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soil reinforcement by geotextile (case study).

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تشـــكـــر

اخترت أن اكتب تشكري باللغة العربية فلا لغة أفضل منها لأخرج و اعبر عن حمدي و سعادتي بإكمال هدا العمل و أن اعبر عن هده التجربة الرائعة التي خضتها.

أولا احمد الله تعالى سبحانه و تعالى حمدا تسعه السماوات و الأرض لأنه وفقني لكل هذا و أعطاني القوة لأكمل دراستي و أطال عمري لأعيش هذه التجربة الرائعة و ليس هذا إلا بداية المشوار .

أقدم هذا العمل المتواضع لوالدي اللذان سهرا علي و على دراستي ... اقل ما يقال عنهما أنهما الأفضل بدون منازع ..و لعائلتي الرائعة من أخواتي و أخواي لكم جزيل الشكر .

اشكر أستاذي الرائع الدكتور منعة لزازي الذي كان والدي الثاني بشهادة جميع الطلبة و اقول له شكرا و ألف شكر لكل ما فعلته و عن مستواك الراقي في التعليم تستحق كل كلمات الشكر و الله وحده يعلم .

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صديقتي و رفيقتي في الجامعة نجوى رباحي .

و لا أنسى إسماعيل خليل و أمين بوعشة بارك الله فيكما و رزقكما من واسع فضله و كل طلبة ماستر 2 جيوتقني و كل من درس معي.

لداعمتي خلف الكواليس دائما و ابدا شايب ستي مدينة.

أساتذتي خاصة الأستاذ مسعد علي الذي أمدنا بالطاقة الايجابية من أول يوم درسنا عنده جزاك الله عن كل كلمة طيبة الله وحده يعلم ما غيرته بداخلنا .

اهدي هدا العمل للأستاذة و رفيقتي في الجامعة الأستاذة حريحري وداد التي لم تبخلني يوما بكلمة داعمة حين جفت الايجابية مني لكي مني جميل الدعاء .

و لكل أساتذتي الأفاضل بارك الله فيكم و جزاكم الله عني خير الجزاء .

و للأستاذ رحمة الله عليه عبد القادر دعوة بعد كل صلاة.

و بعد كل شيء الحمد لله .

الطالبة قمانة مريم .

Abstract The construction of embankments on compressible soil often causes difficult damage (stability and settlement). The use of reinforcing geosynthetics in embankments has attracted the interest of many geotechnical researchers to ensure the stability of embankments and reduce their settlement.
The main objective of this study is to conduct a numerical analysis of the geotextile-reinforced high-fill behavior of the penetrant project linking the geosynthetic-reinforced CHIFFA-BARROUAGIA national road on compressible soil.
The numerical analysis was performed using Plaxis computation code to determine the effect of geotextile on settlement evolution and embankment stability.
Keywords: Stability, fill, settlement, numerical modeling, geotextile.

Résumé :
La construction des remblais sur sol compressible pose souvent des dégâts difficiles (stabilité ettassement). L'utilisation des géosynthétiques de renforcement dans les remblais a suscité l'intérêt des nombreux chercheurs dans le domaine de la géotechnique, afin d'assurer la stabilité de remblai et réduire leur tassement.
L'objectif principal de cette étude est de procéder à une analyse numérique du comportement d'un remblai de grande hauteur renforcé par géotextile du projet de la pénétrante reliant la route national CHIFFA-BARROUAGIA renforcé par géosynthétiques reposant sur un sol compressible.
L'analyse numérique a été effectuée à l'aide de code de calcul Plaxis en vue en de voir l'effet de géotextile sur l'évolution du tassement et la stabilité du remblai.
Mots clés : Stabilité, remblai, tassements, modélisation numérique, géotextile.

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الملخص

غالبًا ما يتسبب بناء السدود على التربة القابلة للانضغاط في حدوث أضرار صعبة (الاستقرار و

الهبوط). أثار استخدام مواد البناء الجيولوجية في السدود اهتمام العديد من الباحثين في مجال الجيوتقنية ، من أجل ضمان استقرار الجسر وتقليل استقراره.

الهدف الرئيسي من هذه الدراسة هو إجراء تحليل عددي لسلوك السد الشاهق المعزز بتكسية أرضية للمشروع المخترق الطريق الوطني (شفة-برواقية)الذي يربط بين المرتفعات ، والمدعوم بمواد صناعية تعتمد على التربة القابلة للانضغاط.

لمعرفة تأثير التكسية الأرضية على تطور الاستقرار واستقرار Plaxisتم إجراء التحليل العددي باستخدام كود الكمبيوتر الردم.

..الكلمات الرئيسية: الاستقرار ، الحاجز ، المستوطنات ، النمذجة العددية ، التكسية الأرضية

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NOTATIONS

The following ratings are used in this submission:

γ_d : Dry weight;

Cu : undrained cohesion;

ΔCu : increasing cohesion;

B : Width of embankment;

H : depth of embankment;

Φ : angle of friction;

F_s : Safety factor;

γ_r : Weight of embankment;

q_{max} : The maximum allowable pressure;

M_{RMAX} : The maximum resistive moment;

M_M : The driving moment;

E : modulus of elasticity;

υ : Fish coefficient;

$\Delta\sigma_z$: Increasing vertical stress to Z depth;

I : influence coefficient;

e_0 : The voids index;
Cs : swelling index;
$\Delta\sigma$: Backfill load;
Rt : the total height of the embankment;.

Hr : hauteur prévue ;
T : pourcentage de tassement prévue ;
$\sigma_{prêche}$: contraint de préchargement ;
Pl : pression limite ;
WL : limite de liquidité ;
WP : limite de plasticité ;
IP : Indice de plasticité ;
W : teneur en eau ;
ε : déformation latérale ;
Pc : pression de consolidation.

GENERAL INTRODUCTION

The use of reinforcements represents a revolution to the concept, generally, used in geotechnical projects. Indeed, not as a new practice, as fibrous elements associated with soils have been in use for thousands of years. Incas, Babylonians and Chinese adopted soil, bamboo and straw in their constructions. In addition, birds, such as the Red Ovenbird, and beavers use similar procedures.

However, it is understood that the rational use of the thechnics began un the 1960's ,with Henri Vidal , who patented and developed a methodology for a system design that went to be called "Terre Armé" . Externally a reinforced soil wall is no different from a conventional structure. The particular issue in internal stability. Much better mechanical performance of the soil. Slopes and landfill heights that would otherwise be impossible, become feasible.

The present memory .....consists to understand the disused technics and its various applications. Methods of stability and bearing ability were also exposed. Presented work is sensed on 03 principal chapters: first, Bibliographic search second, affected to geological and geotechnical context of analysed road project case (NR-PK37+542 in Berouaguia.W of Medea). Thirst, to numerical and stability study and..... Thesis is started with general introduction and ended by general conclusion.

Chapter 1

Bibliographic search

1.1. INTRODUCTION:

Retaining stuctures are indispensable elements of a great variety of engineering projects such as bridges , piers , docks , railways , buidings in general , power plants , dams etc. Its function is basically to withstand earth pressureses , avoiding landslide , allowing the use of the space in front of the slope and its upper part .

For many years , traditional reinforced or cyclopean concrete retaining stuctures , whether anchored or not , were the only available option.Howevere , with increased soil heights to certain or in cases of less competent foundation soils , the cost of these structures increases considerably .

To overcome these difficulties , various types of retaining structures in reinforced soils were developed , which due to its advantages , gradually replaced traditional structures , in most cases. These structures are, in a final analysis,gravity walls where the construction material of the structure consists of soil associated with tensile resistant inclusions.

According to Motchell and Villet (1987),there are cases of retaining structures built with soil reinforced by vegetable fibres long before the emergence of reinforced concrete. There are examples of applications of this association by Mesopotamainz about 5 thousand years ago, such as the Ziggurat of Ur,in modern Iraq Ziggurat were temples builts by layers of mud reinforced with needs and branches,whose heights reached 50 meters or more .The facing was compoced of mud bricks dried in the sun ,held toghether by asphalt grout / mortar. In China , branches of trees were used for reinforcement of earth dams for least a millennium , and the famous great Wall of China has stretches of reinforced soil.Figure 1.1 shows an artistic representation of the Ziggurat of Ur.

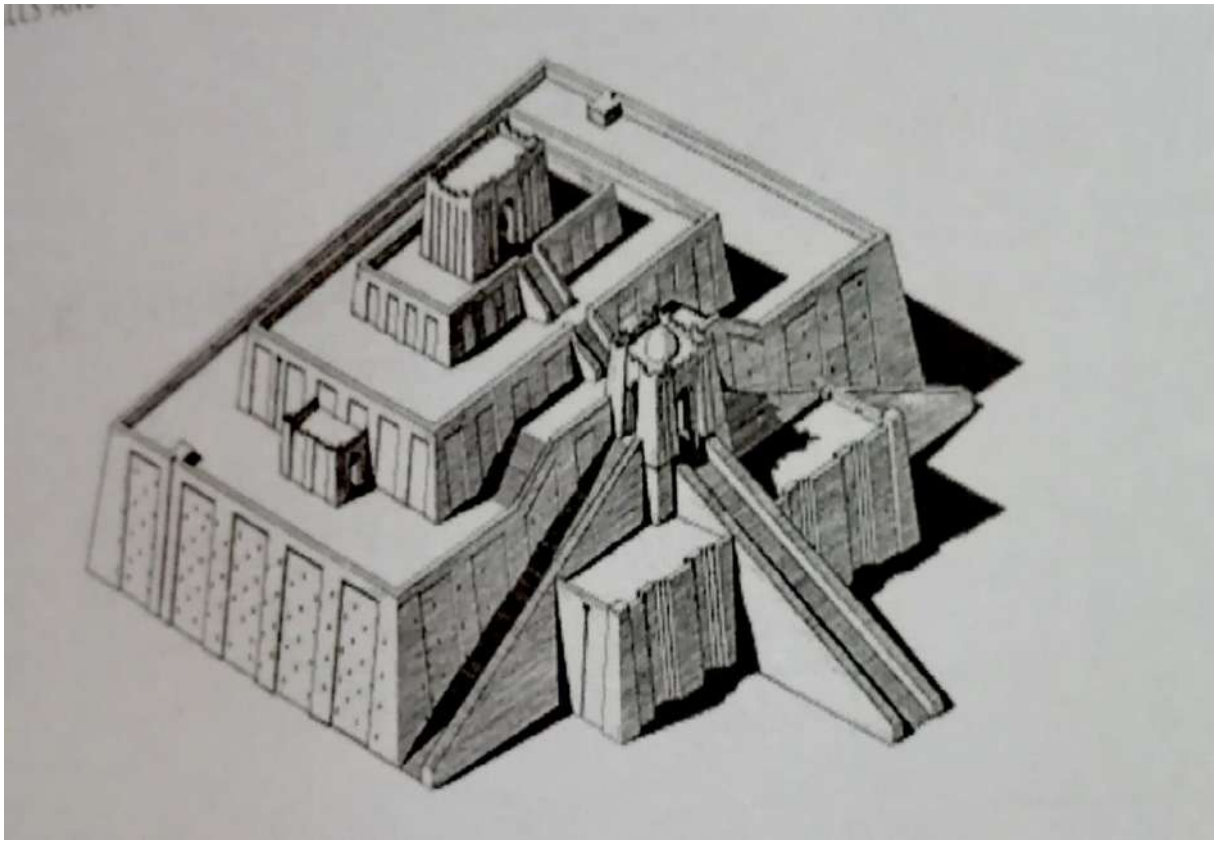


Figure 1.1 : Ziggurat of Ur :perspective

Despite this history , the modern structural concept of reinforced soil for retaining structures was developed only during the 1960s , in France , by Henry vidal (vidal 1969) , which , in the 1960's and 1970's , disseminated a technique with metallic inclusions called -reinforced soil-.This is a registered trademark process, but standardired by ABNT (NBR 9286/86),which consists of a reinforced soil wall with galvanized steel strips and cruciform concrete facing.

