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SOIL REINFORCEMENT IN HIGHWAY CONSTRUCTION

SUMMARY: The concept of reinforced soil as a construction material was first introduced by the French engineer Henri Vidal in the early 1960's with the invention of the Reinforced Earth Technique. This technique was the starting point for a new field of soil improvement, i.e. soil reinforcement and has since led to the development of other techniques for retaining walls, embankments, slopes, dams, foundations, etc. Soil reinforcement has been largely used in highway construction for almost 25 years. This paper presents the main aspects of research, design and construction related to soil reinforcement techniques in this area.

OJAČANJE TEMELJNIH TAL PRI GRADNJI AVTOCEST

POVZETEK: Koncept ojačanja temeljnih tal kot konstrukcijske rešitve je uvedel francoski inženir Henri Vidal v zgodnjih šestdesetih letih tega stoletja z odkritjem postopka za armiranje zemljin. Ta postopek je postal izhodišče za novo področje izboljšanja temeljnih tal, to je armiranje zemljin, kar je dalje privedlo do razvoja drugih novih postopkov za podporne zidove, nasipe, pobočja, pregrade, temelje, itd. Ojačanje temeljnih tal se na široko uporablja pri gradnji avtocest že skoraj 25 let. Pričujoči članek opisuje glavne vidike raziskav na tem področju, pri projektiranju in gradnji, povezanih z ojačanjem temeljnih tal. *(prevod urednika)*

SOIL REINFORCEMENT IN HIGHWAY CONSTRUCTION

by François SCHLOSSER - TERRASOL (France)

1. INTRODUCTION

The concept of reinforced soil as a construction material was first introduced by the french engineer Henri Vidal in the early 1960's, with the invention of the Reinforced Earth technique. This technique was the starting point for a new field of soil improvement, i.e. soil reinforcement and has since led to the development of other techniques for retaining walls, embankments, slopes, dams, foundations, etc. Soil reinforcement has been largely used in highway construction for almost 25 years. This paper presents the main aspects of research, design and construction related to soil reinforcement techniques in this area.

2. HISTORY AND DEVELOPMENT

The Reinforced Earth (R.E.) technique was originally used in France in 1968 with the construction of a highway retaining wall at Incarville. A R.E. retaining wall is composed of a backfill material having a suitable friction angle and reinforced by linear metal strips placed horizontally during wall construction. The facing is made of prefabricated concrete panels attached to the strips. Figure 1 shows the annual surface area of facing used in R.E. structures built around the world. Geographically, Europe represents 33 % of the projects, while the USA and Canada comprise 34 % of the total market. Considering the distribution by project type, 48 % are related to urban highways and 21 % to mountainous highways, comprised essentially of retaining walls and bridge abutments.

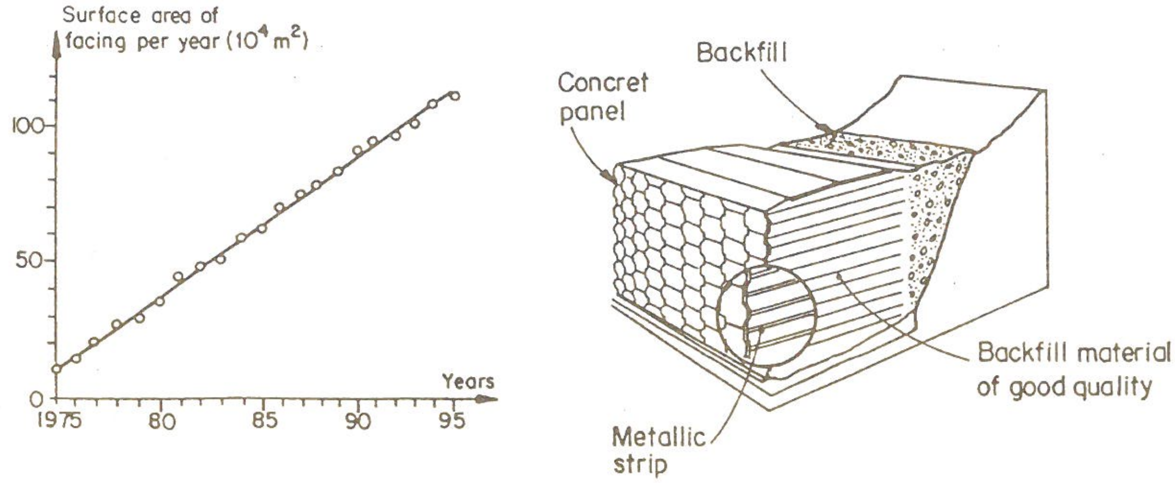


Figure 1 : Development of reinforced earth walls worldwide since 1975

In 1972 the first soil nailed retaining wall was constructed in France for the widening of a railway line in the vicinity of Versailles. As shown on figure 2, soil nailing consists of reinforcing an in-situ soil by placing nails, which are generally metal bars, placed in predrilled boreholes. On the contrary of a R.E. wall, a soil nailed wall is constructed starting from the top and the nails are placed as the soil is excavated. The facing is made by placing shotcrete after each excavation phase.

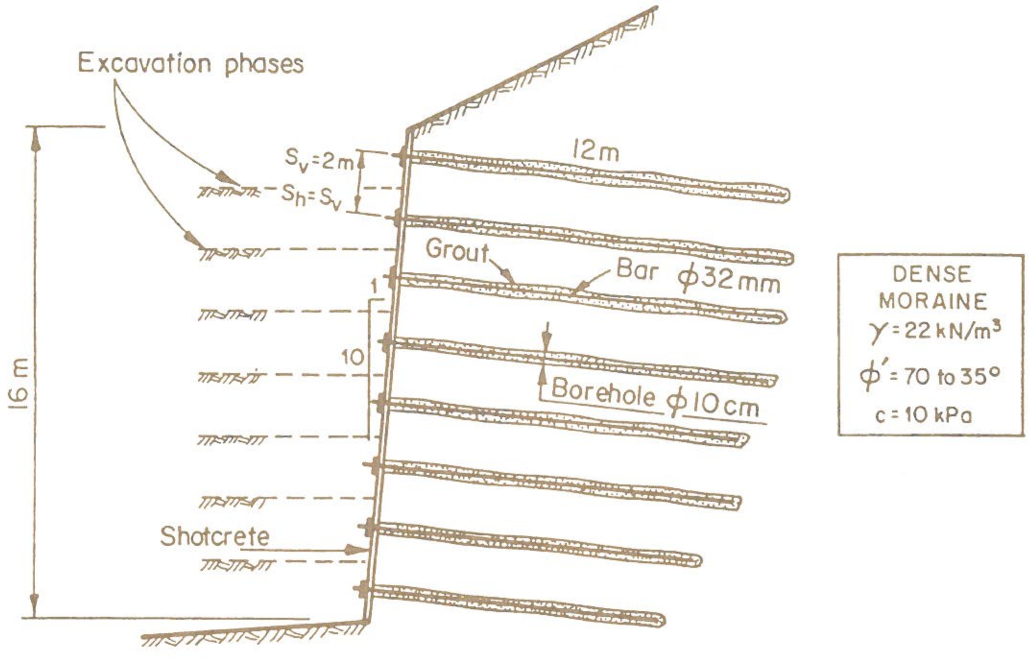


Figure 2 : Soil nailed wall (La Clusaz, 1981) Grouted nails

Two large research projects on Reinforced Earth were performed in Europe form 1967 to 1977, including reduced scale models and full scale experiments. The first project was performed in France at the Laboratoire Central des Ponts et Chaussées (LCPC) and led to the first design methods and specifications. A second project was performed some time later at the Transport Road Research Laboratory in the U.K., which dealt with technological aspects related to the facing and strips.

The initial research projects on soil nailing were essentially performed in Europe, with the first experimental project being performed in Germany on full scale retaining walls and a second project being performed in France from 1976 to 1991. This second experimental project, entitled CLOUTERRE, resulted in the publication of recommandations in 1991 and the edition of an english translation "Recommendations Clouterre 1991" by the U.S. Federal Highway Administration in 1993.

In tunneling, the New Austrian Method enables the construction of linings using radial bolts in the ground and a shotcrete facing. This technique was first used in 1965 and is an interesting example of rock bolting used as a reinforcement technique. More recently, nailing of the excavation face in tunnels has been used in Italy (Lunardi, 1991) for stabilizing the face and reducing the settlements at the ground surface. It appears to be an efficient technique (figure 3).

