Soil Reinforcement

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SOIL REINFORCEMENT

General Report

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I - INTRODUCTION

Soil Reinforcement is a special and recent field of soil improvement. It covers a range of techniques which consist of placing resisting inclusions in the soil.

Among the different conferences devoted partly or totally to soil improvement, those related to this special field have been the following :

- International Symposium on Soft Clays, Bangkok (1977)
- Symposium on Earth Reinforcement, Pittsburgh (1978)
- International Conference on Soil Reinforcement, Paris (1979)
- 8th European Conference on Soil Mechanics and Foundation Engineering, Brighton (1979)
- International European Conference on SMFI, Stockholm (1981)
- Second International Conference on Geotextiles, Las Vegas (1982)
- International Symposium on Soil and Rock Improvement, Bangkok (1982)

Soil reinforcement was pioneered by Henri Vidal who invented and developed in the early sixties the technique of Reinforced Earth. Soil reinforcement is, however, now accepted as a more general concept which includes such techniques as micro-piles, stone columns, in-situ stabilised columns, soil nailing, Texsol, membranes, etc...

Depending on the type of the inclusion two extreme cases can be considered :

1) a "uniform inclusion" where the soil-reinforcement interaction can develop in any point along the inclusion ;
2) a "composite inclusion" which consists of an inclusion reinforced in some particular points where the soil-reinforcement interaction is concentrated. Generally as in the case of anchorages these points are located at the extremities of the inclusions.

In the case of a "uniform inclusion" a relatively high and uniform density of the reinforcements will result in a new composite material called the "Reinforced Soil". The behaviour of the "reinforced soil" mass can be investigated considering a representative sample of the new composite material. This concept is illustrated in Fig. 1. The reinforced earth mass is a composite material and its apparent mechanical properties can be determined from laboratory tests on representative samples. On the contrary, the "ladder wall" invented by Coyne, is a multi-anchorages system where the soil-reinforcement interaction is concentrated in the extremeties of the ties.

These considerations lead to the classification of soil-reinforcement systems presented in table I.

Type of reinforcement / Density of reinforcement	uniform	composite
multiple	reinforced soil	multi-anchorages systems
isolated	membranes piles	anchorages

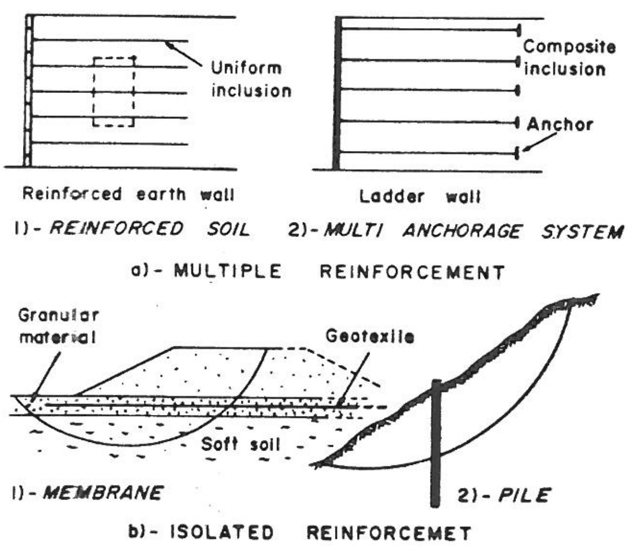


Fig:1 - TYPES OF SOIL REINFORCEMENT SYSTEMS

classification	REINFORCED SOILS						multi anchorages	isolated uniform inclusion
techniques or systems / applications	stone columns	Reinforced Earth	Soil Nailing	Micro-piles	Multimembranes or Multigrids	Special Systems	Ladder Wall	Isolated membrane or grid
FOUNDATIONS	10	1			2	1		8
WALLS		3	3	1			1	
SLOPE STABILIZATION			2					
	2 papers deal with soil-reinforcement interaction 1 paper deal with general applications of geotextiles							

TABLE 2 - CLASSIFICATION OF PAPERS SUBMITTED TO THIS SESSION

Thirty five papers submitted to this conference are related to the Soil Reinforcement. They cover most of the soil-reinforcement systems discussed above and can be classified considering the different techniques and applications as indicated in table2.

The present report summarizes the new aspects of the state of the art and reviews the papers submitted to this conference. It describes briefly the different techniques and develops more particularly the following points :

- Soil reinforcement interaction
- Behaviour and design methods
- Case histories and control

The behaviour and design methods are discussed considering the different types of applications : retaining walls, slope stabilization, shallow foundations with reinforcements, in-situ reinforced soil foundations. The section dealing with shallow foundations with reinforcements has been prepared by the co-reporter Dr. H.M. JACOBSEN.

2 - DESCRIPTION OF THE TECHNIQUES AND SOIL-REINFORCEMENT INTERACTION

2.1 Description of the techniques and major efforts

The inclusions used for soil-reinforcement are resisting elements which are generally either linear or plane. Depending on their relative rigidity with respect to the soil, their behaviour is rather similar to that of a beam or an armour-plate where they are relatively rigid and to that of a threat or a membrane when they are relatively flexible. Consequently, the major efforts mobilized in the inclusions can be of four types : tension, compression, bending and shearing.

Table 3 shows these major efforts considering the different soil reinforcement systems. In fact the mobilization of these efforts depends on a large variety of parameters including the relative rigidity of the inclusions, their orientation, their density, the structure geometry, the construction process, the mechanical properties of the inclusions and the soil, etc...

As shown in table 3, most of the available techniques can be classified as reinforced soil systems. Accordingly these types of systems will be more particularly described considering the different fields of application.

In order to improve soil foundations the soil-reinforcement techniques commonly used are :

- Stone columns. This technique is used in soft grounds and the reinforcing inclusion is a vertical column of highly compacted sand, gravel or agregates. Generally, the installation of the column comprises two main stages: 1) a casing pipe or a vibrating device is driven into the ground down to the designed level driving away the surrounding soil; 2) the system is then drawn up progressively and the cavity is being filled with granular material highly compacted statically or by vibrations. The major role of the column is to increase the resistance and the modulus of the foundation soil, moreover it also constitutes a vertical drain. This inclusion is relatively flexible and can therefore withstand essentially compression. However when the stone column is used to improve the stability of a foundation soil with respect to a general sliding it also increases significantly the shearing resistance of the reinforced soil.
- In-situ stabilized columns. The efficient use of stone columns has led to the development of a similar approach which consists of creating in-situ stabilized columns. Different techniques have been developed including : in-situ lime stabilization (Broms 1975), in-situ stabilization by jet grouting (Yahiro and Yoshida, 1978) ; compaction grouting piles (Baker, 1981). Compared with stone columns these inclusions are generally more rigid and can withstand both compression, bending and shearing.
- Micro piles. This technique consists of installing in the soil small reinforcing grouted piles. Each pile is made of a bar or a tube of a few centimeters diameter surrounded by a grout all along its length. The total diameter is of about 10 to 15 cm. This technique has been already used for about thirty years mainly in foundation soils but it has also interesting applications in slope stabilization, in-situ retaining structures and underpinning. This inclusion is rather rigid, however the mobilized efforts depend essentially on the structural effect of the group of piles and the major efforts are generally tension and compression.

Considering retaining structures, the soil reinforcement techniques commonly used are :

- Reinforced Earth. This technique consists of associating a granular frictional backfill material with flexible, linear, reinforcing strips of a high tensile resistance. The outside facing of the structure is relatively thin and flexible and is usually made of concrete panels. To limit the deformation of the structure, relatively inextensible reinforcement have to be used. Consequently, the inclusions are generally made of steel, galvanized to insure its protection against corrosion.
- Multi membranes and grids. The rapid development of Reinforced Earth has more recently led to use different types of inclusions in soil-reinforcement systems, including mem-

Soil Reinforcement system	Reinforced Soils								Multi composite inclusion		isolated inclusion				
Technique or system / major effort	Stone columns	In situ stabilized column	micro piles	R.E.	Multi membranes or grids	Soil Nailing (slope)	Soil Nailing (walls)	Texsol	Ladder Wall	Multi active anchorages	Membranes or grid	pile (slope)	anchorage		
tension			**	***	***		**	***	***	***	***		***		
compression	**	**	**												
bending		*	*			**	*					**			
shearing	*	*	*			**	*					**			

TABLE 3 - Major efforts in the inclusions of soil-reinforcement systems

branes and geotextiles. The geotextile membranes can withstand only tensile forces. The major difference between steel or strong plastic and geotextile is their deformation characteristics which influence significantly the lateral deformations of the structure. Although some attempts have been done to use geotextiles in retaining structures with vertical facings they are usually used to reinforce embankment slopes and soil foundations. Considering grids they are generally made of metal or strong plastics and can therefore be efficiently used to restrain the lateral deformations of the structure.

- Soil Nailing. It is an in-situ soil-reinforcement technique by passive bars which are either placed in boreholes and grouted or simply driven into the ground. When the technique is used in retaining structures the bars are generally horizontal and the major effort is tension. On the contrary, when this technique is used for slope stabilization the bars are generally vertical and the major efforts are bending and shearing.

- Texsol. This new technique invented by Leflaive (1982) consists of reinforcing a granular backfill material by a continuous fiber resisting to tension.

Among the techniques which have not been classified in soil-reinforcement systems it is interesting to describe the ladder wall system which has been invented in 1926 by Coyne. It is a multi-tied back system associated with a thin facing which can be made either of concrete panels or of a continuous wall. The ties withstand tensile forces which are constant along the ties. The soil-reinforcement interaction is being essentially realized by the passive lateral earth thrust on the anchors. More recently different similar multi anchorages systems have been developed (anchored-earth - Murray 1981).

2.2 Soil-reinforcement interaction

The mobilization of the efforts in the inclusions of the different soil-reinforcement systems described above involve essentially four types of interaction mechanisms : I - lateral friction along the inclusion ; II - lateral earth pressure on the inclusion ; III - passive earth thrust on cross elements of composite inclusions ; IV - passive earth confining pressure on stone columns.

2.2.1 Lateral friction

The mobilization of lateral friction along piles and reinforcing inclusions has already been extensively studied and summarized by several authors (Baguelin et al, 1975 ; Schlosser and Guilloux, 1981 ; etc...). These studies have shown that in compacted granular soils the soil-reinforcement friction depends on a large variety of parameters (surface characteristics of the inclusion ; density and mechanical properties of the soil ; normal stress on the inclusion, etc...) and particularly on the dilatancy behaviour of the soil.

Fig. 2 illustrates the mechanism of soil-inclusion in a dilatant soil. As demonstrated experimentally by Bacot (1981), using a photometric technique on a two-dimensional model, the pull-out of an inclusion induces shear displacements in a zone of the surrounding soil. The volume of this zone depends significantly on the state of the surface of the inclusion. In a compacted granular soil around a reinforcing strip the sheared zone tends to dilate but this volume change is being restrained by the surrouding soil. This restraining effect results in an increase of the normal stresses on the strip.

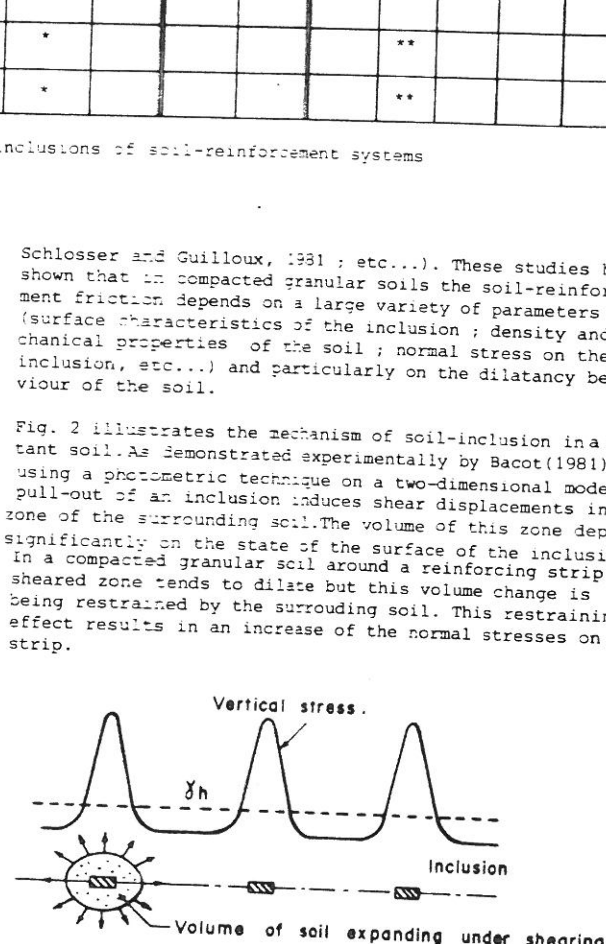


Fig 2 - MECHANISM OF SOIL-INCLUSION INTERACTION IN DILATANT SOIL

This effect of restrained dilatancy on the increase of the normal stresses on the inclusion has been demonstrated experimentally by Wernick (1978). He has carried out pull-out tests on cylindrical steel pipes (2,5 m long and 5 to 10 cm diameter) instrumented with cells to measure the shear stresses τ and the normal stresses σ. The pipes were placed in a large round test bin filled with compacted sand (γd = 17.14 KN m3). Fig. 3a shows a typical stress path during a pull out test (loading and unloading). Starting from Ko conditions the state of stresses attains rapidly a limit equilibrium and the normal stress increases progressively up to a value which is about 8 times the initial value. At failure, due to a strain softening there is a decrease of the mobilized shear stress. The unloading stress path shows that there is no residual dilatancy and consequently the final state of stresses corresponds to the initial one. In order to investigate this dilatancy effect Guilloux and Schlosser (1979) have carried out constant volume direct shear tests on compacted sand (γd = 17.3 KN/m^3). As illustrated in Fig. 3b the stress path followed in this test is quite similar to the one represented in Fig. 3a.