In Italy early 1970's , reinforced soil systems with metal inlys were disseminated around the world. Around that same time , the first application of walls reinforced with polymermic fibres (geosynthetics) came about.
Geosynthetics are one of the newest groups of construction materials, commonly used in work of various sizes ,especially in heavy construction . The term derivers from the junction of ''geo" , referring to the earth , and "synthetic", referring to the polymeric materials used in their manufacture.

The geotextile was the first type of geosynthetic systematically used in geotechnical engineering , having been used as of 1950's , in the United state as drainage , separation or erosion control elements . Its use in Europe began in 1960's and in Brazil , it started in the

1970's. Although there are records of empirical applications of geotextiles in the stabilisation of slopes in Brazil dating back to the 1970's , a rationaldesign of a reinforced soil wall came to be know only in 1986 (Carvalho , Pedrosa , Wolle ,1986).

Based on the philosophy of reinforced soil and the rapid development of the petrochemical industry , in the following decades serveral synthetic materials of high tensile strength and capable of reinforcing the soils began the emerge. The development of geosynthetics was fast.Currently geogrides are most commonly used geosynthetic for soil reinforcement .

Such material is usually a cheaper alternative and easier to employ in comparison to the existing traditional solutions . Therefore , in a matter of four decades , it went from having the status of incipient thechnology to cutting-edge technology , with wide acceptance and growing demand .

In the early 1990's , this industry already moved billions of dollars (Korerner ,1990) . The technique of reinforced soil is used in projects of roads , ports , canals , mining , slope containments and urbanization , among others .

According to Elias , Christopher and Berg (2001) , the retaining structures in reinforeced soil are cost-effictive solutions , which can provide high tolerance for foundation settlements , constructive ease and reduced execution time . The advantage of not requiring skilled labor can also be added .

In addition , it also allows for stable soil slopes , in vertical position , and a good aesthetic finish, if the apporopriate facing system in used , such as segmental blocks or slope re-vegetation

Figure 1.2 presents a cost comparision for some types of retaining structures.

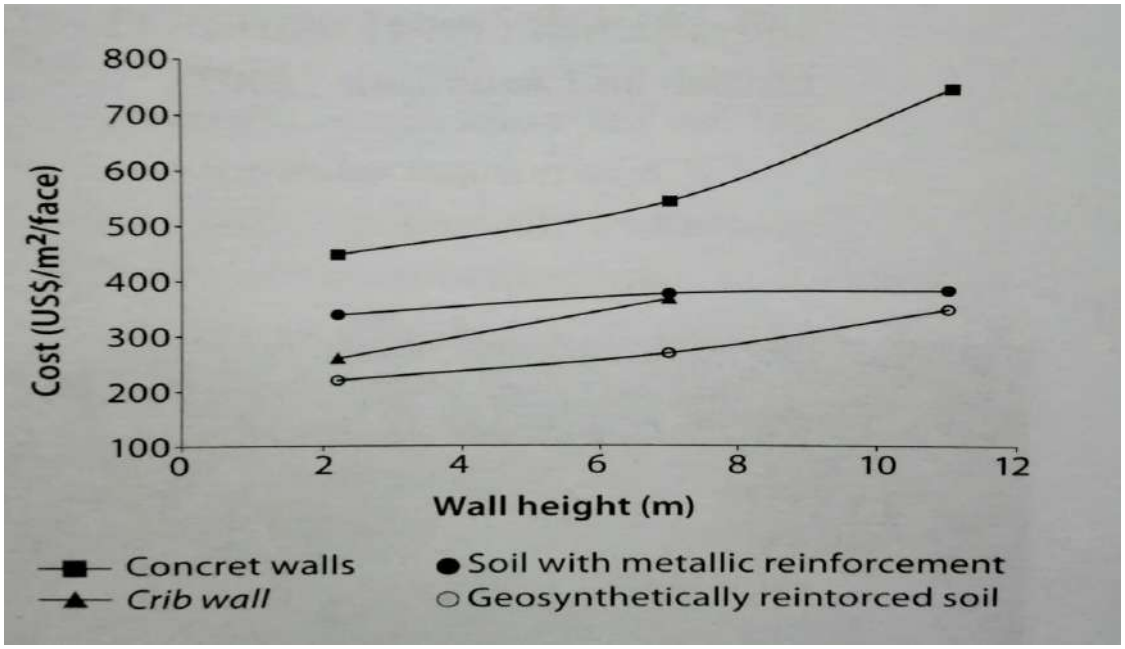


Figure 1.2 :Contruction costs , by face area , according to the height of the wall,for Various retaining solutions (Elias,Christopher ; Berg ,2001)

A tapical arrangement of a reinforced soil wall is represented in Fig 1.3 , which highlights the constituting elements .

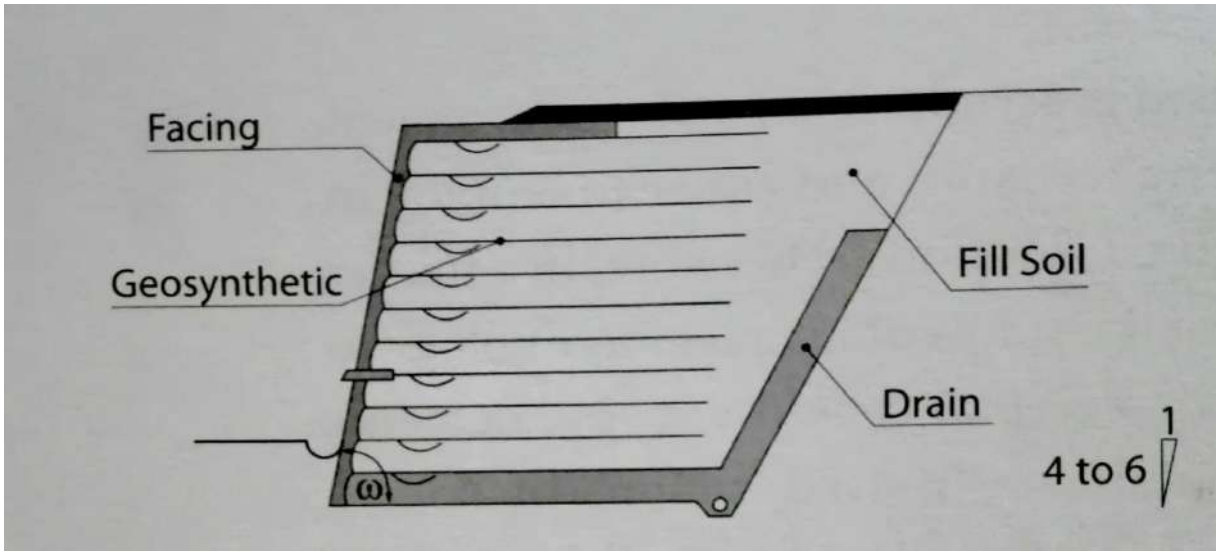


Figure 1.3 :Cross section of typical reinforced soil wall with geosynthetics (Ehrlich , Azambuja ,2003)

Figure 1.4 shows the appearance of a reinforced soil wall built in 2004 for the lowering of a raiway line that crosses the city of Maringa , in Parana . The maximum height of the wall

is 9 metres . A mature residual soil avaible on site , classified as sandy clay was used , with approximatery 40% fines (sieve 200) . The facing of the wall was executed with Terrae-W concrete blocks and the rienforcements are Fortrac polyester geogrids .

Since its first applications , the technique has been studied by several researchers in order to understand the behaviour mechanisms and devlop specification methods .As a result , today there are servel design methods for reinforced soil structures , based on empirical , limit equilibrium or working stress conditions considirations .

The methods lead to significantly different results due to their differing assumptions.[1]



Figure 1.4 Appaearance of a reinforced soil wall with segmental block facing (Brugger,2007)

Thanks to innovative techniques, it is now possible to build on all types of floors. From simple structures to architectural feats, the know-how of engineers is mobilized to analyze, study, reinforce and consolidate soils. The geotechnician probes and determines the characteristics of the soil in order to design, within design offices, the foundations of the structures. Soil reinforcement, although generally invisible, is the cornerstone of any structure built on soil with poor mechanical characteristics. Current technological developments in soil reinforcement are moving towards more economical and environmentally friendly technical solutions.

The need for poor quality land development is increasingly necessary, given the growth of humanity and its activities. Certain types of soil, hitherto neglected because of their poor mechanical characteristics, must be reinforced to ensure the stability of the buildings and the civil engineering infrastructure they are required to support. Not very visible to the layman because they are buried in the ground, **reinforcement techniques are very widespread** and surround us on a daily basis (foundations, retaining walls, road embankments, railways, etc.). More than a technological challenge, the construction of infrastructure in classified unbuildable areas **represents a real strategic, economic and environmental challenge** in many common cases such as:

- to build a high-speed line (HSL) in an area of compressible ground, when this is more economically advantageous than diverting the line (for example, SEA HSL crossing the Dordogne),
- to install warehouses in a port area in which the soils are often highly compressible due to coastal geology (e.g. Le Havre harbor),
- to build an airport on an artificial island in a marine environment (for example, Kobe airport in Japan). [2]

1.2.Techniques adapted to the complexity of the soil and each field of use :

Soils are complex materials made of **a** more or less compact **mineral skeleton** with a natural porosity **where air and water circulate freely**. Soil grains result from the more or less coarse fragmentation of rock into pebbles, gravel, sand or silt, but also from the chemical decomposition of some rock compounds into microscopic clay particles. In permeable soils, such as pebbles, gravel and sand, water circulates easily. This is not the case in low-permeability or even impermeable soils, such as silt and clay. Water circulates 1 million times less quickly in clay than in gravel. In addition, as a result of external stresses such as water inflows, drought or earthquakes, the mechanical characteristics of soils may change and no longer be adapted to their intended use.