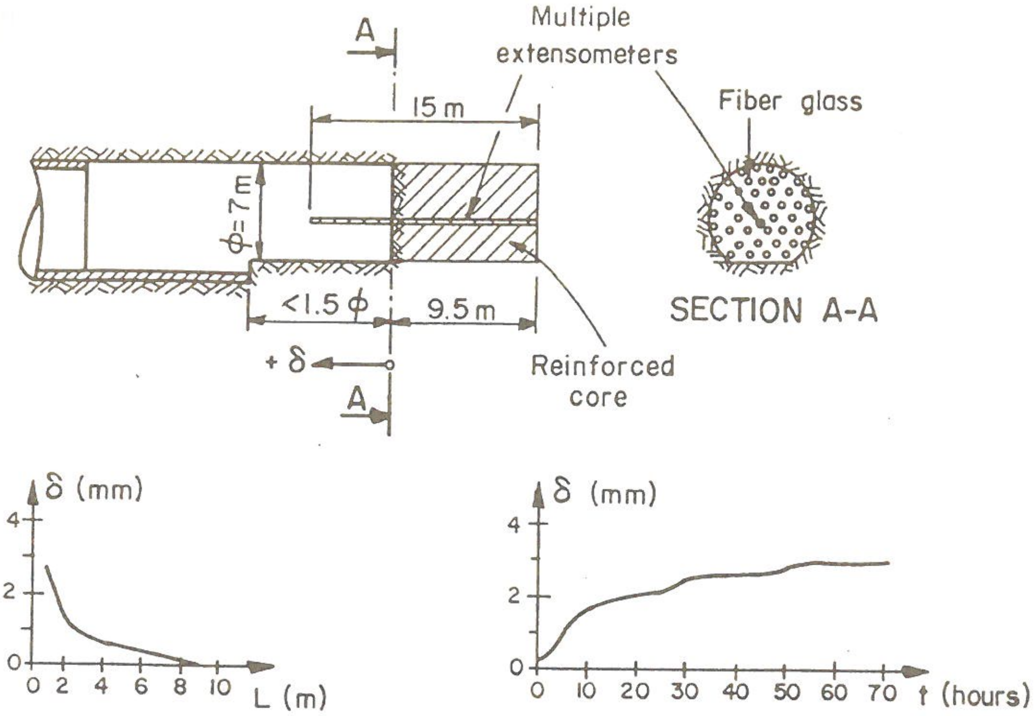


Figure 3 : Nailing of the face of a tunnel effect of a reinforced core (on the displacements)

Micropiles Network shave been extensively used by Lizzi in Italy for underpinning ancient monuments. However, the mechanism of such soil reinforced foundations is not very well known to date and no design method is available. A French National Research Project, entitled FOREVER (Fondations Renforcés par Eléments Verticaux), is presently being performed on this topic.

3. SOIL REINFORCED TECHNIQUES

The concept of soil reinforcement is based on the development of a strong interaction between the soil and the reinforcing inclusions. The most common interaction is friction, but passive pressure may also be mobilised. Frictional interaction requires good mechanical properties of the soil, particularly in terms of the friction angle and drainage : granular soils are the best adapted for this kind of reinforcement. Frictional interaction leads to the development of tensile or compressive forces in the inclusions. Passive pressure is generally associated with bending of the inclusions, which leads to shear forces and bending moments in the inclusions.

The various techniques for soil reinforcement can be classified in regards to the predominant type of forces generated in the inclusions (tension, compression, shear force and bending moment). Another classification can be made depending on the continuous or the discontinuous aspect of the mechanical interaction between the soil and inclusion. Depending on the inclusion type, two extreme cases can be considered (Schlosser et al, 1983):

1) a "uniform inclusion" for which the soil-reinforcement interaction is continuous and can develop at any point along the inclusion;

2) a "composite inclusion" which consists of an inclusion reinforced at specific locations where the soil-reinforcement interaction is concentrated. This interaction is therefore discontinuous along the inclusion.

Figure 4 shows the second type of classification. A discontinuous interaction is developed in multianchored walls, such as the "Ladder wall" invented by Coyne in 1926, where the inclusions are "deadman" anchors or the Anchored Earth technique developed by the Transport Road Research Laboratory, where the inclusions are linear metal bars bent at their extremity.

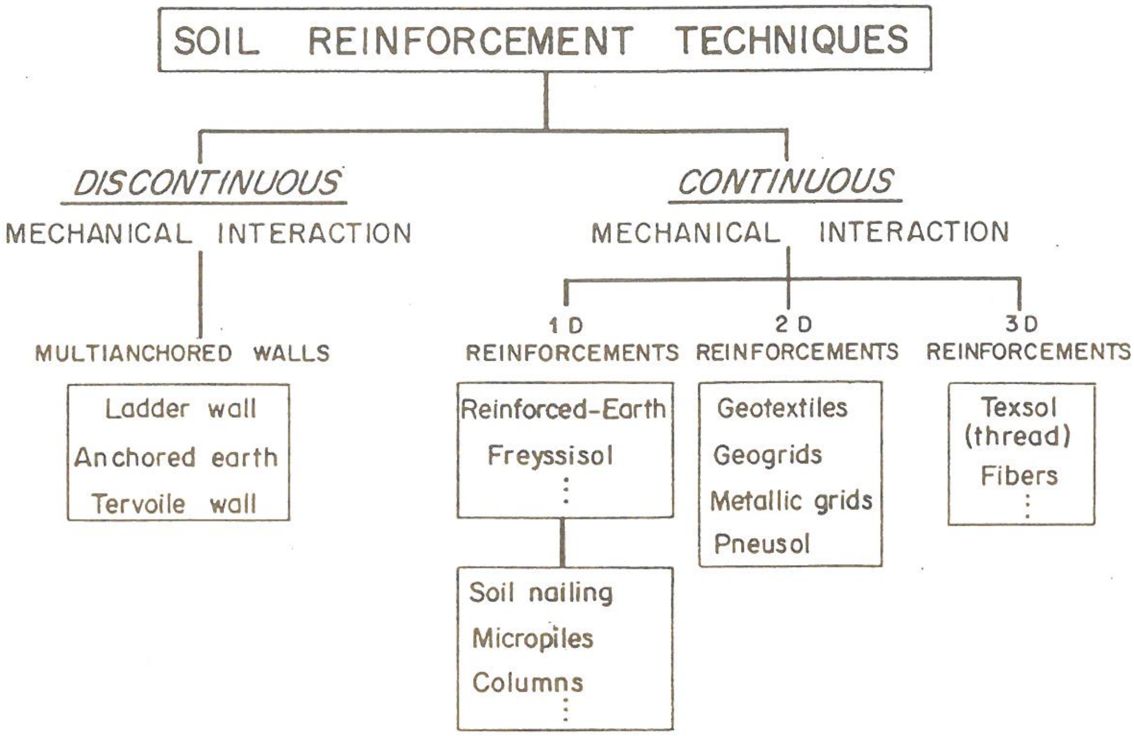


Figure 4 : Soil reinforcement techniques

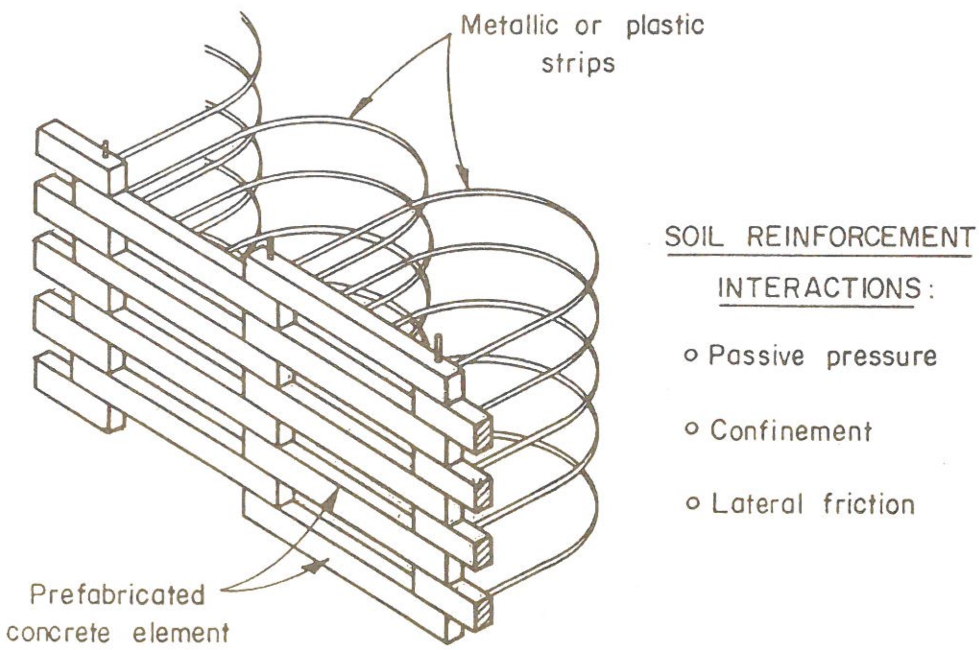


Figure 5 : The Tervoile technique (structural cell)

The Tervoile wall technique incorporates both soil reinforcement and mechanical stabilization similar to that of a crib wall. The facing consists of prefabricated elements and both extremities of the strips are connected to the facing, which creates cells analogous to those existing in a crib wall (figure 5).