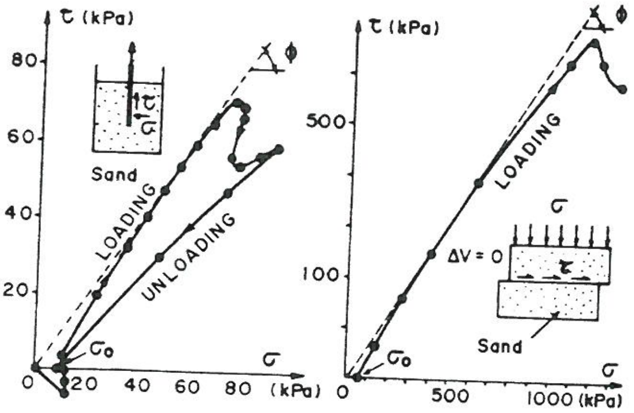


Fig 3 - EFFECT OF RESTRAINED DILATANCY IN COMPACTED GRANULAR SOIL.

It is interesting to note that the constant volume direct shear test represents the extreme case of a restrained dilatancy whereas in the pull-out test a limited expansion is possible. Consequently, the limit state of stresses in the constant volume direct shear test corresponds to the critical state value of ϕ_{cv} angle whereas the limit state in the pull-out test is characterized by the value of ϕ at peak. It is also interesting to note that the increase of the normal stress in this test is larger than in the pull-out test ($\sigma \cong 14\ \sigma_o$).

The paper of Koirumaki (Helsinki Conference) analyses the effect of dilatancy on the friction angle between sand at different densities and aluminium plates.
The author has carried out series of direct shear tests at a constant normal stress. Using the energy equation he has shown that the part of the friction angle due to dilatancy is decreasing with an increase of the normal stress. However, direct shear tests at a constant normal stress do not represent the complex phenomenon of a restrained dilatancy which occurs around the reinforcing strips in actual structures.

In fact as demonstrated by both laboratory model tests and full scale experiments the restrained dilatancy effect decreases with the normal stress. This has led to consider for design purposes an apparent friction coefficient μ^* which is defined as the ratio of the maximum shear stress along the inclusion to the initial normal stress σ_o $\mu^* = \tau max/\sigma_o$ (Schlosser and Guilloux, 1979). This apparent friction coefficient is highly dependent on the dilatancy behaviour of the soil. It can attain values which are much larger than the internal friction angle of the soil and is decreasing with the increase of the normal stress.

Gigan and Cartier (Helsinki Conference) have carried out pull-out tests on driven metal profiles used as reinforcements in a nailed soil retaining wall. They have demonstrated that the values of the apparent friction coefficient determined from these tests agree fairly well with those suggested by Schlosser and Guilloux for the design of Reinforced Earth walls. However it should be noticed that in soil nailing the initial normal stress on the inclusion is difficult to determine because of the geometry of the structure and the inclination of the inclusion. Consequently for practical considerations it has been proposed (Schlosser, 1983) to use the value of the limit shear stress along the inclusion ($\tau max = \mu^* . \sigma_o$) which is approximately constant with depth as demonstrated by different authors and confirmed by the observations of Cartier and Gigan (Fig. 4).

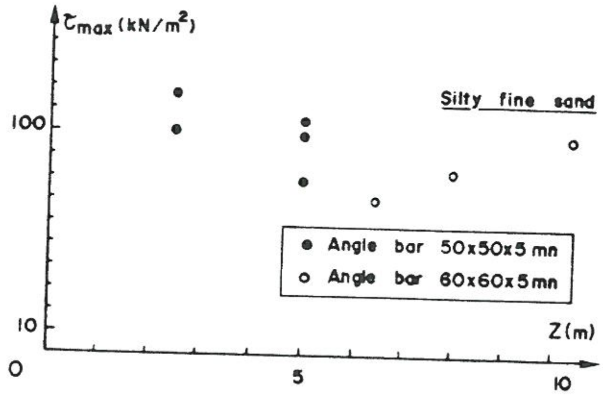


Fig. 4. VALUES OF THE LIMIT SHEAR STRESS ALONG REINFORCEMENT BARS (After Cartier and Gigan, 1983).

Fig. 5 shows that a relatively small displacement (few millimeters) is sufficient to generate the limit shear stress along smooth inclusions. These results agree with observations on both piles and Reinforced Earth. However the mobilization of the soil-inclusion friction in compacted granular soils depends on the volume of the sheared dilatant zone around the inclusion and consequently on the state of the surface of the inclusion. Thus in the case of ribbed strips (Fig. 5b) a large displacement of about 5 to 10 cm is necessary in order to attain the peak value.

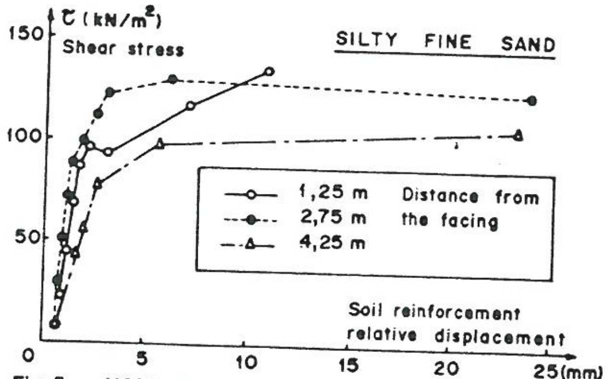


Fig 5a - MOBILIZATION OF THE FRICTION IN A PULL-OUT TEST OF A TUBE.
(Cartier and Gigan, 1983)

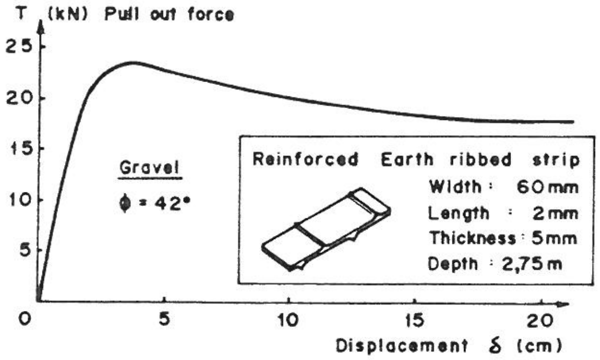


Fig 5b - PULL OUT TEST OF A RIBBED STRIP (Schlosser and Guilloux,1979)

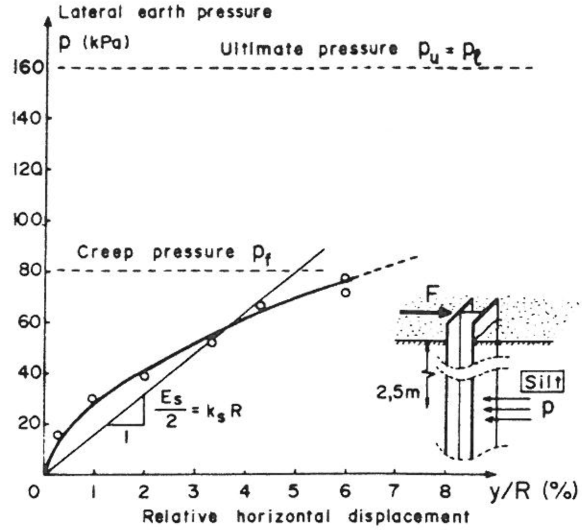


Fig 6 - REACTION CURVE (Baguelin and Jezequel,1972)

2.2.2 Lateral earth pressure on the inclusion

The development of the lateral earth pressure on a linear inclusion located inside of a soil mass requires a relative rigidity of the inclusion as well as a shear zone in the soil. Two different mechanisms can be considered :

1) the mobilization of the lateral earth pressure under static conditions when a static equilibrium is reached.

2) the mobilization of the lateral earth pressure under soil-creep conditions, when inclusions are used in order to decrease the rate of creep of a sliding slope.

The mechanism of lateral earth pressure under static conditions has been widely studied for piles subjected to horizontal loading (Brinch Hansen 1961, Matlock and Reese 1960, Ménard 1962, Broms 1967, Baguelin and Jezequel 1972). Experiments and calculations have shown that the concept of a local reaction curve, relating the pressure p_y at the front of the pile to the relative displacement $\frac{y}{R}$ (R : radius of the pile), is valid, provided that the curvature of the deformed pile is not too small. Figure 6 shows an experimental reaction curve which gives initial and secant values of the soil modulus $E_s = 2k_sR$ (k_s : subgrade reaction modulus) and which is limited by the ultimate pressure p_u. These two parameters are only depending on the soil and not on the pile.

A prediction of the reaction curve has been proposed by different authors and particularly by Menard (1962, 1969) and Matlock (1970). Menard's prediction is based on the pressuremeter curve which appears to be in a good agreement with the reaction curve because of the similarity between the phenomenon of a cavity expansion within a soil and the lateral earth pressure mobilization in an horizontal loading. It is assumed that the ultimate pressure p_u is equal to the limit pressure p_l. Other proposed values of p_u are not so easily related to the mechanical parameters of the soil ; for instance Broms (1964) gives the following formula :

Cohesionless soil : $p_u = p_o \, 3 \, tg^2 \left(\frac{\pi}{4} - \frac{\phi}{2}\right)$
Coherent soil : $p_u = 9 \, c_u$

The similarity between pressuremeter curve and reaction curve suggests that the deformation $\frac{y}{R}$ corresponding to the ultimate pressure p_u would be very high, about 100 %. However the large deformations due to plastic zones around the piles begin at approximately $p = p_f$, which corresponds to a relatively low value of the deformation (y/R =5 to 10%).

Baguelin and Jezequel (1982) have shown experimentally that the lateral earth pressure p is essentially due to the reaction of the soil at the front of the pile and that only a small part is provided by the reaction at the back and by the friction on the two lateral sides.

Considering the mechanism of the lateral earth pressure on a linear inclusion in static equilibrium, it appears that the ultimate pressure p_u is difficult to obtain for inclusion of diameter greater than 2 or 3 cm because of the relatively large required displacement (1 to 1.5 cm). Moreover experiments on piles have shown that the placement method which disturbs the soil around the pile has a great influence on the soil modulus E_s and consequently on the displacements necessary to mobilize the ultimate pressure p_u.

Creeping soil : The lateral earth thrust which develops on relatively rigid inclusions used to stabilize unstable slopes of sliding ground depends mainly on the state of the ground in the immediate vicinity of the potential sliding surface. In the case of a creep state the main role of the inclusion is to reduce the distortion rate $\dot{\gamma}$ in the sheared zone. As shown later the lateral earth thrust mobilized at the ground-inclusion interface depends both on the gradient $d\dot{\gamma}/dz$ of the distortion rate in the sheared zone and on the level of $\dot{\gamma}$. To describe this interaction mechanism it is possible to use the law of the plastic flow of the soil proposed by Leinenkugel (1976) and considered by Winter et al (Helsinki Conference). According to this law the ultimate shear stress τ_l in the sheared zone is related to the distortion rate ($\dot{\gamma}$) by the so called viscosity index I_{v_o}:

$$\tau_l = C_u \, (\dot{\gamma}_o) \left[1 + I_{v_o} \, \ln \left(\frac{\dot{\gamma}}{\dot{\gamma}_o}\right) \right]$$

where : C_u ($\dot{\gamma}_o$) denotes the undrained cohesion corresponding to a reference distortion rate $\dot{\gamma}_o$. The viscosity index can be determined from undrained triaxial shear tests on saturated consolidated soil samples.

By decreasing the distortion rate of the soil in its surrounding the inclusion reduces the ultimate shear stress τ_l mobilized in the shear zone. Considering a representative layer of a thickness dz in the sheared zone the static equilibrium conditions implicates that when the slope is unreinforced the ultimate shear stress τ_l must be equal to the driving shear stress τ_o whereas in the reinforced slope the change of the ultimate shear stress $\Delta\tau_l$ in the soil surrounding the inclusion must be in equilibrium with the lateral earth thrust on the inclusion. Thus, considering a representative segment of the inclusion, as illustrated in Fig. 7, the soil-inclusion interaction can be described by the following equation :

$$p.B.dz = d\,\tau_l \,.\, S^{eq}$$

$$p = \frac{S^{eq}}{B}\,\frac{d\tau_l}{dz} = \frac{S^{eq}}{B}\,\frac{k}{\dot{\gamma}}\,\left(\frac{d\dot{\gamma}}{dz}\right)$$

$$\text{with : } k = Cu\,(\dot{\gamma})\; I_{v_o}$$

where $d\tau_l$ is the average change of the limit shear stress on the equivalent surface of influence S^{eq} of the inclusion and B is the diameter of the inclusion.