Depending on the type of soil considered and the type of structure to be built, it is important to choose **an appropriate reinforcement solution** that is adapted to both the nature of the soil in place and its environment. The problem remains highly complex, as

evidenced by the remains of the past (Pisa Tower, for example). **Two major techniques** can be used to increase the mechanical characteristics of soils: by **modifying the internal structure of the existing soil** and by **reinforcing the soil** by adding inclusions. More specifically, soil improvement techniques can increase the compactness of the existing soil, either by reducing the volume of voids, for example by applying an overload to saturated soil and allowing it to settle by expelling the overpressure water, in this case *consolidation*, either by imposing vibrations in the soil so that it densifies by rearranging the grains *dynamic compaction technique* allowing to densify the soil to a depth of less than 10m, under the impacts of a mass of 15 to 150 tonnes falling from 20 to 40 m). Soil reinforcement techniques themselves involve vertical or horizontal reinforcement elements in the soil. The objective of all these techniques is to allow the construction of a structure without excessive surface deformation or stability defects.[2]

1.3. Soil-reinfocement interaction:

1.3.1 Concepts of reinforced soil:

The association of soil and reinforcements leads the composite materil to have better mechanical properties. The soil , when properly compacted , in general , has good resistance to comrpression and shear . The tensile strength , strength , however , is low or zero .Similarly , to what happens in reinforced concrete , the inclusion of reinforcements makes up for this deficiency.

For example, because they rest angle of these materials , which is about 40° (Fig 3.1A).These same sands , if reinforced , would allow for vertical slopes (Figure 3.1B)

Reinforcements act in similar fashion to a “seam” commiserating the region where the soil would be potentially unstable to the stable region and preventing the collapse of embankment.

Great differences in level can be achieved , according to the quantity and the resistance of reinfocements used. The length and amount of reinforcements are determined by analysis of external and internal stability.

In general , the length of geosynthetics varies between 60% and 80% of the height of the wall.

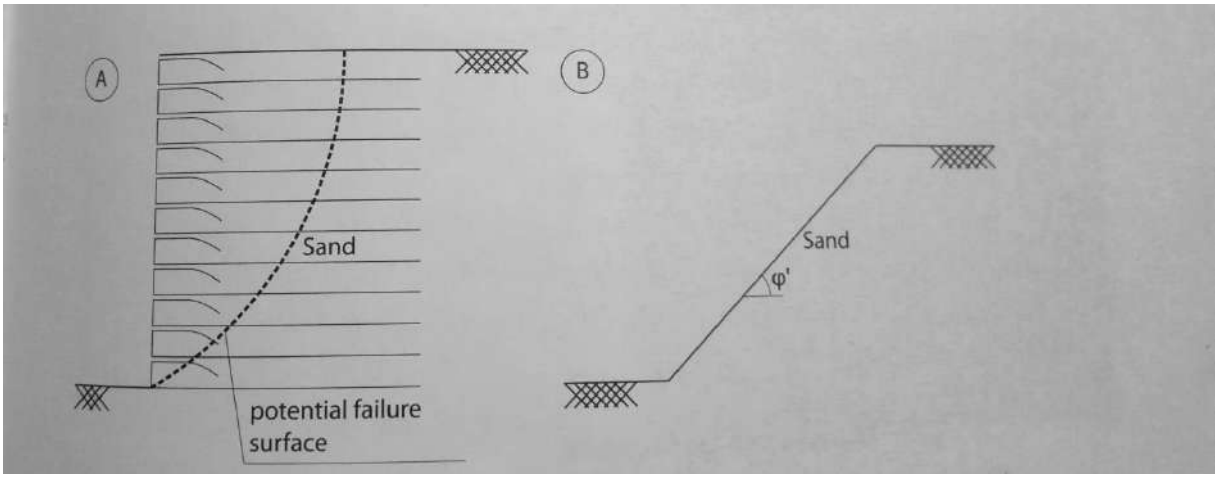


Figure 1.5 Influence of reinforcement on slope stability : (A) reinforced sand ; (B) sand slop (adapted from Ehrlich and Azambuja ,2003)

1.3.2 Characteristics of geosynthetics for reinforcement :

Various types of reinforcements for RSRS canbe used : metallic reinfoecementd , geogrids , woven or nonwoven geotextiles. A wide variety of types of geosynthetic reinforcement purposes , divided into two main categories : geotextiles and geogrids.

Geotextiles are the most versatile geosynthetic In addition to reinforcement the soil , they can be used for draining , filtration and separation.

Geotextiles can be divided into wonen or nonwonen groups , accourding to their manufacturing process .

Geogrids are specially produced for soil reinforcement . They are very open flat plastic structures , in a grid format , hence the name geogrid. Details of the different types of reinforcements are presented below .

1.3.2.1 Metalic reinforcements :

A noted above , the pioneer reinforcement of the modern era were the steel strips used in the Rienforced Earth system . The dimensions of the strips vary according to the demands of each work . The thichness should include an additional value (sacrificial thickness) to compensate for losses due to corrosion . According to the useful life of the work and the degree of recommended by ABNT (NBR 9286/86) , may cause the work to be more expensive . Gabion mesh can also be used as reinforcement with the same restrictions as to corrosion .

1. 3.2.2 Woven geotextiles :

The woven geotextiles are manufactured using convertional weaving processes. Based polymers are melted and forced to pass through an extruder , which grants them of filaments , which take the shaoe of the holes of the extruder , and may be cylindrical or laminetes. Once cooled , the filaments are woven in two directions .
The longitudinal direction of the machine in called warp and the cross section in called welf . The intertwining of the fibres may be in the form of single or multiple filaments and laminetes .

1.3.2.3 Nonwoven geotextiles :

They are basically made with same types of filaments as the previous group , cut into pieces ot continuous . However , instead of the weaving processes , the filaments are launched randomly on a converyor belt , on top or each other . The weight of the product depents on the release rate of the filaments .

The bonding of the filaments , the geotextile can be further pressed into rolls of smaller diameter for storage , which are easiest to transport . Nonwoven geotextiles have great structural complexity , in addition to presenting more isotropic physical and mechanical characteristics than woven geotextiles .

Figure 1.6 shows blown up photos of woven and nonwoven geotextile stuctures .

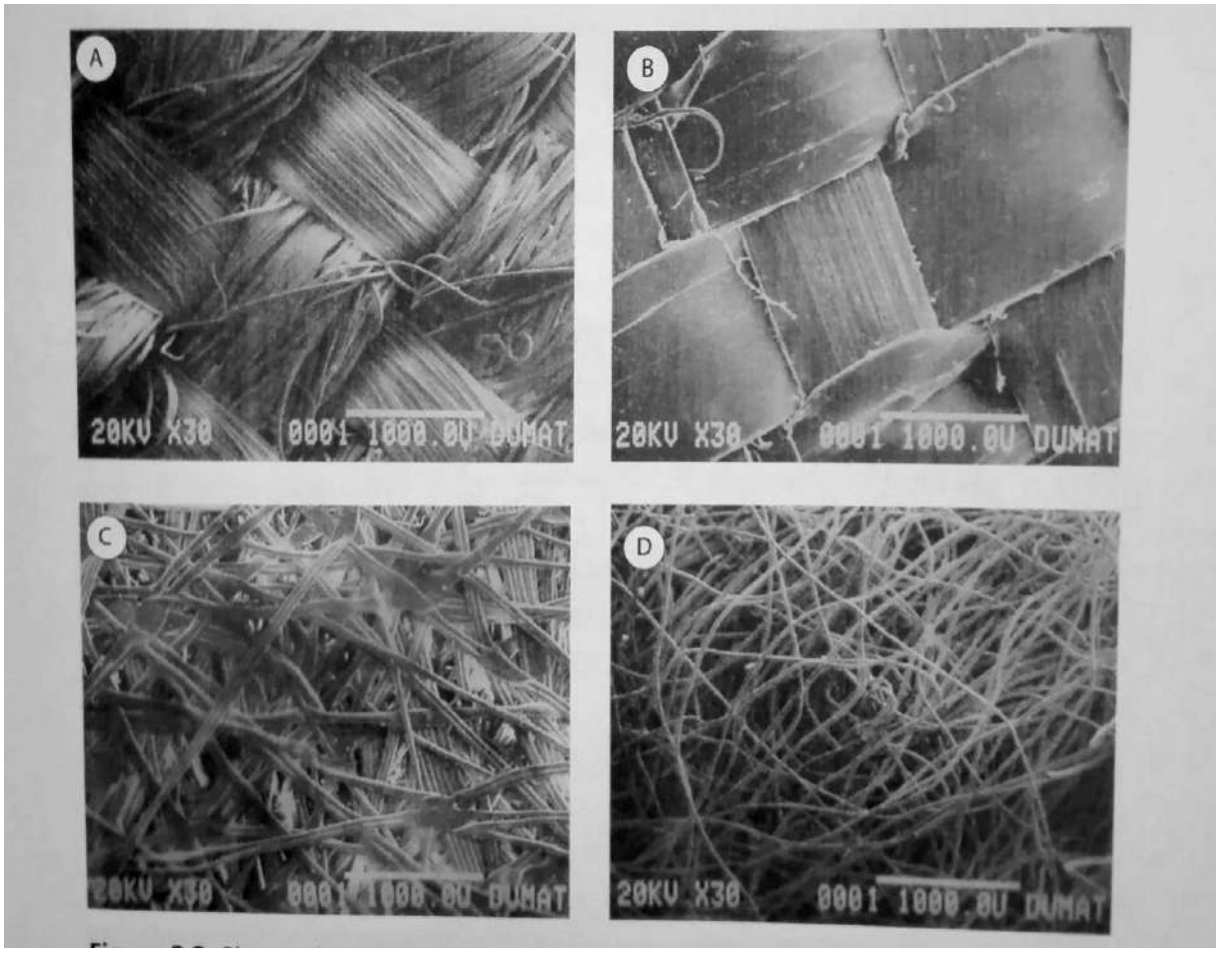


Figure 1.6. photo microscopy of geotextiles with different structures : (A) woven multifilament , (B) woven monofilament , (C) non woven needle punched, (D) nowwoven thermo bonded (Koerner , 1998)

1.3.2.4 Geogrids :

Like geotextiles , the geogrids are supplied in rolls of specific width and length . The two main types are : one-way , when they have high tensile strength in two orthogonal directions.

The commonly used polymers for geogrid manufacture are high-density polyethylene (HDPE), polyester (PET) and polyvinyl alcohol (PVA) , and they are characterized by low deformability and high tensile streangth . Figure 1.7 shows the appearance of a piece of flexible geogrid , made from PVA threads with a polymer protection coating.