The continuous mechanical interaction can be developed a long one-dimensional, two-dimensional or three-dimensional reinforcement. The most common type of inclusion is linear reinforcement, which are used in a large number of techniques : Reinforced Earth, Freyssisol (plastic strips), soil nailing, micropiles, columns (jet grouting, stone columns, deep mixing, etc.). Recently an inexpensive reinforced soil retaining wall has been developed using two-dimensional reinforcement. It associates geotextile sheets with a concrete blocks facing (figure 6). Each sheet is placed between two layers of concrete blocks and the connection is provided by friction. Generally the facing has to be inclined in order to ensure a good internal stability of the wall.

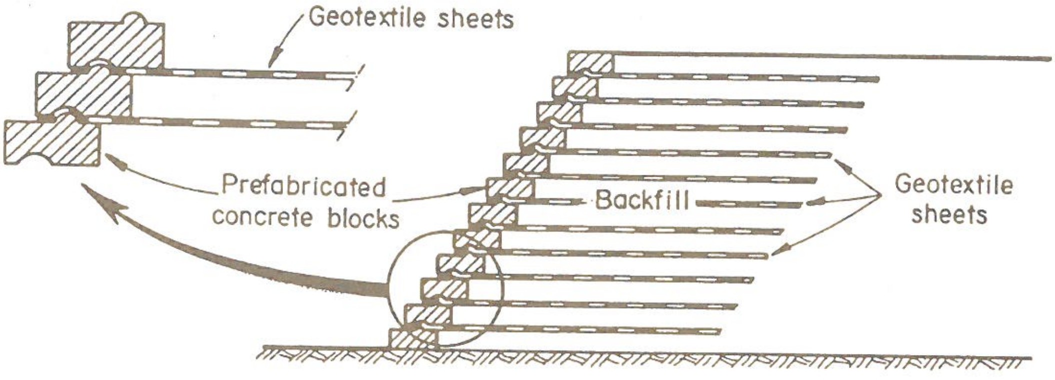


Figure 6 :Reinforced soil wall using geotextile sheets and a concrete block facing

3. MECHANISM, BEHAVIOR AND DESIGN

3.1. Active and resistant zones

Considering the mechanism and the distribution of the tensile forces along the reinforcement, a reinforced soil retaining wall is divided into two zones, the active zone and the resistant zone, as shown for the first time by Schlosser in 1971 (figure 7a). In the active zone, the soil tries to move away from the structure, but is restrained by friction developed along the inclusions. The mobilized shear forces are directed toward the front of the wall, which results in an increase in the tensile force with distance from the facing. Consequently, the maximum tensile force in the reinforcement does not occur at the wall facing but rather at some distance away from the facing. In the resistant zone, the shear stresses are oriented away from the facing and prevent slippage of the reinforcement at the soil/inclusion interface.

From the results of reduced scale models, full scale experiments and instrumented walls, it can be observed that the maximum tensile force line, which separates the active zone from the resistant one, is significantly different from that predicted by the Coulomb failure wedge for classical retaining structures. The presence of horizontal and practically inextensible reinforcement (metal reinforcement) restrains lateral deformations and consequently completely modifies the stress and strain patterns in the soil. In the case of a wall with a vertical facing and a horizontal top, the maximum tensile force line is vertical in the upper portion (figure 7b). Its distance from the facing at the top of the wall is equal to 0.3 H, where H is the height of the wall.

Extensible reinforcement induce lateral displacements, especially at the top of the structure. This leads to the mobilization of an active state of stress and to a maximum tensile force line close to the Coulomb failure plane (figure 7c).

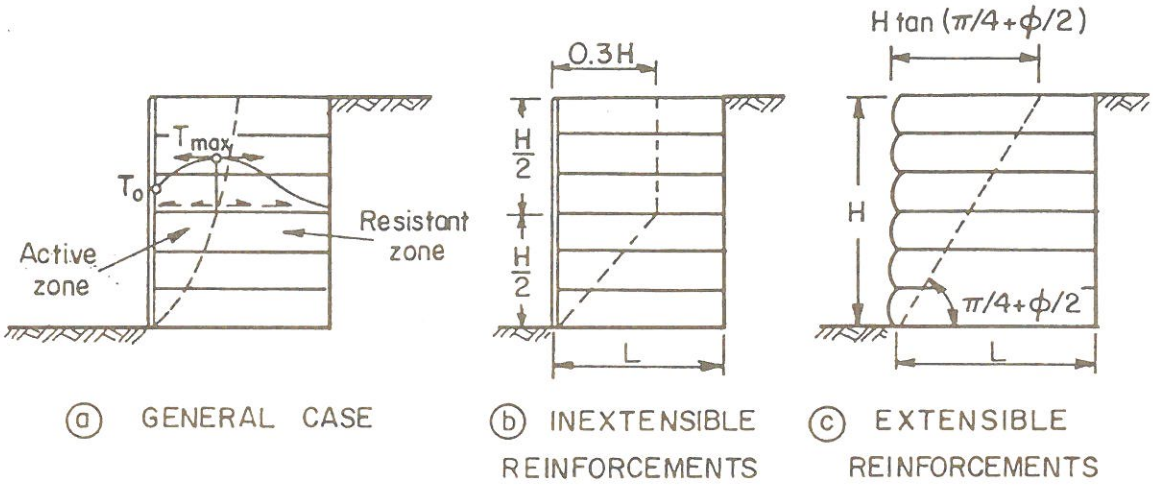


Figure 7 : Maximum tensile force line in a reinforced soil wall

Instrumented soil nailed walls have shown a similar shape for the maximum tensile force line with some slight differences due partially to the inclination of the nails (figure 8).

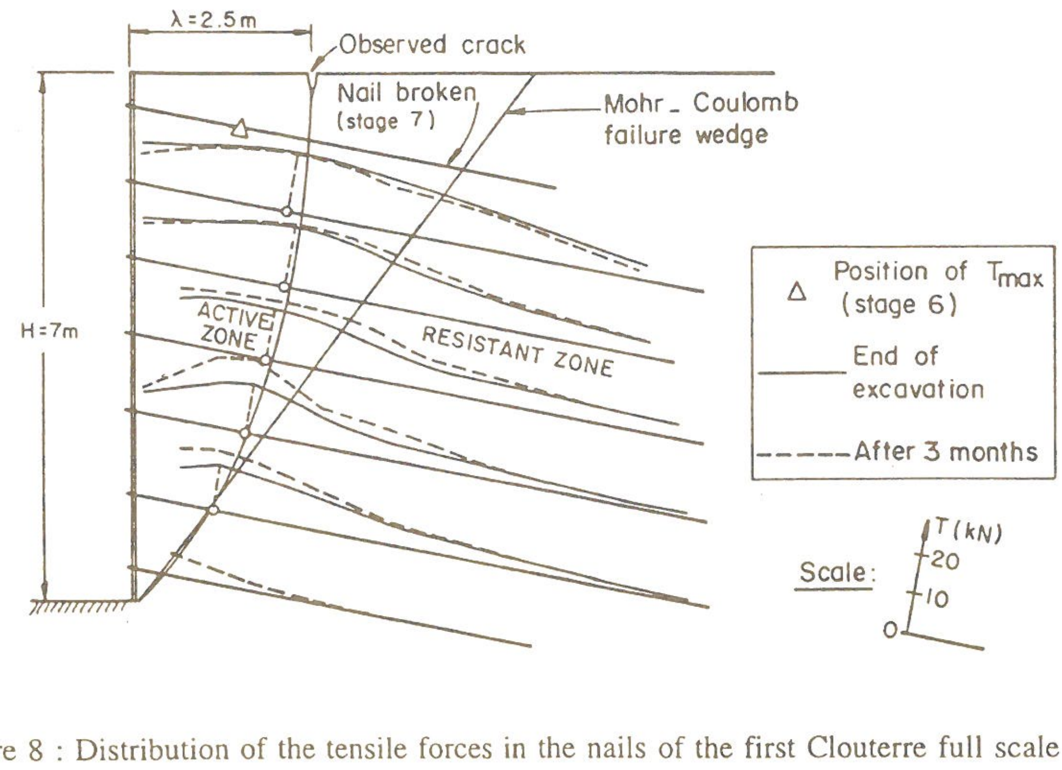


Figure 8 : Distribution of the tensile forces in the nails of the first Clouterre full scale experiment on a soil nailed wall