This equation shows that locally the lateral earth thrust under creep soil conditions depends on $d\dot{\gamma}/dz$ and on $\dot{\gamma}$.

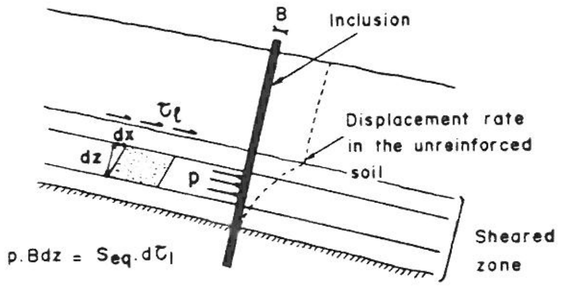


Fig 7 - LATERAL PRESSURE ON INCLUSION IN CREEPING SLOPE .

2.2.3 Passive earth thrust on cross elements of composite inclusions

Cross elements of composite inclusions can be either perpendicular plates as in the case of "Ladder walls" or transversal bars as in the case of grids or of "Anchored Earth" reinforcements. A passive lateral earth pressure is developing against these cross-elements and the mechanims is rather similar to the one described in the previous section.

However it is interesting to know the distribution of the resisting force in a composite reinforcement and particularly the part taken by the cross elements (passive earth thrust) and the part taken by the longitudinal bars (friction). Generally the part due to friction along a smooth longitudinal bar with an anchor at its extremity is small. In the case of grids, Bacot (1981) has shown that friction is the essential phenomenon at low values of the relative soil-reinforcement displacement (0,5 cm) and that passive earth thrust is only mobilized at large values of this displacement. He has performed pull-out tests in a large box filled with compacted sand on different types of reinforcements, 5 m long.

The results are presented on fig. 8. There is a difference in the maximum pull-out force between a smooth bar and a composite bar with very small transversal elements of 2 cm long, due to the "rib effect" and consequently the dilatancy effect. However there is no difference if the length of the transversal bars is increased (fig. 8). Unfortunately, all the pull-out tests have been stopped at a displacement value of about 0.5 cm after the peak has been reached. Only one test has been performed until large displacements but on a slightly different reinforcement (two longitudinal bars spacing of 15 cm and transversal bars, spacing of 20 cm) and with a different soil (gravel). It shows (fig. 9) two stages in the pull-out mechanism : first the friction

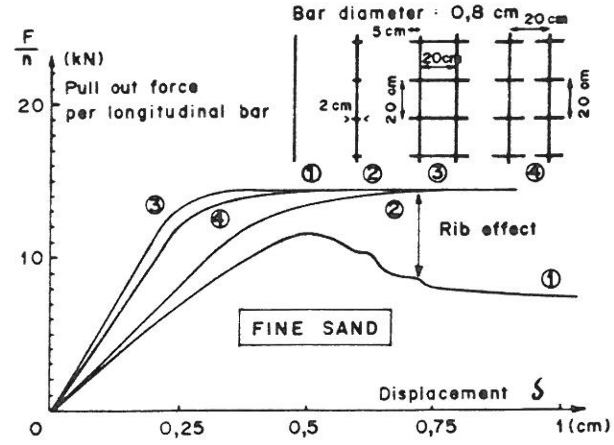


Fig 8 - PULL OUT TESTS ON BARS WITH TRANSVERSAL ELEMENTS (After Bacot, 1981)

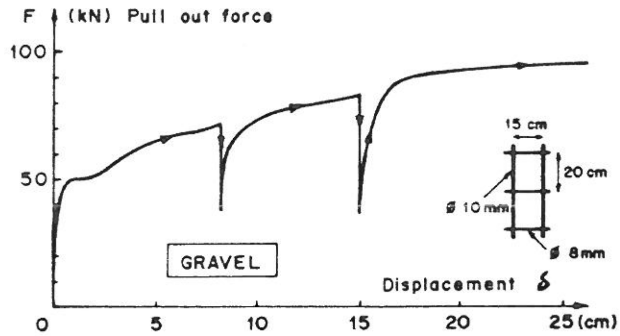


Fig 9 - PULL OUT TEST ON A GRID (Bacot, 1981)

and the rib effect are mobilized at a very small displacement 0.5 cm, leading to a first maximum in the pull-out force displacement curve, then there is a progressive increase of the pull-out force due to the mobilization of the passive earth thrust on the flexible transversal bar leading to a second maximum at a displacement of 25 cm.

2.2.4 Passive lateral confining pressure on stone columns

Stone and sand columns are inclusions which can withstand both compression and shearing. They are generally used as compression resistant reinforcement but their response to loading is mainly controlled by the confining pressure mobilized in the surrounding soft soil to restrain their bulging.

The soil-column interaction can be considered, in a first approximation, as a plane phenomenon and consequently the mobilized confining pressure q is a function of the radial strain ε_r at the interface. Considering an isolated column in a semi-infinite soil this relationship $q = f\ (\varepsilon_r)$ can be approximated by the pressuremeter curve (Hughes et al, 1975). However in practice when a foundation reinforced by stone columns is uniformly loaded the group effect modifies the boundary conditions and requires a zero lateral strain at the limit of the tributary area of each column. This boundary condition can result in a large increase of the mobilized confining pressure. This aspect has been considered by different authors (Priebe, 1976 ; Goughnour et Bayuk, 1979) and has been the basis for the development of the concept of a "unit cell" containing the column and its surrounding tributary soil. In laboratory this concept can be studied in a special oedometer with a central column (Aboshi et al, 1979) but the determination of the relationship $q = f(\varepsilon_r)$ would require the use of a mini-pressuremeter at the location of the column.

3 - BEHAVIOUR OF STRUCTURES AND DESIGN METHODS

3.1 Retaining structures

3.1.1 Reinforced soil retaining structures

The behaviour of reinforced soil retaining structures depends on the extensibility characteristics and on the relative rigidity of the inclusions. Among the techniques mentioned above Reinforced Earth presents the case where the inclusions are linear, inextensible and completely flexible. The behaviour of this system has already been studied in details both on laboratory models and full scale experiments. Fig. 10 illustrates the fundamental aspects of the behaviour of a Reinforced Earth retaining wall. The locus of the maximum tensile forces in the reinforcing strips separates an active zone, close to the facing and a resistant zone. This locus which represents a potential failure surface is quite different from the classical Coulomb's failure plane in retaining walls. The distribution of the maximum tensile forces is also quite different from the trinangular active earth pressure predicted by Rankine's theory. These differences have been explained considering the effect of the inextensible inclusions both on the stresses field (Schlosser, 1969) and on the strains field (Bassett, 1978) which develop in the backfill material. The inclusions restrain the lateral deformations of the structure and maintain the soil in the active zone in a more elastic state of stresses. Consequently, the maximum tensile forces at the upper part of the wall corresponds to the lateral earth pressure at rest.

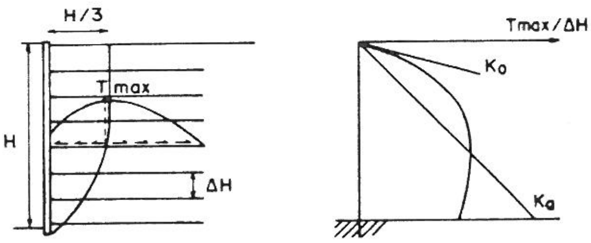


Fig:10 - BEHAVIOUR OF A REINFORCED EARTH RETAINING WALL.

In reinforced earth walls the reinforcing linear strips are usually made of galvanized steel. John and Peteley (Helsinki Conference) present the measurements of tensile forces distributions along linear strips made of paraweb in two reinforced soil structures : the Portsmouth wall - 2,5 m high and the Jersey wall - 8 m high. The paraweb strips are constituted of polyester fibers coated with plastic. The admissible working stress for the paraweb strips is about 200 MPa of the same order, as the admissible working stress for the steel strips which is about 160 MPa. However the paraweb is much more extensible than the steel and consequently it can be reasonably expected that the lateral deformations of the wall with paraweb strips would be larger than those expected in reinforced earth walls.

Fig. 11 presents the maximum tensile forces distribution measured by John and Peteley. In the upper part of the wall this distribution is close to the K_a line. This suggests that the lateral deformation of the structure are large enough to attain the K_a state of stresses in the soil and consequently the tensile forces measured in the paraweb strip in the upper part of the wall are smaller than those measured in steel strips in the upper part of reinforced earth walls. Marczal (Helsinki Conference) reports the results of a full scale experiment on a 6 m reinforced earth wall using relatively inextensible linear strips made of glass fiber reinforced polyester. The measured tensile forces are larger than those calculated considering a K_a line distribution.

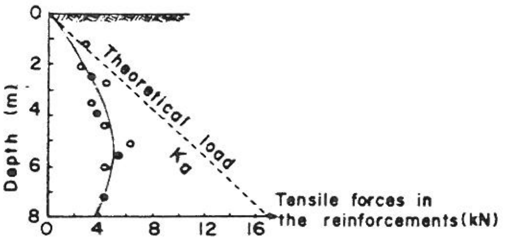


Fig:11 - TENSILE FORCES IN A REINFORCED SOIL WALL. (John and Petley, 1983)

Considering inextensible inclusions a limit analysis method has been developed (Juran, 1977) and used to interpretate the two failure modes : breakage of the strips and sliding of the strips in the resistant zone. Present design methods for Reinforced Earth walls (Schlosser et al, 1979) integrate both these theoretical results and observations on full scale structures.

In-situ retaining structures built using soil nailing present three major differences with reinforced earth walls : 1° - the in-situ soil has generally a cohesion ; 2° - the inclusions when installed as micro piles, present a certain rigidity to bending which affect the behaviour of the structure ; 3° - the construction of the wall is realized as an excavation starting at top and consequently the stress history is different.

A few number of experimentations has been published (Stocker et al 1979 ; Gässler and Gudehus, 1981 ; Shen et al, 1981) but partial observations on actual structures have been frequently reported during the last decade. They have shown the development of active and resistant zones considering the distributions of the tensile forces along the reinforcements. Although the maximum tensile forces line is difficult to determine it seems to be different from that of reinforced earth wall due to some factors, including : larger horizontal displacement at the top ; cohesion of the in-situ soil, inclination of the inclusions and of the facing ; etc...

Cartier and Gigan (Helsinki Conference) present a full scale experiment on a nailed soil retaining wall 5.5 cm high with a vertical facing. The inclusions were driven angle bars of a relatively low bending stiffness and were inclined at an angle of 20° to the horizontal (Hyrpinoise method). The soil was a silty fine sand. Fig. 12 illustrates the lateral displacements of the structure measured with inclinometers and is clearly shown that this displacement pattern is quite different from that of the facings of reinforced earth walls. Similar results have been reported by Gässler and Gudehus (1981) and by Shen et al (1981). They indicate that the progressive failure in a nailed soil wall induces larger displacements at the top of the wall.

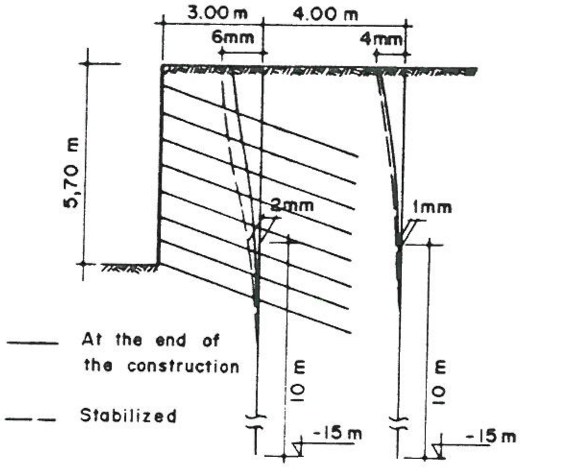
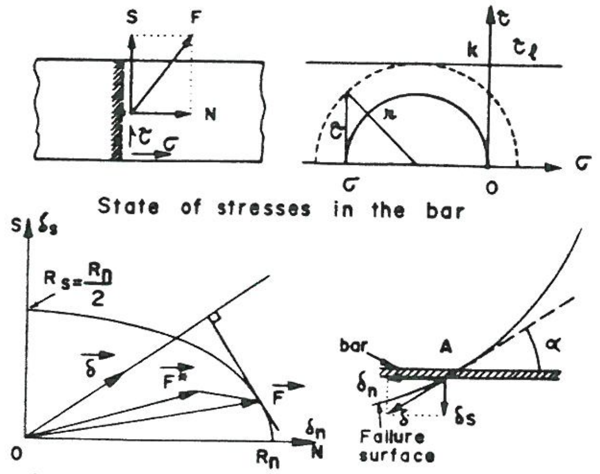


Fig:12 - HORIZONTAL DISPLACEMENTS INSIDE AND BEHIND A NAILED SOIL WALL (Cartier and Gigan, 1983)



Application of the principle of maximum work
Fig: 13.- DETERMINATION OF THE MAXIMUM FORCE IN THE BAR (Schlosser, 1983)

Guilloux et al (Helsinki Conference) report a case history of an instrumented nailed soil retaining wall, 14 m high, with a facing slightly inclined (steep slope of 10/1). Micro pile type reinforcements were used and they were practically horizontal. The soil is a dense moraine. The tensile forces measured in the inclusions show the development of an active and a resistant zones. Under frost conditions the heave effect results in an increase of the lateral earth thrust on the facing and consequently the maximum tensile forces in the inclusions develop at the facing. Guilloux et al present a design method developed by Terrasol (Schlosser, 1983). This design method takes into account failure conditions and considers four failure criteria related to the mobilization of the different efforts in the soil and in the inclusions : 1° - shearing of the inclusion due to the combined effects of the mobilized tensile force, shearing force, and bending moment ; 2° - shear strength of the soil ; 3° - lateral friction along the inclusion and 4° - lateral earth pressure on the inclusion. Calculations are made along a circular sliding surface using a slices method.