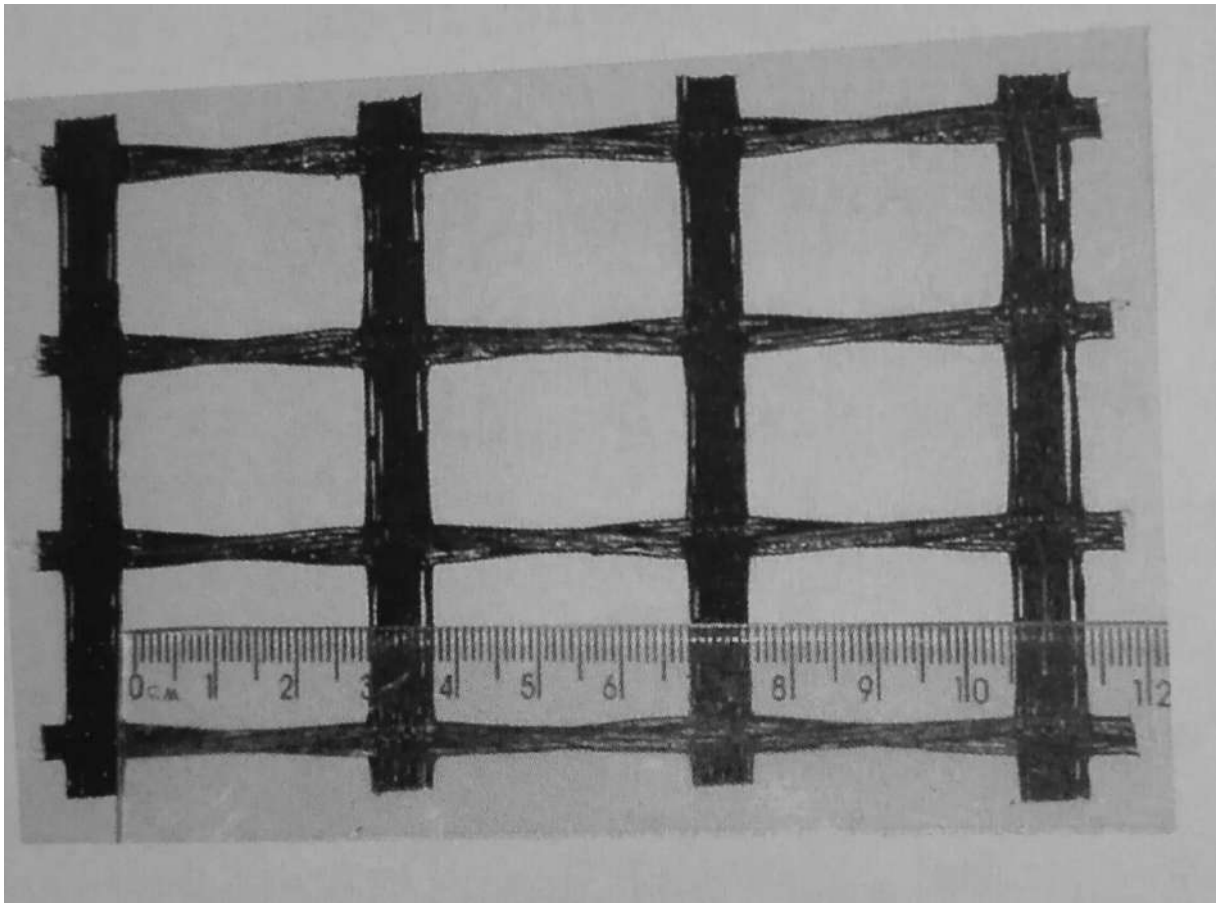


Figure 1.7 photograph of a PVA geogrid

1.3.2.5 Types of polymers :

The vast majority og geosynthetics is made up of synthetic polymers . The molecular structure of a polymer can be compared to the gathering of many molecule chains that are repeated , called monomers. The most commonly used polymers in geosynthetic manufacturing are polyester (PET) , polypropylene (PP) , polyethylene (PE) and polyvinyl alcohol (PVA).

They consist of long chains of molecules arranged in crystalline regions (aligned chains) and amorphous regions (randomly intertwined chains) , obtained by chemical polymerization.

1.3.2.6 Creep and deformability

The non-confined tensile strength is determined in the test of wide-width strip method , with a saple 200mm wide by 100mm long , set between two clamps and pulled until

failure. The load applied at the time rupture divided by the the initial width of the sample body is the tensile strength , expressed in KN/m .

The test is standardized by ABNT (NBR 12824/93) .

Monitoring of load and strains is performed during the wide-width test , on which the distributed load-strain curves of the geosynthetics can be based . The shape of the cureves varies gtreatly depending on the type of geosynthetic , its polymer and structure .

Figure 1.8 shows some typical examples.

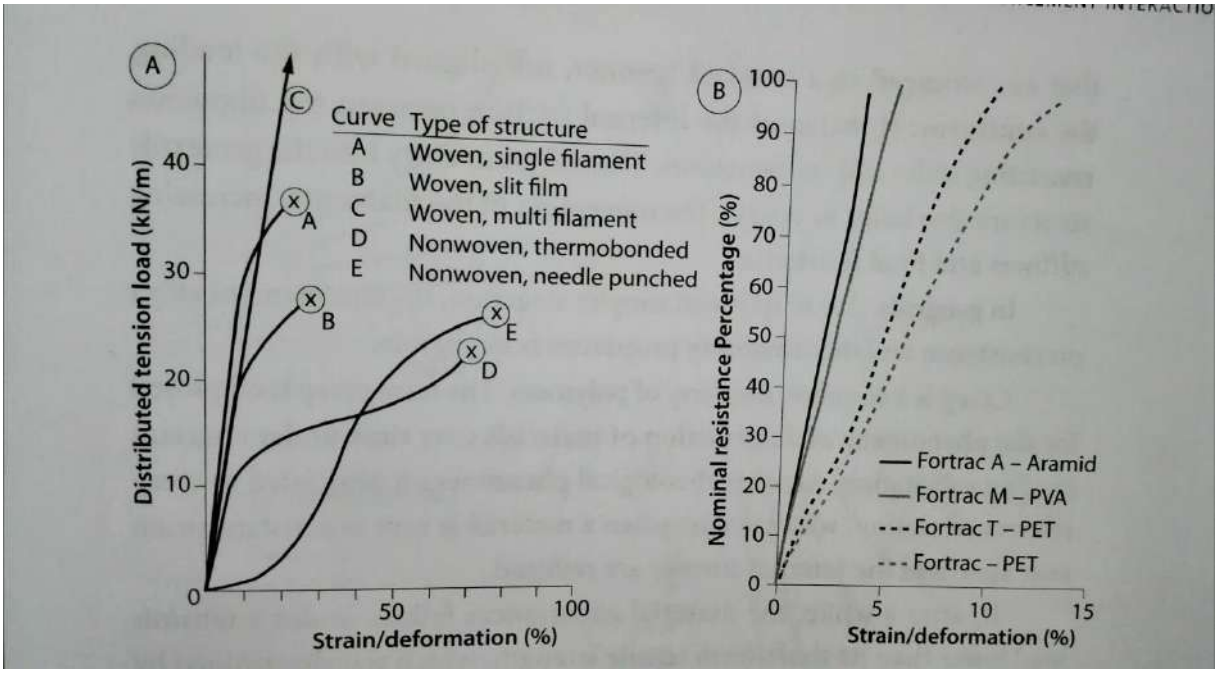


Figure 1.8 Behavior of distributed load-strains of various types of geosynthetic: (A) geotextiles (Koerner ,1990) (B) geogrids (Huesker).

It is observed that woven geotextiles have much greater stiffness than non-woven , due to loose bonding between the nonwoven filaments. The stifnesse of geogrids, in turn , depends mainly on the polymer used in its manufactures .

In general, the curves are not linear and thus different types of stiffness can be calculated . It is custotary to use the initial stress-strain curve and the secant stiffness to the origin , which is the inclination of a straight line connecting the origin to a point on the curve – for example , 26% strain/deformability.

The stiffness is expressed in KN /m and can be defined as follows

$$Jr = \frac{Tr}{\varepsilon r}$$

WhereJr is the reinforcement tensile strength stiffness module ; Tr is the tensile strength per width unit in the wide-width strip test ; and Ɛr is the corresponding strain of the sample in the wide-width strip test .

For many geograds , it is unusual for manufacturers to provide secant stiffness for 2% strain or at failure stress.

Soil confinement generally increases the tensile strength and stiffness or nonwoven geosynthetics have filaments that are arranged in crooked manner , not aligned with the loading , the confinement increases the intenal friction between the filaments, restricting slides and reorientation . The soil grain entry into the geotextile structure also helps to restrict the movement of the filaments , I,creasing stiffness and final resistence .

In geogrids , due to its much simpler structure , the confinement effect on resistance and deformation of materials over time under constant loading solicitation.

Another rheological phenomenon associated to creep is stress relaxation , which occurs when a material is kept at constant strain over time and the internal stresses are reduced .

If , after a while , the matrial experience failure under a tension load lower than its short-term tensile strength , which was determined by wide-width strip testing , this phenomenon is called creep failure. Creep tests are usually confinement . There is evidence that confinement changes the creep properties of non-woven geotextiles (McGown ; Andrawers, Kabir , 1982 ; Beker , 2001) . For woven geotextiles and , in particular for geogrids , the effect of confinement for creep is negligible .

Most creep studies are based on non-confined trials , recently standardized by ABNT (NBR /15226/05) . The sample is held vertically between two clamps and weigths are attached to the lower clamp for stress/tension . The weights ensure constant load during the desired period of time . The strains of sample are recorded at regular intervanals , and the temperature and humidity of the enclosure should be kept constant . The load is expressed in KN/m , by dividing the applied weight by the original width of the geotextile . Tests can be conducted with loads of 10% , 20% , 30% , 40% and 60% of tensile strength . Strains

are calculated based on the initia length of sample . The test results can be expressed in many ways , and the most common are presented in Figure 1.9