One can speculate if the line of maximum tensile forces occurring at the service state is different from that occurring at failure as a consequence of breakage of the reinforcements and whether the behavior is different from that predicted by the Coulomb failure surface. Although there are few full scale experiments on reinforced soil walls which have been pushed to failure, these walls have shown that the maximum tensile force line for inextensible reinforcements at the service state corresponds to the failure surface occurring at the final state. On the other hand, it is probable that this is not the case for extensible reinforcement, especially if the passage from the service state to the failure state is accompanied by large lateral displacements. Figure 9 shows the soil deformations and of the bending of the reinforcement for a soil nailed wall pushed to failure within the framework of the Clouterre french research project. A failure surface cannot develop inside of the wall because it is prevented by the mobilization of the bending stiffness within the inclusion which induces a zone of distorsion in the soil. However, it is observed that the maximum tensile force line is at the same location at the service state and at the failure state.

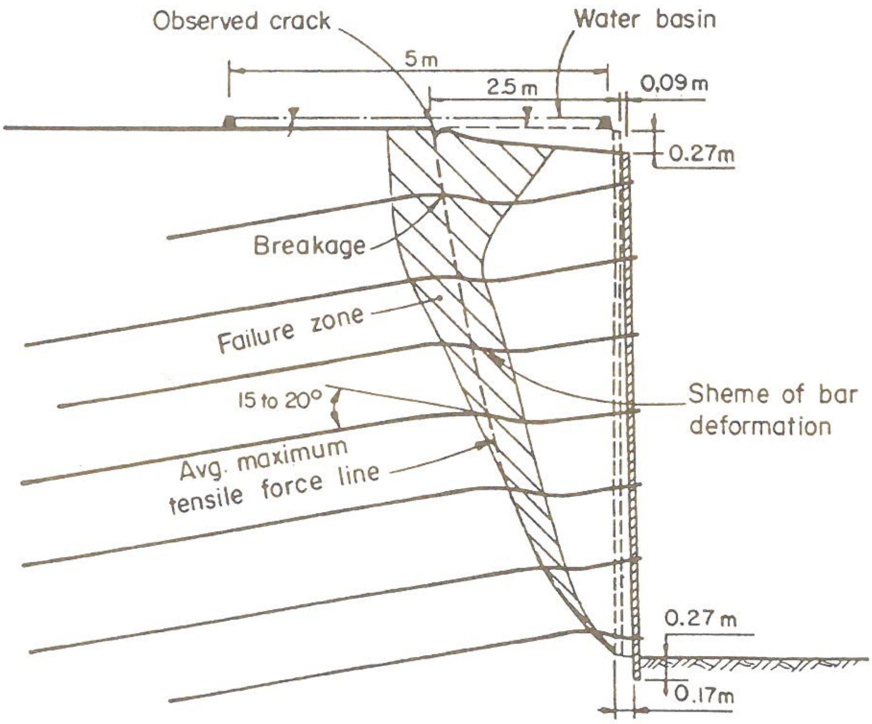


Figure 9 : First Clouterre full scale experiment at failure of a soil nailed retaining wall

In all reinforced soil structures where tension is the primary force generated in the reinforcement, the behaviour shows the development of an active zone and a resistant zone. For instance (figure 10) shows the results of tensile force measurements in the bolts of a lining constructed using the New Austrian Methods (radial bolting and shotcrete facing) for a highway tunnel near Nice. Despite the large scattering of the results, an active zone and a resistant zone can be observed.

3.2 Friction and restrained dilatancy effect

The mobilized resistance along a longitudinal reinforcement can be influenced by the effect of restrained dilatancy, resulting in the development of a 3-D friction mechanism. Pull-out of a linear inclusion (figure 11a) induces shear displacements in the zone of soil surrounding the reinforcement. In a compacted granular soil, this zone tends to dilate but the volume change is restrained by the surrounding soil, inducing an increase in the applied normal stresses on the inclusion. This behavior was first observed from the interpretation of pull-out tests on R.E. strips, which gave values for the friction coefficient significantly greater than the values measured in the laboratory. This has led to the concept of an apparent friction coefficient friction μ^*, which is related to the overburden pressure γh ($\mu^* = \tau/\gamma h$). The values for μ^* obtained from pull-out tests on actual structures are presented on figure 11a, in addition to those obtained from shear tests performed with no volume change, which represents the extreme case of restrained dilatancy. It can be observed that μ^* (or tan ϕ^*) measured in the pull-out or direct shear tests conducted at constant volume, decreases with the initial normal stress ($\sigma_o = \gamma h$), which is similar to the well-known phenomenon of the decrease of soil dilatancy with an increase in confining pressure.

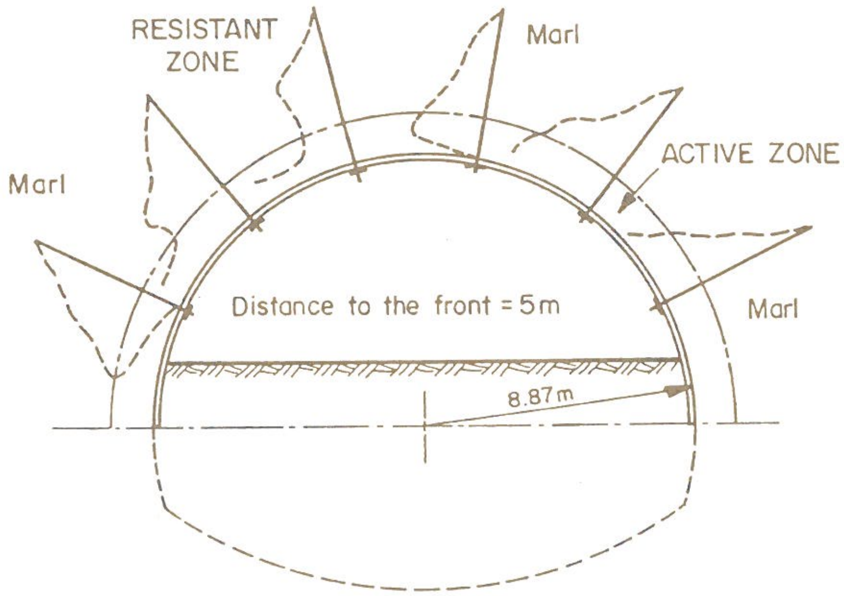


Figure 10 : Tensile forces distribution in the nails of the natm lining in the upper half-section of the las planas tunnel (Nice A8 highway)

The increase in the normal stress on the inclusion during pull-out was initially measured in West Germany by Wernick, using a special cell located on the bar to measure simultaneously the applied shear and normal stresses. Wernick demonstrated that the stress path followed during pull-out rapidly attained the Mohr-Coulomb curve ($\tau = \sigma \tan\phi$), with a maximum increase in the normal stress of approximately four times the initial value. In France, Plumelle was able to determine the approximate volume of the sheared zone surrounding the inclusion