Fig. 13 shows the simplified case of the first criterium when the bending moment can be neglected. The condition $\tau \le k$ (maximum shear stress in the reinforcement) leads to the following formula :

$$(N/R_n)^2 + (S/R_s)^2 \le 1$$

where N and S are the tensile and shear forces, R_n and R_s the resistance of the bar to tension and to shearing.

At failure, the tensile and shear forces mobilized in a bar are determined using the principle of maximum plastic work as indicated on the fig. 13. This principle gives :

$$(\vec{F} - \vec{F}^*) \cdot \vec{\delta} \ge 0$$

where $\vec{F}$ is the real force in the bar, $\vec{F}^*$ a virtual force which respects the failure criterium and $\vec{\delta}$ the displacement of the bar, at a point of a circular failure surface. $\vec{\delta}$ is tangent to the circle.

The third criterium can be written : $N \le N_\ell$, where N_ℓ is the pull-out force of the part of the portion of the bar located beyond the failure surface.

Taking into account the fourth criterium requires to consider the bending moment. It leads to a yield surface in the (N,S) plane which is much more complex than the ellipsa corresponding to the first criterium only. This design method which has been checked on many reinforced soil structures (Schlosser, 1983) (walls with flexible bars, rigid bars, slopes stabilized by micro-piles) is used with the following values of the safety factors : elastic limit stress for the steel, half of the pull-out force for the friction, lateral earth pressure limited to the creep pressure in the pressuremeter test, safety factor of 1.5 on the shear strength of the soil. The values have been chosen in order that all the criteria will be compatible with respect to the considered displacement pattern.

Along with deterministic design methods, statistical approaches are presently developed to overcome the difficulties involved with the determination of design parameters and safety factors. Such a probabilistic approach is presented by Gässler and Gudehus (Helsinki Conference) for the design of nailed soil retaining walls.

3.1.2 Multi anchored walls

The concept of a multi anchored wall has been initiated by Coyne in 1926 with the invention of the "ladder wall" system. Similar systems have been recently developed (Murray, 1981 ; Fukuoka, 1982 ; etc...). Fukuoka described a full scale experiment on a multi anchored wall the facing of which is made of fabric attached to vertical columns. The backfill is a silt. The composite reinforcement consisted of steel anchored rods attached to concrete vertical plates (1 m X 1 m X 0.15 m).

Fig. 14 shows the rotation of the columns, the active earth pressure on the facing and the mobilized passive lateral earth thrust on the concrete plates which is slightly superior to the lateral earth pressure at rest. These results show that in this retaining system the displacement (rota-

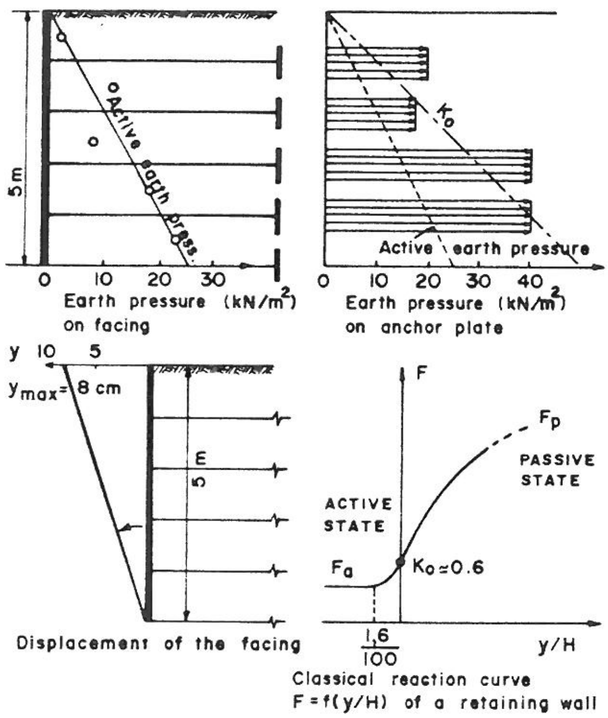


Fig : 14 - BEHAVIOUR OF A MULTI ANCHORED WALL (based on results reported by Fukuoka et al, 1982)

tion) of the facing is sufficient to attain the active earth pressure on the facing. The displacement of the anchor rods results in a mobilization of the lateral earth thrust on the concrete plate. This lateral earth thrust can be predicted provided that the mobilization curve of passive and active earth pressure, shown in Fig. 14, is known.

Chabal et al (Helsinki Conference) present the construction of a dam built with a "ladder wall" system, 21 m high. The down stream facing of the dam was constituted of the anchor plates. Measurements have shown that the tensile forces in the tie rods do not increase linearly with the overburden pressure and are less than predicted. This structure behaves rather like a double facing structure and is therefore different from the classical multi anchored wall. It can be reasonably expected that the state of stresses in the soil is close to K_o.

In-situ multi anchored walls are realized using prestressed active anchorages. The construction process is similar to that of a soil nailing system but the behaviour is relatively different because the prestress efforts restrain the lateral displacements. This system has been recently used for the construction of a 30 m deep retained earth structure which has been thoroughly investigated (Kerisel et al, 1981). However this system will not be discussed in this report.

3.2 In-situ slope stabilization

Four papers submitted to this conference deal with in-situ slope stabilization by soil nailing (Juran et al) by piles (Winter and Gudehus, Cartier and Gigan) and micro-piles (Lizzi). The behaviour of these systems is discussed below.

3.2.1 Nailed slopes

The stabilization of unstable or sliding slopes by nailing consists of placing passive linear inclusions capable of withstanding bending moments vertically or perpendicularly to the failure surface. The inclusions are installed with a uniform density and the construction process is similar to that used to build nailed soil retaining walls. The behaviour of the system depends on several factors, including : the inclination of the inclusions with respect to the failure surface, their density, the relative rigidity of the inclusion and the soil and the actual state of the sliding (static equilibrium, creep, etc...)

The effect of the orientation of the reinforcement has been studied by Jewell (1980). He showed that the development of tensile forces in the reinforcements during a direct shearing of a reinforced soil depends mainly on the inclination of the reinforcement with respect to the sliding surface. The maximum increase of the shear strength of a sand sample reinforced by passive bars or grids is reached when the reinforcement is oriented close to the direction of the principal tensile strain increment which would have occurred in the unreinforced sand at failure. When the reinforcement is oriented in a direction of a compressive strain increment it may result in a decrease of the shear strength of the soil. Fig. 15 shows the experimental and theoretical relationships between the increase of the shear resistance of the reinforced sand $\Delta\tau/\sigma_y$ (σ_y being the applied normal stress) and the inclination angle θ.

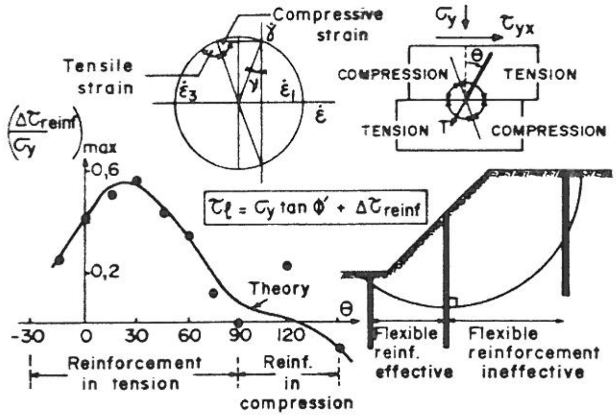


Fig : 15 - INCREASE IN THE SHEAR STRENGTH OF SAND $(\Delta\tau_{reinf}/\sigma_y)_{max}$ Vs THE ORIENTATION θ OF THE REINFORCEMENT (Jewell, 1980)

Jewell showed also that an extremely small displacement is sufficient to generate the soil-reinforcement friction and the corresponding increase $\Delta\tau$ of the average shear stress mobilized along the sliding surface.

These results suggest that reinforcing of an unstable slope in a direction of the compressive strain increment that is in the upper part of the slope, is ineffective and may decrease the shear strength mobilized in this part along the failure surface. However this mechanism ignores the lateral earth thrust on the inclusions, which at larger displacements results in the mobilization of the bending stiffness in the inclusions.

The effect of the rigidity of the inclusions has been studied experimentally by laboratory direct shear tests on a silty soil reinforced by a row of vertical steel bars of different diameters (ϕ = 8 mm and 12 mm) (Juran et al, 1981). The results showed : 1° - a progressive mobilization of the bending stiffness of the bars which results in an apparent cohesion c* of the nailed soil ; 2° - the displacement necessary to mobilize entirely this apparent cohesion is much larger than that required to generate the soil-reinforcement friction. Juran et al (Helsinki Conference) report a finite element analysis of the behaviour of the nailed silty soil in the direct shear tests described above. Fig. 16 shows the comparison of the theoretical mobilization of the global apparent cohesion c* with the experimental results. The curves present a change in the slope at a relative displacement of about 4 %. This change is due to a progressive plastic flow of the soil around the bars. The finite element analysis agrees fairly well with the experimental results. It shows that the global apparent cohesion of the nailed soil is much larger than that corresponding to the sum of the shear forces mobilized in the inclusions. The authors show that this difference is due to the effect of the presence of the inclusion on the stress and strain fields in the soil.

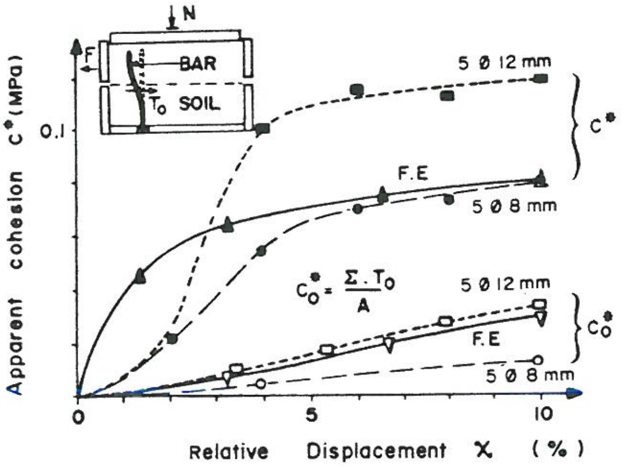


Fig: 16 - MOBILIZATION OF THE APPARENT COHESION OF THE NAILED SOIL (Juran et al, 1983)

Fukumoto (1974 and 1976) reported observations on slope stabilization by soil nailing and particularly an in-situ shear test on a soil mass reinforced by two steel pipe piles. The results show a mobilization of the bending moment similar to that described by Juran et al.

When nailing is used to stabilize a sliding slope the behaviour of the nailed soil is controlled by the creep characteristics of the visco-plastic flowing soil and therefore seems to be different from the behaviour of the same nailed soil under static equilibrium conditions. Ito and Matsui (1975) have developed two approaches to analyse the problem of lateral pressure exerted by laterally sliding soil on a row of rigid piles. They have considered : 1° - a plastic deformation of the soil around the piles, and 2° - a visco-plastic flow of the soil around the piles. The two theories, were applied to predict the total lateral force on instrumented piles in five different sites, and yieldied results of the same magnitude which corresponded fairly well to the experimental observations. Although in the theory of visco-plastic flow the lateral force increases with the viscosity of the soil and with the sliding velocity, it does not change very much with the yield stress of the soil considered as a Bingham's solid. These results suggest that pseudo static considerations can be used in a first approximation to analyse the behaviour of a nailed soil under creep conditions.

Such an approach is proposed by Winter et al (Helsinki Conference). They consider essentially that the lateral earth pressure on the pile is related to the change of the undrained cohesion of the creeping soil due to the decrease of the creep rate.

The uniform density of a nailed slope is also an essential parameter of the behaviour because it controls the group effect. Considering the lateral earth pressure on the reinforcements the group effect results in an apparent inclusion constituted of the reinforcement and its tributary surrounding soil. Consequently the total lateral earth pressure resisted by the group of inclusions is greater than the sum of the earth thrust on the individual reinforcements. When the density is large enough the nailed soil behaves as a monolith. However this group effect which is a common problem in soil mechanics has not yet been sufficiently investigated.

Present design methods for nailed slope under static equilibrium conditions do not consider the group effect. The Terrasol method presented by Guilloux et al (1983) for the design of nailed soil retaining walls can also be used for the design of slope stabilization. A displacement calculation method is proposed by Cartier and Gigan (1983). This method is based on the consideration of circular sliding surfaces and on an increase of the safety factor due to the moments and the shear forces mobilized in the inclusions. To calculate these moments and shear forces it is necessary to know the relative displacement of the pile and the soil and therefore the initial displacement pattern of the slope in the absence of the reinforcements. Except of some particular cases the determination of these displacements is difficult and requires a finite element analysis.

