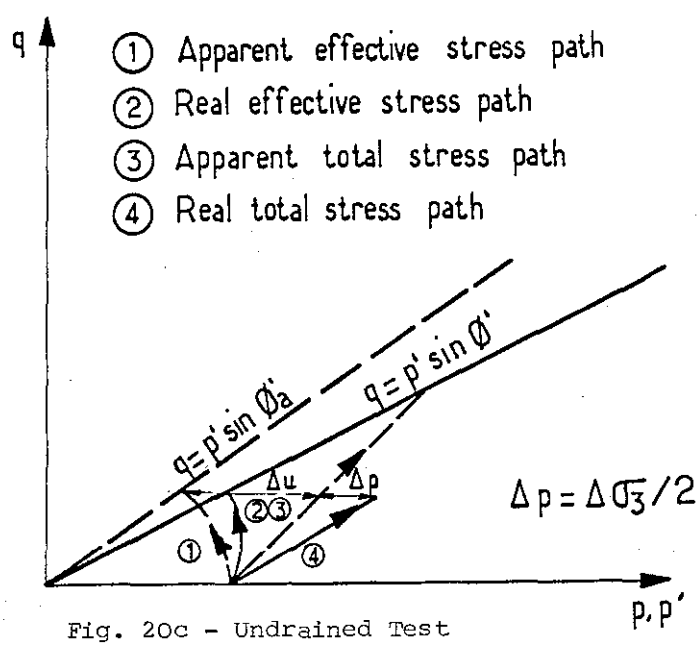


Fig. 20c - Undrained Test

Schematic behaviour of reinforced clay sample in drained and undrained tests

In the case of an undrained test on the normally consolidated reinforced Kaolinite, as the pore water pressure Δu is positive, the increase of the effective lateral confining pressure $\Delta\sigma_3'$ is associated with an increase of the total lateral confining pressure $\Delta\sigma_3$, which is given by the classical Terzaghi's expression :

$$\Delta\sigma_3 = \Delta\sigma_3' + \Delta u \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

Fig. 20c shows the real and the apparent total stress paths in an undrained test. The corresponding efffective stress paths are deduced from the total stress paths by subtracting the measured pore water pressure (Δu). They determine respectively the real internal friction angle ϕ' of the unreinforced Kaolinite and the apparent friction angle ϕ_a' of the reinforced Kaolinite. It is clearly seen that ϕ_a' can be much greater than ϕ'.

Analytical considerations show that the apparent internal friction angle of the reinforced Kaolinite depends mainly on the distribution of the effective lateral confining pressure coefficient at failure A_f of the Kaolinite, and on the isotropic consolidation stress p_0'. Depending on these parameters, the apparent internal friction angle determined from undrained tests by measuring the pore water pressure can be much greater than the one determined from drained tests. This can therefore partially explain the tests results reported by Andersland and Khattak.

On the other hand, Andersland and Khattak indicate that structural effects are also involved in the mechanism increasing the apparent internal friction angle of the reinforced Kaolinite. Individual pulp fibers contain internal pores which allow entry of water molecules and swelling at low pressures. These pores contribute to a high water holding capacity. This fiber structure combined with high water contents and the undrained test condition appear related to high values for the shear strength parameter ϕ'.

The effect of reinforcements on the effective strength parameters of clay samples has been investigated by Ingold. In a paper to this conference, Ingold reports the results of drained and undrained unconfined compression tests on clay samples reinforced using horizontal aluminium discs.

Flexible porous discs 4.75 mm thick and 0.02 thick aluminium foil discs were placed at various spacings normal to the longitudinal axis of triaxial specimens of Kaolin clay.

For porous discs at spacings greater than 40 % of the sample diameter, the strength was reduced compared to the unreinforced clay for shear rates high enough to give undrained behaviour. For smaller spacings the strength was increased. For slow strain rates so that drained conditions prevailed, there was a strength increase for all reinforcement spacings.

The reinforcement, by aluminium foil discs, was found to give a reduction in the soil strength of about 50 % for all spacings tested (1/8 to 1/4 the sample diameter). On the other hand the apparent pore water pressure coefficient A was found to be significantly increased by the reinforcements (Table 1).

Sample	Cell Pressure σ_3 kN/m²	$\sigma_1-\sigma_3$ kN/m²	Δu kN/m²	A	$\Delta\sigma_3$ kN/m²	Strength Ratio
Control	100	52	50	0.96	–	1.00
	200	108	101	0.94	–	1.00
	400	191	201	1.05	–	1.00
Reinforced	100	32	71	2.22	21	0.62
	200	82	126	1.54	25	0.76
	400	151	264	1.75	63	0.79

Table 1 - Results of Multistage Triaxial Tests

$$\Delta\sigma_3 = \Delta u - A(\sigma_1 - \sigma_3) \quad \ldots\ldots\ldots \quad (7)$$

According to Yang's concept the reinforcement effect was interpreted as an increase of the lateral stress $\Delta\sigma_3$ and a failure criterion was established. This failure criterion agrees fairely well with the results of drained tests but it leads to overestimate the undrained strength of the reinforced clay.

In the undrained triaxial tests the increase of the total lateral confining stress $\Delta\sigma_3$ due to the reinforcement effect was determined from the change in the pore water-pressure Δu, using the pore water pressure coefficient A of the unreinforced Kaolin clay (Table 1, Equation 7). However, as the distribution of the lateral confining stress $\Delta\sigma_3$ induced by the reinforcement is generally far from being uniform (Yang, 1972), the author postulates that high values of $\Delta\sigma_3$ generated near the center of the sample induce correspondingly high pore water pressures, which are transmitted through the sample to areas of low $\Delta\sigma_3$. These high transmitted pore water pressures can then cause a reduction in effective stress and hence induce a premature failure leading

to experimental strength ratio values which are much lower than the theoretical ones (the strength ratio is defined as the ratio of the maximum deviator stress exerted on the reinforced clay sample to the one exerted on the unreinforced one).

The reinforcing effects of asbestos and glass fibers on the compressive and tensile strength of compacted cement-treated black cotton soil were determined by Satyanarayana, Sharma, and Shivastava. The tensile strengths were measured using the split test on cylindrical specimens. Increases of between 25 and 55 percent in the tensile strength were measured when one to three percent glass or asbestos fibers were added to the soil and cement mixtures. Increases in compressive strength also resulted from the addition of fibers. The authors note that increased tensile strength resulting from the addition of fibers may have application in pavement structures or in dam cores where tensile stresses might otherwise cause cracking.

SOIL-REINFORCEMENT INTERACTIONS

Introduction

The question of the sliding resistance between soil backfill and tensile reinforcing elements is of greatest importance. Lee (1978) considered it the "most fundamentally important, the most critical and the least understood aspect of reinforced earth in any form". Specific questions raised by Lee were :

- How it is developed ;
- How to measure it ;
- How to quantify or express it ;
- How it varies with certain design factors ;
- How to apply it in practical designs.

Unless shear stresses can be transferred to the reinforcements slip will develop between the soil and reinforcing. On the other hand if the sliding resistance exceeds the tensile strength of the reinforcement, then reinforcement breakage will occur before slip ; a condition that could lead to a catastrophic failure.

The friction mobilization along the reinforcements depends at each point on the local relative displacement of the reinforcement with respect to the soil, and, therefore, it is a function of the overall deformations of the reinforced earth mass. This leads to study of the distribution of the tensile forces along the reinforcements and the evolution of the maximum tensile forces line in the different reinforced earth structures, as previously discussed in this report.

Sliding resistance between soil and reinforcements

The complex mechanism of the soil-reinforcement friction mobilization and the relative influences of the different factors affecting the value of the soil-reinforcement friction coefficient have been the object of full scale experiments, laboratory model studies, and conventional laboratory tests.

Several types of tests have been used to measure the soil-reinforcement friction coefficient in reinforced earth. They include (Fig. 21) :

1) Direct shear test in the direct shear box

In this test the normal stress and the shear stress exerted on the strip surface are considered to be accurately known, and, therefore, this test is expected to give a quite accurate value of the soil-reinforcement friction coefficient. However, this test represents the two-dimensional case of an infinite reinforcement sheet, and it does not represent the different phenomena involved in the complex three-dimensional mechanism of the soil-reinforcement interaction in actual reinforced earth structures.

2) Strip pull-out tests from model, prototype and full scale structures and embankments

This test consists of extracting the reinforcement from the reinforced earth mass and recording the pull-out force-displacement curve. It is an adequate representation of the real phenomenon which actually occurs in reinforced earth structures and gives values of the soil-reinforcement friction coefficient which are used in the current design of structures with respect to failure by lack of adherence. However, because of the dilatancy effect which develops in the granular mass and modifies the distribution of the vertical stresses in the vicinity of the reinforcement, the normal stress exerted on the reinforcement surface is actually unknown. Therefore this test gives only an average apparent friction coefficient f^* which is defined by the ratio :

$$f^* = \frac{\tau}{\gamma H} = \frac{T}{2bL\gamma H} \quad \text{.....} \quad (8)$$

where :

. τ is the average shear stress along the reinforcement ;

. γH is the over burden stress ;

. T is the applied pull-out force ;

. L is the length of the reinforcement ;

. b is the strip width.

3) Strip pull-out tests in the shear box

This test is analogous to the classical pull-out tests mentioned above, however, it enables a better control of the friction mobilization than the classical pullout tests where the reinforcements are embedded in a large mass.

4) Strip pullout from a rigid rotating model wall

This test has been adopted by Hausmann and Lee (1978) in order to determine the actual mobilization of the soil-reinforcement friction in the reinforced earth walls. The authors pointed out the fact that the soil-reinforcement friction mobilization is a function of the overall deformation of the reinforced earth mass. Accordingly a rigid model wall was rotated about a knife edge support attached to the base of the box containing the sand backfill, and the curve of the applied moment-versus the rotation angle was recorded. Although a rigid wall rather than a typical element wall was used in the tests, the authors indicated that the results are believed to be relevant to reinforced earth design, because the overall deformation pattern reported for experimental walls is essentially that of a rotation rather than translation. This test gives the overall safety factor of the structure with respect to failure by reinforcement slippage.

5) Strip pull-out test during vibration-model scale

Available information on the factors affecting the values of the apparent friction coefficient f^* has been reviewed and summarized by Schlosser and Elias (1978) and by McKittrick (1978). The data provide a clear indication that both peak and residual values of f^* are functions of the soil density, effective normal stress, geometrical factors and surface roughness. Although

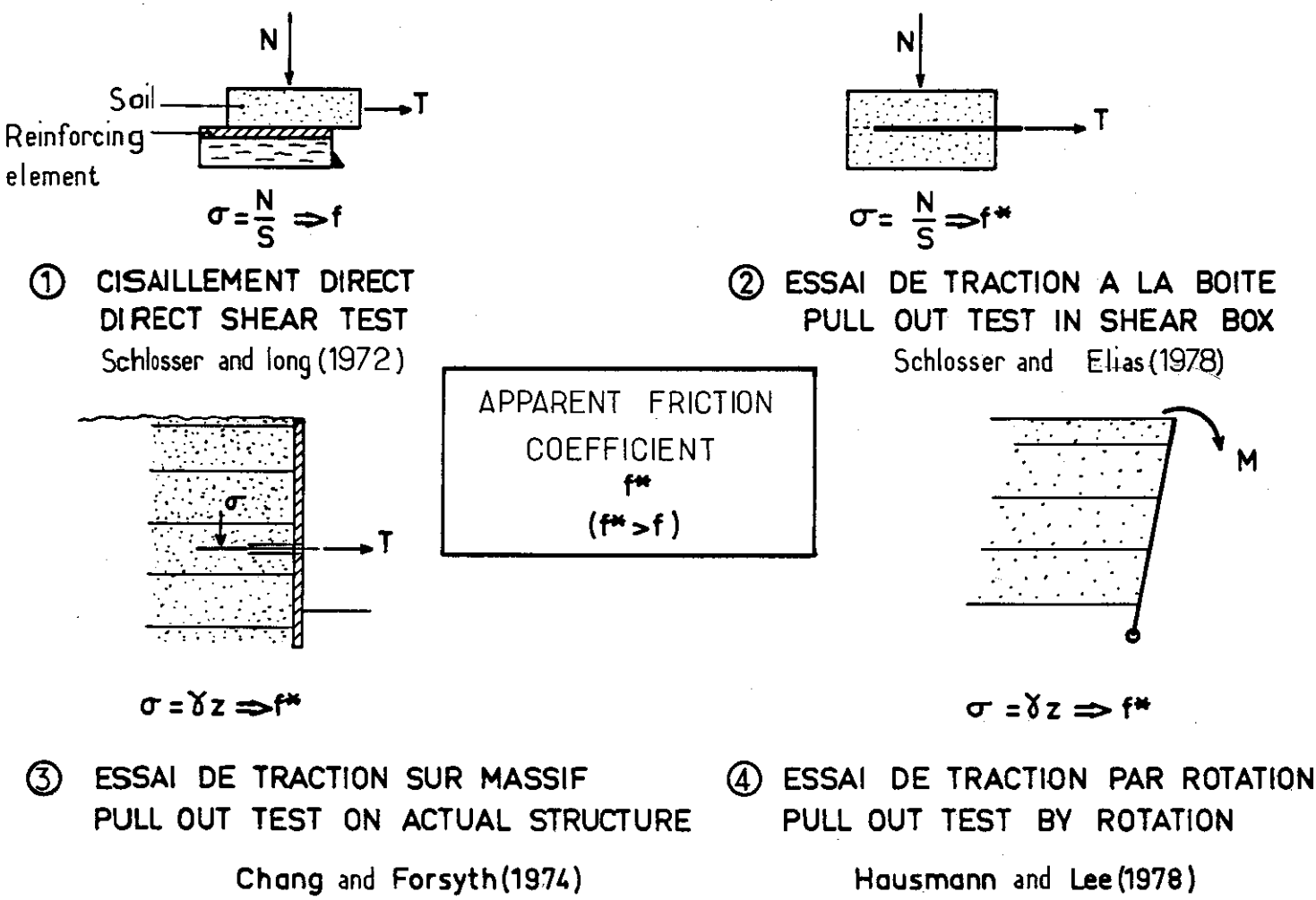


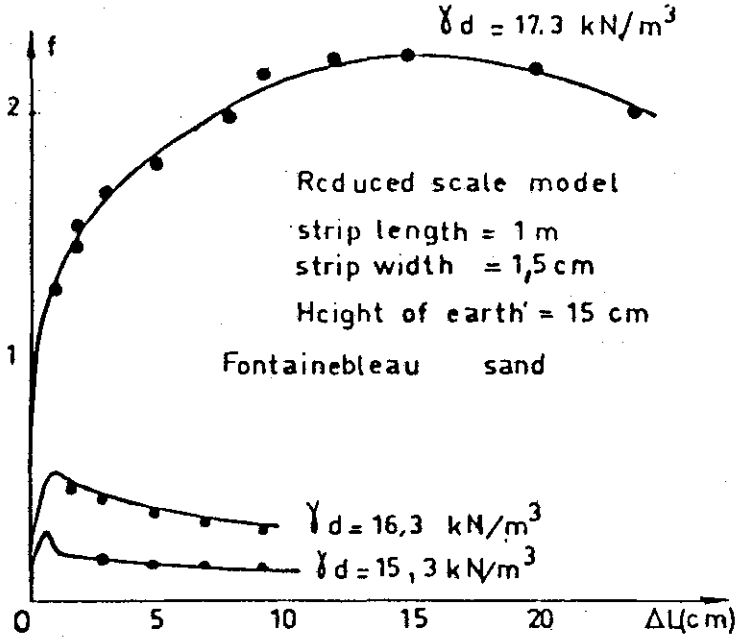
Fig. 21 - Measurement of soil-reinforcement friction

the available data on f* may appear inconsistent and conflicting in many respects, the authors present evidence to show that the factors affecting f* are qualitatively consistent with established soil behaviour theory.

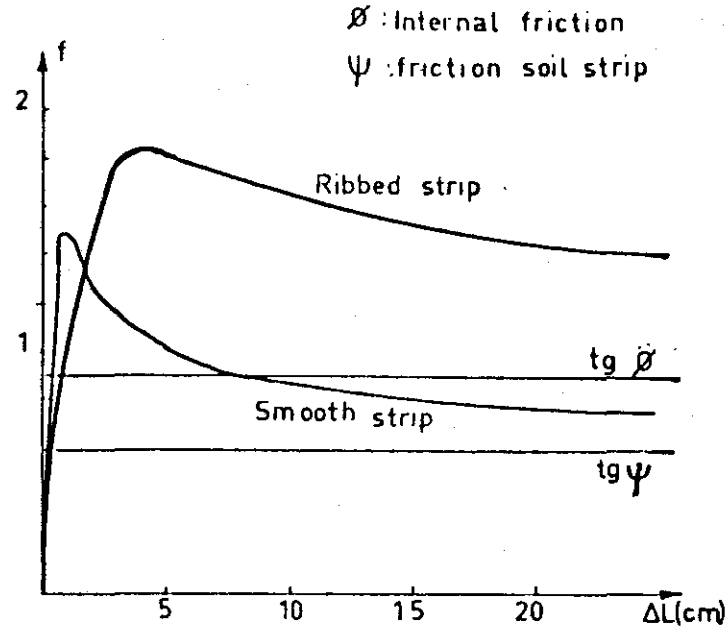
Among the most important findings are :

1- Values of f* deduced from the results of strips pull-out tests are usually significantly greater than values obtained from direct shear tests. This is due, at least partly, to the restriction of volume change (suppressed dilatancy) associated with the pull-out test.

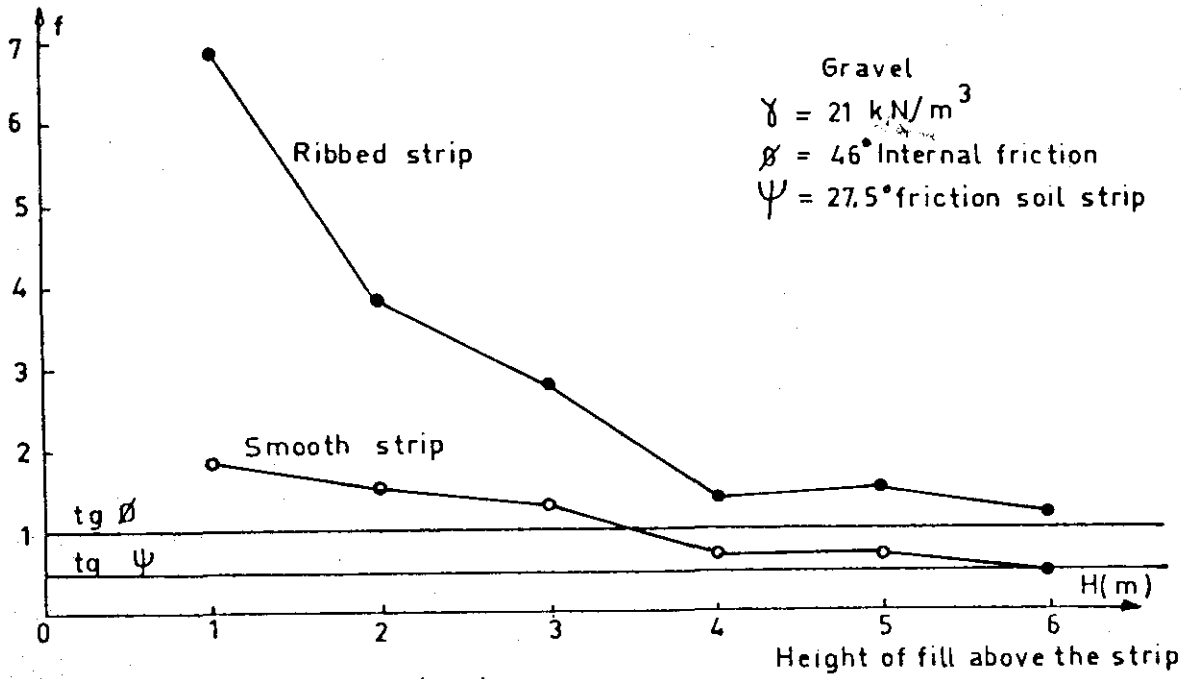
2- Increased soil density gives increased f* for two reasons :



a) Pull out test of a strip buried in sand influence of the density



b) Influence of the nature of the surface in pull out tests of strips



c) Influence of the overburden pressure in pull out tests of strips

Fig. 22 - Factors affecting the value of the apparent coefficient of friction (Alimi et al., 1977)

a) soil friction angle increases with density increase ;

b) denser soils are more dilatant.