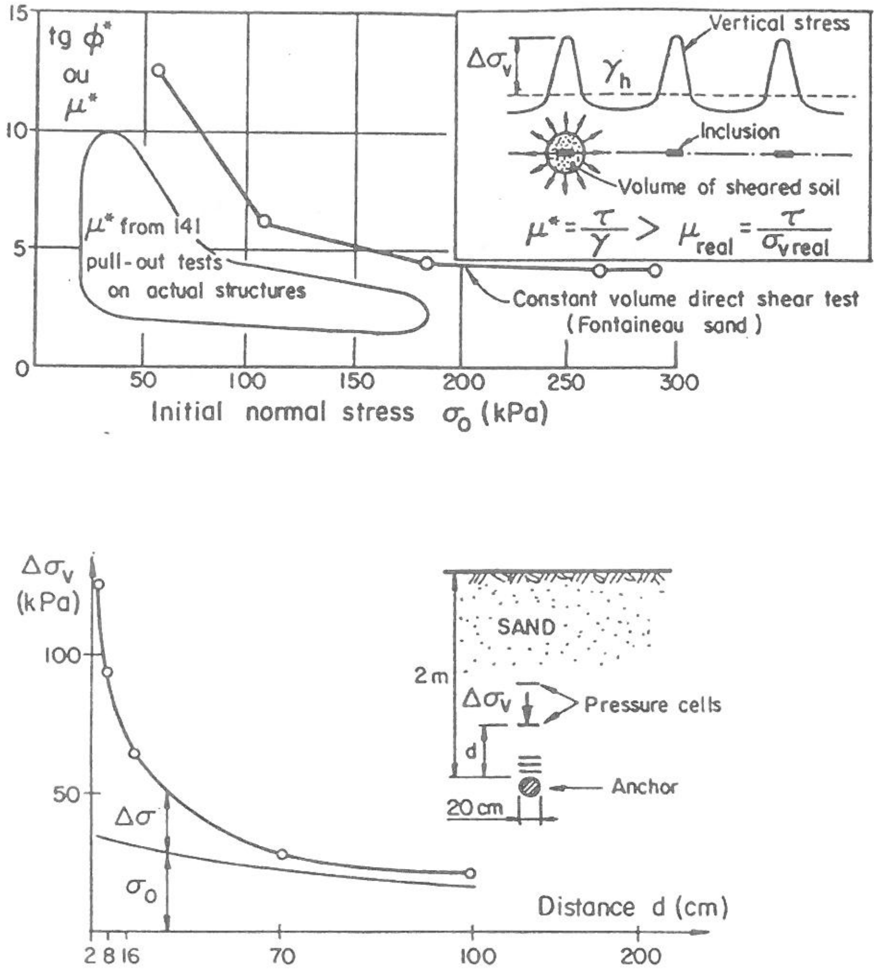


Figure 11 : Restrained Dilatancy Effect on the Soil-Linear Inclusion Friction

during pull-out. He used a tie-rod placed in a compacted Fontainebleau sand embankment and pressure cells located at different distances above the tie rods. As shown on Figure 11b, the influence of restrained dilatancy above the tie-rod has an effect on the applied normal stress for a relatively large distance from the inclusion (more than 3 diameters above the tie-rod).

In soil nailing, the shear stresses acting along the nails is not determined by considering an apparent friction coefficient, but rather by predicting the limit frictional shear stress τ_l, similarly to the skin friction developed along piles. It is interesting to note that τ_l is practically independent of the depth, since there is a compensation between the decrease in dilatancy and the increase of overburden pressure with depth. For backfilled reinforced soil walls, in accordance with specifications for Reinforced Earth, the apparent friction coefficient μ* is considered to decrease linearly with depth until 6m, after which it becomes constant.

3.3. Passive resistance

Passive resistance of the soil is mobilized when the inclusion is loaded in tension and includes protruding elements (large ribs, discs) and/or transversal members (bar mats), or when the inclusion has some bending stiffness and is sheared or bent along the zone of potential failure. Generally in these cases, the overall resistance is mobilized by a combination of both friction and passive resistance. To analyze the respective importance of these two phenomena, Bacot and Schlosser et al. performed pull-out tests on bar mats. In addition, Morbois and Long measured the mobilization of the pull-out resistance for rods equipped with circular transversal passive anchor plates (figure 12). It is interesting to note that while the resistance mobilized by friction reaches a maximum for very small displacements at the head of the rod (= 5 mm), the displacements required to mobilize passive earth pressure are much greater (over 25 mm). As a consequence, when both mechanisms occur, the friction is always completely mobilized before the passive resistance, indicating that the soil reinforcement friction is the most important interaction phenomenon.

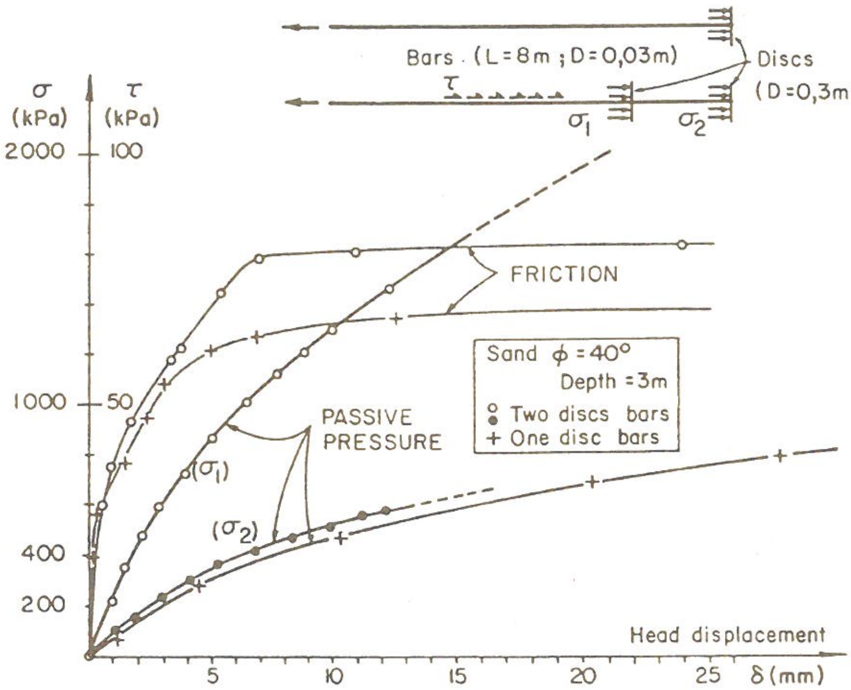


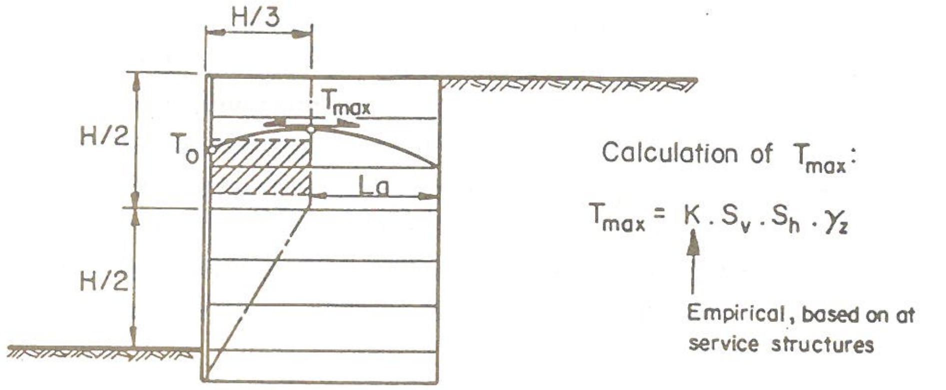
Figure 12 : Mobilization of Friction and Passive Pressure on Bars Equipped with Anchoring Discs (Morbois and Long, 1984)

5. DESIGN AND CONSTRUCTION

5.1. Design methods at the service state

The design of Reinforced Earth structures by the local equilibrium method can be considered a calculation method at the service state since the maximum tensile forces in the reinforcement are calculated empirically from measurements on actual structures taken prior to failure. In this method, the maximum tensile forces are determined from the value of the earth pressure coefficient (K), obtained from full scale experiments (French and U.S. specifications). The deformations, especially the lateral displacements, for walls designed using this method are generally observed to be very small and without consequence on the structure.

For a Reinforced Earth wall, the two following design criteria have to be checked at each level of reinforcement (figure 13) :



(1) $T_{max} \leqslant n \cdot A \cdot \sigma_e$ BREAKAGE OF THE REINFORCEMENTS

(2) $T_{max} \leqslant \int_0^{La} n \cdot 2\mu^* \, b\gamma_z \, . dx$ PULL OUT OF THE REINFORCEMENTS

Figure 13 : At the service state design method for Reinforced Earth walls

1) Failure by breakage of the reinforcement :

$$T_{max} \leq n.A.\sigma_e$$

where T_{max} is the maximum developed tensile force in the reinforcing strip, σ_e is the maximum allowable tensile stress, or elastic limit, of the reinforcing material and n is the number of strips per linear meter in the layer of reinforcement.