Under creep conditions Winter et al (1983) propose a pseudo static design method based on the principles described above (see 2.2.2). The lateral earth pressure on each inclusion corresponding to the decrease of the creep rate from V_1 to V_2 is given by :

$$p = \lambda \cdot \frac{S}{h} \ln \frac{V_1}{V_2}$$

where S and h are respectively the tributary area per inclusion and the effective height of the resisted lateral pressure. λ is a parameter depending on the creep characteristics of the soil.

3.2.2 Piles

One or two rows of large rigid piles are often used to stabilize land slides (Yamada et al, 1971 ; Fukumoto, 1972 ; Kerisel, 1976 ; Sommer, 1979). The behaviour of this system is different from that of a nailed slope because the row of piles usually constituted a relatively rigid screen and consequently an element of discontinuity in the displacement pattern of the slope. Usually the piles are located at the toe of the slope and consequently the stabilization is progressive starting from the lower part of the slide. However, as in Soil Nailing, the bending stiffness of the pile is the essential parameter.

Three types of design methods have been developed. The first one (Brinch Hansen, 1960) considers a rigid plastic soil and assumes that the passive lateral earth pressure on the pile is entirely mobilized at both sides of the sliding surface. This method is usually applied for thick piles.

The second method is an elasto-plastic approach as proposed by Juran et al (1981) and Cartier and Gigan (1983) This method requires an adequate determination of the relative displacements between the pile and the soil. This approach is more adapted for the design of rather flexible piles.

The third approach, as described by Winter et al (1983),is used for the stabilization of creeping slopes. The first and the third approaches have been considered by Sommer (1979) in the analysis of a sliding slope stabilized by a row of rigid instrumented piles, 3 m diameter. The 10 m high clayey slope was sliding at a rate of 14 mm/month. Fig. 17 shows the design and measured lateral earth pressure on the piles. The measured earth pressure on the piles corresponds to an increase of about 5 % of the total shear resistance along the sliding surface (soil shear strength + shearing efforts in the piles) but this slight increase was sufficient to reduce the sliding rate to about 10 % of its initial value.

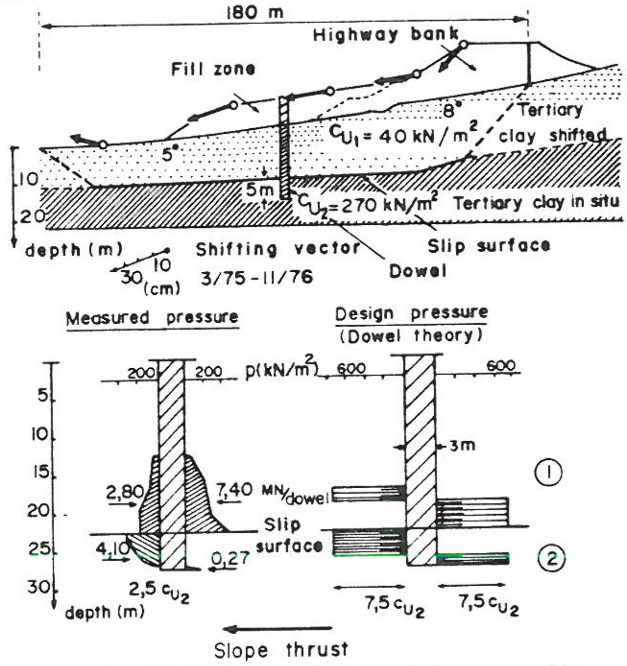


Fig:17-STABILIZATION OF SLIDING SLOPE (Sommer,1979)

Cartier and Gigan (1983) have used more flexible inclusions in three rows of poured in place concrete piles (40 cm diameter reinforced by H200 metallic profiles to stabilize an unstable slope under a railway embankment. The slope was progressively sliding at a rate of 10 cm/year. The measurement of the displacement of the pile enabled to calculate the shearing efforts and the bending moments in the pile and to demonstrate that an increase of about 7 % of the safety factor was sufficient to reduce the sliding rate to 2.5 mm/year.

It is interesting to note that in the two cases described above the lateral earth pressure on the pile was significantly inferior to the creep pressure of the soil.

3.2.3 Micro-piles to stabilize land slides

In a paper to this conference Lizzi describes the multiple applications of micro-piles in slope stabilization, retaining structures and foundations. The author distinguishes the cases of stiff and loose soils. In the first case micro-piles are uniformly installed all along the slope and have mainly to create in-situ a monolithic rigid block of reinforced soil sufficiently deep below the critical failure surface. In the second case the micro-piles are concentrated in the lower part of the slope to create an in-situ gravity wall.

The main difference between this system and soil nailing is that the behaviour of micro-piles is significantly influenced by a structural effect (Lizzi et al, 1979 ; Schlosser et al, 1979) which is due to the particular arrangement of the micro-piles. The complex soil-piles interaction is usually large enough to create a monolith but it has not been yet sufficiently investigated. This is the main reason for the fact that present design methods consider almost only aspects involved with the external stability of micro-piles.

3.3 Shallow foundations with reinforcements by H.M.JACOBSEN

Six of the contributions to this session are concerned with the influence of soil reinforcement on the ultimate bearing capacity of a shallow footing. Many papers have in recent years dealt with this problem. It would seem possible to find a main tendency in the behaviour of such a footing by comparing all these papers, but most likely such procedure would prove difficult due to deficient data on the research reported. For instance, the depth to the first layer of reinforcement or the density of sand may be missing; or the triaxial friction angle may be given without any information on stress levels, which differ very much from triaxial to small scale model tests; or the ultimate bearing capacity of a corresponding model footing on an unreinforced sand could be omitted. Another difficulty is that different failure criteria have been used, for instance maximum load or load at differently specified relative settlements. Below follows an assessment based on existing papers containing sufficient data for the analysis.

3.3.1 Bearing capacity of footings on reinforced sand

Special interest seems to concentrate on model tests on inclusions in a sand mass without a weaker subsoil. These models imploy one to six layers of reinforcement.

The displacement vector field has been studied very carefully by photogrammetric or stereo-photogrammetric techniques. The testbox has then a wall made of thick glass to permit photography or a two dimensional material (steel pins) has been used. The vector field is analysed in order to find trajectories of zero extensions or of principal tensile strains. The latter is very important because the most effective orientation of a limited number of inclusions is supposed to be that of the principal tensile strains, Andrawes et al. 1978.

The displacement field depends on the material of the inclusions. Synthetic materials have normally lower in-soil friction than the soil itself and the displacement will have a tendency to follow the surface of the inclusions. A layer of steel rods can have an in-soil friction equal to the soil friction, Andrawes et al. 1978, and the displacement will either go through or away from the inclusion. When using flexible inclusions as for instance geotextiles, the failure could develop in the soil before activation of the reinforcement. In that case the zero extension trajectories are nearly identical to the so-called deformation characteristics used in the theory of plasticity. The observed vector fields are normally in close agreement with the rupture figure for the bearing capacity problem, fig. *18* and *19* Therefore, it seems reasonable to use the theory of plasticity as a starting point for analysing the bearing capacity of reinforced soil, at least when flexible materials are used. In the absence of a better theory it could be used for stiff reinforcements too. However, observations of displacements above the inclusions, Andrawes et al. 1978, seem to indicate, that the upper part of the soil has to be regarded a special layer.

Some inappropriate locations of inclusions are mentioned in several papers and fig. 3 displays corresponding displacement fields / rupture figures. The inclusion in fig. 20a has a relatively smooth surface, and slippage between sand and inclusion will reduce the bearing capacity from that of the soil alone, Andrawes et al. 1978. At 3b

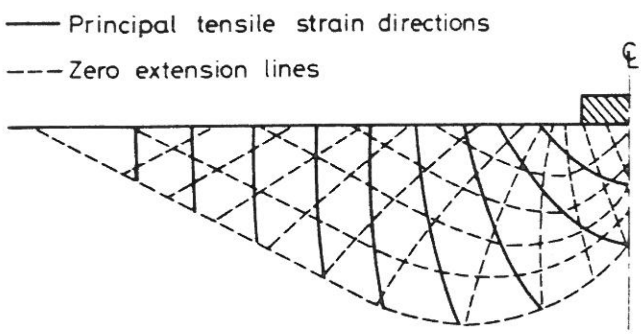


Fig: 18 _ OBSERVED TRAJECTORY DIAGRAM OF TENSILE PRINCIPAL STRAIN AND ZERO EXTENSION LINE FOR SAND ALONE, (Andrawes et al, 1983)

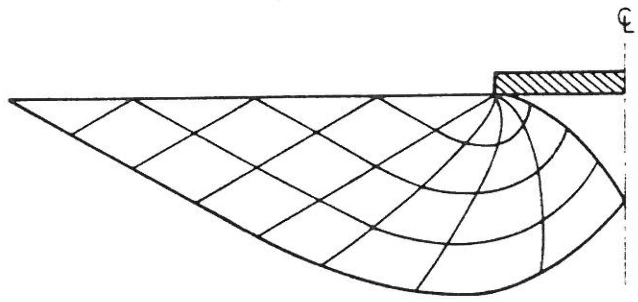


Fig: 19 _ RUPTURE FIGURE FOR BEARING CAPACITY OF A FOOTING ON A UNLOADED SAND SURFACE, (After Lundgren and Mortensen, 1953)

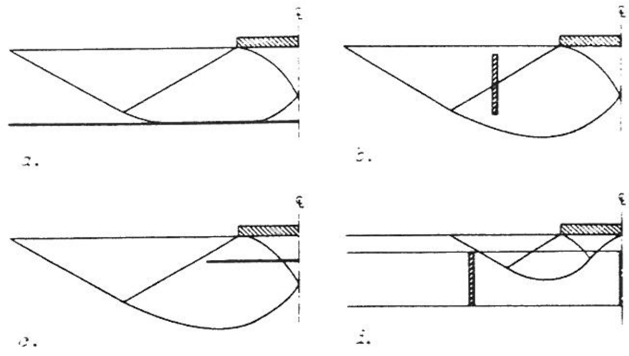


Fig: 20 _ INAPPROPRIATE LOCATIONS OF INCLUSIONS

and 20c too short inclusions are placed in the lines of principal tensile strains. At 20d a PVC-grid is used in a model test; although the displacement field looks very different from that of the sand alone, the bearing capacities are observed to be nearly equal as should be when using the theory of plasticity.

The optimal location of a single horizontal layer which gives the highest improvement factor on the ultimate or residual bearing capacity is studied in some papers (Table 1). The maximum benefit occurs when the geotextile is situated at a depth of 0.25B - 0.5B, where B is the width of the footing. The optimal length of the inclusion depends on the material. Extensible inclusions transfer their stresses into the soil over a limited area and the optimal length L is found to be L = 5B, Fragaszy et al. 1983. Rigid inclusions, e.g. by steel, require a bigger anchor length, McGown 1979. The use of more layers have been studied by many authors (Table 2). Two or three layers have a favourable effect on the bearing capacity even with a vertical spacing Z of 0.5 - 0.75B.

The most important results of all these efforts are improvement factors, which depend on number of layers, vertical spacing af the layers, depth to the upper layer, strength of soil, inclusion materials, etc. Therefore it is complicated to achieve a general idea of the reinforcement mechanism or at least to make a convenient comparison.

In one of the papers Denver et al. 1983 propose to use the theory of plasticity in the simplest possible way when calculating the influence of a PVC-grid. This idea can be used also to analyse test results with horizontal reinforcement layers, when introducing an angle of distribution α.

First a single layer of reinforcement is considered. The reinforcement and the sand above is assumed to be a homogeneous material which is stronger than the subsoil. Failure occurs when the footing penetrates the upper reinforced layer into the weaker subsoil. The stress distribution through the upper sand layer is then assumed to follow lines inclining α to the vertical (fig. 4). The corresponding hypothetical footing on the subsoil has a width B*, which is

$$B^* = B(1 + 2\tan\alpha\ D/B)$$

and a bearing capacity, which can be expressed as,

$$q^* = \frac{1}{2}\gamma B^* N_\gamma s_\gamma + \gamma(\delta + D) N_q s_q d_q$$

where γ is the sand density, N_γ and N_q bearing capacity factors, s_γ and s_q shape factors, d_q is a depth factor, and δ is the settlement at failure. The surface of the sand is assumed to be unloaded and the load on the footing vertical.

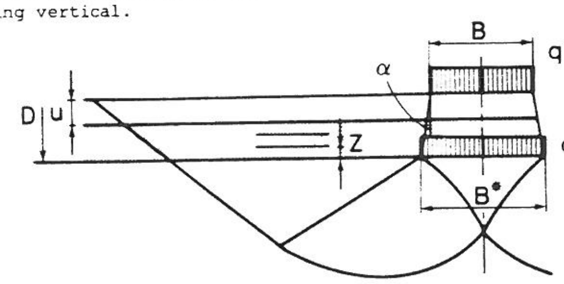


Fig: 21 _ PENETRATION OF REINFORCED SAND

The load on the real footing can then be calculated from

$$q = \frac{A^x}{A} q^x$$

where A and A^x are areas of the real footing and the hypothetical footing respectively. It is assumed that the resulting force on the two inclined distribution lines is horizontal.

A depth factor of $d_q = 1 + 0.35D/B$ is used when the results are analysed. Observations of vector fields seem to show that sand grains in the upper layer move towards the footing, which means that the horizontal pressure may be too small to establish the normal shear forces along the rupture lines in the upper parts of the rupture figure. The use of a depth factor different from unity can therefore be discussed, but its influence on tanα is rather small.