Fig. 22a illustrates the results obtained by Alimi et al. (1977) from pull-out tests on strips embedded in models of sand embankments of low, average and high densities. In the case of low and average densities the peak value of the displacement-pull-out force curve is noticeably accentuated, it is obtained for a small displacement, and the residual value is about 50 % of the peak value or less. Inverse phenomena occur in the case of high density.

3- Reinforcements with ribs, deformations, or otherwise roughened surfaces give grater values of f* than smooth strips. Soil-to-soil properties control behaviour rather than soil-to-reinforcement values. The effect of surface roughness is illustrated in Fig. 22b.

4- The value of f* decreases with increased overburden pressure. This effect is particularly remarkable in the case of ribbed strips. Examples are shown in Fig. 22c. Reasons for this are :

a) The main reason is that increased overburden pressure suppresses dilatancy. As dilatancy is essentially responsible for the high values of f*, the decrease of the dilatancy leads to values of f* close to tg ϕ for the ribbed strips and to tg ψ for the smooth strips (ϕ and ψ are respectively the internal friction angle of the sand and the soil-reinforcement friction angle determined by a direct shear test).

b) In addition, most granular soils exhibit curved failure envelopes, with decreasing friction angles associated with increasing effective stress.

The above discussion is restricted to a consideration of granular soils of all types generally found acceptable for use in reinforced earth structures. The influence of soil fines (particles < 15μ) on the soil-reinforcement friction is discussed in session n° 3.

Review of Conference Papers

A number of the papers included in Session 1 deal with questions of forces in reinforcements, shear stresses between reinforcements and soil, and the influences of various factors on these forces and stresses.

Guilloux, Schlosser, and Long describe shearbox and pull-out tests on laboratory models with smooth reinforcement strips.

A group effect was observed for uniformly spaced reinforcements as shown in Fig. 23. In this figure d/b is the ratio between spacing and strip width and C_{eff} is the ratio of group capacity to single strip capacity times the number of strips.

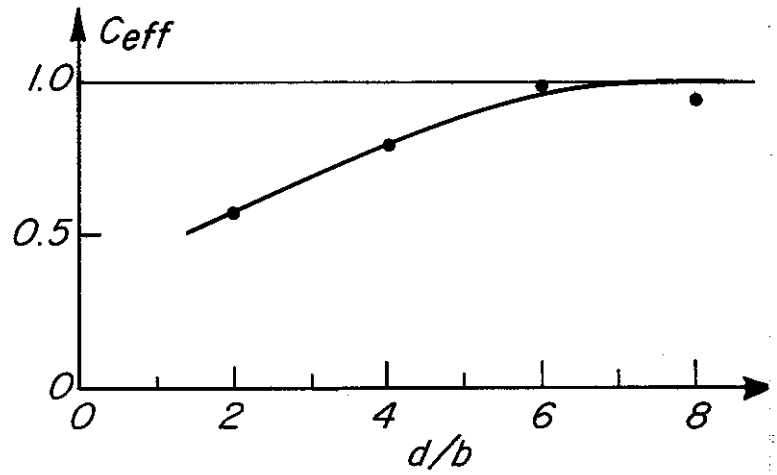


Fig. 23 - Coefficient d'efficacité d'un groupe d'armatures (Guilloux, Schlosser, Long)

Some pullout tests were done by Guilloux et al. using instrumented strips. The great influence of sand density on tensile forces and their distribution is shown in Fig. 24. It is hypothesized that in loose sand the reinforcements act as rigid elements ; whereas, in dense sand they act as deformable elements and the mobilized friction is more important at the beginning of the reinforcement than near its end.

The stress-displacement relationships for sand-sand shearing in the shear box and sand-reinforcement shearing in pullout tests were quite different. It was concluded that with smooth strips the contact planes are parallel to strip surfaces. As the volume variation is very small, dilatancy effects cannot be mobilized.

To explain the high values of the apparent friction coefficient f* obtained with the compacted soils, Guilloux et al. consider the hypothesis that the normal stress exerted on a reinforcement increases during pullout because the dilatancy is restrained. In order to demonstrate this hypothesis shear tests were done under constant volume conditions on highly compacted sand. These tests represent an extreme case and are expected to give the maximum increase of the normal stress which can be obtained. Very large increases of the normal stress were observed, and the stress-displacement curves showed an accentuated peak. On the other hand the sand-reinforcement shear tests gave stress-displacement curves without maxima and with a relatively small increase of the normal stress.

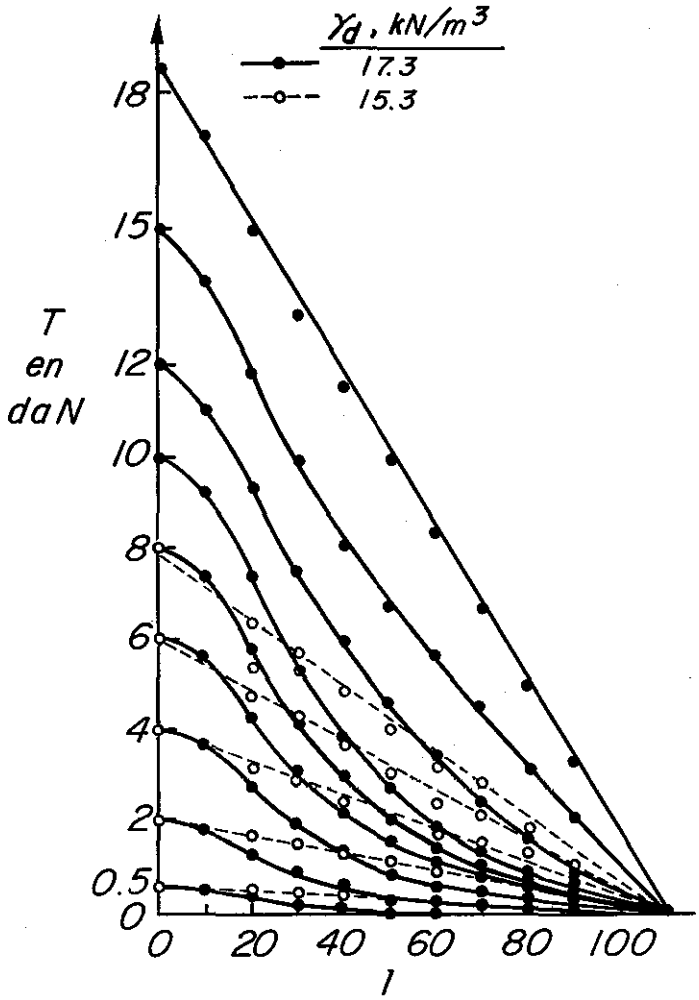


Fig. 24 - Répartition des tractions dans une armature lors d'essais de traction à différentes densités

The results of these tests are summarized in Fig. 25. They account for observed differences between the smooth reinforcements in which soil-to-reinforcement friction controls, and the ribbed reinforcements, in which soil-to-soil friction controls.

Holm and Bergdahl studied the total overturning stability of a fabric reinforced earth retaining wall model. The wall, 180 cm long, was made of four wooden planks placed upon each other in a frame of U shaped steel beams to a total height of 672 mm. The backfill used was a uniform medium fine dry sand of a uniform density of 1.59 t/m^3 and an internal friction angle of 40°.

At the lower edge of the wall an horizontal axis was mounted. The wall was rotated around this axis, the movements in the backfill were studied and measured, and the applied turning force was recorded versus the rotation angle of the wall. The tests results were consistent with the results of similar tests Hausmann and Lee (1978).

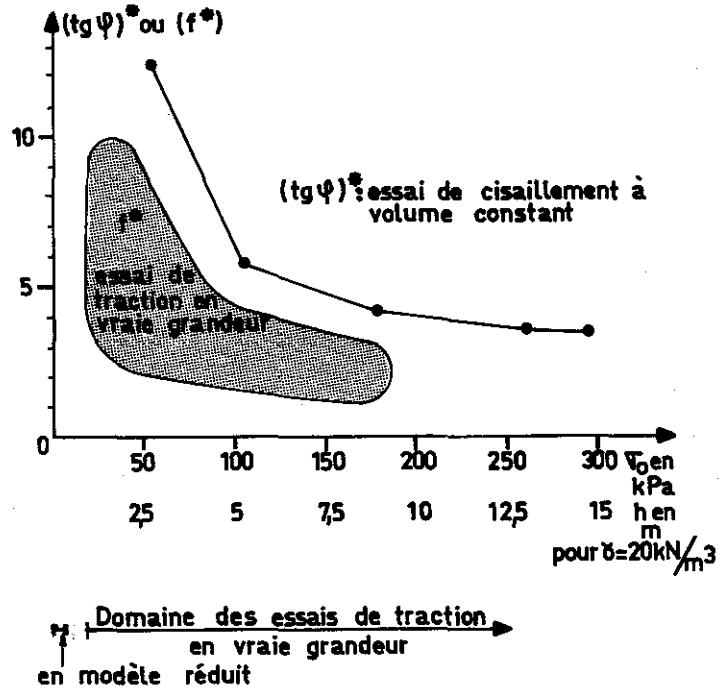


Fig. 25 - Variations du coefficient de frottement apparent en fonction de la contrainte initiale (Guilloux, Schlosser, and Long)

By comparing the measured anchor force at the wall without a fabric with the anchor forces at the wall with fabric reinforcements, the total overturning stability of the wall was calculated. It was found out that in order to get a safety factor of 1.5 the necessary length of fabric is about 40 % of the total height of the wall.

Based on the agreement between measured and calculated forces and movements a design method was proposed for fabric reinforced earth retaining walls.

The results of different types of laboratory tests to evaluate friction between soil and tensile reinforcement are reported by Shen, Mitchell, DeNatale, and Romstad. Direct shear tests, pullout tests, and tests on model walls with both rigid and flexible facings were made using a uniform, fine, river sand and 0.1 mm thich polished steel reinforcing strips of various lengths and widths. Tests on lacquer coated steel were done to account for the lacquer coatings used over strips to which strain gages had been attached.

The authors note that the "apparent angle of skin friction" will be affected not only by test method but also by soil arching, dilation, boundary conditions, soil compaction, and the length, width, and undulations in the strips. The angles of skin friction determined by the different test types are compared in Table 2.

Material	Direct Shear	Pullout	Model
Uncoated		12.5° (1.27 cm wide)	
Steel		14.7°	
Against	21°	(2.54 cm wide)	
Sand		15.4° (5.08 cm wide)	
Coated		20.3° (2.54 cm wide)	21.1° (Rigid Facing)
Steel		Variation	21.9°
Against		with Normal	(Flexible Facing)
Sand		Pressure from 19° to 26°	

Table 2 - Summary of angle of skin friction values measured by SHEN et al.

General conclusions given by Shen et al. are :

1- The skin friction angle determined by direct shear tests is greater than that obtained in pullout tests. This is a somewhat surprising finding in relation to previous studies wherein the reverse result was obtained.

2- Pull-out test results agreed with model test results. This suggests that pull-out tests, and not direct shear tests, should be used to determine soil-strip interaction.

3- In-situ frictional behavior at the soil-strip interface is influenced by many factors. Of particular importance in this regard is that undulations of a strip within the backfill may be responsible for an increase in the friction angle of several degrees.

The value of the friction coefficient mobilized along reinforcements in contact with the fill material was studied by Schlosser and Guilloux. This paper extends that of Schlosser and Elias (1978) published previously. The apparent friction coefficient f^* is taken as the shear stress τ on the reinforcement divided by the vertical stress σ_v, which is assumed to equal the overburden stress γH ; i.e. :

$$f^* = \tau/(\gamma H) \quad \text{.....} \quad (9)$$

Schlosser and Guilloux studied factors influencing f^* and recommend values for f^* to be used whenever the fill material conforms to the usual reinforced earth specification that no more than 15 percent by weight be finer than 15 μm. As the reinforcements actually used were rigid with respect to the soil, the friction mobilized in the resistant zone can be considered approximately uniform. The factors found to govern f^* are the same as listed previously ; i.e., soil density, nature of reinforcement surface, nature of fill material, value of normal stress acting on the reinforcements.

For the design of ribbed reinforcements in practice it is generally assumed that f^* is constant along the reinforcement. However, f^* varies with depth from high values at the top of the wall to a value of $\tan \phi$ at a depth of 6 m as shown in Fig. 26. From experience it has been found that f^*_o, the value at the top of the wall, depends on grain size distribution according to :

$$f^*_o = 1.2 + \log CU \quad \text{.....} \quad (10)$$

where :

$CU = D_{60}/D_{10}$ = coefficient of uniformity.

The results support further the conclusion that ribbed reinforcements mobilize the dilatancy resistance of the soil and full generation of soil-soil friction.

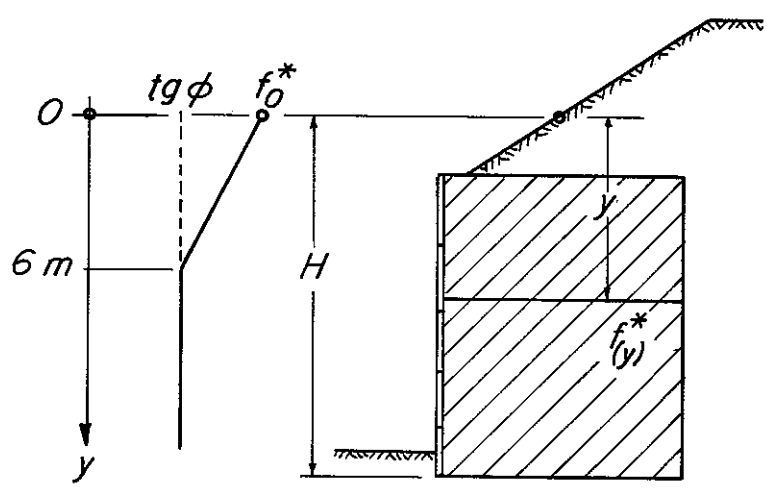


Fig. 26 - Variation of friction coefficient with depth (Schlosser and Guilloux)

Concern over corrosion effects on the long-term durability of tensile reinforcements has led to investigation of the suitability of other materials. Ingold and Templeman note that some form of polymer is about the only alternative material at the present time. Polymer nets and fabrics suffer the disadvantages of high extensibility and low strength as well as undesirable creep characteristics in many cases.

Ingold and Templeman did pullout tests of five types of reinforcement : a woven fabric (Terram RF/12) ; two net structures (Netlon 1168 and FBM 5) ; sand coated mild steel, 0.8 mm thick ; and plain mild steel, 0.8 mm thick. The sand used was coarse to medium with some fine gravel. Results in the form of shear stress vs. normal stress are shown in Fig. 27 and apparent angle of bond stresses vs. normal stress in Fig. 28.

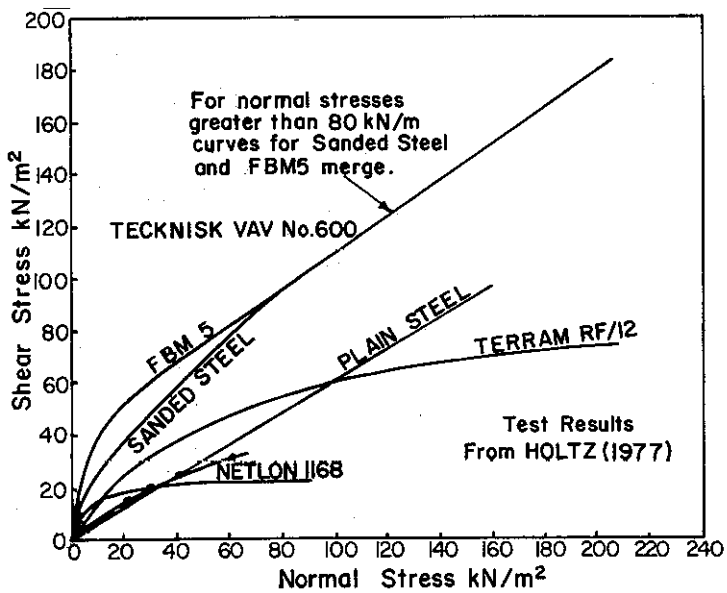


Fig. 27 - Interface shear resistance for several reinforcing materials (Ingold and Templeman)

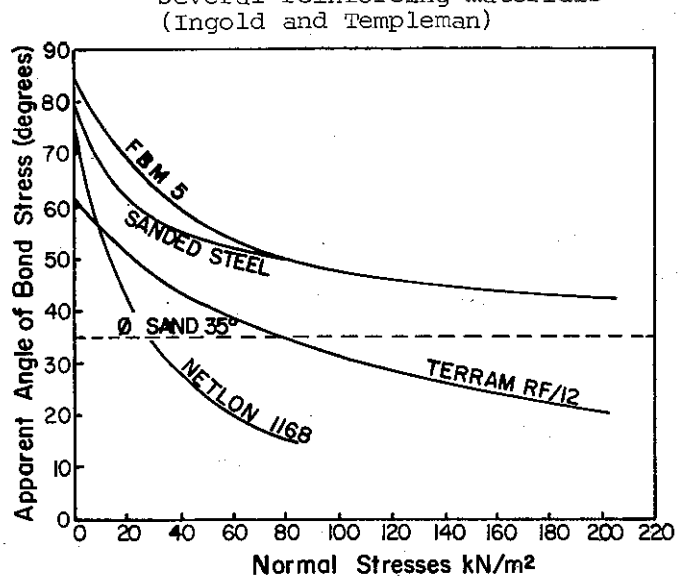


Fig. 28 - Apparent friction angles between sand and several reinforcing materials (Ingold and Templeman)

It may be seen that the apparent bond angle decreases significantly with increases in normal stress. The very large values of bond angle for low normal stresses suggest that some mechanism over and above simple dilatancy is active. Perhaps the inclusion of soil grains within the fabric openings could be a factor. Shear box tests were also done, and in general, different values of resistance (usually lower, at least at low normal stresses) were obtained.

Two types of tests were used by Delmas, Gourc, and Giroud to study soil-fabric interactions in the laboratory :

1- Friction tests where one soil layer is displaced with respect to the other one and the fabric is adherent to the supporting soil ;

2- Pull-out tests where the fabric is displaced between two soil layers and where each soil layer is fixed with respect to the other one.

These tests were done in a shear box of large dimensions (0.40 x 0.25 x 0.20 m).