The maximum tensile force T_{max} in a reinforcement is equal to:

$$T_{max} = \sigma_{H.}S_v.S_h = K\sigma_v S_v S_h$$

where σ_v is the vertical stress at point of the maximum tensile force, calculated using Meyerhof's method based on the equilibrium of the portion of the Reinforced Earth mass above a given reinforcement under the effect of its weight, and the active earth pressure exerted on the wall by the embankment. S_v and S_h are the horizontal and vertical reinforcement spacings and K is an empirically determined coefficient which varies linearly with depth from K_o at the top to K_a at a depth of 6 m, and remains constant for larger depths ($K = K_a$).

The local equilibrium method can be adapted to the other techniques using backfill material reinforced by inclusions such as: metal or synthetic grids, or geotextiles. Christopher et al. (1990) used this procedure to evaluate the effect of the reinforcement's extensibility, represented by a stiffness factor (S_r), which is defined by the following formula :

$$S_r = EA/S_v.S_h$$

where : E is the Young's modulus of the reinforcement, A is the cross-sectional area of the reinforcement, S_v and S_h are the reinforcement spacings.

The effect of the stiffness factor on the earth pressure coefficient (K) has been experimentaly studied from the measurements made on a large number of full-scale experiments and observations made on actual structures. A schematic representation of the proposed K values obtained from this study is presented on Fig. 14 as a function of the depth from the top of the wall. At the top of the wall, where overstresses caused by the compaction process occur, the K coefficient decreases with a corresponding increase in the reinforcement's extensibility until it attains a value of K_a for geotextiles. It can be observed that the value of K decreases with the depth and in general, attains the active state (K_a) for depths greater than 6 m. However, the K coefficient for the metal grids (welded wire mesh and bar mats) is always superior to K_a, even at large depths. This is probably caused by an increased effect of the soil compaction, which does not decrease with depth. The value of the K coefficient determined as a function of the stiffness factor is given by the empirical formula given on figure 14 where S_r is in MPa and Ω_1 and Ω_2 two geometrical factors.

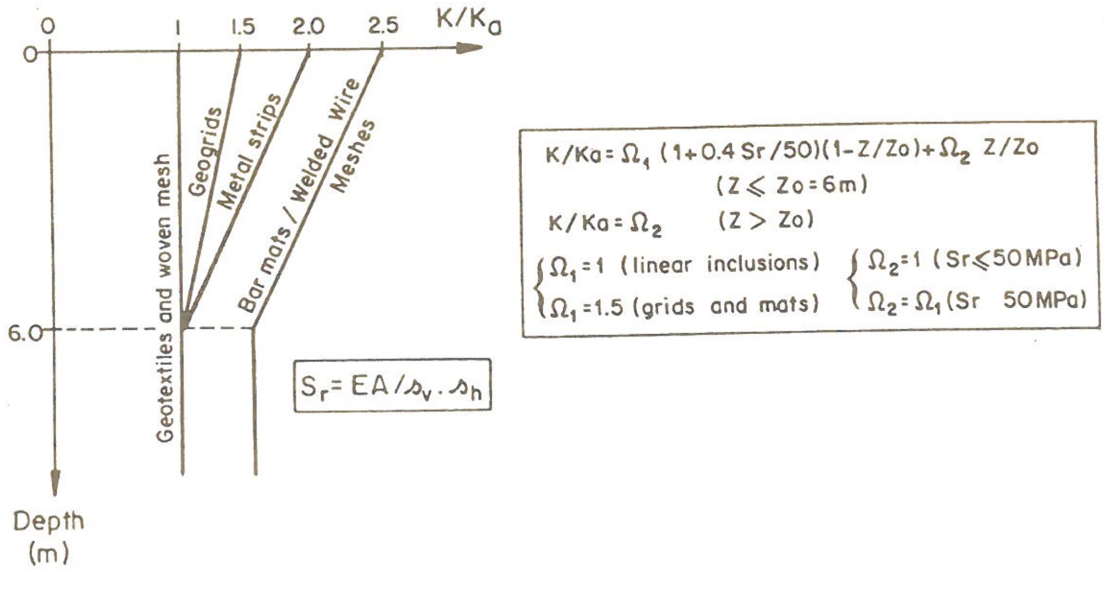


Figure 14 : Variation of K with depth and reinforcement type (Christopher et al, 1990)

The study of the maximum tensile forces generated in the reinforcement, as a result of the interface lateral friction developed in the resistant zone, requires that one knows beforehand the maximum tensile force line. Christopher et al. (1990) has stated that it is necessary to first distinguish between inextensible and extensible reinforcement, since the latter can lead to large lateral displacements and consequently, the maximum tensile force line will shift approximately to the Mohr-Coulomb failure surface. For inextensible reinforcements on the other hand, the maximum tensile force line is vertical at the top of the wall and much closer to the facing.

2) Failure by pull-out of reinforcement :

$$T_{max} \leq 2bn \int_{xo}^{L} \tau(x)\,.dx \quad with \; = \; \tau(x) = \; \mu*.\tau_v(x)$$

where x_0 is the width of the active zone at a given level, L is the total length of the reinforcement, $\tau(x)$ is the mobilized shear stress along the soil/reinforcement interfaces at point (x), $\mu*$ the apparent friction, $\sigma_v(x)$ the vertical effective stress on the soil-reinforcement interfaces at point (x), and b is the width of the strips.

The apparent coefficient $\mu*$ decreases linearly with depth until 6 m because of the dilatancy effect and remains constant for larger depths.

5.2. Design methods at the ultimate limit state

Design at the Ultimate Limit State is used for soil nailing because the geometry of the structure, may be highly variable : inclination and length of the nails, inclination of the facing, etc.

The procedure employed is analogous to the classical stability analysis used for unreinforced soil slopes and retaining walls, where the stability of a soil mass is calculated with respect to a potential failure or sliding surface. It consists of determining the global stability of the reinforced soil volume considering the driving forces (i.e. weight of the soil) and the resisting forces of both the soil and the reinforcement. The resisting forces are comprised of two different types : the stresses (σ, τ) generated in the soil and the forces (T_n , T_c) generated in the reinforcement, which are applied at the point of intersection of the inclusions with the failure surface.

Grouted metal bars commenly used in soil nailing generally have a bending stiffness, which can have a beneficial effect on the overall resistance, since the bars are able to develop both bending resistance and shear forces, addition to the tensile force.

For the stability analysis, four failure criteria related to the following phenomena are to be considered, as presented on figure 15

1) complete plastification of the bar across a section, leading to the breakage of the bar;

2) soil-bar friction interaction;

3) soil plastification caused by the normal reaction pressure p between the soil and the bar;

4) complete mobilization of the shear resistance of the soil along the failure surface (Mohr-Coulomb criterium).

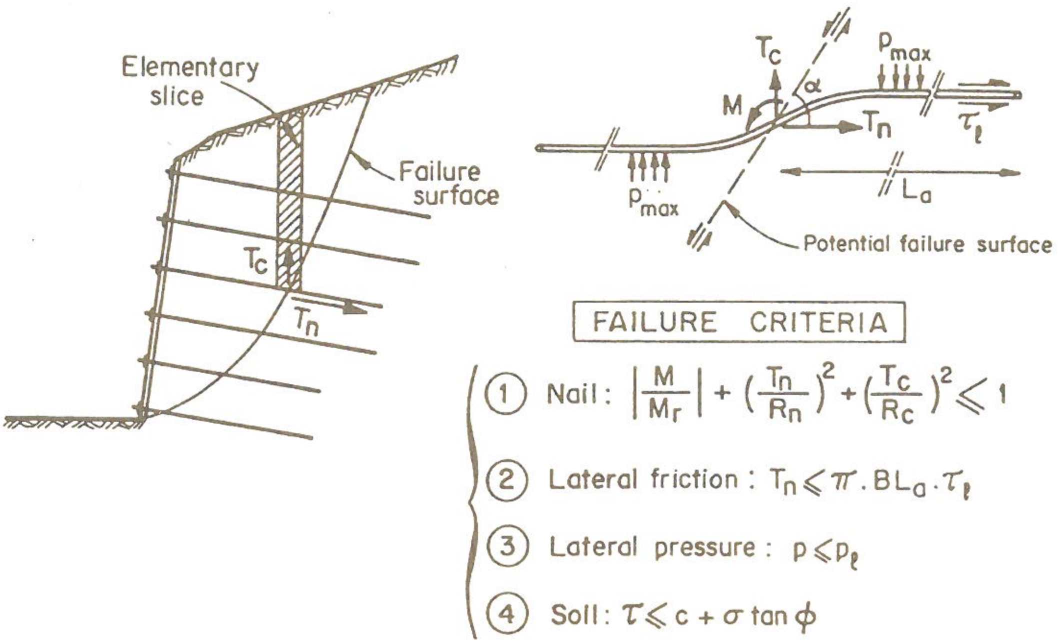


Figure 15 : Design at ultimate limit state in soil nailing

The resistance of the nail (criterium 1) can be expressed by a relation between the bending moment M, the normal force Tu and the shear force T_n as indicated on figure 16.