In tables 1 and 2 are mentioned some of the most complete test series. The friction angle is calculated by backanalysing tests on pure sand using bearing capacity factors from Lundgren and Mortensen 1953. The friction angles may therefore differ slightly from those in the papers quoted in table 4 and 2. They depend strongly on the stress level or the size of the model; i.e. the smaller the plates, the higher the friction angle.

The calculated values of tanα for a single layer of reinforcement are plotted against u/B in fig. 5. The reinforcements consist of geotextiles, woven or non-woven; of polypropylen rods or of rope fiber strips. A certain scatter is observed, but in spite of that there seems to be a tendency for tanα to decrease when the relative depth of the layer u/B increases from 0.1 to 1.

Tests with several layers (2-6) can be treated the same way. The upper layer now includes all inclusions, even grids. tanα is plotted also against u/B in figure 23. The result is rather astonishing, because of the limited scatter, taking into consideration that different types of inclusion and test procedures have been used. For u/B > 0.5 tanα is nearly constant, but for u/B < 0.5 tanα may assume higher values, which means that the use of reinforcements is much more effective, when the uppermost layer is situated at a small depth below the footing. The knowledge on this particular point is still insufficient and further experiments are needed.

Table 4. A single horizontal layer of reinforcement. Subsoil : Sand.

ANDRAWES McGOWN WILSON - FAHMY 1983	Geotextile, non-woven Sand: n = 0.34 Model: L = ∞ B = 0.12 m	u/B q/γB δ/B φ tanα	 250 0.16 49° 	0.125 335 0.19 0.33	0.250 330 0.21 0.00	0.500 310 0.20 -0.16	0.750 270 0.15 -0.22	1.00 235 0.14 -0.24
VANISEK 1983	Geotextile, woven Sand: γ = 15 kN/m³ (?) Model: L = ∞ B = 0.04 m	u/B q kN/m² δ/B φ tanα	 150 0.13 49° 	0.375 250 0.22 0.10	0.75 260 0.22 -0.06	1.125 175 0.15 -0.19		
AKINMUSURU AKINBOLADE 1981	Rope fiber strips Horizontal spacing x/B Sand γ = 17 kN/m³ Model: B = L = 0.1 m δ/B = 0.10 (?)	u/B x/B q kN/m² φ q/q₀ tanα	 91 42.5 	0.5 0 2.35 0.10	0.5 0.5 2.15 0.00	0.75 0.5 1.17 -0.23		
MILOVIC 1977	Polypropylen rods Sand γ = 15 kN/m³ Model: D = 0.6 m	u/B q kN/m² δ/D φ tanα	 400 0.1 41.5 	0.25 660 0.1 -0.03	0.5 830 0.1 -0.07	0.75 1100(?) 0.1 -0.06		

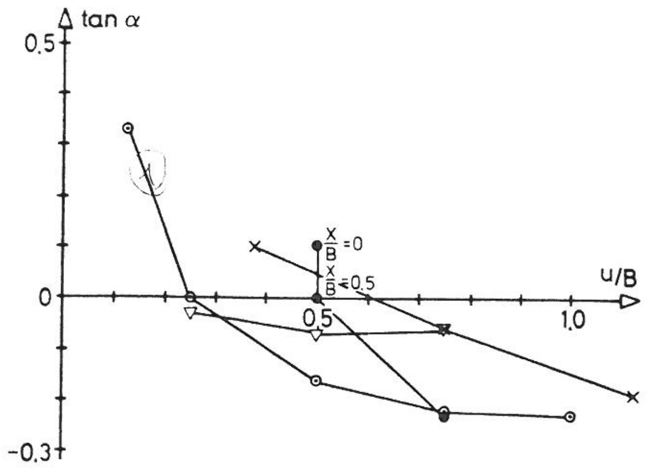


Fig: 22_ TAN. α FOR A SINGLE LAYER OF REINFORCEMENT

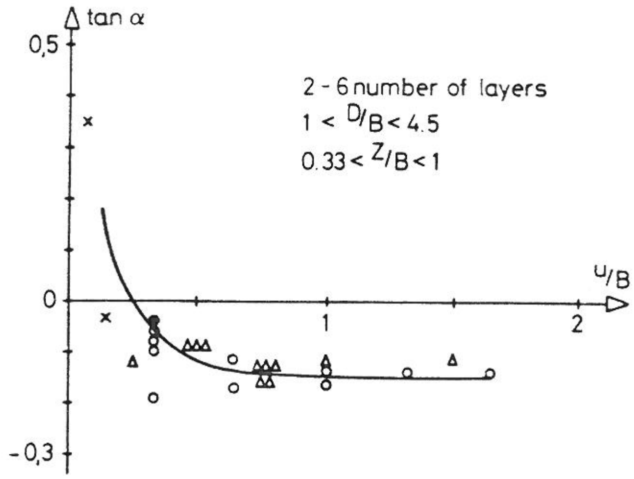


Fig: 23_ TAN α FOR 2-6 LAYERS OF REINFORCEMENT

Table 5. Test results with N horizontal layers (N > 1). Subsoil : Sand.

BINQUET LEE 1975	Aluminium strips	u/B		0.33	0.66	0.33	0.33	0.33
		D/B		1.33	1.66	1.00	1.67	2.00
	Sand: γ = 15 kN/m^3	q kN/m^2	83	186	162	116	214	255
		N		4	4	3	5	6
	Model: L = ∞ B = 0.076 m	δ/B	0.08	0.07	0.07	0.07	0.07	0.07
		ϕ	42°					
		tanα		-0.10	-0.12	-0.19	-0.08	-0.06
		u/B		0.66	1.00	1.00	1.33	1.67
		D/B		1.33	1.67	2.00	2.00	2.33
		q kN/m^2		116	128	148	141	120
		N		3	3	4	3	3
		δ/B		~0.07	0.07	0.07	0.07	0.07
		ϕ	42°					
		tanα		-0.18	-0.16	-0.13	-0.14	-0.14
FRAGASZY LAWTON ASGHARZA-DEH-FOZI 1983	Aluminium strips	u/B		0.33				
		D/B		1.00				
	Sand γ = 15.4 kN/m^3	q kN/m^2	75	200				
		N		3				
	Model: L = ∞ B = 0.076 m	δ/B	0.1	0.22				
		ϕ	42°					
		tanα		-0.04				
DENVER CHRISTENSEN HANSEN STEENFELDT 1983	PVC-grid	u/B		0.15		0.07		
		D/B		1.08		0.47		
	Sand γ = 15.5 kN/m^3	q kN/m^2	75	244	85	390		
		δ/D	0.10	0.10	0.10	0.10		
	Model: Circular	D m	0.065	0.065	0.15	0.15		
		ϕ	44°		41°			
		tanα		-0.04		0.35		
AKINMUSURA AKINBOLADE 1981	Rope fiber strips Horizontal spacing 0.5	u/B	0.75	0.75	0.75	0.75	0.5	0.5
		D/B	1.25	1.75	2.25	2.75	2.5	3.5
		q kN/m^2	142	136	139	139	223	191
	Sand γ = 17 kN/m^3	N	2	3	4	5	5	5
		δ/B	~0.1	~0.1	~0.1	~0.1	~0.1	~0.1
	Model: B = L = 0.1 m	ϕ	40°					
		tanα	-0.16	-0.16	-0.13	-0.13	-0.09	-0.09
		u/B	0.25	0.50	0.75	1	1.5	0.5
		D/B	2.25	2.50	2.75	3.00	3.50	4.5
		q kN/m^2	175	182	139	116	114	155
		N	5	5	5	5	5	5
		δ/D	~0.1	~0.1	~0.1	~0.1	~0.1	~0.1
		ϕ	40°					
		tanα	-0.12	-0.11	-0.13	-0.12	-0.11	-0.09

The above analysis seems to show:

(i) It is possible to find the improvement of bearing capacity when reinforcing a pure sand layer. The normal approach for penetration of a layer into a weaker soil can be used, but with different pressure distribution.

(ii) The "distribution angle" α seems to be nearly independent of inclusion material, number of layers, density and strength of sand, and reinforcement depth within the wide range of parameters used in the tests.

(iii) The "distribution angle" α seems to depend mainly on the relative depth of the upper layer of reinforcement.

The negative value of α indicates that a better method be proposed in the future.

The basic idea in the proposed calculation method is that failure occurs in the subsoil as normal, but that the reinforced sand beneath the footing acts as a block during the penetrating process.

In the model tests two additional modes of failure were observed. The inclusion was pulled out or broke during failure. The reinforcement ties always broke approximately under the edge or towards the center of footing, Binquet and Lee 1975. The tensile forces have been determined in an undamaged geotextile, Andrawes et al. 1983, and show the same tendency (fig.24).

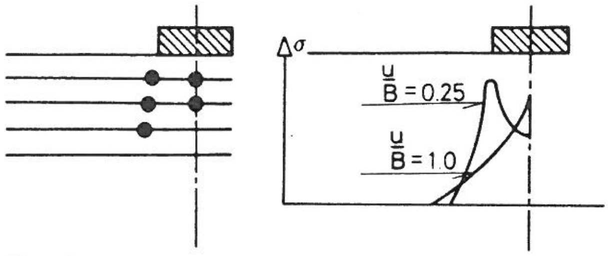


Fig. 24_ POSSIBLE LOCATION OF BREAKES, (Binquet and Lee, 1975). AND TENSILE STRESSES IN A GEOTEXTILE, (Andrawes et al, 1983)

In the model tests the bearing capacity is only slightly influenced by these phenomena, but in practice the tensile forces in the inclusions are much higher.

For design purposes it is very important to be able to calculate tensile forces in the inclusions. Design methods based on the theory of elasticity, Schlosser and Long 1974, or the theory of plasticity, Binquet and Lee 1975, have been proposed, but have not until now been compared with relevant experimental datas.

3.3.2 Settlement of footings on reinforced sand

Settlement of footings on reinforced sand are strongly dependent on the properties of inclusion material such as flexibility and surface roughness, the number of reinforcement layers, and properties of the sand.

When using a flexible, non-woven geotextile the load-settlement curve is not influenced by the reinforcement until a certain value of the settlement is reached, according to Andrawes et al. 1983, 0.08B. For any other material reduced settlements are observed even for very small loadings. For woven geotextiles the improvement seems to be up to 100%, Vaniček 1983, for aluminium strips 50-500%, Binquet and Lee 1975, for steel rods and PVC-grids even 500-1000%, Milovic 1979, Denver et al. 1983.

The effect of repeated loadings has been dealt with in two papers. Denver et al. mention that when using a PVC-grid as reinforcement the settlements are reduced by a divisor of 2-5. Patel and Paldas use a composite reinforcing element. They make a distinction between the elastic component δ_e and the plastic component δ_p of the settlement. They find that the elastic component is unchanged but the plastic component is reduced by a divisor of more than 2.

3.3.3 Use of small scale model tests

All the tests mentioned until now are small scale model tests. However, their practical application is rather limited because the laws of similarity are not fulfilled. Three major problems should be mentioned.

The stress field in model and in prototype should be similar. In the mentioned small scale model tests the stresses are much smaller than in the prototype. The friction angle depends very much on stress level and is much higher in model tests (table 4 and 5) than in prototype. The stress distribution is scaled incorrectly probably leading to an improper failure mechanism. The stress problem may be overcome by exposing the model to an acceleration field in a centrifuge, Kim et al. 1983, Ovesen and Krarup 1983. The small scale model tests can be used to describe a certain phenomenon as for instance the relationship between tanλ and u/B (fig. 23), but before practical utilization it should be controlled by centrifuge tests.

The inclusions have to be carefully scaled down if tensile forces or breaks in the inclusions should be studied. Strictly speaking the dimensions should be scaled down; an inferior alternative is to scale down the strength of the material, Ovesen and Krarup 1983. In neither of the tests mentioned above scaling has been introduced.

The grain diameter, the thickness of inclusion, and the stress-strain curves are normally not scaled down, and the settlements of the model are normally not similar to the settlement of the prototype. Carefully described case studies are therefore very valuable. In very small models the scaling effect from grain size can be observed on the failure loads too, but such small models are normally not in use anymore.

3.3.4 Reinforced footing on soft subsoils

5 papers submitted to this conference concern the influence of the geotextile located at the surface of a soft subsoil under an embankment or road layers on the deformation and on the general stability.

Kebs et al (1983) have carried out centrifugal tests and Boutroup et al (1983) have performed a finite element analysis. These two studies provided similar qualitative results which enable to explain the mechanism : the geotextile membrane restrain the lateral displacement of the subsoil and consequently the settlement of the loaded surface is practically uniform. The amplitude of the total settlement is only slightly reduced. However as shown by Jewell (1982) an extension of the geotextile under side berms results in a restrain of the lateral displacements under a larger surface and consequently may decrease the total settlement. Boutroup et al have shown that under undrained conditions (ν = 0,5) the tensile forces in the geotextile are larger than those developing under drained conditions (ν = 0.33).

Gourc et al (Helsinki Conference) have carried out experimental study of the behaviour of a road sand layer loaded by a slab and based on a clayey subsoil. A geotextile membrane was placed at the subsoil surface. They have also shown that under a static loading the membrane modifies the displacements pattern in the subsoil and consequently reduces the total settlement and increases the bearing capacity.