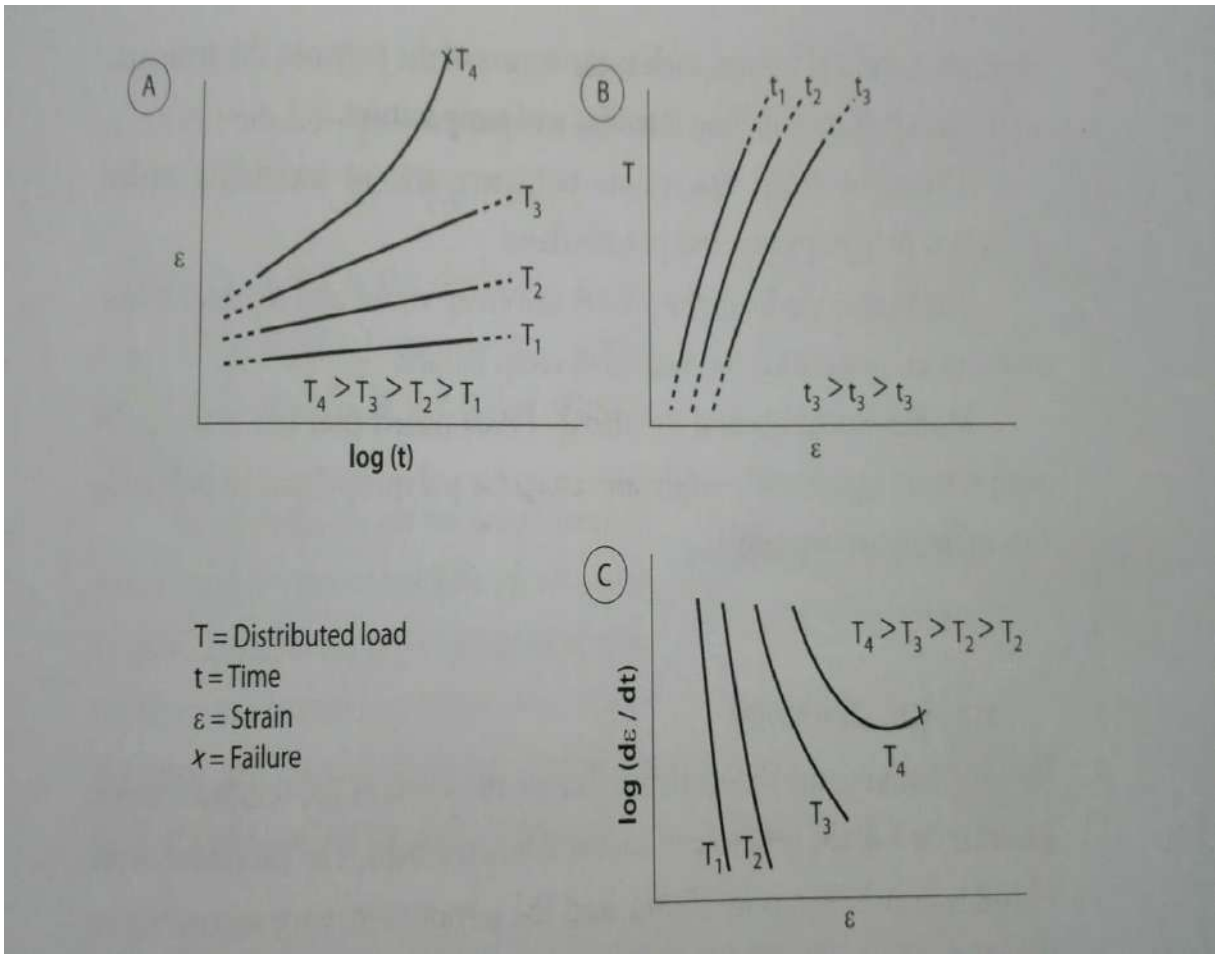


Figure 1.9 Idealized creep graphs type: (A) semi-logarithmic , (B) isochrones , (C)sherby-dorn (Carruffi et al .1997)

In general , the designer should prevent reinforcements from reaching the ultimate failure state or from suffering excessive strains that effect the appearance or use of the structure . Watts , Brady and Greene (1998) suggest that limiting the strain imposed on the reinforcement is sufficient to meet both criteria .

In practice , two approaches can be used to solve the problem :

- Reduse the short-term tensile strength by dividing it by a reduction factor that takes into account the type of polymer and considers the useful lifetime of the work and acceptable strains in an impmicid and empirical manner . This method is used when there are no available results for creep tests .

- Establish a maximum strain valus allowed for the reinforcement and determine the load corresponding to the useful lifetime of the work in the isochronous graph.

In both cases , the burden will be further diminished by other reduction factors in order to consider installation damage , variability of the material andchemical and biological degradation .

The strain / deformation value to be used for the reinforcement depends on the work's responsibility , the properties of the soil and tolerable strains for the structure .

It is worth mentioning that creep behavior of a geosynthetic depends on serveral factors, such as nature of the polymer , the structure of the geosynthetic , loading intensity and temperature .

The most creep susceptihble polymers are , in ascending order , polyster , polypropylene and polyethylene .

The higher are load , the greater the creep strain and the higher the strain rates , increasing the chanses of creep failure .

Muller-Rochholz and Reinhard (1990) noted that the increase in temperature significantly accelerates creep for polypropylene . In poyster , this is influence is negligible .

***The concept of reinforced soil was used for the first time by Henri Vidal, as part of the technique of Armed Soil that he invented anddeveloped in the early 60s. This concept has since been extended, according to other authors, to various techniques such as: micropiles, ballashed columns, columns in stabilized soil, soil nailing,Texsol, membranes, etc.

Depending on the type of inclusion used, two extreme cases can be considered in soil reinforcement :

1. The case of uniform inclusion " where the phenomena of soil-inclusion interaction can develop throughout the inclusion.

2. The case of" composite inclusion", which consists of enhanced inclusion at particular points where The soil-inclusion interaction is concentrated. Generally, as in the case of anchors, these points are located at the ends of the inclusions.

In the case of a uniform inclusion, a relatively high and uniform reinforcement density will lead to a new composite material called "Reinforced Soil". The corhportance of a "reinforced soil " can be studied on a representative sample of the new composite material. This concept is illustrated in **Fig. 1.10** . The armed soil mass is a composite material and its apparent mechanical properties can be established from laboratory tests carried out on representative samples. Conversely, the (ladder wall " invented by Coyne, is a multi-anchor system where The ground-inclusion interaction is concentrated at the ends of the tie rods.

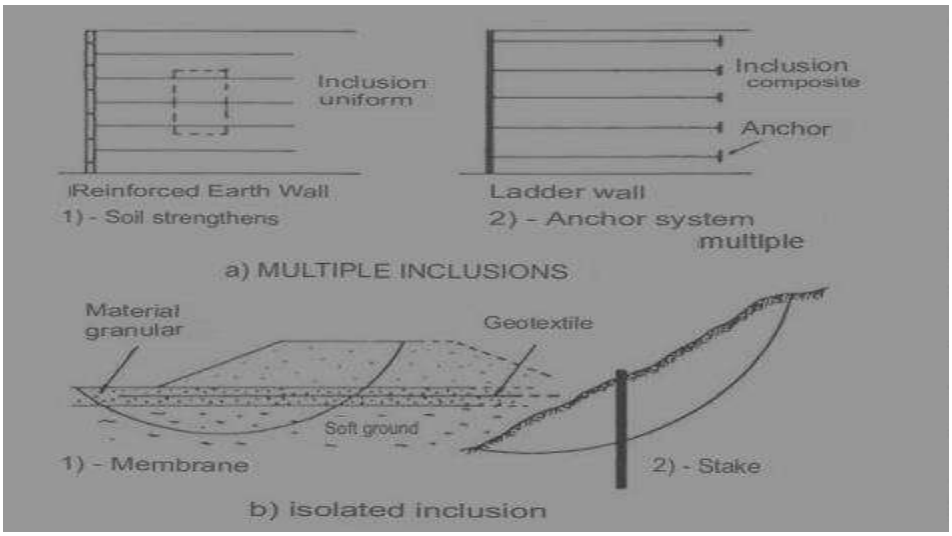


Figure . 1.10: Types of reinforced soil structures

Type of reinforcement / The density of thereinforcement	Uniform	Composite
Multiple……..	Reinforced soil	Multi-anchor systems
Isolé……	Pile membranes	Anchor

Table.1.1 : These considerations lead to the classification of soil systems-inclusions shown in

Thirty-five of the papers presented at this conference relate to soil strengthening.

They cover most of the reinforcement systems mentioned above and can be classified by considering the different techniques and applications referred to in table II below.

This embarrassing report summarizes the new aspects of current knowledge and reviews the communications submitted to the conference. It briefly describes the various techniques and develops in particular the following points:

-soil-inclusion interaction.

-behavior and methods of calculation.

-case studies and control.

Behavior and calculation methods are treated by considering the different types of application: retaining walls, stabilizing slopes, reinforced surface foundations,reinforced foundation soil in place. The section on reinforced surface foundations was prepared by the co-rapporteur, Dr. H.M. Jacobsen.

classification	Reinforced soils						Multi-anchoring	Uniform isolated inclusion
Techniques or systems Application	ballasted columns	Armed soil	Clouding of the soil	Micro-piles	Multi membrane	Special systems	Scale Wall	Membrane or insulated wire mesh
Fondations	10	1			2	1		8
walls		3	3	1			1	
Slope stabilization			2					
1 communications related to soil-inclusion interaction 2 communication related to the general application of geotextiles								

Table 1.2 : classification Reinforced soils

1.4. Description of techniques and interaction soil inclusions :

1.4.1 . Description of techniques and main efforts:

The inclusions used for soil reinforcement are resistant elements that are generally either linear or planar .Depending on their relative rigidity relative to the ground, their behavior is quite similar to that of a beam or plate when they are relatively rigid, and that of a wire or membrane when they are flexible .As a result, the main forces mobilized in the inclusions can be of four kinds: traction, compression,
bending and shearing . Table III shows these main efforts by considering the various soil reinforcement systems. In fact, the mobilization of these forces depends on a large number of parameters, including the relative rigidity of the inclusions, their orientation, their density, the geometric structure, the construction process, the mechanical properties of the inclusions and the soil, etc.
As shown in Table III, most of the techniques that can be used can be classified in soil reinforcement systems. This is why these systems will be more particularly described, taking into account the different fields of application.

When it comes to improving foundation soils, commonly used soil reinforcement techniques are:

-The ballast columns. This technique is used in soft soils; the reinforcement inclusion is a vertical column made of sand, gravel or heavily compacted aggregates. Generally, the placement of the column comprises two main phases :

1. a tube or vibrator is driven into the ground to the desired level by pushing the surrounding ground back;

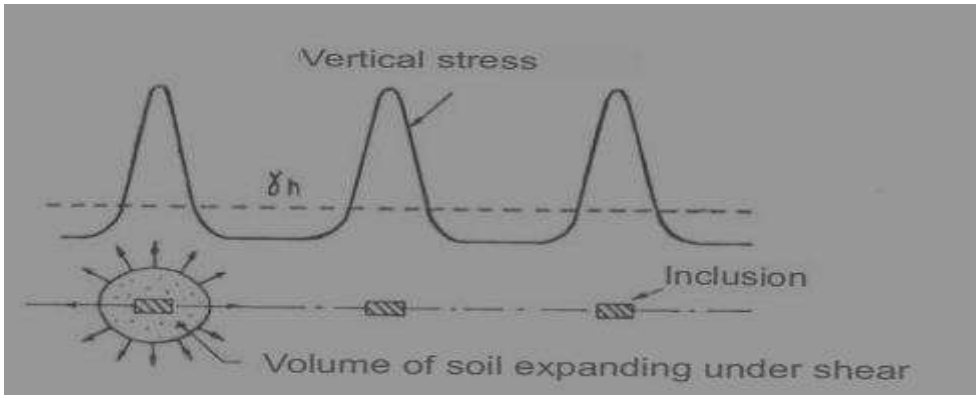


Figure 1.11: Mechanism of soil-inclusion interaction in a dilating soil.

the system is then gradually reassembled while the cavity is filled with a granular material strongly compacted statically or by vibration.