Many types of supporting soils (placed in the lower part of the box) and of covering soils (placed in the upper part of the box) were used as well as many fabrics. The different results of the friction tests indicate the influences of the grain size distribution and the angularity of each soil layer, the nature of the fabric, the normal stress, and the role of the deformation of the fabric which is adherent to the grains of the soil.

Fig. 29 shows that the apparent friction angle increases to a constant value with increasing normal stress. This is attributed by the authors to the fact that the friction is controlled by interactions between the geotextile and individual soil particles. When the fabric is pushed by the soil grains it deflects, and the frictional contact area increases. When the value of normal stress reaches some value

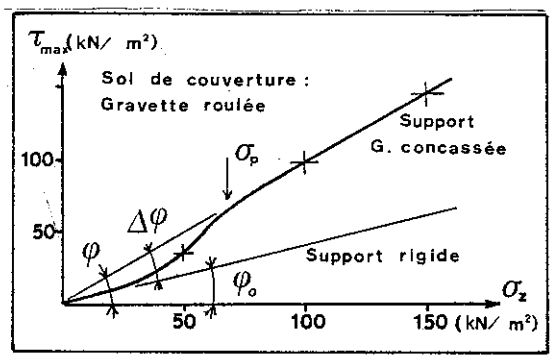


Fig. 29 - Courbe donnant la contrainte tangentielle en fonction de la contrainte normale dans un essai de frottement sur géotextile NT 400 (Delmas, Gourc, and Giroud)

high enough, the geotextile goes around all grains and increasing the stress has no further effect on this friction angle.

The pull-out tests, done for only one type of fabric, show that the fabric maintains a relatively plane form and that the results are close to those obtained with a smooth support in the friction tests. Reinforcement strip length is important, because for a long strip a large displacement is necessary at the pulled end to mobilize friction at the free end.

A theoretical approach to the pullout tests, which takes into account the deformations of the fabric and of the soil and is based on the theory of elasticity, is presented by Delmas et al. and is in partial agreement with the experimental results.

Very complete details of a series of model tests designed to evaluate the comparative behavior of aluminum and non-woven polypropylene fiber fabric (Mirafi 140) strips as reinforcements are reported by Tumay, Antonini, and Arman. The experimental arrangement, as shown in their Fig. 6, enabled evaluation of both pressures on the model wall and the pullout resistance of the reinforcements. Length of reinforcements and relative density of the sand backfill were included as additional variables in this study.

Although the elongation of the fabrics was much greater than that of the aluminum for a given tensile stress, the apparent friction between the sand and fabric was about three times greater than for the aluminum and fabric. It is probable that the rough surface texture of the fabric develops an interlocking with the sand. Increase in relative density of the sand resulted in a substantial increase in the resistance of the fabric ; whereas, little improvement was observed in the case of the aluminum strips. Tests to failure showed that construction of a safe wall to a given height could be accomplished using shorter lengths of fabric reinforcement than aluminum reinforcement.

As used by Tumay et al. reinforcement concentration referred to reinforcement spacing in the horizontal plane divided by the length of reinforcement. In their test program only the length was varied. Although the results of this study are encouraging as regards the potential for use of fabrics as reinforcements, it is suggested that potential creep effects which were not referred to by the authors, be given attention in subsequent studies.

Soils suitable for earthwork construction using tensile reinforcements have been restricted almost exclusively to granular materials with upper bounds on the percentage of fines and plasticity. Table 3 is an illustration of a typical specification. Fine-grained soil backfills present difficulties associated with poor drainage, slow development of effective stress transfer, low friction angles, and time-dependent creep and stress relaxation phenomena. These problems are discussed by ASCE (1978), Lee (1978), McKittrick (1978) and elsewhere. Nonetheless, a study of bond failure of reinforced clay in triaxial compression was carried out by Ingold as discussed previously.

Sieve Size	Percent Passing
6"	100
3"	75-100
No. 200	0-25
and P.I. < 6	
Or, if percent passing No. 200 is greater than 25 percent, and percent finer than 15μ is less than 15 percent, material is acceptable if : ø = 30°, as determined by AASHTO T-236 P.I. < 6	

Table 3 - Minimum Specifications for Selected Backfill

While recognizing these difficulties and the inherently poor performance anticipated with tension-reinforced fine-grained soils, it is significant that Hashimoto (this Conference, Session 5) was successful in the use of a loam for backfill of a temporary reinforced earth wall in Japan. The reinforced earth technique was chosen for reasons of time and space available. Construction was done in the usual way except that gravel drains were used behind the wall facing and horizontal drains were included.

Measurements on the completed structure showed that the line of maximum stresses was similar to the one ordinarily observed. The measured friction coefficient between soil and reinforcements was in the range of 0.38 to 0.41. For design f* was taken as 0.24. The settlement measured was 0.91 m ; whereas 0.74 m was predicted. The structure was subjected to a small earthquake four months after its completion. No adverse affects were noted.

Further consideration of the utilization of poorer quality soils has been made by Snaith, Bell, and Dubois, who point out that embankment construction costs could be reduced through use of low grade fills or steepened side slopes, and embankments could be constructed over poor foundation

soils if reinforcements could be used to strengthen the materials. To this end the applicability of fabric inclusions has been studied. They note that fabric requirements for road and railway embankments are between those for vertical walls, where low elongation, high modulus reinforcing strips are required, and those for membranes over soft ground, where high elongation materials are useful.

The functions and requirements of fabrics in embankments are listed by Snaith et al. as :

(1) : Strengthening against rotational shear ;

(2) : Anchorage, dependent on fabric to soil adhesion ;

(3) : Interlayer friction ;

(4) : Cross plane permeability ;

(5) : High in-plane permeability for drainage ;

(6) : Separation between fines and granular layers.

Shear box tests on clay specimens with fabric inclusions were used to evaluate the reinforcing effects. As would be expected, the fabric gave a reinforcing effect when loaded in tension, and the greater the deformation, the greater the reinforcing effect. When the plane of the fabric coincides with the shear plane, a reduction in shear resistance can be obtained if the adhesion between soil and fabric is less than the strength of the soil. This should be considered when selecting shear surface geometry for analysis of embankment stability. Furthermore, soil-fabric adhesion and fabric-fabric friction for woven fabrics was observed to depend on the slip direction relative to the weft and warp direction.

ANALYSIS METHODS

Several analysis procedures for use in the design and stability evaluation of reinforced structures have been developed.

As noted previously, the classical Rankine and Coulomb's theories provided the first basis for computation of lateral pressures to be resisted. Fig. 30 illustrates the L.C.P.C. (1974) basis for static design of a reinforced earth wall. The zone of reinforced fill ABCD behind the wall is assumed to behave as a cohesive composite body. These first design methods for reinforced earth walls were elaborated on the basis of the first observations on laboratory reduced scale models of walls, uniformly reinforced, built up to failure at L.C.P.C.

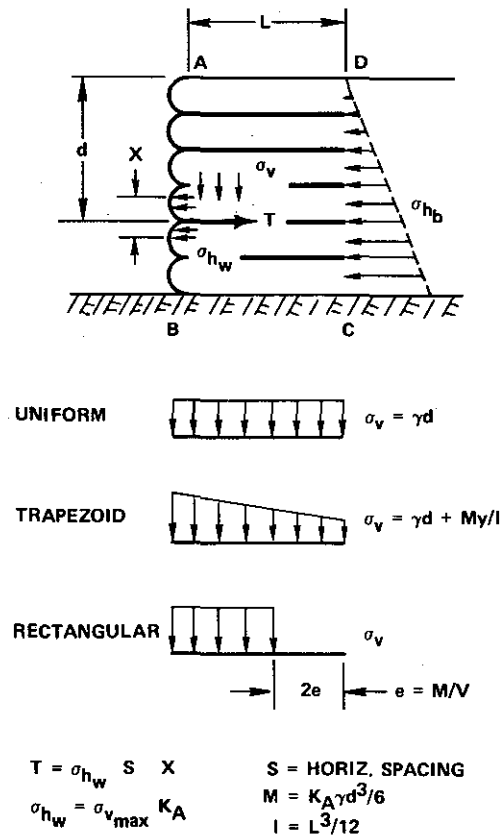


Fig. 30 - Original LCPC basis for static design of a reinforced earth wall

The first two-dimensional laboratory models developed at L.C.P.C. in 1969 (Schlosser and Vidal, 1969) showed (Fig. 31) that the critical height (H_f) of the wall at failure by reinforcement breakage is directly proportional to the tensile resistance R_T of the reinforcement and to the inverse ratios of their vertical spacing ΔH and of the backfill density γ . These results led to the following expression :

$$H_f = H_o + \frac{R_T}{K \cdot \gamma \cdot \Delta H} \quad \text{.....} \quad (11)$$

where :

. H_o is the critical height of an unreinforced wall supported by the facing itself ;

. K is the inverse of the coefficient of proportionality.

By substituting $K = K_a$, Rankine and Coulomb's classical solutions were obtainned. The experimental results of tests on the two-dimensional models agreed fairly

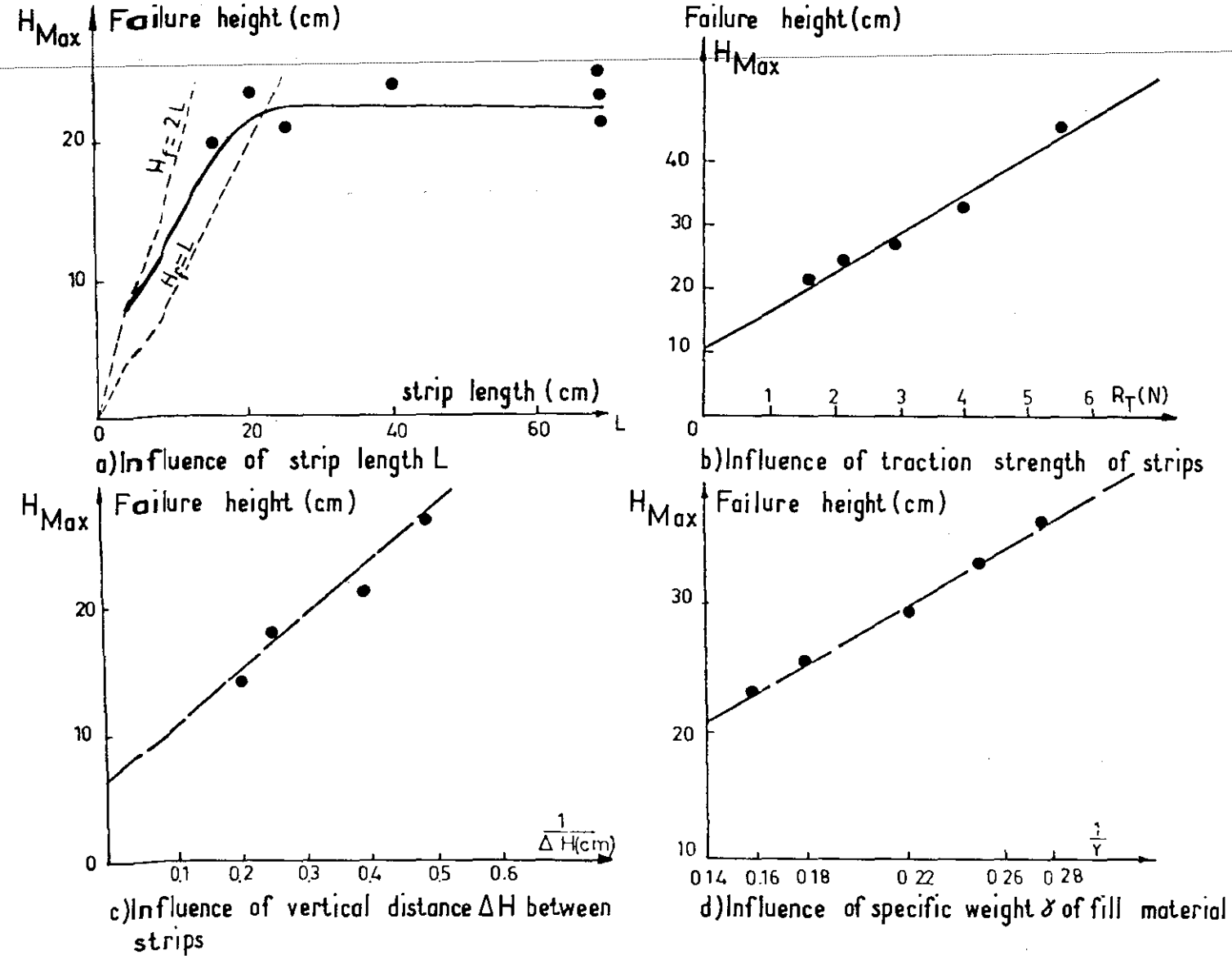


Fig. 31 - Influence of some factors on failure height of bidimensionnal model tests of Reinforced Earth walls (L.C.P.C., 1972)

well with equation (11). Consequently, a first design method was based on it.

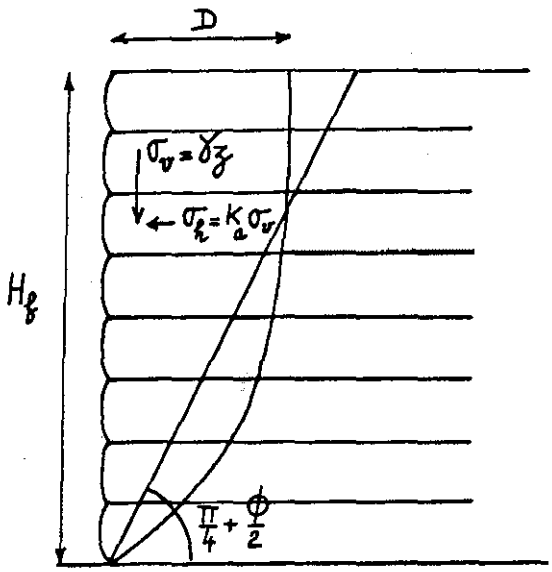
However, although three-dimensional model studies at U.C.L.A. (Lee et al., 1973) confirmed that Coulomb's theory could be considered an adequate approach to the design of reinforced earth retaining walls, Bacot in 1970 and investigators at L.C.P.C. in 1973 (Binquet and Carlier, 1973) showed that Coulomb's theory leads to a large underestimation of the wall critical height. The experimental critical height was found to be approximately double the theoretical value. They also pointed out that the observed failure surface was essentially different from the classical Coulomb's failure plane.

In a paper presented in this conference, Soydemir and Espinosa describe reinforced earth wall model tests, wherein the vertical spacing of reinforcements was varied while the horizontal spacing was maintained constant at 6 in (152,4 mm). The model walls were built up to failure which occured either by reinforcement slippage or by reinforcement breakage. Their findings were generally similar to those previously obtained by Lee et al. (1973), and general agreement was found between the experimental results and Coulomb's force model. For all failures, a failure wedge was clearly identified by means of placing very thin layers of colored sand during backfilling. Failure wedges were characterised by a failure plane at 45° + (ø/2) at the bases becoming slightly more vertical with increasing height. Through video tape recordings, it was observed that the failure wedge deve-

loped progressively, initiating at the lower most region similar to local yielding and gradually moving upward.

The analysis of the observations on the failure mechanism in the different model studies of reinforced earth retaining walls, illustrated in Fig. 32, shows a fundamental relationship between the observed shape of the failure surface passing through the breakage points of the model reinforcement strips and the critical height of the model wall at failure.

As previously indicated in this report the full scale experiments on actual reinforced earth structures confirmed that the locus of the maximum tensile forces is basically different from Coulomb's failure plane. They also showed that the coefficient K characterising the state of horizontal stresses within the backfill material varies with depth from K_o at the top of the wall to a value which is less than K_a at the lower part of the wall. The high values of K at the top of the wall are mainly due to the effects of compaction and to the presence of the reinforcements which restrain the lateral deformations and maintain the equivalent K_o state of stresses as illustrated in Fig. 11. Fig. 33 illustrates the variations of the ratio K/K_a as a function of the height of the embankment above



$$H_f = H_o + \frac{R_T}{\Delta H . \gamma . k} \quad \text{(LCPC 1971)}$$

$$k = K_a \quad (\text{Rankine or Coulomb})$$

Failure by reinforcement breakage

Fig. 32 - Models of reinforced earth walls

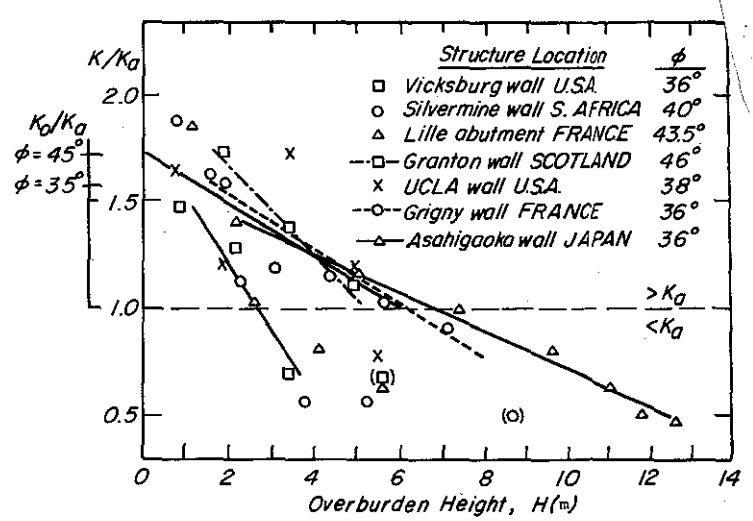


Fig. 33 - Earth pressure variation in reinforced earth structure (Schlosser, 1978)

the considered reinforcement strip in seven experimental structures. The full line indicates the variation of the ratio K/K_a adopted by the French specifications for the design of reinforced earth walls.