For practical considerations of design purposes the conventional method is usually based on a circular sliding stability analysis taking into account the tensile forces mobilized in the reinforcements (Jewell, 1982). Quast et al (Helsinki Conference) propose that when a relatively deformable geotextile (admissible strain of 5 %) is used the deformation of the reinforcement follows the displacements of the soil along the potential failure surface. Consequently the tensile forces are mobilized in the direction of the failure surface.

3.4 In-situ reinforced soil foundation

3.4.1 Stone columns

Reinforcing soft foundation soil by sand or stone columns have mainly three reasons :

1° - to increase the bearing capacity of the foundation soil.
2° - to reduce settlement and accelerate consolidation.
3° - to increase the slope stability of a supported embankment.

Thus the column has both a reinforcing role offering high resistance to compression and to shearing and a drainage role when it is realized in a fine saturated soft soil.

3.4.1.1 General considerations on the behaviour

The most common use of stone columns is to increase the bearing capacity of a rather large foundation soil. Generally the density of the columns is relatively high and as piles they transfer the load to a firmer bearing layer. However the behaviour of the column is different from that of a pile since the mechanism of interaction is that of a restrained expansion in the surrounding soft soil as explained in 2.2.4.

Full scale experiments have shown that under the effect of surface loading by embankment (Vautrain, 1977 ; Aboshi et al, 1979) and rigid footing (Goughnour and Bayuk, 1979) the vertical displacements of the ground surface are practically uniform. Consequently the distribution of the load is characterized by a vertical stresses concentration on the column. The stress concentration ratio n = σ_c/σ_s (where σ_c and σ_s are the vertical stresses respectively in the column and in the soft soil) is a fundamental parameter which depends on several factors including the replacement factor a = A_c/A as defined in Fig. 25.

Both laboratory studies (Aboshi et al, 1979) and full scale experiments have shown that the value of n is generally 3 to 5 at the ground surface. However as shown by Vautrain (1977) it can reach values as high as 50 in depth in the case of a very soft layer.

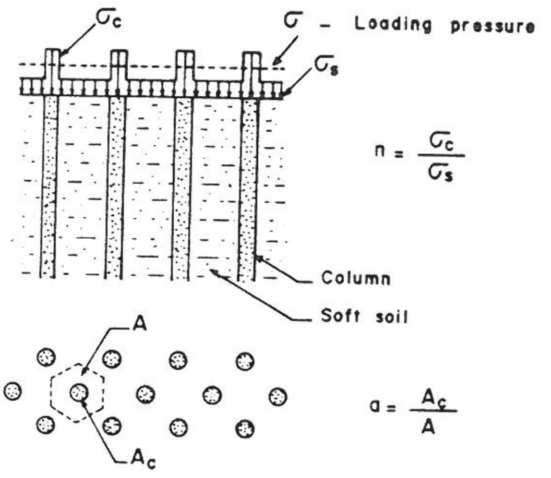
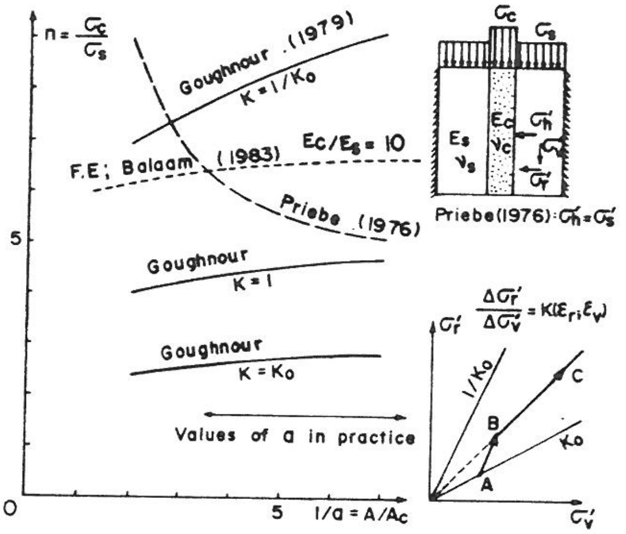


Fig 25 - DESIGN PARAMETERS : STRESS CONCENTRATION RATIO AND REPLACEMENT FACTOR.



Effective stress path in a unit cell assumed by Goughnour et al,1979.

Fig 26 - THE UNIT CELL CONCEPT - COMPARISION BETWEEN DIFFERENT MODELS AND FINITE ELEMENT ANALYSIS.

If the soils are assumed elastic n is equal to the ratio of the modulus of deformation of the column and the soil ($n = E_c/E_s$).

Goughnour and Bayuk (1979) have shown that the shear stresses generated at the soil-column interface are rather small and do not practically affect the state of stresses in the soil.

The installation of the stone column causes an initial compression of the surrounding soft soil and thus increases the value of K_o (Goughnour et al, 1979).

3.4.1.2 Modelling of the behaviour (bearing capacity)

Two types of models have been developed. The first (Poteur, 1973 ; Hughes et al, 1975 ; Aboshi et al, 1979) did not take into account the group effect and considered a single, incompressible, rigid-plastic column in a semi-infinite rigid-plastic soft soil. The available radial confining pressure σ_r can be determined from a triaxial compression test ($\sigma_r = 2\,Cu + \sigma_s$) or from a pressuremeter test ($\sigma_r = pl$).

The second type of models (Priebe, 1976 ; Goughnour and Bayuk, 1979) consider the behaviour of a "unit cell" containing a single column and its surrounding tributary soil. It is assumed that this unit cell is confined by a rigid frictionless wall and that the vertical strains at any horizontal level are uniform. These models are quite similar to an oedometer with a central column and provide a more rational basis for the design.

Priebe assumed that the column is rigid-plastic and incompressible whereas the soft soil in the unit cell is elastic. He also assumed that the state of stresses in the soft soil is isotropic ($K_o = 1$) and therefore $\sigma_r = \sigma_s$. He showed that under these conditions the stress concentration ratio $n = \sigma_c/\sigma_s$ is a function of the Poisson's coefficient ν and of the replacement ratio $a = A_c/A$. As shown in Fig.26 n decreases with 1/a.

Goughnour et al (1979) assumed that the stone column is linearily elastic, perfectly plastic at failure and incompressible in the plastic state. The soil confined within the unit cell is assumed to have a non linear elastic behaviour following an effective stress path which depends on the vertical and the radial strains ε_v and ε_r and on the problem geometry. When the replacement ratio a approaches 1 the ratio K of the radial to the vertical effective stresses approaches $1/K_o$. During the loading the effective stress path is assumed to be bilinear as shown in Fig. 26 and the K coefficient varies between K_o and $1/K_o$.

Depending on the state of deformation the column can be either in an elastic state or in a state of a contrained plastic equilibrium. In the latter case n is function of the replacement factor a and of the assumed value of K. The theoretical variations of n with 1/a for different values of $K = K_o$; 1 ; and $1/K_o$ are shown in Fig. 26 assuming $K_o = 0.6$.

It is interesting to note that in the range of interest for practical considerations $4 < 1/a < 9$ the two models provide similar results considering K = 1, which agree fairly well with experimental observations (n = 3 to 5).

Balaam and Poulos (Helsinki Conference) have performed a finite element analysis of the behaviour of stone columns. They have considered that both the columns and the clay are elastic, perfectly plastic materials obeying a Mohr Coulomb's failure criterium and a law of plastic flow which is characterized by a dilatancy angle. The soil-column interface is simulated using contact elements which allow for pure adhesion, pure friction and adhesion-friction taking dilatancy into account. The "unit cell" concept has been considered for the investigation of the reinforced foundation soil under both rigid and flexible foundation rafts uniformly loaded. The authors have shown that for the geometry of stone columns generally used the solutions for uniformly loaded flexible foundations are nearly equal to the analytical elastic solutions obtained by Balaam and Booker (1981) for uniformly loaded rigid foundations.

They have calculated the variation of the ratio σ_s/σ with the replacement factor a for different values of the ratio of the elasticity modulus E_c/E_s. From these results the value of n is approximately constant with 1/a and varies from about 6 to 30 when the modular ratio E_c/E_s varies from 10 to 40 (Fig. 26).

Wallays et al (Helsinki Conference) have considered the "unit cell" concept and proposed the formulation of a new model assuming that the soil and the column are both linearly elastic and perfectly plastic at failure and that their compressibility at the plastic state can be adequately predicted from Vesic's solution. Both rigid and flexible foundation rafts have been discussed. Their approach to estimate the settlements and the stresses in the column and in the soft soil is similar to that conventially adapted for elasto-plastic analyses. The column is assumed to be in a contained state of plastic equilibrium when the resulting vertical strain is larger than that calculated for the column in an elastic state. No application of this model is presented.

Although the first type of models provide simple solutions and can be related to simple tests like the pressuremeter test the assumption of a complete plasticity of the soft soil between the columns does not correspond to the actual state of the confined soft soil in the reinforced foundation. Moreover the predicted value of n under undrained conditions depends on the level of loading. This theoretical results do not agree with field and laboratory observations (Aboshi et al, 1979). Therefore to the general reporter's opinion further development of models based on the "unit cell" concept as suggested by Wallays et al are necessary in order to obtain more appropriate design methods.

3.4.1.3 Design methods

The behaviour of a large foundation soil reinforced by stone columns involves essentially two aspects of design:

1° - in the central part of the loaded surface the soil displacement is essentially vertical and the design should provide an estimation of the settlement reduction ratio :

$$\beta = \frac{\text{settlement of reinforced foundation soil}}{\text{settlement of untreated soil}}$$

2° - at the extremeties of the loaded surface the lateral displacement can be as large as the vertical settlement and the design should provide an estimation of the local stability with respect to sliding.

Different design methods have been proposed to estimate β. Both empirical methods (Greenwood, 1970 ; Thorburn, 1975) analytical solutions (Priebe, 1976 ; Aboshi et al, 1979 ; Goughnour and Bayuk, 1979) and finite element analyses (Balaam et al, 1977 ; Morgenthaler et al, 1978) have been developed.

The main design parameters are the stress concentration ratio n and the replacement factor a.

A simple solution based on the assumption of a uniform settlement has been used by Aboshi et al according to which

$$\beta = \frac{1}{1 + (n - 1)\, a}$$

More sophisticated methods are based on the "unit cell" concept.

Priebe (1976) considered an incompressible column and an oedometric settlement in the elastic soil contained in the unit cell, consequently his solution provides the same value for β.

Goughnour (Helsinki Conference) develop a theory based on the "unit cell" model (Goughnour et al, 1979) discussed above. The incremental analysis follows two steps : first, the column is considered to be in a contained plastic state of equilibrium and all the volume change is accommodated by the soft compressible soil. Then, the column is assumed to be linearly elastic and its vertical strain is calculated. The actual vertical strain at any level is the larger of those calculated for the two stages. Goughnour provides useful curves for predicting β .

Van Impe and De Beer (Helsinki Conference) propose a simple design method for the estimation of the settlement reduction ratio β based on a similar "unit cell" concept and considering respectively the two cases of (1) rigid-plastic incompressible columns which, for the sake of simplification, are replaced by stone walls with equivalent area, and (2) linearly elastic column. In the two cases the soft soil is assumed to be elastic. However the authors note that the second case does not generally correspond to stone column practice. Their solution for the first case predicts stress concentration ratios which are significantly inferior and settlement reduction ratios which are significantly superior to those predicted by Priebe's solution (fig. 27 and 28).

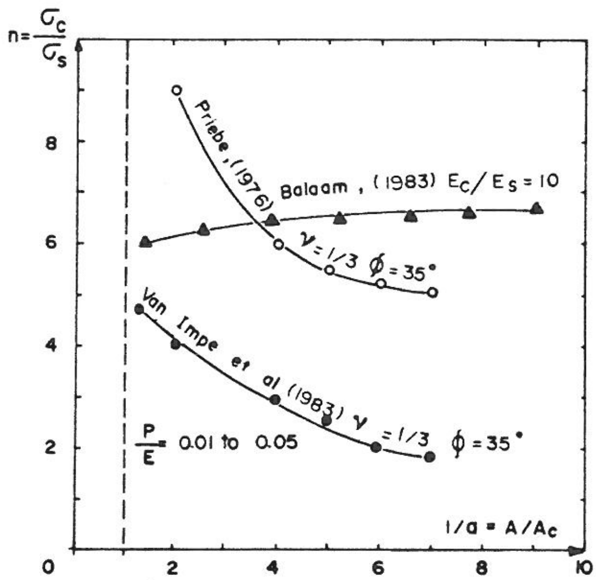


Fig. 27 PREDICTED EFFECTS OF a ON n

A finite element analysis has been carried out by Balaam et al (1983). The authors indicate the finite element solution agree fairly well with elastic solutions obtained by Balaam and Booker (1981) uniformly loaded rigid foundation. As shown in Fig. 28 the settlement reduction ratios β obtained for a range of the modular ratio E (column)/E(soil) ≅ 10 to 40 agree reasonably well with Priebe's solution and the predicted values of β are quite smaller than those predicted from Greenwood's empirical curves. The author also shows that these finite element solutions are generally in a good agreement with observations on actual sites and they can therefore provide a rational basis for design purposes.