From the detailed analysis of the soil/inclusion interaction mechanisms, the combination of the four critera previously mentioned determines a domain of admissible forces which can be represented in the (T_n, T_c) plane at the point where the nail intersects the potential failure surface, and the point where there is no bending moment (figure 16). Using this multicriteria theory and applying the law of maximum plastic work, it is then possible to determine the forces in the nails (T_c, T_n) along the potential failure surface (figure 17).

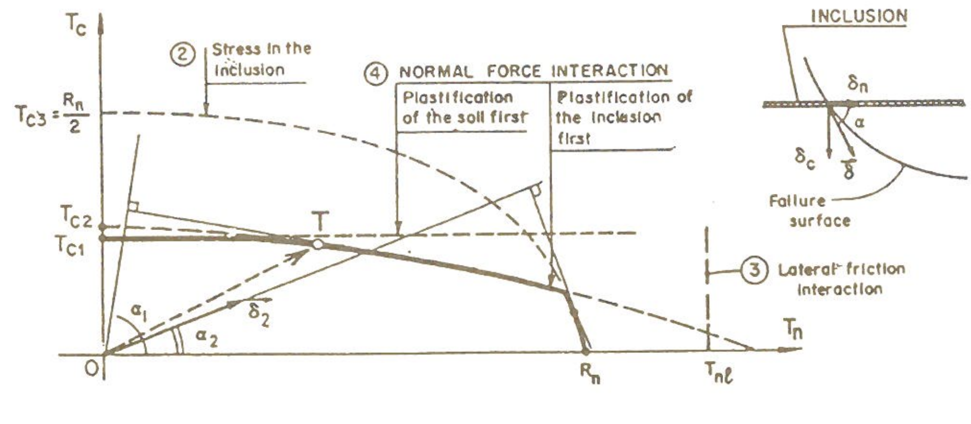


Figure 16 : Principle of maximum work

6. The TALREN program

In 1980, Terrasol (France) developed the stability analysis program Talren, using the slices method and taking into account the effect of various types of inclusions (nail, anchor, brace, reinforcing strip, geotextile, pile, micropile, sheetpile, etc.) placed in the soil. Its development was carried out concurrently with experimental research on soil-inclusion intereaction and the design of actual structures.

This program, which has been marketed worldwide, is now considered as a very comprehensive stability analysis program for geotechnical structures along potential failure surfaces. It considers all kinds of hydraulic or seismic data and can analyse circular and non circular failure surfaces.

The forces in the reinforcing elements which are to be considered at the intersection with the failure surfaces are as follows (figure 17) :

. the axial force for nails, anchors, braces or reinforcing strips;

. the shear force and bending moment for nails working in compled tension/shear or pure shear (piles and micropiles work in shear and bending when used for slope stabilization).

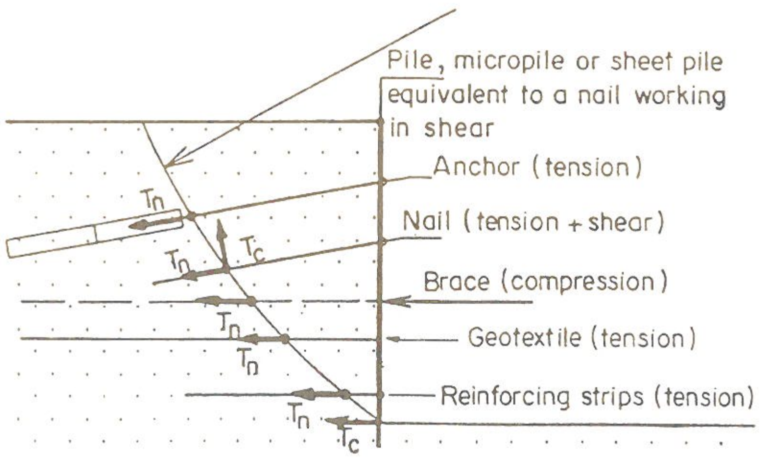


Figure 17 : Forces for the various types of reinforcing elements

Talren takes into account the multicriteria theory which associates the failure criteria related to the soil, reinforcement and the soil/inclusion interactions (lateral friction, lateral pressure between the reinforcements and the soil).

In the stability analysis performed by Talren, the stability of the geotechnical structure can be represented by a classical global safety factor (F.S.) or by partial safety factors in an analysis at the Ultimate Limit State.

The analysis method at the Ultimate Limit State consists in comparing, along the potential failure surfaces, the shear stresses generated by the loads, to the maximum shear resistance. Each parameter is factored by a weighing factor (for loads) or by a partial safety factor (for resisting forces). The static equilibrium is thus given by :

$$\Gamma_{s3}\ \tau \leq \tau_{max}$$

To change the expression into an equality, an additional coefficient Γ_{max} is incorporated, which gives :

$$\Gamma\Gamma_{s3}\ \tau = \tau_{max}$$

where :

$$\tau = \tau(\Gamma_{s1}G,\ \Gamma_Q Q) + \tau_R \left(\frac{FR}{\Gamma_{mR}}\right)$$

$$\tau_{max} = \tau_{max}\left(\frac{\tan\phi}{\Gamma_\phi}\ \frac{c}{\Gamma_c}\right) + \tau_{maxR}\left(\frac{FR}{\Gamma_{mR}}\ \frac{\tan\phi}{\Gamma_\phi}\right)$$

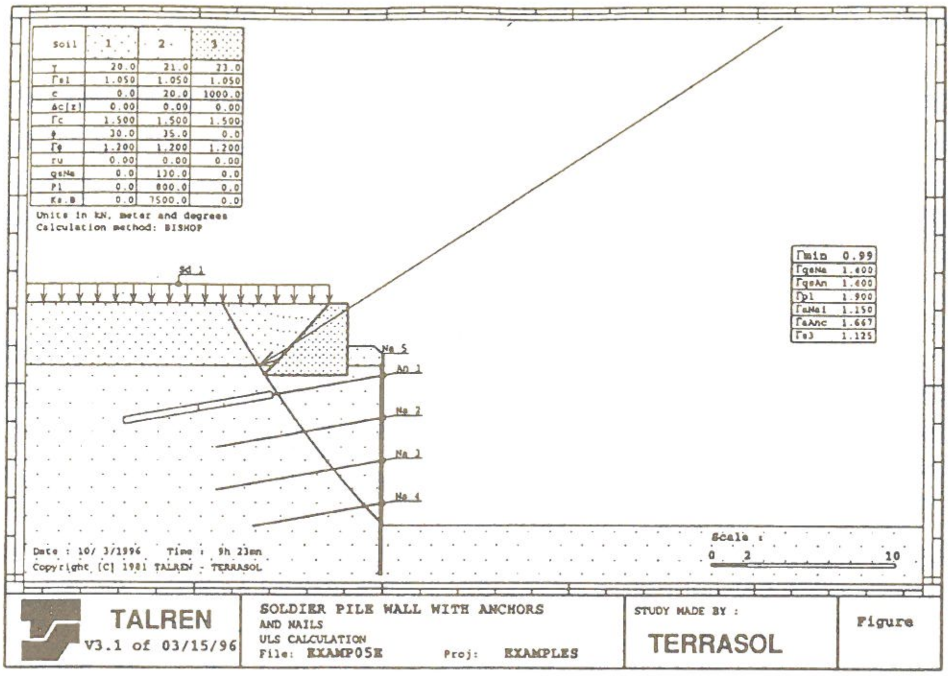
Γ_{s3} = coefficient related to the uncertainty of the analysis method;
Γ_{s1} = weighing factor on the soil unit weights;
Γ_Q = weighing factor on the loads;
Γ_{mR} = partial safety factor on the reinforcement effet (Γ_{qs} for the soil/inclusion lateral friction, Γ_{ml} for the soil limit pressure, Γ_{mR} for the tensile or compressive strength of the reinforcement);
Γ_ϕ = partial safety factor on the soil internal friction angle (tan ϕ);
Γ_c = partial safety factor on the soil cohesion;
Γ = additional coefficient to obtain an equality and whose minimum value Γ_{min} for the set of failure surfaces should be greater or equal to 1.