The main role of the column is to increase the strength and modulus of the foundation soil, in addition the column acts as a vertical drain. This inclusion is relatively flexible and mainly resists compression. However, when ballast columns are used to improve soil stability
with respect to general sliding, they also significantly improve the shear strength of the soil.
Stabilized floor columns. The efficiency of the ballasted columns has led to the development of a fairly similar technique which consists in creating columns in stabilized soil.When compared to ballasted columns, these inclusions are generally stiffer and can withstand both compression, bending and shear.

System of strengthening	Reinforced floors								Inclusion multiple and composites		Isolated inclusion		
Technical or system / Major effort	Column ballast	Columns of stabilized soil	Micro piles		Membrane or wire mesh multiple	Nailing of soils (slopes)	Nailing ground (retaining wall)	Texsol	Ladder wall	Anchor multiple active	Membrane or wire mesh	Pile (slopes)	Anchor
Traction			**	***	***		**	***	***	***	***		***
Compression	**	**	**										
Bend...		*	*			**	*					**	
Shear	*	*	*			**	*					**	

Table 1.3: Major efforts in soil reinforcement system inclusions

Micro-piles: This technique consists in putting small injected piles into the soil. Each pile consists of a bar or a tube of a few centimeters diameter surrounded by coulis along its entire length. The total diameter is approximately 10 to 15 cm. This technique has been used for the last hundred years, mainly in foundation soils, but it also has interesting applications for slope stabilization, support structures and undercover recesses. These are rather rigid inclusions in which the efforts mobilized depend so much on the structural effect of the pile group. The main forces are traction and compression

If we consider support structures, the most commonly used soil reinforcement techniques are:

Armed soil: This technique consists of the combination of a rubbing granular material and flexible linear shields of high tensile strength. The outer face of the structure is relatively thin and flexible; it is generally made up of concrete panels. Steel shields which are relatively inextensible to the embankment floor and galvanized to prevent corrosion are used to limit structural deformation.

Multimembranes and grills: The rapid development of the armed soil has recently led to the use of all kinds of inclusions in soil reinforcement systems, including membranes and geotextiles. The latter can only withstand traction efforts . The main difference between steel or rigid plastics and geotextiles resides in their deformability which has a significant influence on the lateral movements of the structures. Although there have been some attempts to use geotextiles in vertical-facing support structures,instead, they are used to reinforce slopes of slopes and foundation soils. Metal or rigid plastic meshes are used efficiently to limit lateral deformation of the embankments

The clouding of the soil :It is a technique of in situ soil reinforcement using passive bars, either sealed in boreholes or simply beaten in the ground . When this technique is used for support structures, the bars are generally horizontal and the main force is traction. On the contrary, when this technique is used for stabilization of slopes, the bars are generally vertical and the main forces are bending and shearing.

The Texsol: This new technique, invented by LEFTAIVE (19821), consists of reinforcing a granular backfill material using one or more tensile-resistant continuous wires.

1.5.Structure behavior and design methods:

1.5.1Retaining structures :

1.5.1.1 Reinforced soil retaining structures :

The behavior of retaining structures in reinforced soil depends a lot on the deformability and the relative stiffness of the inclusions. Among the techniques mentioned above, the

Armed soil represents the case where the inclusions are linear, inextensible and completely flexible . The behavior of this material has already been studied in detail both on scale models in laboratories and on real-scale experiments. **Figure 10** illustrates the characteristic aspects of the functioning of a Reinforced soil wall. The geometric locus of the maximum tensile points in the reinforcement separates an active zone, near the facing, from a resistant zone behind the wall.This site, which represents a potential rupture surface, is very different from the classic COULOMB rupture plan behind the retaining walls .The distribution of maximum tensile forces is also very different from the triangular distribution of thrust as it results from RANKINE's theory. These differences can be explained by studying the influence of inextensible inclusions on the stress fields (SCHLOSSER, 1969) or deformation fields (BASSETT, 1978) that develop in the massif .The inclusions limit the lateral deformations of the wall and keep the soil of the active zone in an elastic or pseudo-elastic state. This is the reason why the maximum tensile forces in the upper part of the wall correspond to the lateral pressure of the soil at rest.

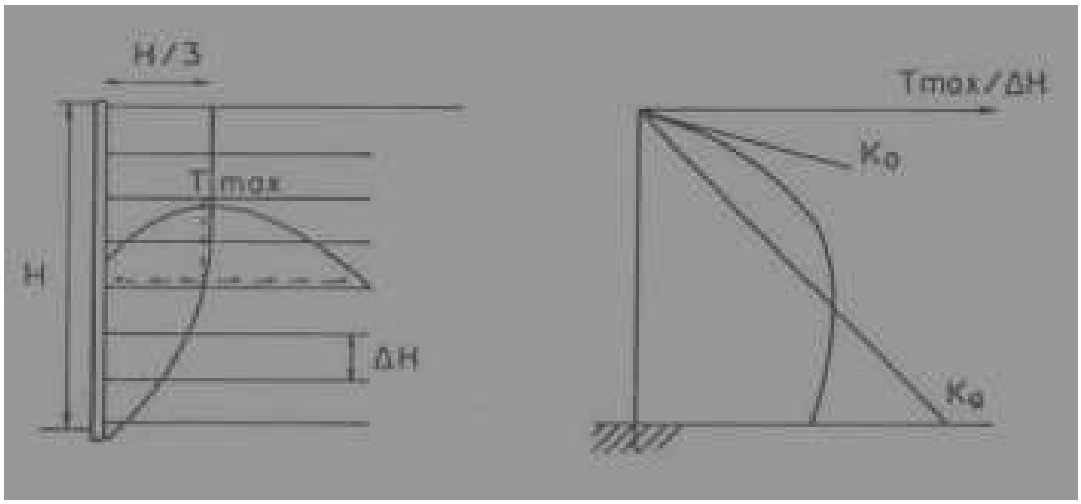


Figure 1.12 Behavior of a retaining wall in reinforced soil

In Reinforced soil walls, the reinforcements are usually made of galvanized steel. JOHN and PETLEY (Helsinki conference) present tensile force measurements in the synthetic armafors (paraweb) of two reinforced soil walls: the Portsmouth wall (2.5 m high) and the Jersey wall (8 m high).The paraweb is made of polyester fibers coated with plastic material. The permissible stress of the paraweb is about 200 MPa, which is the same as the permissible stress in steel which is about 160 MPa . But paraweb is much more

deformable than steel, which naturally leads to greater lateral displacements of the walls than in the case of conventional Reinforced soil walls .

Figure 1.13 shows the distribution of the maximum tensile forces measured by JOHN and PETLEY . In the upper part of the wall, this distribution is close to the K_a line corresponding to the Rankine thrust, which suggests that the lateral deformation of the wall was large enough to put the soil in a state of limit equilibrium.

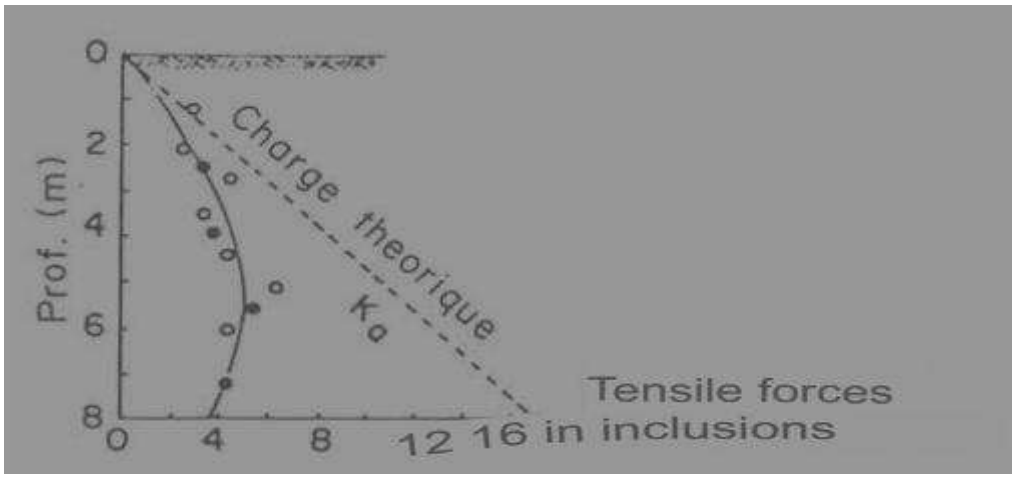


Figure 1.13 Tensile forces of a reinforced soil wall (John and Petley, 1983).

MARCZAL (Helsinki Conference) reports on the results of a full-scale experiment on a 6 m high Reinforced soil wall with relatively inextensible glass fibre reinforcement coated with polyester . The tensile forces measured are greater than those calculated by considering a distribution along the line Ka . In the case of inextensible inclusions, JURAN (19771 developed a theoretical analysis at limit equilibrium and used it for the two modes of rupture of the Reinforced Soil : break by break of the armatures and break by default of adhesion . Current methods of computation of armed soil (SCHLOSSER et al., 1979) take into account both these theoretical results and the numerous observations on true-size works.

The retaining structures carried out in situ using the ground nailing technique have three important differences from Reinforced soil walls:

1. the soil in situ generally has cohesion;

2. inclusions, which are generally bars sealed in boreholes, have a certain bending stiffness, which can influence the behavior of the structure;

3. The wall is built from the top, excavating, and a very different loading story results.

Only a few experimental results have been published (STOCKER et al., 1979; GASSLER and GUDEHUS, 1981; SHEN et al., 1981) but there have been many partial observations on real works over the past decade. These results showed the development of active and resistant zones within the reinforced soil. Although the maximum tractions line is generally difficult to determine, it seems that there is a certain difference with the Reinforced soil walls and this due to certain factors among which: a greater horizontal displacement at the head, a cohesion of the ground in place, an inclination of the inclusions and the facing, etc. .

CARTIER and GIGAN (Helsinki Congress) present a full-scale experiment on a retaining wall in nailed ground 5.5 m high with vertical facing. The inclusions were beaten angles inclined by 20 to the horizontal and exhibiting relatively low bending stiffness (Hurpin method). The ground was a fine loamy sand. Figure 12 shows the lateral movements of the structure measured at the inclinometer; it is clear that the movements of the wall are very different from those observed in reinforeced soil walls . Similar results were reported by GASSLER and GUDEHUS (1981) and by SHEN et al. (1981). They indicate that the construction mode of a nailed soil wall results in greater displacements at the head of the wall.