The large difference between classical limit analysis methods, generally adopted for the design of retaining walls, and the observations on reinforced earth model walls and on actual structures was anlysed by

	Modèl	$\frac{H_f}{H_{Rankine}}$	$\frac{D}{D_{Rankine}}$
LCPC (1969) Long - Schlosser	2-Dimens.	1,2	1
Bacot (1970)	3-Dimens.	≃ 2,0	≃ 0,60
UCLA (1972) Adams - Lee - Vagneron.	3-Dimens.	≃ 1,0	≃ 1,0
LCPC (1973) Binquet	3-Dimens.	1,9	0,60

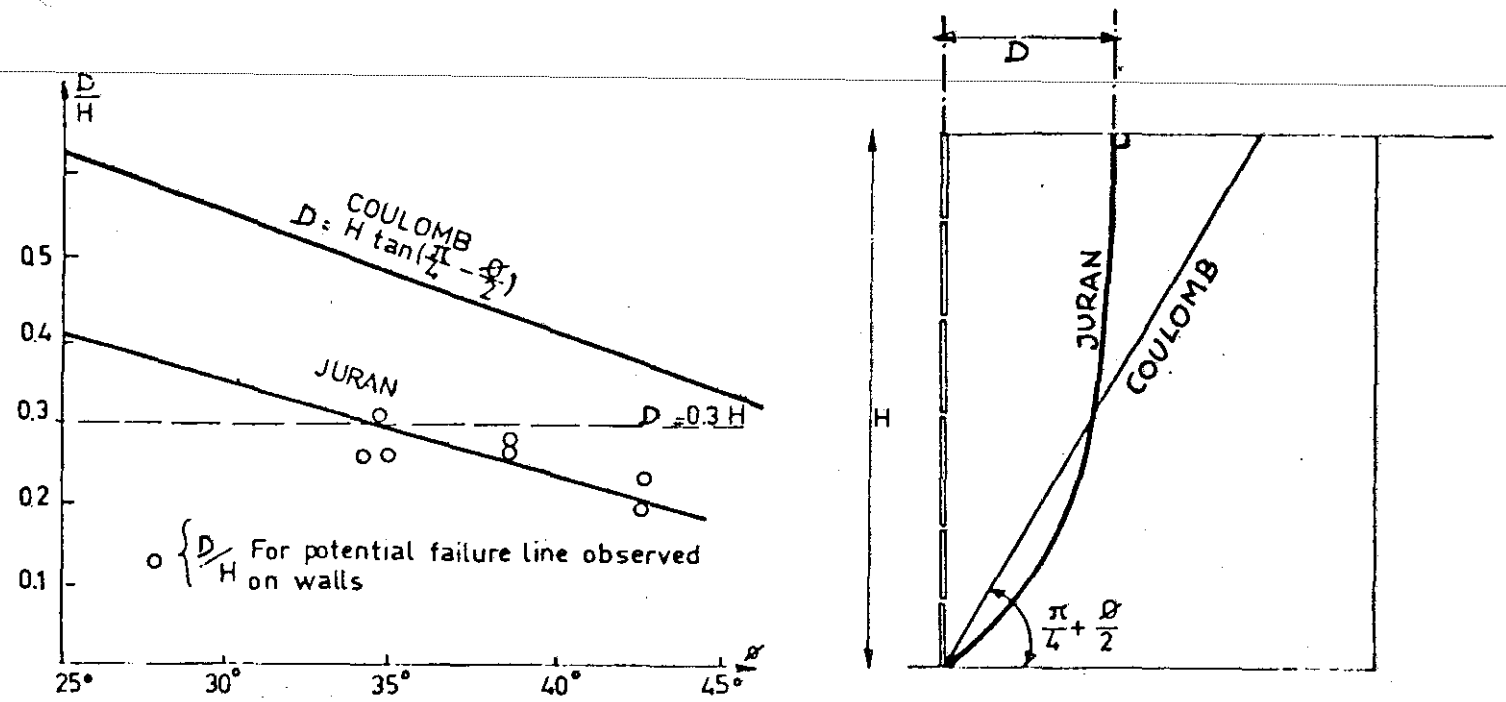


Fig. 34a - Theoretical determination of maximum tensile force line (Juran and Schlosser, 1978)

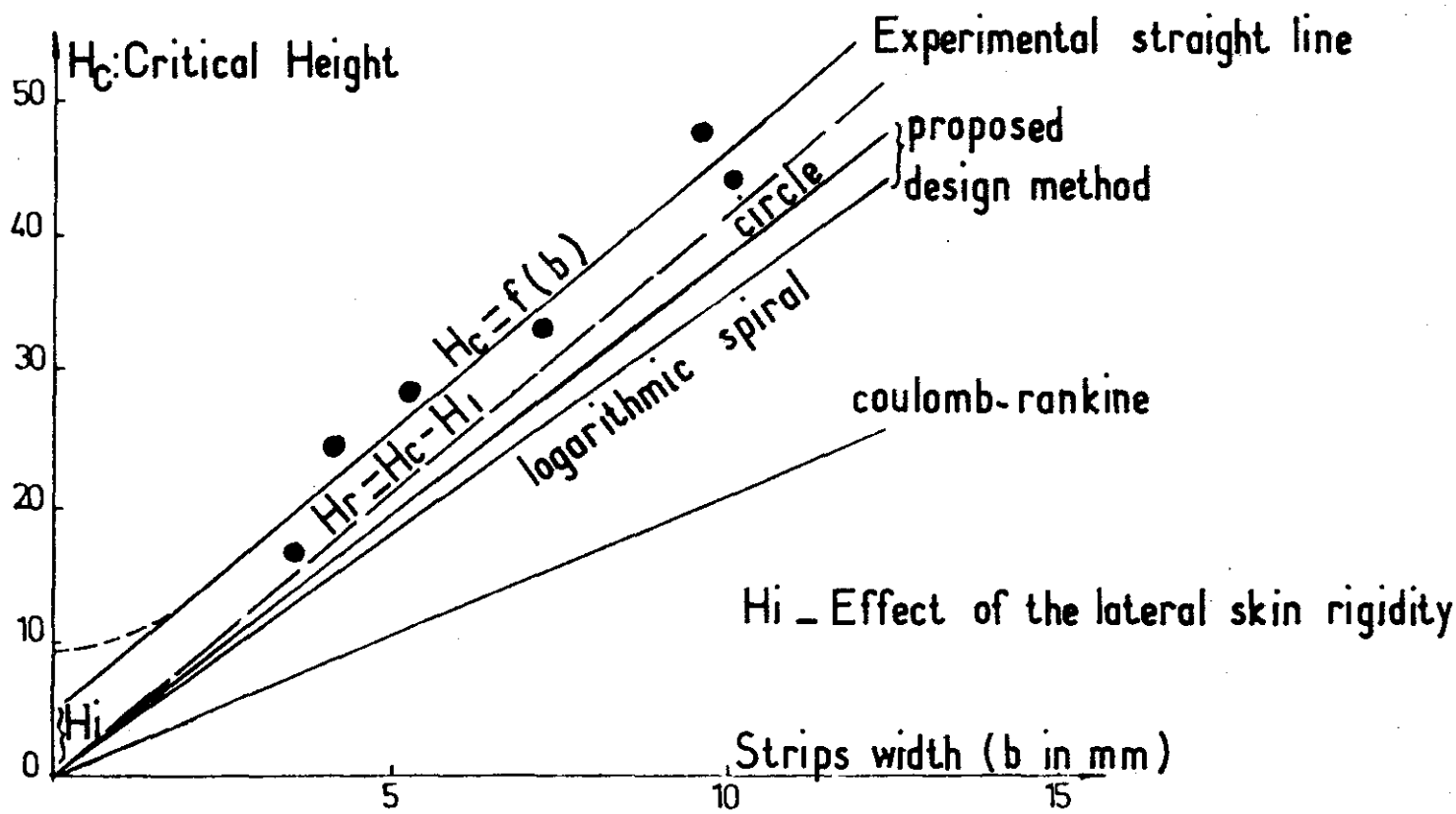


Fig. 34b - Experimental and theoretical critical heights of model walls (Juran and Schlosser, 1978)

Juran (1977). Juran indicated that the reinforcements restrain the lateral deformations of the active zone which are necessary to generate soil resistance to shearing. Therefore, the active zone is rather in an elastic state of stresses which is incompatible with Rankine's theory. This restraining effect is particularly important at the upper portion of the active zone where the soil is actually maintained at a K_o state of stresses. He concluded that the failure mechanism involves a rotation of a quasi-rigid block limited by an extremely thin zone where the soil resistance to shearing is entirely mobilized. This thin failure zone, separating the rotating active zone from the resistant zone along the locus of the maximum tensile forces must be orthogonal to the embankment free surface to comply with the kinematical conditions of zero lateral displacement at the top of the wall. For theoretical analysis, this thin failure zone was considered to be either a logarithmic spiral or a circle. On the basis of this failure mechanism, Juran developed a limit analysis method for the design of reinforced earth walls and bridge abutments (Juran and Schlosser, 1978). As illustrated in Fig. 34, this limit analysis design method gives a reasonably accurate evaluation of the model wall critical height and of the geometrical shape of the locus of the maximum tensile forces in actual structures.

The application of this limit analysis method to the design of actual structures assumes that the rotation of the active zone in the structure is large enough to generate the soil resistance to shearing along the potential sliding surface passing through the maximum tensile forces line. However, this assumption is rather restrictive and should be considered with caution. Alternatively, in order to analyse the stress and deformation fields which develop in reinforced earth structures under normal working conditions which are different from those prevailing at failure elastic methods of analysis have been used.

Finite element methods have been useful for the study of both model and prototype walls. Both composite body and discrete material approaches are being used. In a discrete representation (Al Hussaini and Johnson, 1978), the reinforced system is treated as a heterogeneous body ; the soil and each reinforcing member are separately represented. In a composite representation (Herrmann and Al Yassin, 1978) the reinforced system is modeled as a locally homogeneous orthotropic material. The composite material properties assigned to the continuum elements reflect the properties of the matrix material and the reinforcing members and their composite interaction. Although each approach is inexpensive for analysis of simple two-dimensional systems, for large two-dimensional and three-dimensional systems, only the composite approach is likely to be economically feasible.

Analysis plays an additional role in the evaluation of the overall stability of reinforced earth systems. McKittrick (1978) describes two case histories where gross foundation failures developed under reinforced earth walls that themselves remained intact.

Several papers submitted to this Conference contain concepts, data, or other information related to analysis aspects of earth reinforcement.

Juran and Schlosser present a new design method for determination of the tensile forces in the reinforcements of reinforced earth walls and bridge abutments. This method is based on the kinematical and experimental consideration of the locus of the maximum tensile forces. This locus is a potential failure surface for the reinforcement and a sliding surface for the soil. Kinematical considerations and observations on actual structures and on reduced scale laboratory models led to use of a logarithmic spiral for this locus. The spiral passes through the facing toe and is vertical at the embankment free surface.

The tensile forces are determined considering overall equilibrium of the active zone situated between the facing and the locus of the maximum tensile forces. The soil reaction along the failure surface is determined by integration of Köter's equation. It is assumed that the horizontal shear stresses on each horizontal plane, localized in the center of any soil layer between two reinforcements, are zero, and thus the tensile forces may be determined by the horizontal equilibrium of each soil layer containing the considered reinforcement at its center.

The theoretical results are in a very good agreement with experimental observations on the locus of the maximum tensile forces as well as with the distribution of the values of these tensile forces with depth.

Phan, Segrestin, Schlosser, and Long propose a method for analysis of the stability of Reinforced Earth structures using circular slip surfaces. Both the Swedish circular are method and Bishop's method of slices are sused. In addition to the mobilization of soil friction along any trial slip surface the tensile forces in the reinforcements must be considered. At failure each tensile force is taken as the lesser of the tensile strength of the reinforcement or the sliding resistance of the reinforcement. Application of this method requires a choice of an adequate factor of safety to be applied either to the friction angle of the soil or to the limiting tensile forces in the reinforcements, or to both of them.

Measurements of reinforcement tension and soil strain were made during and after construction of 35 model reinforced earth walls using five different configurations of reinforcement spacing and panel size by Osman Finlay, and Sutherland. Soil expansion near the wall face decreased with distance into the fill and changed to compression over part of the fill height at about half the reinforcement length. This behavior was consistent with the mechanism proposed by Juran and Schlosser (1978) which considers the co-existence of active and resistant zones in the backfill. Maximum tensions were recorded at some distance behind the wall face.

A new method of analysis, termed the Energy Method, is introduced. It is based on the equilibrium of external work due to earth pressure and internal strain energy stored in the reinforcements. This method yields the following relationships :

Maximum reinforcement tension at depth L :

$$T = \sqrt{\frac{6\,K_a^{2.5}}{L}}\;\gamma \cdot h \cdot \Delta H \cdot S\sqrt{H-h}$$

Maximum reinforcement tension in wall :

$$T_{max} = \sqrt{\frac{8\,K_a^{2.5}}{9\,L}}\;\gamma \cdot \Delta H \cdot S \cdot H^{1.5}$$

Critical Wall Height :

$$H_c = \left(\frac{R_t}{\gamma \cdot \Delta H \cdot S}\sqrt{\frac{9\,L}{8\,K_a^{2.5}}}\right)^{0.67}$$

Safety factor against reinforcement pull-out :

$$S.F. = \frac{2 \cdot b \cdot f \cdot L^{1.5}}{S \cdot \Delta H\,\sqrt{6\,K_a^{2.5}\;(H-h)}}$$

WHERE :

h = fill height above reinforcement ;
H = total fill height above base of wall ;
ΔH = vertical reinforcement spacing ;
L = reinforcement length ;
S = horizontal reinforcement spacing ;
γ = unit weight of soil ;
K_a = active earth pressure coefficient ;
R_t = tensile strength of reinforcement material ;
b = reinforcement width ;
f = reinforcement-soil friction coefficient.

Results by the Energy Theory agree well with both Juran and Schlosser's (1978) limit theory and experimental results. Osman et al. indicate also its advantage of easy application.

The "composite material" approach was adopted for the finite element analysis of reinforced earth walls by Al-Yassin and Herrmann. A detailed description of the methods used for parameter selection is presented. Comparisons are presented between the predicted and observed behavior of three walls. In each case the agreement between predicted and measured tensile stress distribution along the reinforcing strips was excellent. Fig. 35 is an example for one of the walls studied. On the other hand, failure conditions were not predicted well.

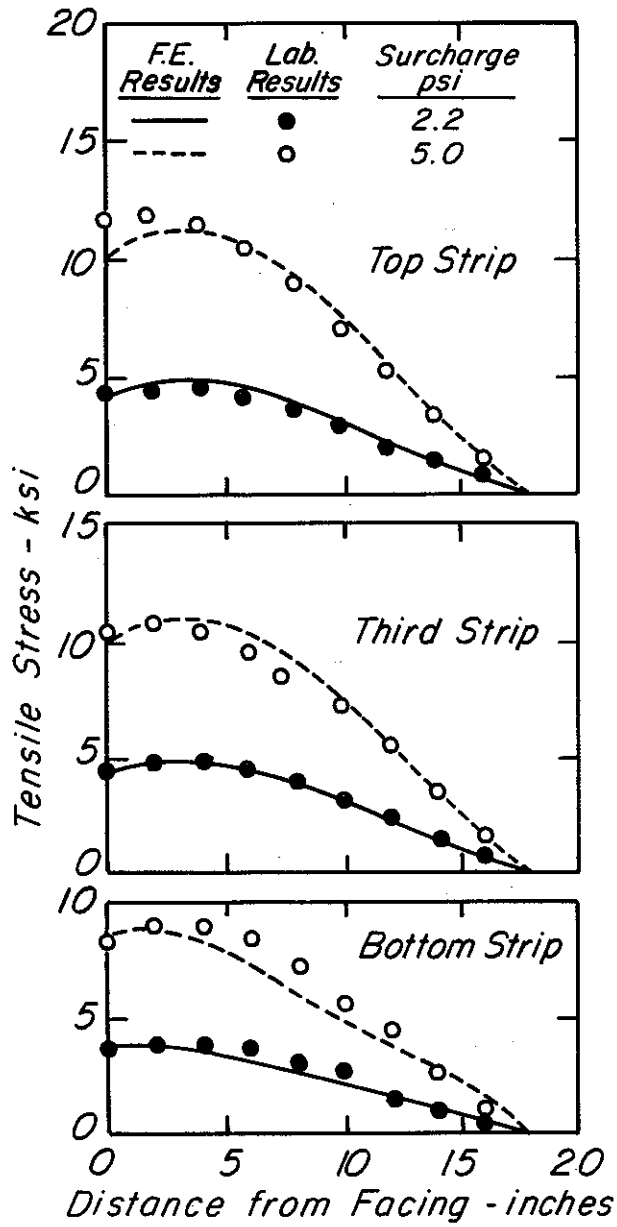


Fig. 35 - Tensile stress distribution along strips - UCD wall (Al-Yassin and Herrmann)

The authors conclude that to overcome these difficulties improvements are needed in the characterization of soil properties near failure and in laboratory procedures to measure the coefficient of friction between reinforcing members and soil.

The consequences of the placement of reinforcing strips in the backfill of a conventional type of retaining wall were analyzed by Saran, Talwar, and Prakash. The pressures acting on the back of the wall and their distributions were evaluated by assuming that the wall moves laterally under the influence of a lateral thrust and that a Coulomb wedge of reinforced soil separates from the rest of reinforced soil backfill when active conditions are reached. The forces considered in the analysis are shown in Fig. 36.

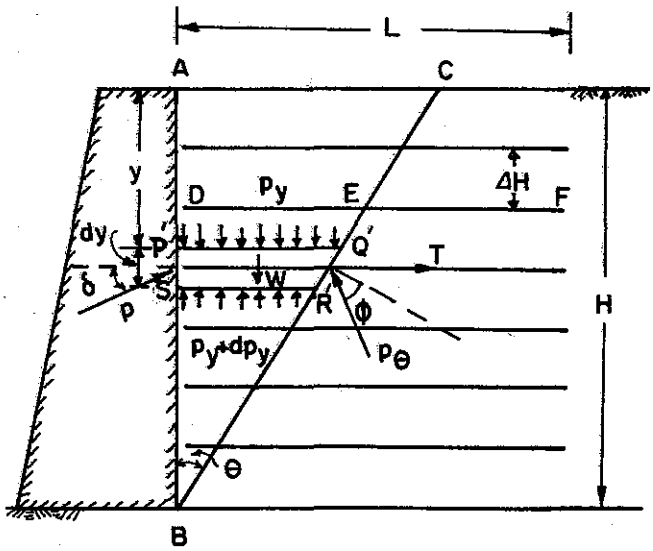


Fig. 36 - Forces acting on the element P'Q'R'S' for analysis of a reinforced backfill behind a conventional retaining wall (Saran, Talwar, and Prakash)

The results of the analyses indicate that substantial reductions in the magnitude of the total thrust acting on the wall (Fig. 37) and the height to its point of application (Fig. 38) result when reinforcing strips are used, provided the length of reinforcement is greater than 0.6 times the wall height. It is possible that the actual failure mode would be consistent with the coherent gravity structure hypothesis rather than the tie-back structure hypothesis (McKittrick, 1978) used by Saran et al. Nonetheless, the general patterns of behavior established appear reasonable and demonstrate the potential usefulness of tensile reinforcement in gravity wall backfills.