Slip circle analysis of local stability of foundation soil reinforced by stone column under embankments are generally done according to two approaches. The first method (Aboshi et al, 1979) considers the shear stresses mobilized in the

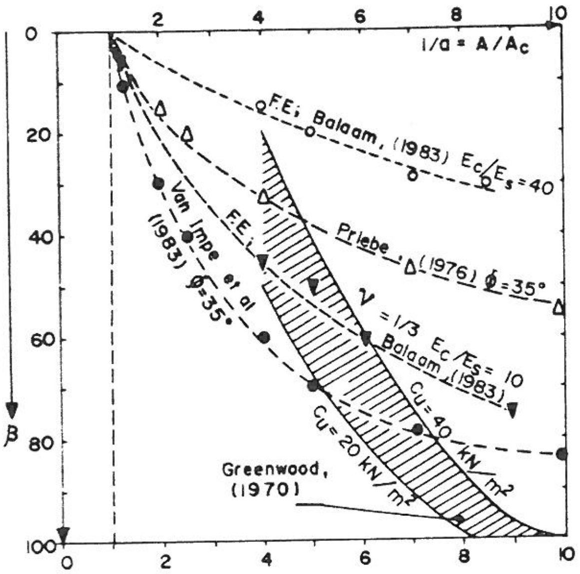


Fig. 28 PREDICTED EFFECTS OF a ON β

columns along the failure surface taking into account the stress concentration ratio n. The second method (Priebe, 1976) considers equivalent shear strength characteristics (ϕ^* and C^*) of the composite reinforced soil. No paper dealing with this aspect of design have been submitted to the conference.

3.4.2 Micro-piles

Micro-piles have been extensively used during the last twenty years for underpinning and reinforcement of foundation soil. A remarkable description of the different applications have been recently edited by Lizzi (1982). The behaviour of foundation soil reinforced by micro-piles has been analyzed by Lizzi and Carnavale (1979) and Schlosser and Juran (1979). As no papers dealing with the behaviour and design has been submitted to this conference these aspects will not be developed in this report.

4 - CASE HISTORIES AND CONTROL

4.1 Case histories

In the field of soil reinforcement practical experience has almost always preceeded and initiated developments of appropriate design and analysis methods.

Furthermore, although during the last decades both laboratory studies and finite element analyses have been carried out, the difficulties involved in modelling reinforced soil systems have clearly showed that full scale experiments are required in order to properly analyse the behaviour of the structure. Consequently most of the presently available design methods tend to integrate empirical considerations derived from both full scale experiments and observations on actual structures.

Among the different techniques discussed in this report Reinforced Earth has made the object of an extensive research on both laboratory models and full scale experiments. Full scale experiments on ground reinforcement techniques: soil nailing, micro-piles, stone columns, etc... are much more limited and their interpretation is generally rather difficult because of local heterogeneities of the in-situ soil. Consequently accumulated past experience based on detailed case histories on structure monitored to control their performance is of a particular interest.

Several papers submitted to this conference describe full scale experiments and case histories related to the different applications of reinforced soil systems : Reinforced Earth and nailed soil retaining walls with instrumented reinforcements, slope stabilization by soil nailing where inclinometers were used to control the decrease of the sliding rate, shallow foundations with instrumented reinforcements and membranes and foundation soil under tanks, silos and rafts where loading tests on isolated columns and on groups of columns were carried out to control the load-settlement behaviour of the reinforced foundation soil.

It is interesting to note that the application of soil-reinforcement systems as Reinforced Earth and soil nailing in retaining structures and slope stabilization generally involves rational analytical or semi-empirical design methods and a significant attempt has been done to compare the observations on full scale structures with theoretical predictions. The main results have been summarized above.

On the contrary, the use of stone columns presents significant problems for the design stages because of the difficulties involved in modelling the effect of the stone columns installations on the surrounding soil and the behaviour of the stone column itself. Consequently current design is mainly based on empirical considerations and it is usually recommended to carry out loading tests on single columns and on a group of columns before the final design. Thus, although six papers to this conference report detailed case histories on application of stone columns under tanks, silos and foundation rafts (Colleselli et al, Green et al, Majorana et a, Romana, Bhandari) as well as under embankment (Sceiro et al) no attempt has practically been done to compare site observations and results of loading tests with theoretical predictions based on the available analytical models discussed above.

A particular application of hammer compacted granular piles in reinforcing loose cohesionless deposit in India is described in the paper of Ranjan and Rao. A relatively economical local technique has been developed to install and compact the sand/stone piles with manual labour. Both isolated and in group piles, plain and skirted, has been loaded. The authors have shown that single piles and groups of 2 and 3 piles increase significantly (from 164 % up to 427 %) the ultimate bearing capacity of the untreated subsoil. A further increase (290 % up to 566 %) is obtained when the piles are skirted. The reduction in settlement is found to be 76 % for a group of 3 or 4 piles and its increases to 86 % when the piles are skirted. This improvement is attributed partially to the confinement provided by the rigid skirt to the granular piles which results in a significant resistance against the bulging of the group of granular piles (Fig. 29).

Two papers presented to this conference describe particular applications of soil-reinforcement in off-shore and coastal structures.

Jewell and Wishart (1983) describe the use of cellular mattresses of reinforcement grids with a sand-tight geotextile facing in containing and reinforcing off-shore hydraulic fills. Mattresses may be placed and filled underwater from a barge reducing the slopes of the fill to about 1:3. Vertical reinforcement polymer grids and a containement facing made of polymer grids on both sides of a geotextile filter membranes were used in a small field scale construction. This trial has shown that the outward face deflections of the 3 m high mattress were about 15 to 18 cm and that the sand was rapidly filled (a rate of about 0.10 m^3/s) and

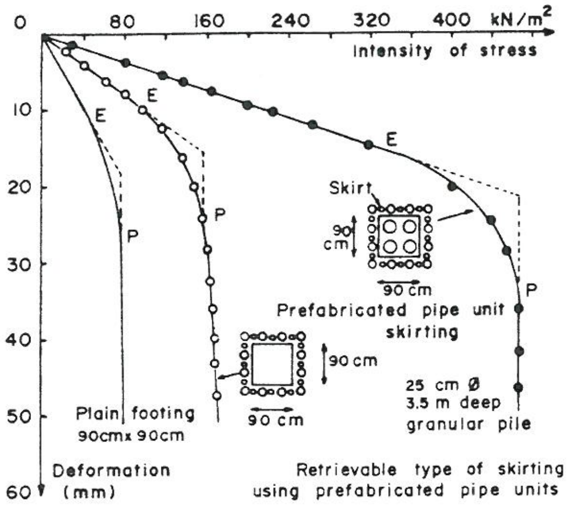


Fig: 29 - STRESS-DEFORMATION BEHAVIOUR OF SKIRTED GRANULAR PILE GROUP AND PLAIN FOOTING AT SITE-I (Ranjan and Rao , 1983)

and successfully contained at the mattress face. After the placing of the mattresses by layers the placing of the sandfill in the core of the island could be carried out contineously.

Simon and Perfetti (1983) describe the underwater use of a non woven geotextile as a reinforcing separator membrane between a compressible clayey mud layer, 5.5 m thick, and a backfill material which supports the 500 m long X 350 m wide runway of the Marseille International Airport.

The geotextile fabric was placed in the critical zones at the mud-embankment interface. It played essentially reinforcing and separating roles but had practically no effect on the amplitude nor on the time of the embankment settlement. It protected against contamination of the selected fill material, prevented the penetration of the soft soil and related localized penetration failures and thus reduced differential settlements, provided a more uniform distribution of stresses at the mud surface and enabled an efficient control of the consumption of the fill material.

4.2 Control

Control in soil reinforcement concerns generally different aspects.

1° - control of the soil when it is a backfill material this is specially the case of reinforced earth.
2° - control of the inclusion when it is realized in-situ that is specially the case of stone columns.
3° - control of the durability of the inclusions considering permanent structures : corrosion of steel and degrability of plastics and fabric materials.
4° - control of the available limit stress of soil-inclusion interaction : pull-out tests in reinforced earth and particularly soil nailing retaining walls, compression loading tests in stone columns.
5° - control of the improvement : measurements of the setlements in reinforced soil foundations ; measurements of the displacements in slope stabilization ; in-situ measurements of the improvement of the mechanical properties of the soft soil around columns.

Some papers presented to this conference deal with these different aspects.

Batelino et al (1983) reported results of an interesting experiment of a Reinforced Earth wall 3,5 m high built with a clayey silt backfill.This material contains 80 % finer than 80 μ and 40 % finer than 15 μ. This is much a larger portion of fines than that admitted in the specification for Reinforced Earth structures.

As shown in Fig. 30 there has been a lateral displacement of the facing during and after the construction due to the creep of the soil. These displacements which have been restrained at the base and the top of the wall by the structure attained 3,5 cm in the middle. In the abscence of restraining effect at the top of the wall this displacement could have been much larger and special precaution would have been required.

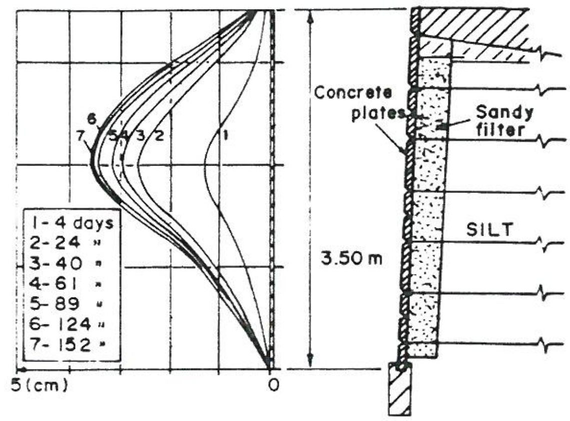


Fig: 30 - LATERAL DISPLACEMENT OF THE FACING PANEL AT DIFFERENT TIMES AFTER THE END OF CONSTRUCTION . REINFORCED EARTH WALL WITH A SILTY BACKFILL .
(Battelino, 1983)

Cartier and Gigan (1983) have reported pull-out tests of tubes used to determine the soil-reinforcement friction value in a nailed soil retaining wall (Fig. 5a). The procedure of these pull-out tests has been similar to that of pile loading test as suggested by Bustamante (1977) for prestressed ground anchors. In this test the load is controlled and applied incrementally by steps of 4 or 8 mm. This test enables the determination of a creep load and an ultimate load. This procedure is quite different from that generally used in reinforced earth and soil nailing where the pull-out test is a displacement controlled test which is realized at a constant displacement rate of few millimeters per minute. The displacement controlled pull-out test enables the determination of the peak value of the residual value and of the strain softening effect which is of a particular interest for design specification.

Several papers submitted to this conference (Colleselli et al, Ranjan and Rao, Bhandari) describe loading tests carried out on both isolated stone columns and groups of stone columns to control the load-settlement behaviour and the ultimate bearing capacity of the reinforced foundation soil. These loading tests generally show that the behaviour (equivalent modulus of deformation and ultimate bearing capacity) of a single pile is similar to that of a group of 3 or 4 piles loaded under the same surface area with the piles being located at the extremities of the loaded surface. When the stone column is realized with a vibro replacement method which densifies and improves the surrounding soft soil the efficiency of the treatment and the increase of the soil modulus and strength characteristics are generally determined by cone penetration tests, SPT or any other appropriate in-situ test.

CONCLUSION

As a conclusion of this report, the following points can be mentioned :

1° - There are large differences in the state of knowledge about the behaviour of the various reinforcement techniques. Thus, the behaviour of Reinforced Earth is rather well known whereas the available information about a foundation soil reinforced by micro-piles is still very limited. Consequently, further research is needed both in laboratory and in-situ on full scale structures.

2° - The deformability of the reinforcement appear to be an important design parameter which governs the behaviour of the reinforced soil system and structures. The nature and mechanical properties of the various types of materials being used have therefore to be selected according to the specific objectives and site conditions of the reinforcement.

3° - The design methods presently available are essentially based on at failure limit analyses. An attempt has been made to use criteria corresponding to at working stresses conditions but this development is still rather limited.

4° - The difficulties involved in modelling reinforced soil systems and assessing the effect of installing inclusions on their surrounding soils have led to integrate empirical considerations in the design methods. Moreover in the field of reinforced foundation soils (micro-piles and stone columns) loading tests have to be integrated in the design procedures to ensure a safe construction. Also for soil nailing pull-out tests are required in order to evaluate the available soil-reinforcement friction.

5° - The values of the parameters used in the design are usually based on empirical considerations or restrictive assumptions and consequently require a direct measurement on the site. Thus ground reinforcement systems often have to be monitored in order to control their performance. Moreover, as the method of construction affects generally the behaviour of the structure, specifications are needed for reinforced soil systems.

PAPERS SUBMITTED TO THIS CONFERENCE

ANDRAWES K.Z. et al - The behaviour of a geotextile reinforced sand loaded by a trip footing.
BOUTROUP E. et al - Analysis of embankments on soft ground reinforced with geotextiles.
BALAAM N.P. and POULOS H.G. - The behaviour of foundations supported by clay stabilized by stone columns.
BHANDARI R.K.M. - Behaviour of a tank founded on soil reinforced with stone columns.
BATTELINO D. - Some experience in reinforced cohesive earth.
BATEREAU C. et al - Improvement of ground though the application of geotextile.
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