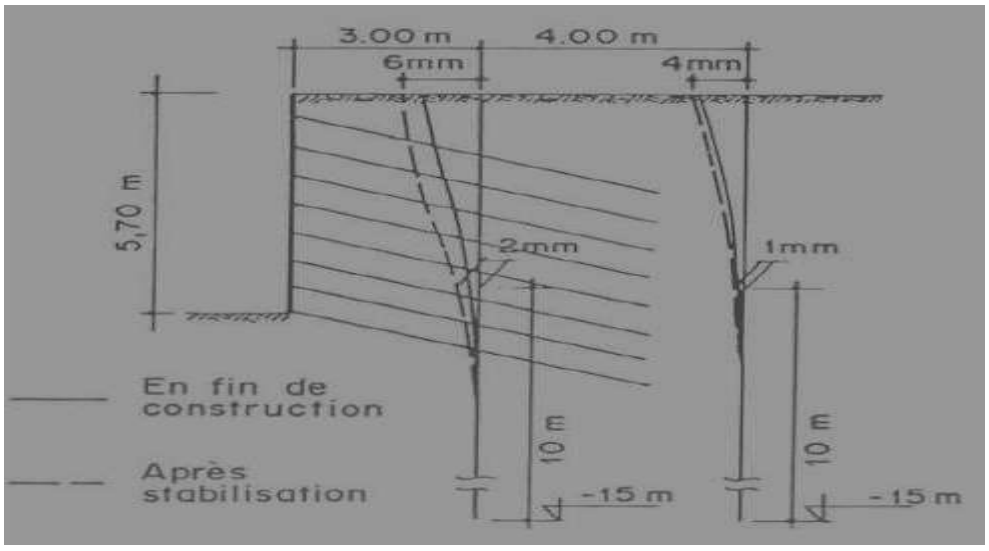


Figure.1.14. Horizontal movements in and behind a nailed soil wall (Cartier and Gigan, 1983).

GUILLOUX et al. (Helsinki conference) reports the case of a 14 m high nailed ground retaining wall with a slightly inclined facing (slope 10/1). The reinforcement consisted of nearly horizontal sealed bars . The soil was a compact moraine. The tensile forces measured in the bars show the development of an active zone and a resistant zone . Under the action of frost the swelling of the soil caused an increase in the thrust on the facing and as a result the tensile forces evolved to become maximum on the facing.

GUILLOUX et al. also present a method of dimensioning nailed soil structures developed by Terrasol (SCHLOSSER, 1981) . This method of calculation at the limit equilibrium considers four rupture criteria related to the mobilization of different forces in the soil and in the inclusions:

1. the failure of inclusion due to the combined effect of tensile, shear and bending forces;
2. soil shear strength;
3. lateral friction along the inclusion;
4. the lateral pressure of the soil on the inclusion. The calculations are made by the slice method considering a circular failure surface

Figure 1.15 shows the simplified case for the first criterion, when the bending moment is neglected . The condition $\tau < k$ (maximum shear stress in reinforcement) leads to the following expression :

$$(N/R'')2 + (S/R,)z<1$$

where N and S are the normal force and the shear force, Rn and Rs are the tensile and shear resistances of the bar.

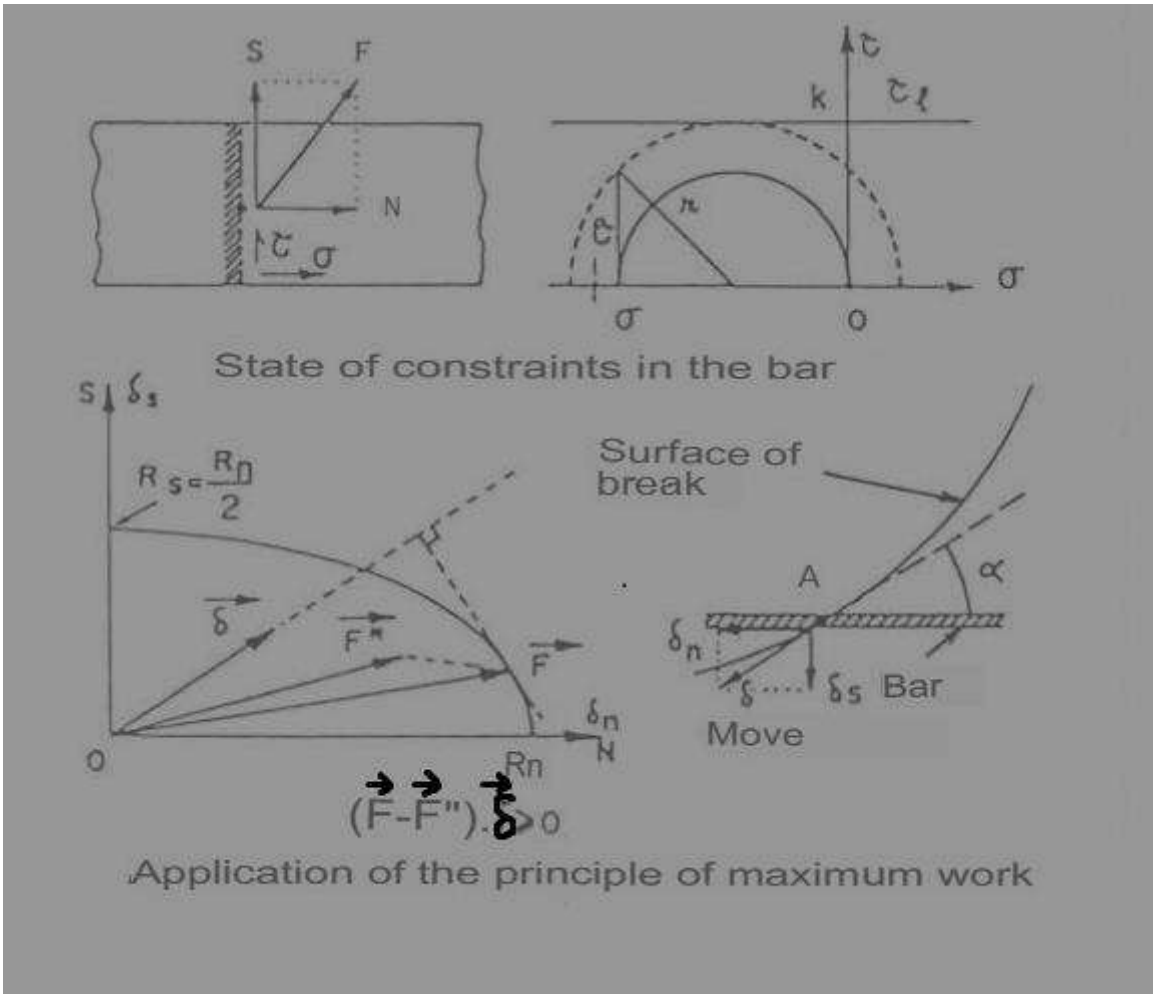


Figure .1.15. Determination of tensile and shear forces (Schlosse, 1983)

At break, the tensile and shear forces mobilized in a bar are determined using the principle of maximum work as shown in Figure 13. This principle gives :

$$(F- F'').\delta > 0$$

where F is the actual force in the bar, F" is a virtual force that does not violate the criterion of repetition and δ is the displacement of the bar, assumed equal to the displacement of the ground at the meeting point with the fracture circle.

The third criterion relating to lateral friction can be written: N<Ni where Ni is the tearing force of the bar portion located beyond the fracture surface. For the fourth criterion it is necessary to take account of the moment of flexion. This leads, in the plane (N,S), to a much more complicated flow surface than the ellipse corresponding to the first criterion alone.

The dimensioning method has been fixed on many reinforced soil structure (SCHLOSSER, 1983) (walls with flexible bars, rigid bars, slopes stabilized by micro-piles) . It is normally used with security factors corresponding to the following: steel working at the elastic limit, lateral support limited to half of its maximum value, lateral pressure of the ground limited to the creep pressure in the pressiometer test,safety coefficient of 1.5 for the shear strength of the soil.These values have been chosen so as to make all the criteria compatible with regard to the displacement field that develops in the reinforced massif.

In parallel with these deterministic calculation methods, statistical approaches are currently being developed in order to overcome the difficulties encountered during the determination of the calculation parameters and the safety coefficients. Such a probabilistic approach is presented by GASSLER and GUDEHUS (Helsinki Congress) for the calculation of retaining walls in nailed soil.

1.5.1.2 Walls with multiple anchors:

The concept of a multiple anchor wall was first considered by COYNE in 1926 with the invention of the ladder wall. Similar systems have recently been developed (MURRAY, 1981; FUKUOKA, 1982; etc.). FUKUOKA described a full-scale experiment on a wall with multiple anchors whose facing consisted of a geotextile fixed to rigid vertical elements. The backfill was loam. The wall was reinforced by anchors made up of tie rods connected to vertical concrete panels (1m x lm x 0.15m).

Figure 14 shows the rotation of the vertical elements of the facing, the soil pressure on the facing and the soil pressure on the concrete slabs, which is slightly greater than the lateral soil pressure at rest. These results show that, in this support system, the displacement (rotation) of the facing

is sufficient to reach the limit state of thrust behind the facing. On the other hand one does not mobilize the stop on the anchoring panels and the pressure which is exerted there can be calculated on condition of knowing the curve of mobilization of the active and passive pressures of the ground (fig. 14).

CHABAL et al. (Helsinki Congress) present the construction of a 27 m high dam built using the ladder wall technique. The downstream facing of the dam consisted of the anchoring panels . Measurements showed that the tensile forces in the tie rods did not

increase linearly with the weight of the earth and were lower than expected. This work behaves more like a double-sided structure; it is quite different from the classic wall with multiple anchors. One can reasonably expect that the state of the stresses in the soil will be close to the state K_0.

there is a technique similar to nailing, but in which the anchors are active, that is to say prestressed. The wall construction process is similar to that of nailed ground walls but the behavior is relatively different because the pre-stressed anchors greatly restrict lateral movements. This technique was recently used for the construction of a support structure 30 m high which was the subject of a experimentation (KÉRISEL et al., 1981); but it will not be dealt with in this report.

1.5.2 Stabilization of slopes in situ :

Four communications submitted to this congress call for the stabilization of slopes in situ by nailing (JURAN et al.), By piles (WINTER and GUDEHUS, CARTIER and GIGAN) and by micro-piles (LIZZI '). The behavior of these systems is discussed below.

1.5.2.1 Nailing:

The stabilization, by nailing, of unstable or moving slopes consists in placing passive linear inclusions capable of withstanding bending moments. Inclusions are beaten or sealed bars which are most often placed vertically and in a uniform distribution. The behaviour

of such a system depends on several parameters including:

The inclination of the bars with respect to the fracture surface, their density, the relative stiffness of the bars with respect to the ground and the current state of sliding (static equilibrium, sliding speed, creep, etc.)

The effect of reinforcement orientation was studied by JEWELL (1980). It showed that the intensity of the tensile force that develops in a flexible inclusion during direct shear at the box depends mainly on the inclination of the inclusion

relative to the slip surface. The maximum increase in the shear stress of a bar- or mesh-reinforced sand sample is obtained when the inclusion is oriented in the direction corresponding to the maximum extension at rupture in the unreinforced sand sample.

When the inclusion is oriented in the direction of a compression strain, this may result in a decrease in the shear stress of the soil. Figure 15 shows the experimental and theoretical values of increasing

the shear strength of a sand sample strengthened with respect to the Θ angle of inclusion from the vertical .