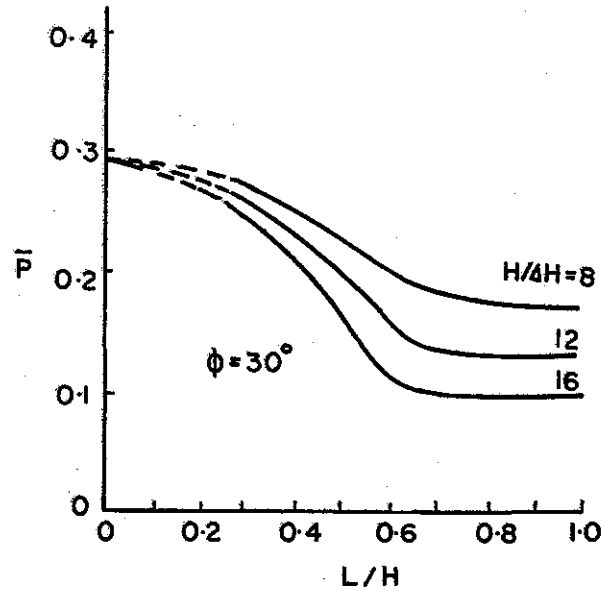


Fig. 37 - Non-dimensional total pressure $\bar{P}$ acting on a retaining wall with a reinforced backfill (Saran, Talwar, and Prakash)

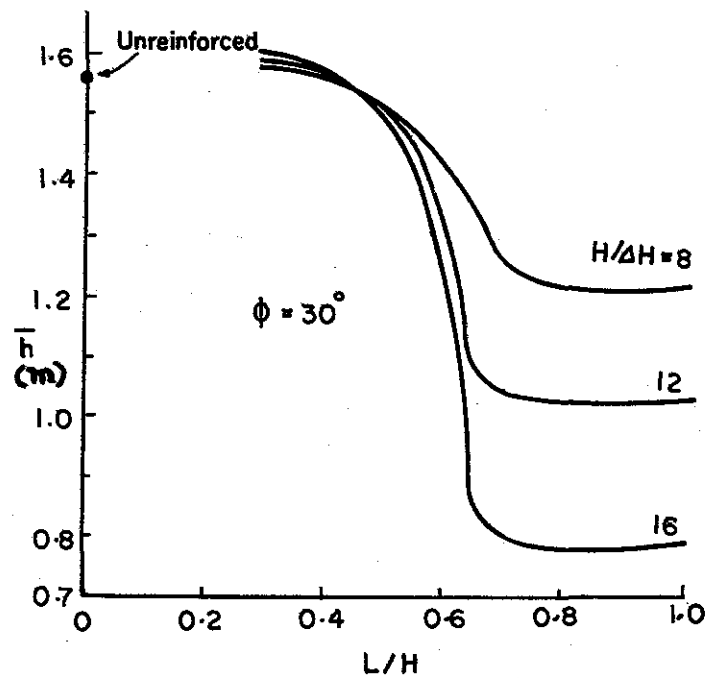


Fig. 38 - Action of resultant force for a reinforced backfill behind a conventional retaining wall (Saran, Talwar, and Prakash)

DESIGN METHODS FOR REINFORCED EARTH SYSTEMS

The design of Reinforced Earth walls requires selection of reinforcement strip sizes and spacings such that :

1) their cross sectional area can carry the maximum tensile forces with an adequate factor of safety and allowance for corrosion ;

2) their surface area and length are adequate to pick up the horizontal stresses from the active zone and transfer them to the resistant zone.

A design method that is consistent with latest findings from field measurements and theoretical analyses has been adopted by the French specifications for the design of reinforced earth structures (Schlosser, 1978). It is given in Fig. 39, from McKittrick (1978).

The design method currently used in France and described in a paper to this Conference by Schlosser and Segrestin is similar to that shown in Fig. 39. It is based on the results of full scale experiments on reinforced earth walls.

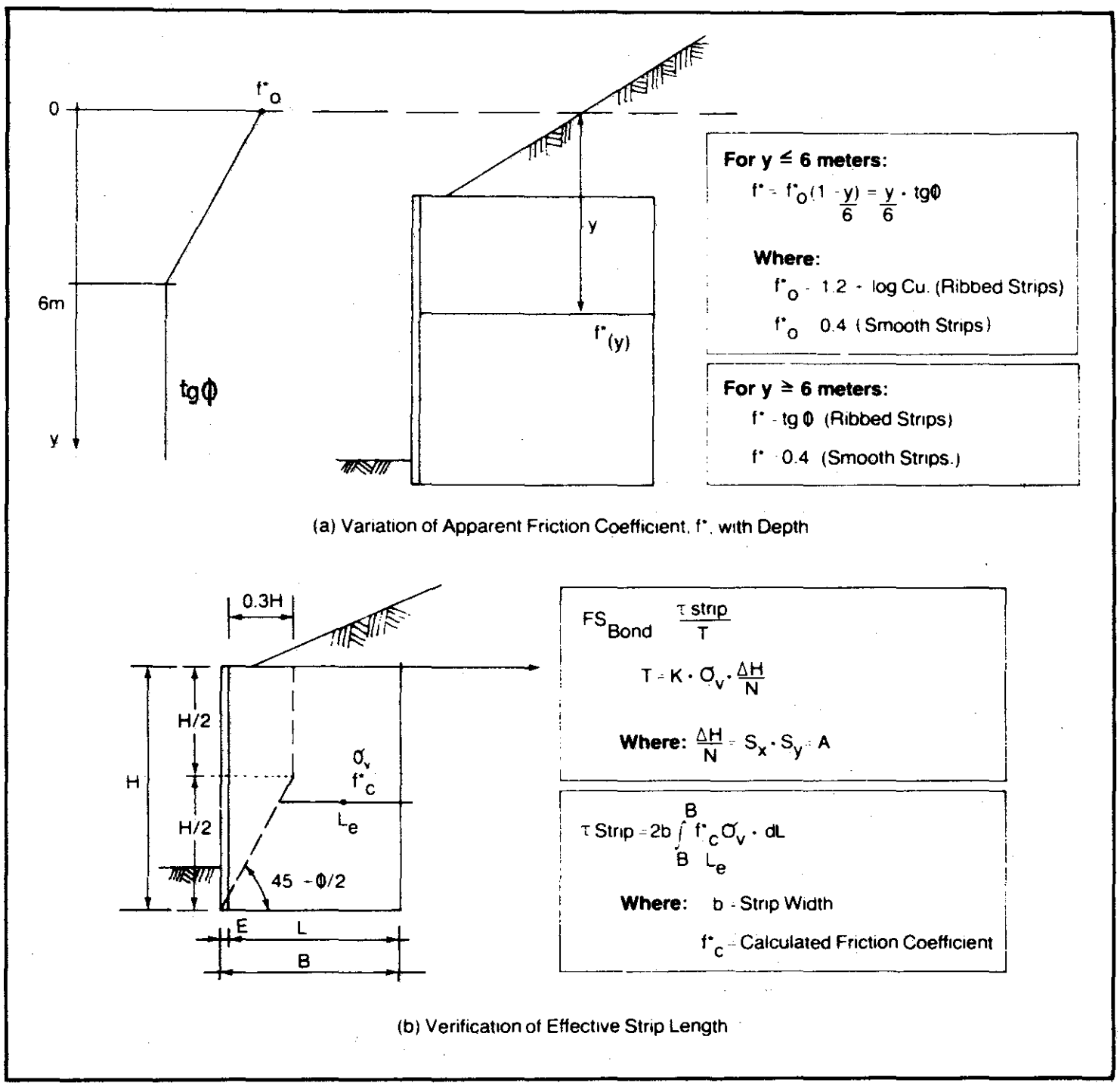


Fig. 39 - Design methods for evaluating safety against failure by lack of adherence (McKittrick, 1978)

They show first of all that the locus of the maximum tensile forces is very different from the classical Coulomb's plane (inclined at $(\pi/4 + \phi/2)$). This locus is vertical at the upper part of the wall, and its distance from the facing is always less than 0.3 H. In the case of loaded walls the locus of the maximum tensile forces is modified by the presence of the load and has a tendency to approach the application point of the load.

The maximum tensile force in the reinforcement is calculated considering the local vertical stress σ_v on the locus of the maximum tensile forces and introducing an empirical coefficient K characterizing the state of stresses within the embankment material. The values of K were determined on the basis of full scale experiments (Fig. 33). It is close to K_o at the top of the wall and decreases with the depth to a value which is less than K_a at the lower part of the wall (Fig. 40).

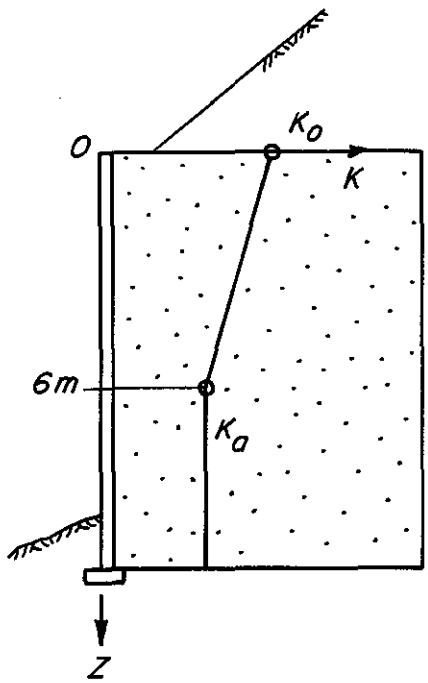


Fig. 40 - Earth pressure distribution in a reinforced earth structure

For a loaded wall (e.g., for bridge abutments) the vertical stress is calculated considering a spread of the load at two vertical to one horizontal with depth. The authors indicate that this load spread angle leads to conservative results. The presence of the reinforcements gives a more elastic behaviour to the mass and therefore, as demonstrated by full scale experiments, a spreading angle of 1:1 is more reasonable to characterise the response of the reinforced earth structures to vertical loading.

The required lengths of reinforcement are determined using the locus of maximum tensile forces and the apparent friction coefficient f* deduced from the actual state of knowledge on soil-reinforcement as reported by Schlosser and Guilloux and Guilloux, Schlosser, and Long in papers to this Conference reviewed earlier in this Report.

Segrestin shows that failure surfaces in reinforced earth walls are inclined at an angle of $(45 + \phi/2)$ degrees to the horizontal whenever failure is due only to breaking of reinforcements. When failure is due in part to slip of reinforcements, the inclination is less than $(45 + \phi/2)$. A computer program has been developed for design of reinforced earth walls based on equilibrium of active zones defined by the failure planes.

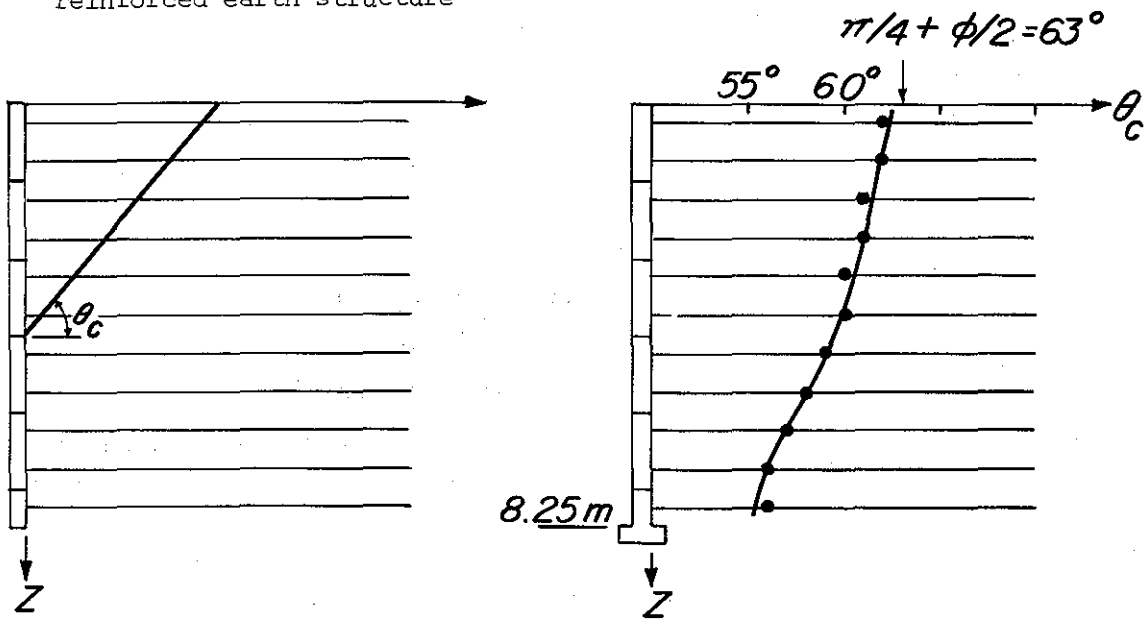


Fig. 41 - Optimum angles of failure planes as a function of the intersection point of the failure planes with the facing (Segrestin)

The maximum tensile force in each reinforcement is the lesser of the resistance of the reinforcement in tension and the sliding resistance of the reinforcement. The variation of the optimum angles of the failure plane as a function of the position of the point of intersection of the failure plane with the wall facing is shown in Fig. 41.

Calculation of the safety factor of each reinforcement of a given wall leads, generally, to greater values than those obtained by the method of local equilibrium, as shown in Fig. 42. The design of a structure by Segrestin's failure wedge method depends strongly on the density of reinforcements at the top of the wall. Thus the method cannot be used in practice for a reliable design ; however, it can be a useful method of verification.

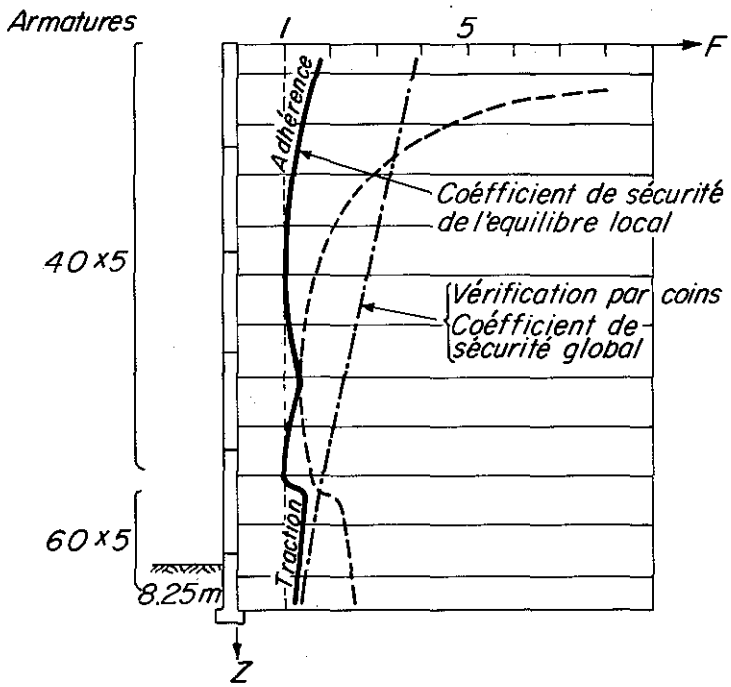


Fig. 42 - Vérification du dimensionnement usuel par la méthode des coins, coefficient de sécurité (Segrestin)

A new design method for reinforced earth walls has been published recently (Department of Environment, UK, 1978) and is described by John. The basic concept of this new method is to determine first the critical failure plane which corresponds to the maximum of the total tensile forces and then its location. John notes that several design methods give similar results for tension design, but that they give widely differing results for adherence design. The new Doe method appears very conservative in this regard.

Model tests were used to observe failure patterns. It was found that the failure surface approximates to a logarithmic spiral regardless of whether the reinforcement is long or short. The shape of the failed zone seems quite similar to that associated with the coherent gravity structure hypothesis (Juran and Schlosser, 1978 ; McKittrick, 1978).

The paper by Popescu presents a comprehensive summary of several methods that have been used for the design of reinforced earth walls. Analysis methods are first reviewed, followed by design considerations for internal stability considering both the lengths and spacings of reinforcements and skin elements. The overall external stability of the reinforced structure is also discussed. This paper will be of interest to those desiring a comparison of design methods that have been in common use.

SPECIAL PROBLEMS

Several papers included within Session 1 deal with important special aspects of earth reinforcement that don't fall conveniently within the preceding sections of this Report.

Vaults in Reinforced Earth

The paper by Behnia and Long deals with two-dimensional model tests of reinforced earth vaults. The soil was simulated by steel cylinders piled up in parallel array and the reinforcements were made of aluminium foil 9 μm thick. All the reinforcements were identical and regularly distributed around the circular vault. The vault was built up inside the model, and a pressure p was applied to the horizontal surface of the mass 40 cm above the center of the vault. No lateral deformation of the soil was permitted.

Two modes of failure of the vault were investigated :

1) lack of adherence, which develops when the reinforcement length is too small ;
2) breakage of the reinforcements.

A characteristic of reinforced earth vaults is that lack of adherence can be progressively compensated by the deformation and the decrease of the radius of the opening which leads to an increase of the effective reinforcement length. However, there is a critical value of the initial reinforcement length, L_c, so that for $L<L_c$ the

vault is unstable and failure occurs by lack of adherence.

Benhia and Long show experimentally that P_r (p at failure by breakage of the reinforcements) is given by the following formula :

$$P_r = g(\phi,\ f)\ \frac{R_T}{R^2} \cdot \frac{1}{\sigma}$$

where :

ϕ = friction angle ;

f = soil-reinforcement coefficient of friction ;

R_T = tensile resistance of a reinforcement ;

R = radius of the vault ;

σ = angle of each element of the vault (see their Fig. 1).

The state of stresses is characterized by the ratio :

$$K = \frac{\sigma_t}{\sigma_r}$$

on the vertical axis (σ_t = tangential stress, σ_r = radial stress). K is found to decrease from K_p at the vault to K_o at the extremety of the reinforcements (see their Fig. 9).

σ_t and σ_r are maxima around the middle of the reinforcement, where the tensile force in the reinforcements is a maximum. The critical length L_c has been found to be proportional to $R \cdot \sigma$.

A theoretical approach for determination of P_r has been developed. It consists of studying the equilibrium of the central and vertical element of the vault, which is found experimentally to be the most critical one. However, there is a large difference between the experimental and the theoretical values. The authors indicate that this difference is probably due to the uncertainty of the exact value of f.

Reinforcement of Railway Ballast

Mazur presents results of full scale laboratory tests of cyclic point loads on a railway sleeper resting on a 40 cm thick sand ballast. Reinforcement consisting of one or two sheets of wire mesh placed horizontally at different depths was used. The sand base was placed over a Winkler foundation and gages were installed to measure the stresses at the bottom of the sand layer.

The results of Mazur's tests show that in loose sand the settlements are reduced by the reinforcement, and the stresses at the base are more uniformly distributed. These results are analogous to observations on the diffusion of vertical stresses induced by vertical loads on reinforced earth structures as shown by Juran, Schlosser, Long, and Legeay (1978).

Bearing Capacity of Reinforced Ground

Stefani and Long, in the first of two papers, present the results of tests on small scale, two-dimensional models of footings over uniformly reinforced sand, as shown in Fig. 43. The depth of reinforcement and length of reinforcing strips were sufficient to ensure failure by rupture of reinforcing strips before slip between reinforcement and soil.

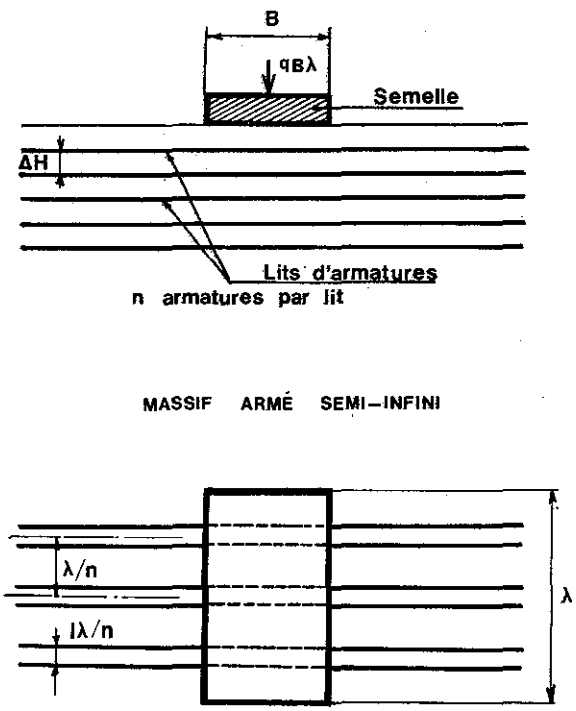


Fig. 43 - Semi-infinite reinforced earth mass (Stefani and Long)

The results show that the bearing capacity q_r can be defined by the first rupture of the reinforcements followed by failure developing progressively in the reinforced mass.

Two cases can be distinguished :

1) Low density of the reinforcements. The first breaking of the reinforcement occured before the mobilization of the bearing capacity q_{or} of the sand. There was a progressive failure of other reinforcements, and the final bearing capacity was slightly higher than q_{or}.