Talren benefits of years of experience acquired in the area of designing reinforced soil structures and the program has been calibrated on failed walls, either occuring naturally or artificially pushed to failure the main cases of back-calculations of failed walls are as follows:

1978 Madrid (Reinforced Earth wall) : provoked failure by accelerated corrosion
1981 Les Eparris (soil nailed wall) : repair after failure
1984 Orchard station (soil nailed wall) : repair after failure
1985 Experimental soil nailed wall CLOUTERRE : provoked failure

The design example given on figure 18 deals with the case of a "soldier pile" retaining system composed of temporary vertical micropiles retained by a row of prestressed ground anchors at the top and by three rows of nails. The piles and nails withstand shear forces and bending moments in addition of compressive and tensile forces, while the prestressed ground anchors can only withstand tensile forces.

The differences in the uncertainties on the strengths of the various components (friction angle, cohesion, lateral friction, tensile strength, etc.) make it necessary to consider different values for the partial safety factors. For instance, the value of the partial safety factor on applied to the friction angle is 1.3 and the factor applied to the cohesion is 1.65.



Soil	1	2	3
γ	20.0	21.0	23.0
Γs1	1.050	1.050	1.050
c	0.0	20.0	1000.0
Δc(z)	0.00	0.00	0.00
Γc	1.500	1.500	1.500
φ	30.0	35.0	0.0
Γφ	1.200	1.200	1.200
ru	0.00	0.00	0.00
qsNa	0.0	130.0	0.0
Pl	0.0	800.0	0.0
Ks.B	0.0	7500.0	0.0

Figure 18 : Use of Talren for a stability analysis of a soldier wall

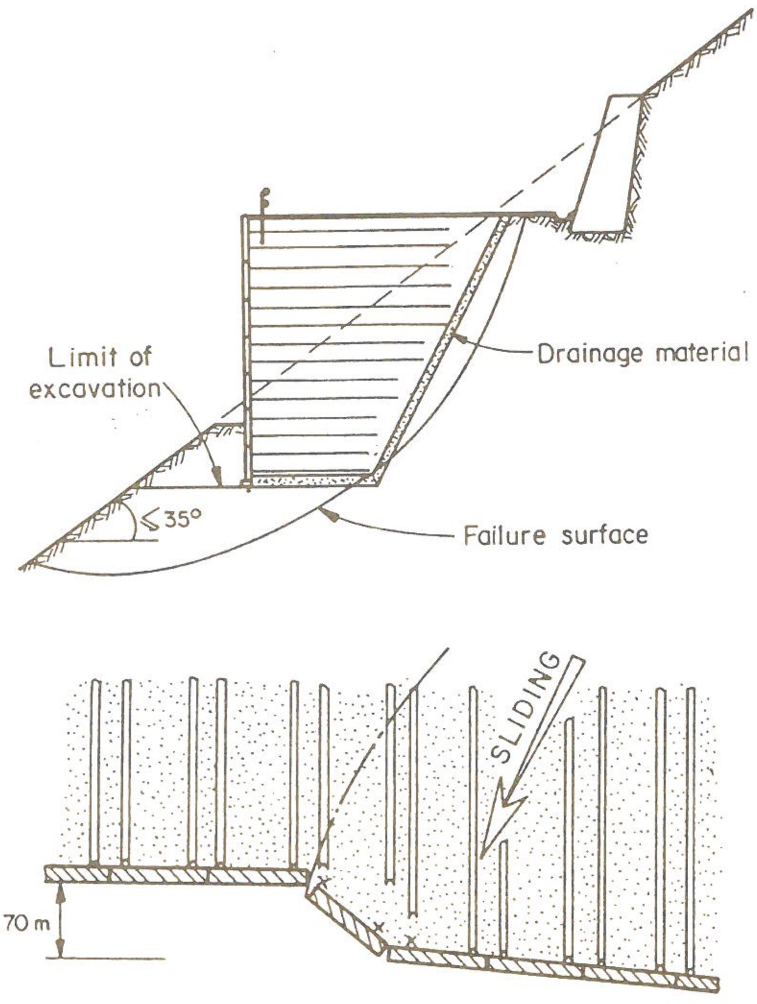


Figure 19 : Reinforced Earth wall construted for the highway access to the Frejus tunnel

7. Typical examples of reinforced soil structures in highway construction

The use of reinforced soil structures presents advantages compared to classical structures. For retaining structures these advantages are as follows :

. low cost;
. rapid and easy construction;
. simultaneaous execution with other earth works;
. large differential settlements accepted.

These advantages result from the fact that the main component of a reinforced soil structure is the soil mass.

An example of a large Reinforced Earth wall built in 1977 for the access highway to the Frejus tunnel in the Alpes in shown on figure 19a. This highway had to be constructed on a steep mountainous slope, inclined at approximately 30° to 35° with respect to the horizontal. In addition, the slope was sliding at a very great depth and at a rate varying from 04 cm to 30 cm per year. The adopted solution was to locate the highway at the top of a very large Reinforced Earth wall. In order to limit the volume of the excavations, the length of the strips were reduced to a minimum at the bottom of the wall.

Just after the construction of the wall, a large differential horizontal movement occured in the slope at the limit of the sliding zone. This resulted in a shearing of the wall which was particularly visible along a narrow zone in which the facing panels were highly damaged (figure 19b). The repair was relatively easy to perform by dismanteling and partially reconstructing the wall in this zone.

A recent example of soil nailing is the widening of the highway A12 in the south-west vicinity of Paris. The old highway had been constructed in a certain areas zone by excavating in the Fontainebleau sand. At this location, two superimposed soil nailed walls were constructed for the widening work without any disruption or reduction of the traffic. The figure 20 shows the cross-section of the walls, whose

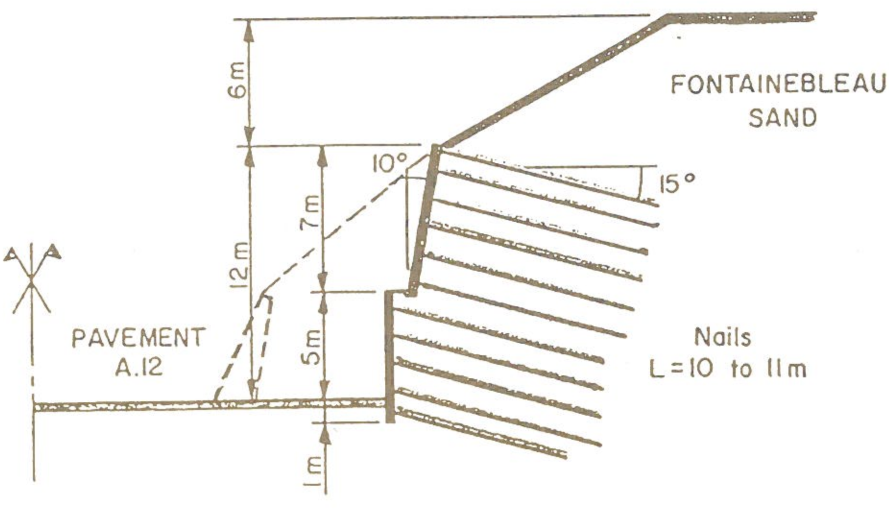


Figure 20 : Enlargement of the A.12 highway near Paris

nails were metal bars, placed in predrilled boreholes and grouted. Some problems occured with local instability of the sand during the excavation phases. It was therefore necessary to excavate over a reduced section and limit as much as possible the delay between the excavation and the construction of the facing. A particular attention was given to the construction of the facing whose shotcreting was made by phases in thickness and furthermore, the facing was constructed with a relatively dry concrete in order to obtain the best aesthetic aspect possible.

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