2) High density of the reinforcements. The failure of the reinforcements occured simultaneously, and the bearing capacity q_r was much greater than q_{or}.

For a given width of the slab the reaction modulus K increased with the resistance of the reinforcement to tension for a given value of the settlement. Stefani and Long could consider another parameter which is an elasticity parameter related to the density of the reinforcements in the sand. In the same way the Reaction Modulus K increases with the width of the slab.

The bearing capacity q_r increased with increased resistance of the reinforcement to tension. The curves comprise a first part, which is a straight line starting at the origin corresponding to the breaking of the first reinforcement which occurs before the soil reaches its failure state. This part is followed by a second straight line which corresponds to the complete breaking of the reinforcements. This second line intersects with the axis of q_r with an initial coordinate corresponding to the bearing capacity of the unreinforced sand q_{or}.

Stefani and Long review four methods which can be considered in order to determine the bearing capacity. The first is based on dimensional analysis, the second is based on the equilibrium of the wedge under the slab, the third uses the values of the pseudo-cohesion of the Reinforced Earth, which is assumed to be isotropic, and the fourth assumes a circular sliding surface. However the authors do not propose any specific reliable formula.

In a second paper (Model Studies) Stefani and Long present more specific details for tests where the width and depth of the reinforced zone are limited. The authors show that except for the failure mechanism there are no noticeable differences between the behavior of such a foundation raft and the behavior of the semi-infinite reinforced mass, providing that an adequate proportion between the width of the loading slab and the depth of the reinforced zone is maintained. This means that if the failure wedge can be entirely developed in the mass, the bearing capacity q_r is practically the same as in the case of the semi-infinite reinforced soil.

A proposed design method based on the hypothesis of a failure wedge with inflexion of the reinforcements which enables taking into account a vertical contribution of their resistance to tension leads to results which are close to the experimental results.

Stefani and Long give also some results of tests on three-dimensional models of a reinforced foundation soil. The variations of the bearing capacity with the resistance of the reinforcements per unit of depth is analogous to the variations obtained on the two-dimensional models.

Milovic presents some results of field load tests on rigid circular foundations of diameter D = 60 cm, placed on reinforced soil. In these tests the layer beneath the foundation was reinforced either by polypropolene cords of 15 mm diameter or with steel bars of 12 mm diameter. This reinforcement was placed in only one or two orthogonal directions. In the first group of field tests the behavior of a multilayer system was examined, where the upper layer was made of gravel and the lower layer of sand. The thicknesses of the gravel layer tested were : H_1 = 15 cm, H_1 = 30 cm and H_o = 45 cm. In the second group of field tests the upper gravel layer was reinforced by polypropolene cords of 4 ø 15 mm in 2, 3 and 4 lines. In the third group the upper gravel layer was reinforced by steel bars 4 ø 12 mm in 1, 2 and 3 lines.

The field load tests showed that reinforcing the upper part of the multilayer system reduced the settlements and that this reinforcing effect increases with the applied stress level. Using the theoretical solutions for the stresses and displacements due to rigid circular foundations on a multilayer system and the results obtained by field load tests, an attempt was made to find the equivalent modulus of the multilayer system with a reinforced upper part. Fig. 44 illustrates the comparison between the equivalent modulus of the unreinforced multilayer system and the equivalent modulus of the one reinforced by the steel bars for different applied stress levels.

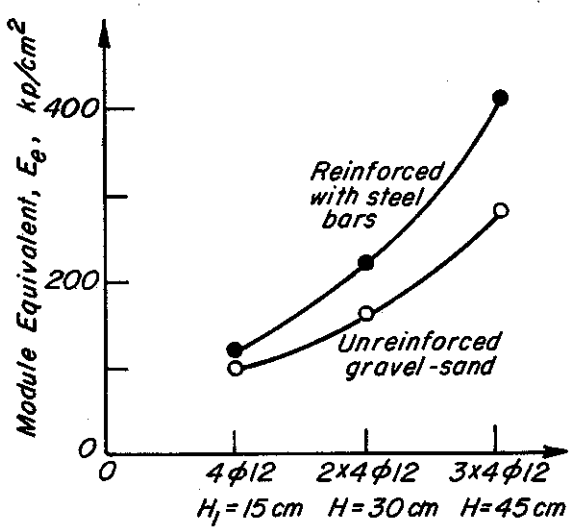


Fig. 44 - Valeurs des modules E_e obtenues par des essais sur place (σ = 1,5 + 5,0)

The paper is limited to experimental observations, and the author does not give theoretical analysis in order to explain the mechanism of the soil-reinforcement

interaction which actually contributes to reduce the settlements and to increase the bearing capacity of the multilayer system.

Behavior of Reinforced Earth in Repeated Loading

Madani, Long, and Legeay report a laboratory investigation on the behavior of reinforced earth in repeated triaxial compressive loading. Tests were carried out on 100 mm diameter by 200 mm high samples of Fontainebleau sand at an average density, reinforced with circular discs (200 mm diameter). The discs were of a high tensile resistance so that failure could only occur by lack of adherence. About 10^6 load cycles were applied.

Tests were done at a constant lateral stress with a sinusoidal dynamic component of the vertical stress. It was assumed that the frequency did not affect the behavior, and a frequency N of 4 Hertz was chosen.

The influences of the following parameters on the permanent deformation εp were studied :

σ_1 : vertical stress ;

$\Delta\sigma_1$: dynamic component of the vertical stress ;

σ_3 : lateral stress ;

n : number of applied cycles ;

ΔH : vertical spacing of the reinforcements.

Principal results of these experiments were :

1- For stresses less than that to cause failure, strain increases continuously with number of cycles. Strains in the reinforced sand are smaller than for the unreinforced sand, and smaller strains are associated with smaller spacings between reinforcements, as shown in Fig. 45.

2- For a given static stress σ_1 the shape of the strain vs. number of cycles curve depends on $\Delta\sigma_1$. For a small σ_1 the strain reaches a maximum. For high σ_1 the incremental strain increases with each cycle and failure occurs after a few cycles. Between these cases the strain increases roughly in proportion to the number of cycles.

3- The ultimate strength is not much influenced by the repeated loading.

4- The elastic modulus is higher after cyclic loading.

Although the study is intended only as a

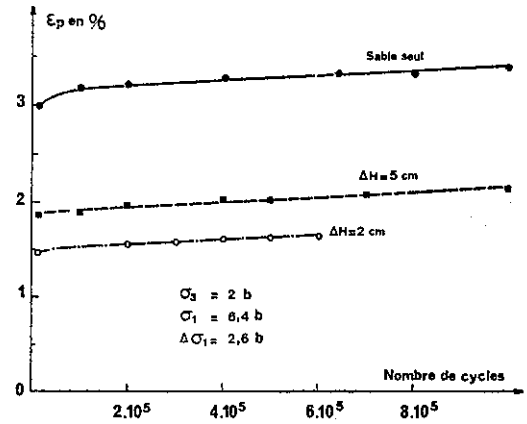


Fig. 45 - Influence of reinforcements on strain caused by repeated loading (Madani, Long, and Legeay)

preliminary investigation of the fatigue behavior of reinforced sand samples, the results indicate a consistency with previously established characteristics of unreinforced soils under repeated loading.

Effects of Seismic Loading

Previous studies (e.g., Richardson and Lee, 1975) have established that provided there is no reinforcement breakage, a reinforced earth wall will not collapse suddenly, but will simply move outwards during strong seismic shaking. It will be deformed but still stable at the end of the event. Recommended magnitudes of seismic reinforcement forces are given by Richardson and Lee (1975).

Double-sided model walls with reinforcing strips attached to one side only were subjected to simulated earthquake motions by Hausman and Wolfe. The tests were designed to give failure by reinforcement slippage rather than by rupture.

In tests with vertical motion only virtually no cyclic non-permanent outward deformation of the wall was observed, and there were no significant increases in measured reinforcement forces. Horizontal base motions caused both deflections and reinforcement force increases. Bi-directional shaking was also studied. Both one and two-sided deformations were observed, but there were no catastrophic failures. An example of a one-sided deformation is given in Fig. 46.

Observed failure plane inclinations were similar to those of single walls. The smaller deformations observed in double-sided walls than in single-sided walls were attributed to a possible reinforcing effect from

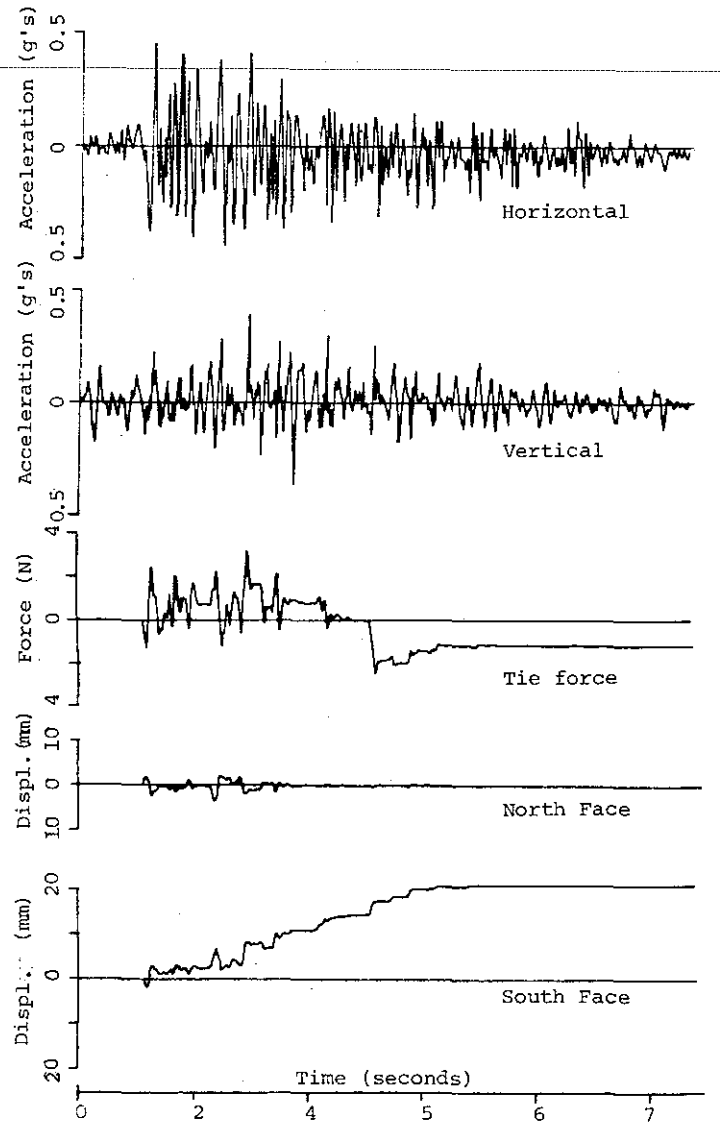


Fig. 46 - Wall response time histories (test No. O) (Hausman and Wolfe)

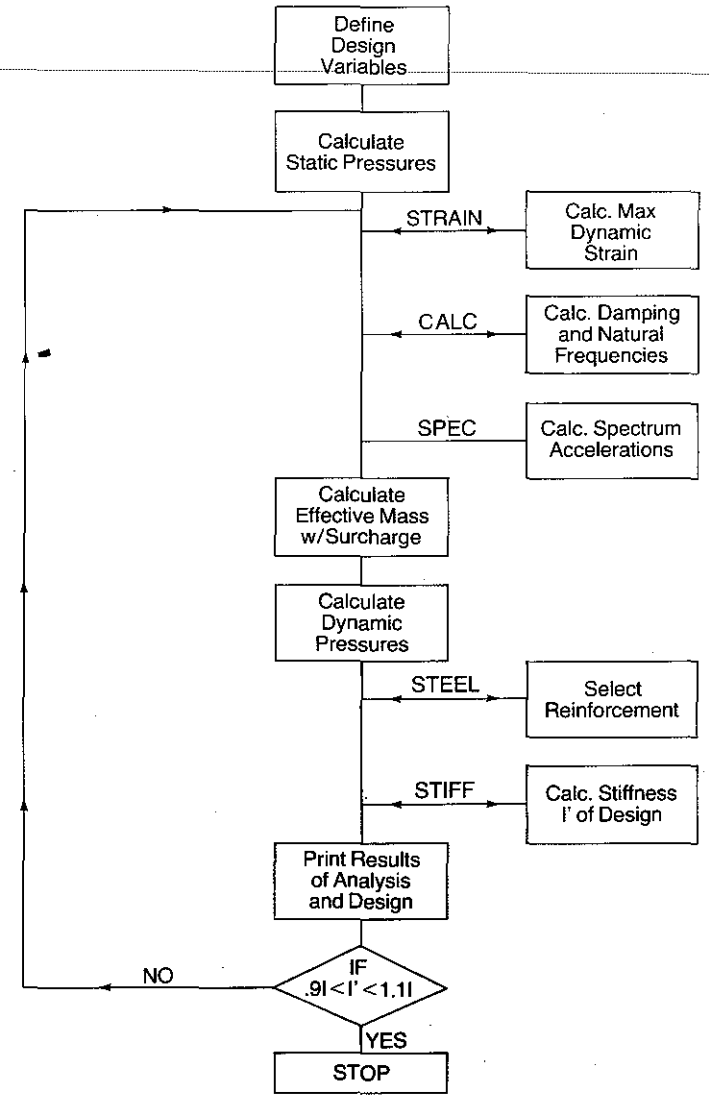


Fig. 47 - Flow Chart of Seismic Design Procedure

the strips connected to the opposite face. As this effect was small, however, Hausman and Wolfe recommend that double-sided walls with reinforcements connected to one side only be designed in the same manner as single walls.

Design of Reinforced Earth structures for the Trans Alaska Pipeline terminal at Valdez required consideration of ground motions corresponding to earthquake magnitudes up to 8.5. The design procedure developed for this is described by McKittrick and Wojciechowski. During a seismic event, the total forces in a structure are assumed to be the sum of the static forces acting before the event plus the dynamic forces. The dynamic forces are calculated using procedures developed initially at UCLA and modified by the Reinforced Earth Company. Response spectrum techniques with empirically-defined parameters are used.

A flow chart for the seismic design procedure is shown in Fig. 47. In application the calculated response represents an upper bound. An initial stiffness consistent with the density and length of reinforcement strips needed to resist the static stresses is assumed. The response of this structure is determined and reinforcement design is carried out to satisfy the combined static and dynamic forces. The stiffness of the resulting structure is calculated and compared with the initially assumed value. The process is repeated until the compacted stiffness is in the range of 0.9 to 1.1 times the initially assumed value.

McKittrick and Wojciechowski report that the effect of inclusion of the dynamic forces in the design is to increase the density of reinforcing strips required near the top of the structure, but strip lengths remain about the same as for static design alone. The method has the advantage, com-

pared to pseudo-static methods, that dynamically induced deflections are computed as well.

CONCLUSION

The state-of-knowledge concerning the mechanisms, behavior, and design methods for earth reinforcement as it existed at the start of the Conference has been summarized briefly. Attention has been directed mainly at reinforcement for construction of reinforced earth walls ; however, some consideration has been given also to other applications of tension elements.

By the start of this Conference evidence was accumulating and analyses were supporting the concepts that Reinforced Earth walls were not analogous to conventional gravity walls as regards the development of earth pressures and modes of failure, especially the shape of the failure zone. Furthermore, the concept that the reinforcements acted as "ties" was being replaced by mechanisms that address the effects of the reinforcements on the strain and stress state within the soil. Accordingly, the adherence or frictional characteristics between the soil and reinforcements become the most critical part of a reinforced earth system.

The models developed within the last few years, especially those at L.C.P.C., which treat the reinforced soil mass as a coherent structure with active and resistant zones, that represent a failure surface of the type shown in Fig. 4, and that establish tensile force distributions along the reinforcements in agreement with both measured and theoretical (F.E.M.) values are generally supported by the contributions to this Conference. Support is provided also for pressure distributions and reinforcement-soil friction values that vary with depth. Although these variations are expressed somewhat empirically for use in analysis and design, they are supported qualitatively by known aspects of soil behavior.

The analysis, design, and behavior of the classical Vidal type of Reinforced Earth wall seems now quite well explained. Safe, economical walls can be assured in most cases if suitable soils and foundation conditions are available. The search continues for suitable reinforcing materials other than metal strips. Contributions to this Conference have provided useful data or the frictional characteristics of soils in contact with fabrics and synthetics. Several papers have pointed up the need for care in selection of the type of test for evaluation of the friction between soil and reinforcement. It is clear that choice of friction value must give consideration to the magnitude of normal stresses. Reasonable values can be chosen for many cases using accumulated data and experience.

The use of inclusions in cohesive soil for reinforcement, while showing promise in some cases, appears to have limited general application unless ways can be found to assure rapid drainage and high adhesion and friction between the soil and reinforcement.

Although only few papers in the session were concerned with applications of earth reinforcement other than for walls, it is believed that significant advances are still possible for uses of tensile reinforcement relating to bearing capacity, embankments, etc. The fundamental mechanisms relating to soil-reinforcement interactions and the effects of these interactions on the stress and strain fields within the reinforced soil mass should be especially helpful in the study of such problems.

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Discussion

Questions de M. BOLLE à M. GUILLOUX

1) Quel est le rapport entre les dimensions de l'appareil de cisaillement et des armatures ? N'y a-t-il pas un effet d'échelles ?

2) Les essais de traction en vraie grandeur ne sont-ils pas influencés par une modification de la répartition des contraintes (effet de voûte dû à l'arrachement d'une seule armature) ? En réalité, dans un ouvrage réel, le comportement est plutôt de type bidimensionnel.

M. GUILLOUX

La boîte de cisaillement utilisée en laboratoire pour les essais de frottement était une boîte standard, de dimensions 100x100 mm. Mais je pense que les seuls essais effectivement interprétables sont les essais de cisaillement interne dans le sable ; le cisaillement sable-armature en laboratoire représente probablement très mal ce qui se passe dans un ouvrage, puisque l'armature occupe toute la section de la boîte, et que le phénomène est donc uniquement bidimensionnel, alors qu'il est tridimensionnel dans les ouvrages. La boîte avait donc 100 mm de côté, alors que les armatures ont 40 ou 60 mm de large.

M. KERISEL

Donc vous admettez qu'il y a un effet d'échelle ? Ou un effet tri-dimensionnel qui est absent ?

M. GUILLOUX

L'effet tridimensionnel est effectivement absent dans l'essai de cisaillement sable-armature, mais je pense qu'il joue un rôle important dans l'essai de cisaillement interne.

La deuxième question concerne l'effet des armatures en place sur celle qui est extraite lors d'un essai de traction en vraie grandeur. La figure 2 de notre communication concerne l'effet de groupe étudié sur modèle réduit par extraction d'un groupe de quatre armatures simultanément, puis de chacune des armatures séparément. Les résultats montrent que l'effet de groupe, important pour un faible espacement des armatures, devient négligeable lorsque l'espacement des armatures atteint 6 à 8 fois la largeur, ce qui est toujours le cas dans les ouvrages.

M. KERISEL

Ce n'est pas exactement la question posée par Monsieur Bolle. Il vous pose la question de savoir si, lorsqu'il y a un groupe d'armatures, et que vous tirez sur une armature, l'effort de traction mobilise ou non des effets de voûte sur les autres armatures, donnant par conséquent des résultats supérieurs à la réalité, alors que le Professeur Mitchell a parlé de l'effet de groupe, qui est analogue à celui dont on parle à propos des pieux et qui varie en fonction de leurs distances. Ce sont deux questions différentes.

M. GUILLOUX

Puisque l'on a constaté que le frottement mobilisé en extrayant un groupe de 4 arma-

tures était le même que celui mobilisé en extrayant les 4 armatures séparément, je pense que l'on peut admettre que dans ces conditions, l'effet de voûte éventuellement mobilisé par l'armature extraite sur les autres armatures est sans influence sur le résultat.

M. SCHLOSSER

Nous ne pouvons que regretter que Monsieur Long du Laboratoire Central des Ponts et Chaussées n'ait pas pu assister à notre Colloque, car ayant dirigé toutes les recherches expérimentales faites sur les modèles réduits, il aurait sans doute été le plus à même de répondre à la question de Monsieur Bolle, tant sur un éventuel effet d'échelle que sur l'espacement minimal à respecter entre les armatures pour ne pas perturber l'essai de traction.

M. KERISEL

C'est une question qui est assez analoque à celle de l'essai de charge sur un pieu en mobilisant un effort de traction sur de des pieux voisins. Il faut qu'ils soient suffisamment espacés pour qu'il n'y ait pas d'interaction.

Question de M. LEGEAY à M. SCHLOSSER

La force de traction maximale dans une armature dépend de la charge verticale, mais aussi (pour une même charge) du niveau auquel elle est située dans le mur (observations sur modèles et nombreux ouvrages). Nous pensons que ceci est dû, bien sûr, à la fondation, mais aussi à l'influence du parement et de la butée en pied. Le parement ne joue pas qu'un rôle local, mais un rôle important sur la distribution des efforts. Or aucune méthode de dimensionnement (ou de calcul des efforts) n'en tient compte. Qu'en pensez-vous ?

M. SCHLOSSER

Je pense que Monsieur Legeay a en partie répondu à la première partie de sa question et que, ce qu'il souhaite, c'est une confirmation. J'ai, pour ma part, été très intéressé sur ce sujet par deux communications présentées au Symposium de l'ASCE de Pittsburgh en avril 78. La première, du Docteur Bassett, montre, d'un point de vue théorique, que la surface de rupture potentielle dans un mur en terre armée est verticale. La deuxième, du Docteur Bolton, rapporte des essais en centrifugeuse sur des modèles de murs en terre armée ; elle confirme expérimentalement le résultat de Bassett en montrant que la ligne des tractions maximales dans les armatures est verticale. Si, dans la partie inférieure du mur, la ligne potentielle de rupture s'incurve pour venir passer par le pied du mur, il faut y voir à coup sûr l'influence de la fondation. Il y a, en particulier, le frottement sur la base, l'encastrement, qui restreignent les déformations et, par suite, modifient l'état des contraintes. Ce point est clair.

La deuxième partie de la question de Monsieur Legeay est relative au parement. Il est bien certain que les parements utilisés, ainsi que les espacements verticaux entre les lits d'armatures influent sur la surface de rupture, notamment dans la zone qui est en pied de mur. Je m'explique. Le parement a une résistance propre qui permettrait de construire un mur sans armatures sur une hauteur de une à deux écailles. Par ailleurs, le parement joue un rôle mécanique d'autant plus grand que l'espacement relatif entre les lits d'armatures est plus important. Ces deux éléments font qu'au pied du mur on retrouve un état de contraintes et de déformations analogue à celui qui se développe derrière un mur classique, d'où les déviations par rapport à la ligne de rupture théorique verticale.

M. KERISEL

Je remercie Monsieur Schlosser et voudrais lui poser une petite question complémentaire. Monsieur Bassett confirme ce qu'a écrit Monsieur Josselin de Jong sur les déformations imposées par un corps étranger dans un sol : le corps étranger étant ici l'armature, qui impose une déformation relative horizontale nulle, et même une vitesse de déformation horizontale nulle. Ceci permet d'avoir la déformation conjuguée, mais non pas d'avoir le tenseur des contraintes, étant donné que ce tenseur des contraintes fait avec le tenseur des vitesses de déformation relative un certain angle qui est variable avec la dilatance. Alors, si l'on constate que la ligne de rupture est une verticale dans le massif à partir d'une certaine distance de la base, et qu'au contraire elle rejoint le pied du mur dans la partie inférieure, cela veut dire, à mon sens, que l'angle que fait le tenseur des contraintes avec le tenseur des déformations relatives, varie selon qu'on considère la partie inférieure du mur ou la partie supérieure ; c'est-à-dire que les conditions de dilatance ne sont pas les mêmes. Cela

veut dire aussi probablement qu'il y a des forces extérieures sur la base de la fondation, frottement, et même un certain encastrement ; j'aimerais bien savoir ce qu'en pense Monsieur Schlosser.

M. SCHLOSSER

Je pense que les deux analyses se rejoignent. Il est clair qu'il y a des conditions cinématiques précises au bas du mur pour la surface potentielle de rupture. Celles-ci sont en relation directe avec les déplacements, et, par suite, avec les déformations dans la zone inférieure du mur. D'une façon générale, il y a dans cette partie une diminution des déplacements qui entraîne une concentration des contraintes. Je ne suis pas capable de dire quelle est la variation correspondante de l'angle de dilatance, mais il est bien certain que les conditions de dilatance, liées notamment à la valeur de la contrainte moyenne, ne sont plus du tout les mêmes.

M. MURRAY

A great deal has been said concerning the locations of the peak tensions on the reinforcements and how in general the points do not conform to a plane surface of failure. However, the majority of the results were obtained from structures which were subjected to their working load and which were not therefore close to failure. I would like to refer to some centrifuge model tests carried out by Bolton and Choudhury on the UMIST centrifuge in which tension failures were induced in the reinforcements. Clearly, for reinforcements of uniform strength, the point at which the reinforcement fails conforms to the location of maximum tension in the reinforcement. The results of this series of tests produced failures in the reinforcements at locations corresponding to a plane surface of failure (Fig. 2).

It could well be, therefore, that in some cases the location of the peak tension at failure may propogate from a point near the base of a structure, say, and develop upwards along a plane surface of failure. In which case it would seem prudent to consider this condition in the design.

M. SCHLOSSER

Il n'y a qu'une apparente contradiction entre les mesures de traction faites par le Docteur Bolton et l'observation des surfaces de rupture réelles. En effet, les mesures de traction permettent de déterminer la surface de rupture potentielle et celle-ci n'a pas de raison d'être la même que la surface de rupture réelle, influencée par la progressivité de la rupture. Si l'on se réfère par exemple aux essais qui ont été effectués à l'appareil triaxial, on trouve bien que ces deux surfaces sont différentes. Lorsque la rupture se produit par cassure des armatures, il se développe des plans

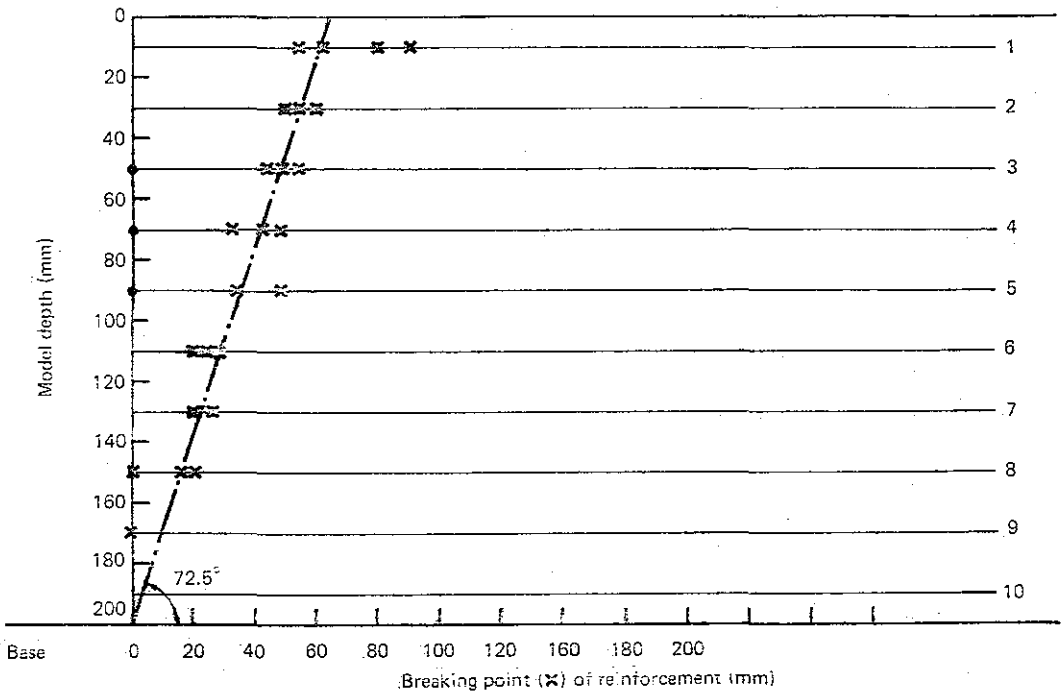


Fig. TENSION FAILURES INDUCED IN CENTRIFUGE MODEL TESTS
(after Bolton and Choudhury)

de rupture inclinés à $(\frac{\pi}{4} + \frac{\varphi}{2})$ sur l'horizontale. Cependant un examen détaillé des armatures, juste avant le pic de la courbe effort-déformation, montre que les amorces de rupture ne se situent pas sur le plan final de rupture, mais sur un cercle qui est le même pour tous les disques : la surface de rupture potentielle est un cylindre circulaire. Dans le dimensionnement des ouvrages, c'est la surface de rupture potentielle correspondant aux tractions maxima qu'il faut prendre en compte.

Il faut cependant noter que dans un bon nombre d'ouvrages, on a observé des surfaces de rupture réelles verticales en tête, et donc assez proches des surfaces de rupture potentielles. C'est le cas d'un mur de 6 m de hauteur, poussé à la rupture par corrosion des armatures, et qui fait l'objet d'une communication à ce colloque. On l'a également observé dans des modèles réduits, mais, comme je l'ai montré dans le rapport général, les résultats des modèles réduits montrent beaucoup de dispersion et celle-ci peut expliquer le résultat apparemment contradictoire présenté par Monsieur Murray.

Question de M. SCHARLE à M. MITCHELL

Pr Mitchell in his conclusions, mentioned that he prefers the pull-out experiment. I think that boundary conditions in pull-out testing, are generally not real ones. The question is : why do you prefer this pull-out test ?

M. MITCHELL

I think basically I arrived at that conclusion because the values of apparent friction that were deduced from pull-out tests, seemed mostly to be in agreement with the values that were obtained from model or full-scale wall tests. It was purely a practical choice on my part, an engineering solution, if you wish, based on the agreement between the results of those test types and the performance of the wall. I am not necessarily saying that the pull-out test results are theoretically or fundamentally the best in terms of defining a particular behavior coefficient. In the same regard it is important to note when we are concerned with the differences between full-scale walls and model tests, and these differences have been cited several times today, that we are dealing with quite different magnitudes of confining pressures in our structures. It has been established by work already completed and reported that the magnitude of confining pressure is an extremely important variable, because of its effect on dilatarcy.

M. MARIOTTI

La communication de Monsieur Mazur présente des essais effectués sur du ballast de voie ferrée armé posé sur du sable. Selon la communication, on constate que pour des sables compactés l'efficacité de l'armature est remarquable alors que pour des sables non compactés l'efficacité de l'armature du ballast est faible.

Ces résultats tendent à donner une impression décevante sur le renforcement des assises de voie ou autres assises avec le principe de couches armées ; je le regrette, car je suis persuadé que les couches granulaires armées offrent un moyen intéressant pour améliorer sérieusement soit la portance des sols sous semelles, soit la stabilité des assises de chaussées souples ou de voies ferrées.

Je pense que les échecs éventuels des expériences sont dus au fait que l'on ne s'est pas préoccupé, préalablement à l'expérience, d'un paramètre important, c'est celui du rapport entre la déformabilité de l'armature et celle du sol environnant. Il faut, en effet, comme dans le béton armé, qu'il y ait évidemment un rapport harmonieux entre le module de déformation de l'armature et celui du sol environnant, sinon on peut donner lieu à des contraintes qui sont localement incompatibles avec la résistance disponible. Le sol armé sollicité en flexion est à considérer comme un matériau composite dans lequel le transfert des efforts se fait par frottement sol-armature ; le choix des matériaux sol et armature, le dimensionnement des sections doivent être définis de façon que l'on maîtrise les contraintes à tous les niveaux.

Je pense que des recherches s'imposent dans ce domaine et je crois qu'en partant d'un examen rationnel du problème, tenant compte de la déformabilité du type d'armature et du sol environnant, on arriverait à contrôler plus largement l'efficacité du système. Dans le même ordre d'idée, je pense que dans le cas du mur en terre armée, il y a également une grande importance à accorder à la déformation du sol de fondation vis-à-vis du comportement interne de la terre armée ; il me semble que l'on ne maîtrise pas bien le comportement interne de la terre armée pour des déformations importantes du support, ce qui peut arriver fréquemment ; on peut alors aboutir à des

échecs qui pourraient être évités, soit en modifiant la déformabilité de la structure même de terre armée. J'évoque cette question, car j'ai eu récemment à réfléchir sur ce genre de problème, pour un cas réel.

M. KERISEL

Monsieur Mariotti souligne l'intérêt d'étudier les déformations du sol de fondation des massifs en terre armée par rapport à la déformation propre du massif, qui est certainement moins importante. Ce matin, le Professeur Mitchell nous a montré un exemple d'adaptation d'un mur en terre armée sur terrain mou. Il nous a indiqué très justement que ce qui est arrivé à ce mur en terre armée serait arrivé à n'importe quel mur de soutènement, et nous le croyons bien volontiers. La question que pose Monsieur Mariotti, c'est de savoir s'il n'y a pas lieu de faire des études de comportement de ces murs en terre armée par rapport à la déformabilité du sol, en deça de la rupture. Est-ce que le Professeur Mitchell veut bien répondre ?

M. MITCHELL

I think that absolutely one needs to study the overall stability of the system composed of the wall and its foundations, and the illustration that I showed this morning was an instability, not of the wall alone, but of the system. The point I was trying to make was that this is like designing any type of retaining wall or structural foundation. One needs to be sure that there won't be a slip surface passing beneath the reinforced earth and through the native ground.

Question de M. KENTER à M. MITCHELL

In slip circle analysis the influence of the reinforcement is often included by means of an additional cohesion. The direction of this "cohesive force" is assumed either horizontal or tangential to the slide-plane. Which direction is more realistic ?

M. MITCHELL

My understanding of the application of the tensile forces in the stability analysis by the slip circle method is that one notes the location of the strips and includes a force in the appropriate direction. If you are using a method of slices stability analysis, you then have an additional force for each slice. I don't think you would necessarily call it a cohesion or anything of that sort, it is just an additional force to be taken into account in the system. That is my understanding of the way this would be done. One could note though that if you have a circular arc cutting across these reinforcements, there will be some tendency for them to deform which could give you a component in a non-horizontal direction. I don't know how to take that into account.

Question de M. KENTER à M. SCHLOSSER

In the conference Fabrics in Soils of 1977 Al-Hussaini showed a case in which an embankment, reinforced with synthetic strips failed, most probably due to long elongation of the strips. The Rankine type design method didn't seem to hold for a structure with synthetic strips, although it did seem to hold for a structure with synthetic sheets.

Does this conclusion still hold nowadays ?

M. SCHLOSSER

Il s'agit de savoir si un ouvrage, comme un remblai renforcé par du géotextile, se rompt suivant le mode Rankine ou suivant le mode de rupture mentionné dans le rapport général. La réponse est, me semble-t-il dans la nature des déplacements et des déformations. Dans le cas des ouvrages en terre armée classique, où la déformabilité des armatures est faible comparée à celle du sol, le mode de rupture est très différent de celui de Rankine ou de Coulomb. Par contre, si la déformabilité des armatures est suffisamment importante pour que se développent dans le mur des déplacements analogues à ceux que l'on observe derrière les murs de soutènements classiques, on observe alors une surface de rupture proche d'un plan inclinée à $(\frac{\pi}{4} + \frac{\varphi}{2})$ sur l'horizontale. Je pense d'ailleurs que cette question rejoint en partie la question que m'avait posée Monsieur Murray à propos de la surface de rupture potentielle et de la surface réelle. Il faut analyser les déplacements de l'ouvrage. Pour répondre plus précisément à la question posée, je pense que, dans le cas d'un remblai renforcé avec du géotextile, il faut s'attendre à une rupture du type

Rankine lorsque les déformations sont importantes.

Question de M. MURRAY à M. INGOLD

This paper makes a useful contribution to understanding the behaviour of reinforced specimens of clay tested in the triaxial apparatus. The apparent reduction in strength produced in reinforced specimens undergoing rapid shear is of particular interest, but should be considered with some care before attempting to extrapolate such behaviour to full-scale reinforced earth structures. As the author has pointed out, the distribution of $\Delta\sigma_3$ induced in a specimen was strongly non-uniform (Fig. 6). Very steep stress gradients were therefore created and as a consequence the net effective stress reduced when porewater pressures attempted to equalise over any section. Such stress gradients are not normally produced at full scale and there will generally be a gradual increase in effective stress as consolidation takes place and presumably a corresponding increase in strength. This behaviour is to some extent confirmed by two full-scale studies involving cohesive fills reported in this conference (Hashimoto, Murray and Boden). The writer would be grateful for Mr Ingold's views on this and also how he might consider applying such triaxial test results to the design of reinforced earth structures.

M. INGOLD

Under axi-symetric stress conditions horizontal reinforcement induces an additional confining pressure $\Delta\sigma_3$. If the reinforcement is both continuous and impermeable then when combined with a saturated clay there is a reduction in compressive strength. Deviations from these condition would be expected to affect the behaviour of the reinforced earth.

For reinforced earth structures, such as walls, stress conditions are plane strain. The reinforcement is generally either discontinuous and impermeable, i.e. metal strips, or continuous and permeable, i.e. fabrics. Furthermore, the fill material is generally not saturated. This deviation alone causes very significant changes in behaviour. Tests carried out at the Ground Engineering Laboratory indicate that the strength ratio rises from approximately 0.5 for fully saturated clay to approximately 3.0 for clay with a degree of saturation of 80 percent. In the case of the reinforced clay wall reported to this conference by Messrs Murray and Boden, (pages 569), the implied average degree of saturation of the clay fills used varied between some 74 and 92 percent.

On the question of the effects of total stress gradients it would seem likely that the low gradients operating in a full scale wall with discontinuous reinforcement would be far less deleterious than those generated in a small scale sample tested under the conditions reported. In the case of the wall constructed at the Transport and Road Research Laboratory it appears that the beneficial effects associated with a low degree of saturation far out weigh the deleterious effects associated with an adverse total stress gradient.

In view of the lack of correspondence between the conditions prevailing in the reported triaxial tests and those in a reinforced earth wall it would be inappropriate to apply the triaxial test results to wall design.

Question de M. MURRAY à MM. HAUSMANN et WOLFE

In their very interesting paper the authors have described how double-sided reinforced earth walls appear more able to resist seismic forces when the reinforcements are connected only at one end. However, if I understand their approach correctly, then each side of the wall is designed independently and the amount of reinforcement produced on a vertical section is twice that employed when the reinforcements are attached at both ends (Figs. 1b and 1c).

Possibly an alternative approach which could reduce costs and still avoid the type of brittle failure which was observed with double-sided walls would be to use a lower modulus reinforcement - such as fabric say - and to retain the attachment at both ends. I would be grateful for the authors' comments on such an approach.

MM. HAUSMANN et WOLFE

The authors attempted to show that attaching the strips at only one end can, with appropriate design, ensure that a double-sided wall fails in a non-catastrophic mode so that it is likely to remain servicable even under prolonged seismic forces.

Mr Murray's suggestion of using reinforcement able to undergo large deformation before tension failure is well worth further investigation as the amount of reinforcement required is considerably less, for a narrow wall, if it can safely be attached at both ends. However, a large single-sided model wall described by Al-Hussaini and Perry (1976), which was constructed with neoprene coated nylon fabric reinforcing strips, failed at a height lower than that predicted by conventional theory and standard design criteria will have to be critically reviewed before membrane type reinforcing strips are adopted.

(Ref : Al-Hussaini M.M. and Perry E.B., "Effect of Horizontal Reinforcement on Stability of Earth Masses", U.S. Army Corps of Engineers, Technical Report 5-76-11, Sept. 1976).

Questions de M. INGOLD à MM. DELMAS et al.

1) In their evaluation of the coefficient A, equation 8, the authors take $\varepsilon_z = 0$. Does this imply shear at constant volume ? What value is given to dz ?

2) In Fig. 10 the authors show a peak pull-out force of approximately 5 kN. Assuming the width of the fabric sample used to be the width of the box i.e. 250 mm then this pull-out force relates to a load per metre width of 20 kN which appears to equal the R_T value given in Table 3. In view of this could the authors confirm that the peak load measured was limited by bond failure rather than tensile failure ?

3) Was the total thickness of the sand sample used in the pull-out test 40 mm, as used in the friction test, or was it the full depth of the box, i.e. 200 mm ?

4) In equation 8, the authors give an expression for the vertical stress σ_z. When x = 0 the constant λ assumes some non zero value, presumably $\bar{\sigma}_z$, the applied vertical. In fig. 13, however, the authors show σ_z decreasing to zero at x = 0. Would the authors please comment ?

MM. GOURC et GIROUD

Let us deal successively with the four points.

1) It is not true to state that $\varepsilon_z = 0$ in equation 8. The assumption concerning ε_z in equation 8 is $d\varepsilon_z/dx = 0$. This means that the loaded upper plate of the shear box remains horizontal during the test.

It is true to state that $\varepsilon_z = 0$ implies shear at constant volume. But such an assumption is valid only if the density of the granular material is the "critical density". This is the case for the sand used in the tests (ρ_d = 1 690 kg/m³), and it has been checked that the volume of this sand does not change during the shearing tests. Therefore the assumption $\varepsilon_z = 0$ has been made only for the numerical example presented in the last six lines of the paper.

In this numerical example, dz = 40 mm. It is reminded that the geotextile is in the middle of the soil layer, whose thickness is 2dz. Note that it would have been more suitable to write Δz instead of dz since it is not a differential.

2) It is true to state the pull out tension as measured in the test (Fig. 10) is approximately 20 kN/m. The values of R_T given in Table 3 are the minimum values obtained in series of uniaxial tests (strip tests). In the shear box, the state of stresses of the geotextile is triaxial. According to tests conducted by Mc Gown, triaxial strength is larger than biaxial strength. And it is well known that biaxial strength is larger than uniaxial strength. Therefore the strength of the geotextile in the shear box is much larger than the value of 20 kN/m as indicated in Table 3. A value of 25 to 30 kN/m is more likely. This confirms that the peak load measured was limited by bond failure rather than tensile failure.

In fact, after the completion of the test, no symptom of failure was observed on the geotextile. Moreover, it is very likely that the failure, if any, would occur in the portion of the geotextile which is actually pulled out of the box. This is because in this portion, the state of stresses is somewhat between uniaxial and biaxial (see in Fig. 3 necking of the geotextile outside the box). To prevent this type of failure, an anti-necking device has been recently added on the portion of the geotextile outside the box.

3) In the friction test, the total thickness of sand is 200 mm (100 mm above and 100 mm underneath the geotextile). In the pull out test, the total thickness of sand is 80 mm (40 mm above and 40 mm underneath the geotextile). The reason for a smaller thickness in the pull out test is to prevent excessive displacement

of sand. Such a displacement would result in a tilting of the loading plate of the shear box. Tilting of the loading plate would change the distribution of the stress σ_z applied by the loading plate. In a multilayer soil-goetextile system, tilting inside a layer cannot occur. Therefor it must be prevented in laboratory tests.

4) It is true to state that for $x = 0$, σ_z is not equal to 0. However, in the numerical example presented at the end of the paper, σ_z is very small for $x = 0$. In this example, the numerical values are :

dz : 0.04 m ;

$A = 13.39\ m^{-1}$ (from equation 8) ;

hence, for $x = l = 0.3$ m :

$\sigma_z = 55.5\ \lambda$.

Therefore, σ_z for $x = l$ is 55 times larger than σ_z for $x = 0$. This is the reason why σ_z seems equal to 0 for $x = 0$.

Question de M. MURRAY à M. JOHN

What was the length height ratio of the model ?

M. JOHN

For the series of models referred to in the paper, the width to height ratio was 0.5. The object of these tests was primarily to observe the planes of adherence failure and secondly to determine the critical height at which the safety factor fell below 1.0. Careful consideration was given to the effect of the width to eight ratio on these two observations. In the case of adherence failure of a reinforced earth model the tank side friction reduces the forces which make the wall stable as well as those causing it to fail. The effect on the safety factor is therefore likely to be less than observed in tests on other types of retaining wall. Calculations carried out by Bransby and Smith indicate that although there is an 11 % reduction in active pressure when the width to height ratio is 0.5, the effect on the failure plane should be negligible.

Before finalising the width of my model tank a series of tests was carried out using a tank whose width ratio could be varied between 0.3 and 3.0. These confirmed that although the critical height of the model increased as the tank became narrower, there was no alteration of the failure plane. It was concluded that a width to height ratio of 0.5 was an acceptable compromise between construction speed and side friction.

(Ref : Bransby P.L. and Smith I.A.A., The effects of side friction in model retaining wall experiments, Report No. CUED/C-Soils TR 17 (1973), University of Cambridge Department of Engineering).

Question de M. GASSLER à M. JOHN

Did you see that the figures 2a, b, c show a good example of the phenomenon of progressive failure and that you have to fullfill the model laws, when you are making the analysis for a full scale wall ?

Did you therefore make use of a model sand 07 granis diminuished in the geometric scale ?

M. JOHN

Model tests can generally be divided into two categories. The first type is concerned with predicting the behaviour of a specific prototype structure under working conditions from the behaviour of a model. The second type is an experiment where the model is considered as a prototype structure in its own right. In this case the model is used to test the theories of design and analysis so that they may be improved and applied with more confidence to other structures.

The series of model tests described in my paper fall into the second category for which the model laws do not have to be applied so rigidly as for the first category. These model laws, stated by Roscoe, are as follows :

1) Use the same soil at the same state for the model as for the prototype.
2) Use the same pore fluid.
3) Scale the selfweight of the pore fluid and each particle of soil inversely to the modelling scale.
4) Maintain geometric similarity.

If the grain size is modelled as suggested in the question it becomes impossible to comply with law one above, for any specific

structure. However the soil in a full size structure corresponding with the fine to medium sand used in my models would be a fine gravel which could well be used in a full scale wall.

Figures 2b and 2c demonstrate how errors can occur if failure is not observed at the instant of collapse. The important plane is the primary one in figure 2a because provided the reinforcement is sufficient to prevent this developing, the secondary planes do not have to be considered since they only occur after collapse has been initiated.

(Ref : Roscoe K.H., Soils and model tests, J. of strain analysis, Vol. 3, 1, 1968, pp. 57-64).

Question de M. MURRAY à M. FINLAY

How is the energy method applied with more complex situations - say a concentrated load for example ?

M. FINLAY

The energy theory expressions presented in the paper were derived on the assumption that only the self weight of the wall was acting.

When more complex loading situations arise it is necessary to obtain mathematical expressions for the deflected shape of the wall and the pressure distribution behind the wall. These equations can be incorporated in the generalised energy relations and formulae similar to those presented in the paper can be obtained for the particular loading case.

Question de M. JURAN à M. FINLAY

Your method is based on energy considerations and therefore it is mainly based on elasticity while the method we have presented is based on limit analysis. Would you consider that your method predicts failure behaviour or that the method we have proposed may be also used to predict the structure behaviour before failure ?

M. FINLAY

The proposed method of analysis establishes energy relations from the elastic deformations of the wall facing and the ties. Since it is based on an assumption of elasticity in the reinforced earth mass, it can be used to predict the wall behaviour prior to failure.

In model studies it is useful to predict the wall behaviour at failure, and an elasto-plastic assumption has been adopted which allows relationships to be obtained for predicting the wall behaviour at failure.

Thus the energy method can be used to predict wall behaviour under working conditions as well as at failure.

Question de M. HOSHIYA à M. HAUSMANN

The seismic stability of a reinforced earth structure may depend greatly upon the type of ground motion. Could you give us comments on this point ? (Conceivably I believe that an impact type of acceleration force with quite high intensity and with rather short duration may be most severe against the structural stability).

M. HAUSMANN

In their test series with double-sided walls the authors did not study any other types of motion than that described in the paper.

In most model studies carried out previously, sinusoidal type horizontal shaking was used and the effect of various frequencies and base accelerations were studied. Hausmann (1978) carried out dynamic tests with a rigid rotating model wall to which reinforcing strips were attached. Observing the rate of outward movement of the wall at or near the resonant frequency, it was found that as the acceleration increased to a "critical" value, the wall would move so rapidly, that overturning failure was imminent. This critical acceleration increased with increasing length of reinforcement.

(Ref : Hausmann M.R., "Static and Dynamic Behaviour of Model Wall with Reinforcement", Symposium on Soil Reinforcing and Stabilising Techniques in Engineering Practice, Sydney, Australia, October, 1978).

Commentaire de M. INGOLD

I would like to clarify a few points concerning the derivation of the expression for average strength ratios in my paper. At present there is no quantitative information available regarding the gradient of radial shear stress across a horizontal disc of reinforcement in a cylindrical sample of reinforced earth. Since by definition the free surface of a cylindrical sample is a principal plane it follows that the radial shear stress at the periphery must also be zero if equilibrium is to be maintained. This reasoning leads to the conclusion that there must be a steep negative radial shear stress gradient near the periphery. This fact is not taken into account in the theory presented, it was assumed, for the sake of simplicity, that radial shear stress increases linearly from the centre of the sample. In the paper the average value of the strength ratio was determined by integration over the radius of the sample. However, in view of the uncertainties mentioned above an average value could equally well be determined by integration over the area of the sample. In this case, Equation 16 (page 60) becomes :

$$\frac{\sigma_3' + \Delta\sigma_3'}{\sigma_3'} = \frac{2\alpha Ka}{\tan\delta}\left(e^{\tan\delta/2\alpha Ka} - 1\right) \ldots \quad (16a)$$

Similarly Equation 24 (page 61) becomes :

$$\frac{(\sigma_1 - \sigma_3)\, r}{2\ Cu} = \frac{2\alpha}{\tan\delta}\left(e^{\tan\delta/2\alpha} - 1\right) \ldots \quad (24a)$$

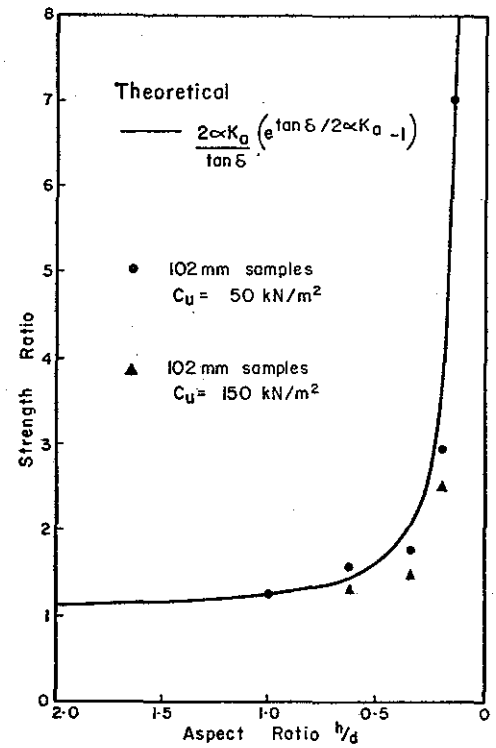
This approach can, of course, be extended to the undrained condition. If it is again assumed that there is a linear mobilisation of shear stress across the disc radius then Equation 29 (page 61) becomes :

$$\frac{(\sigma_1 - \sigma_3)\, r}{2\ Cu} = 1 + \frac{\mu}{8\alpha} \ldots \quad (29a)$$

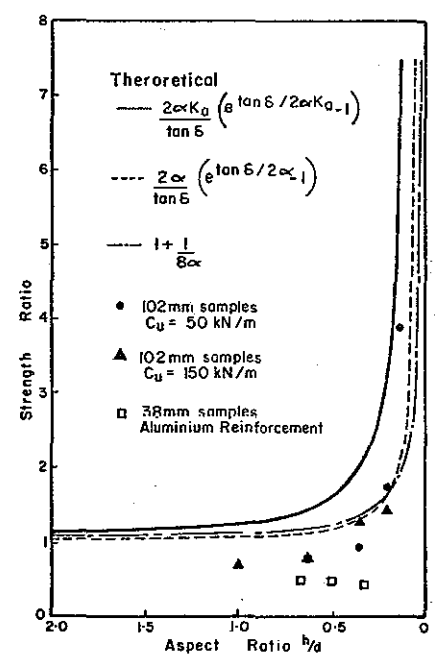
At large vertical strains it is possible for the clay to undergo plastic flow. Under this circumstance it could be argued that the upper limit for the average shear stress mobilised is μCu. Based on this assumption and averaging over the area of the sample Equation 29 (page 61) becomes :

$$\frac{(\sigma_1 - \sigma_3)\, r}{2\ Cu} = 1 + \frac{\mu}{6\alpha} \ldots \quad (29b)$$

As can be seen from figures 1a and 1b integration over the area of the sample infact gives a better fit between the theory and test results.



(a) DRAINED SHEAR TESTS



(b) RAPID SHEAR TESTS

Commentaire par MM. GOURC et GIROUD

En réponse à une question qui nous a été posée plusieurs fois, nous apportons une

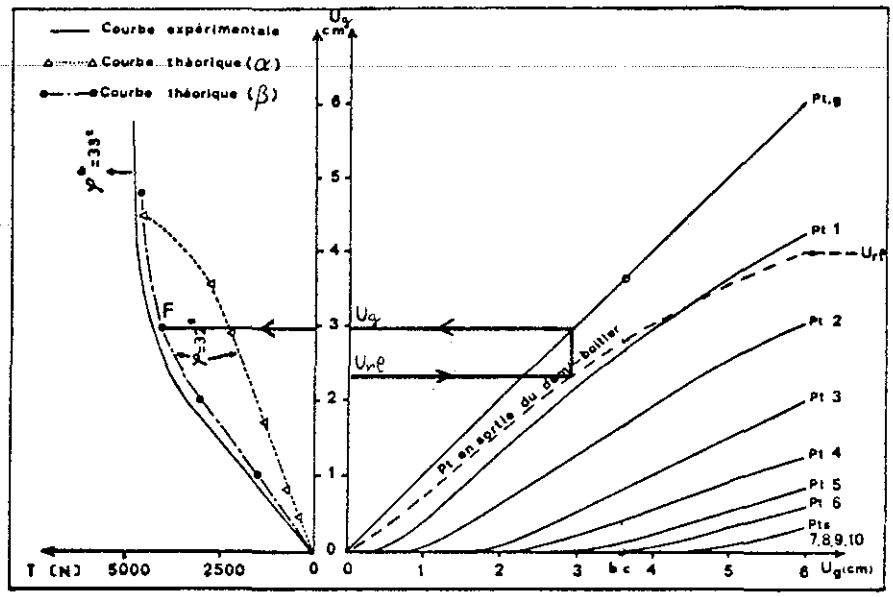


Fig. Essai de traction d'un géotextile nontissé HT400 entre deux couches de sable (σ_z = 50kN/m2).

précision concernant la figure 10 de notre article. La question porte sur le mode d'obtention de la courbe β sur la partie gauche de la figure 10.

Il convient d'abord de préciser deux définitions :

U_g : déplacement de l'extrémité du géotextile ;

U_{rl} : déplacement, à un instant donné, du point du géotextile qui se trouve à l'extrémité de la boîte.

La construction de la courbe β se fait ainsi. Etant donné une valeur de U_{rl} (par exemple U_{rl} = 2.4 mm sur la figure ci-jointe), on en déduit graphiquement la valeur de U_g (soit U_g = 3.0 mm sur la figure ci-jointe). D'où T_1 par la formule 11 (avec A = 13.39 m^{-1} d'après la formule 8). Soit, T_1 = 3 700 N, d'où le point F sur la figure ci-jointe.

Commentaire de M. MURRAY

In their paper entitled "Theoretical study of traction forces in slabs of reinforced earth structures", Messrs Juran and Schlosser have presented comprehensive details of measurements of peak tension on the reinforcing elements for both model and full-scale structures from a range of sources. Although the results generally relate to structures operating at their working load, the authors have assumed that the locus of peak tension corresponds to the failure surface and have shown that such a surface can be reasonably approximated by a logarithmic spiral.

Clearly, curved failure surfaces are likely to arise more generally with all types of retaining structure but nonetheless the simple plane surface is still widely used for conventional design because solutions to many problems can be more readily obtained and also because it is widely held that frequently little is lost in terms of accuracy. It is the writer's contention that this could well be true also in the case of reinforced earth. Moreover, the conditions in a structure at the working load may be rather different from those at failure. As shown in the figure, the tension failures of reinforcements in a series of centrifuge model tests were coincident with a plane surface of failure and it is possible that in some cases the position of the locus of peak tension could be altered significantly as failure approaches.

Conclusions de M. KERISEL

Mesdames et Messieurs, nous voici arrivés au terme de cette session n° 1, qui aura été particulièrement chargée, comme vous le constatez. Je pense qu'on ne peut être qu'extrêmement frappé par la clarté des exposés des rapporteurs généraux, par la pertinence des questions posées, et il vous apparaîtra tout de suite qu'il est impossible de résumer une séance aussi riche en quelques phrases. Je voudrais d'abord remercier non seulement les rapporteurs généraux, mais aussi Monsieur Murray et Monsieur Baguelin qui m'ont assisté dans ma tâche. Il me paraît impossible d'autre part de clore cette séance sans avoir une pensée particulière pour un auditeur anonyme et modeste qui a été présent dans cette salle, tout l'après-midi, j'ai nommé Monsieur Vidal. Il nous a déclaré ce matin : "J'ai passé 5 ans à me convaincre, et j'ai passé ensuite 5 ans à convaincre les autres". Si j'étais à sa place, assis dans cette salle, je constaterais certainement que ce qu'il a semé a produit une moisson d'une richesse exceptionnelle, moisson d'idées, moisson de recherches nouvelles auxquelles ont conduit les idées qu'il a répandues. Et avant de lever la séance, je pense être votre interprète à tous, en lui exprimant notre reconnaissance et notre chaleureuse sympathie. (Applaudissements nourris).