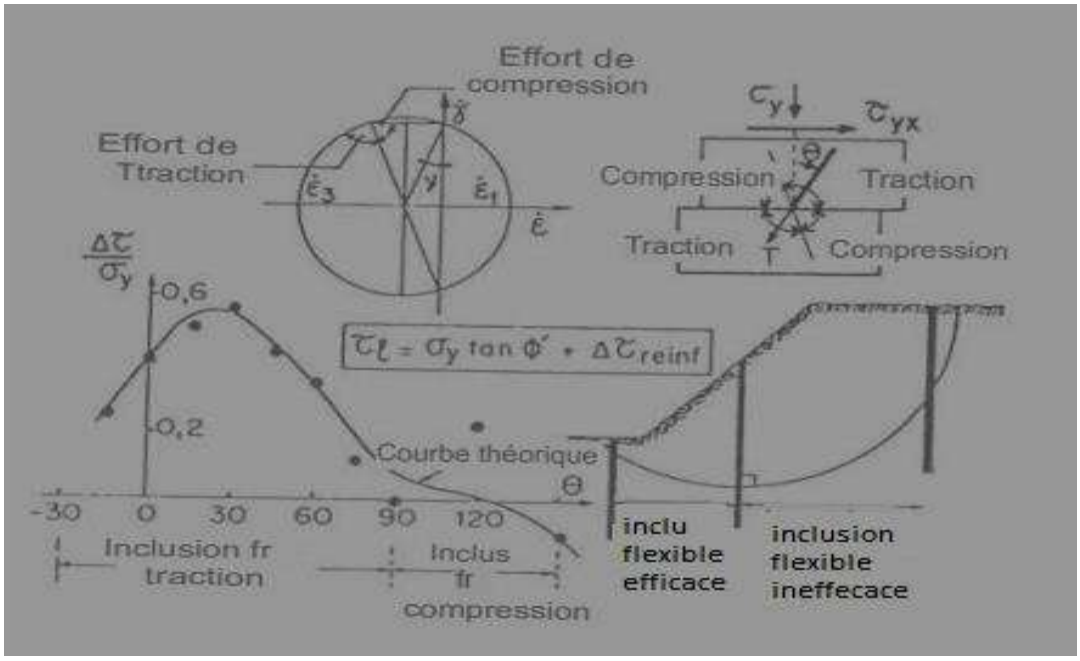


Figure 1.16. Augmentation de I'effort de cisaittement du sable Δτ/σ, en fonction de l'orientation de l'inclusion (Jewel, 1980).

JEWELL has also shown that a very small displacement is sufficient to mobilize the tensile strength of the inclusion and obtain the corresponding increase Δτ in the average shear stress along the sliding surface.

these results suggest that it would be inefficient to want to reinforce the upper part of an unstable slope by flexible vertical inclusions, that is to say oriented in a direction in compression.

the result was even a decrease in the shear stress mobilized along the fracture surface. Such reasoning does not, however, take into account the action of the lateral pressures of the soil on the inclusions which has the effect of mobilizing the flexural strength of the inclusions.

The influence of a rigidity of the inclusions was investigated experimentally in the laboratory in direct shear tests on loamy soil reinforced by a row of vertical steel bars of variable diameter: 8 to 12 mm (JURAN eI âl., 1981). The results showed that:

1. the progressive mobilization of the resistance to flexion of the bars gives the soil a visible cohesion C*;
2. the displacement necessary to mobilize this apparent cohesion is greater than that necessary to mobilize the soil-inclusion friction. JURAN et al. (Helsinki Congress) present ananalysis by

the finite element method of these shear tests. FIG. 16 shows a comparison between the theoretical mobilization of the overall apparent cohesion C* and the experimental results. The curves show a bend for a relative displacement of about 4%, a bend which reflects the appearance of plasticity of the soil around the bars. The finite element analysis is in fairly good agreement with the experimental results. It shows that the apparent overall cohesion of the nailed ground is indeed greater than that corresponding to the sum of the cutting forces mobilized in the bars. This difference is due to the fact that the presence of inclusions modifies the fields of deformations and stresses in the soil.

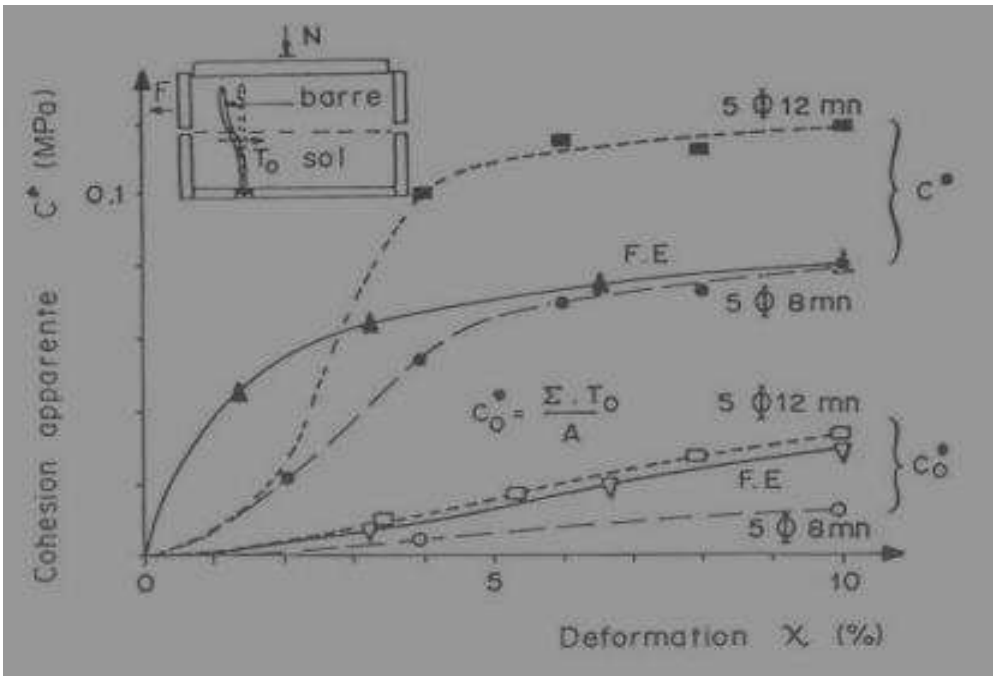


Figure 1.17 Mobilization of apparent cohesion in nailed soil (Juran et al., 1983).

FUKUMOTO (L974 and L976) reported observations on the stabilization of slopes by nailing and more particularly an in situ shear test on a mass of soil reinforced by two tubular steel piles. The results show a mobilization of the bending resistance of the piles similar to that described by JURAN et al.

When nailing is used to stabilize a moving slope, the behavior of the nailed block is governed by the creep properties of the soil since it is a visco-plastic flow. It therefore seems a priori different from the behavior of the same nailed bed, but in static equilibrium. ITO and MATSUI (1975) proposed two approaches to analyze the problem of the lateral pressure exerted by a moving ground on a row of piles resistant in bending. They considered on the one hand a plastic deformation of the soil around the piles, and on the other hand a

visco-plastic flow of the soil around the piles, and on the other hand a visco-plastic flow of the soil around the piles.

Both methods were applied to predict the forces exerted on piles at five different sites; they gave fairly similar results which corresponded relatively well to the experimental observations.Moreover, in the visco-plastic flow method, the force on each pile increases with the viscosity and with the sliding speed of the soil, but does not vary much with the

elastic limit of the soil considered as a Bingham solid. These results indicate that one can, as a first approximation, analyze and dimension the reinforcement by nailing of a moving slope from pseudo-static considerations. This is what has been proposed by WINTER et al. (Helsinki Congress) in a method where the lateral pressure exerted by the soil on the pile is calculated with an undrained cohesion varying as a function of the sliding speed.

Slope nailing density is also an important parameter of the behavior as it controls the group effect. The effect of this is to involve the soil in the reinforcement, which is to consider thicker apparent inclusions, each consisting of the bar and part of the surrounding soil. Thus, the total effort that the group of bars resists is greater than the sum of the ground effort on each bar.

When the density is high enough, the nailed ground behaves like a monolith. But this group effect, which we encounter quite successfully in soil mechanics, has not yet been studied very much.

Current design methods for nailed slopes under static equilibrium conditions do not take the group effect into account. The Terrasol method presented by GUILLOUX et al. (1983) for the design of retaining walls in nailed soil can also be used for slope stabilization. CARTIER and GIGAN (1983) proposed a method which takes into account a circular sliding surface and which calculates the increase in the safety coefficient due to the moments and the frank forces mobilized in the inclusions. To calculate these moments and these shearing forces, it is necessary to know the relative displacement of the inclusion with respect to the ground and therefore the initial field of displacements in the slope in the absence of any reinforcement. Apart from a few particular cases, the knowledge of these displacements is difficult and requires in general a finite element analysis.

For slopes nailed in motion, WINTER et al. (1933) propose a pseudostatic calculation method which is based on the principles described previously (**see 2.2.2.).** The lateral pressure exerted by the soil on each inclusion and corresponding to a decrease in the sliding speed from V1 to V2 is given by the formula:

$$p=\lambda.\frac{s}{h} ln \frac{V1}{v2}$$

where **s** and **h** are respectively the section of the apparent inclusion and the effective height of inclusion which resists lateral pressure; **λ** is a parameter which depends on the creep characteristics of the soil.

1.5.2.2. piles :

Landslides are frequently stabilized by one or two rows of long piles rigid enough to withstand large bending moments (YAMADA et al., 1977; FUKUMOTO, L97Z; KÉRISEL, L976; SOMMER, 1979). The behavior of this reinforcement differs from that of a nailed slope because the row of piles constitutes a relatively rigid screen and consequently an element of discontinuity in the field of displacements of the slope. Usually, the piles are placed at the foot of the slope and this results in gradual stabilization, starting with the lowest part of the slip. But, like in ground nailing, the bending strength of the piles is an essential parameter.

Three types of dimensioning methods have been developed.

The first (BRINCH HANSEN, 1960) considers a rigid plastic soil and assumes that the lateral pressure of the soil on the pile is fully mobilized on both sides of the sliding surface. This method is usually applied to very rigid piles.

The second is of the elasto-plastic type. It was proposed by JURAN et al. (1981), CARTIER and GIGAN (1983) and requires knowing the displacements of the soil in relation to the pile "This method is well suited to fairly flexible piles.

The third, described by WINTER et al. (1983), relates to the stabilization of moving slopes.

The first and third methods were used by SOMMER (L979) in his analysis of a moving slope stabilized by a row of rigid piles 3 m in diameter. a layer of clay 10 m thick slid at a speed of 14 mm per month.

Figure 17 shows the calculation and measurements of the lateral pressure exerted by the soil on the piles. The resistance due to the piles contributed only about 5% of the total shear resistance along the sliding surface (stress ofground shear + shear forces in piles), but this was found to be sufficient to reduce the sliding speed to about 10% of its initial value.

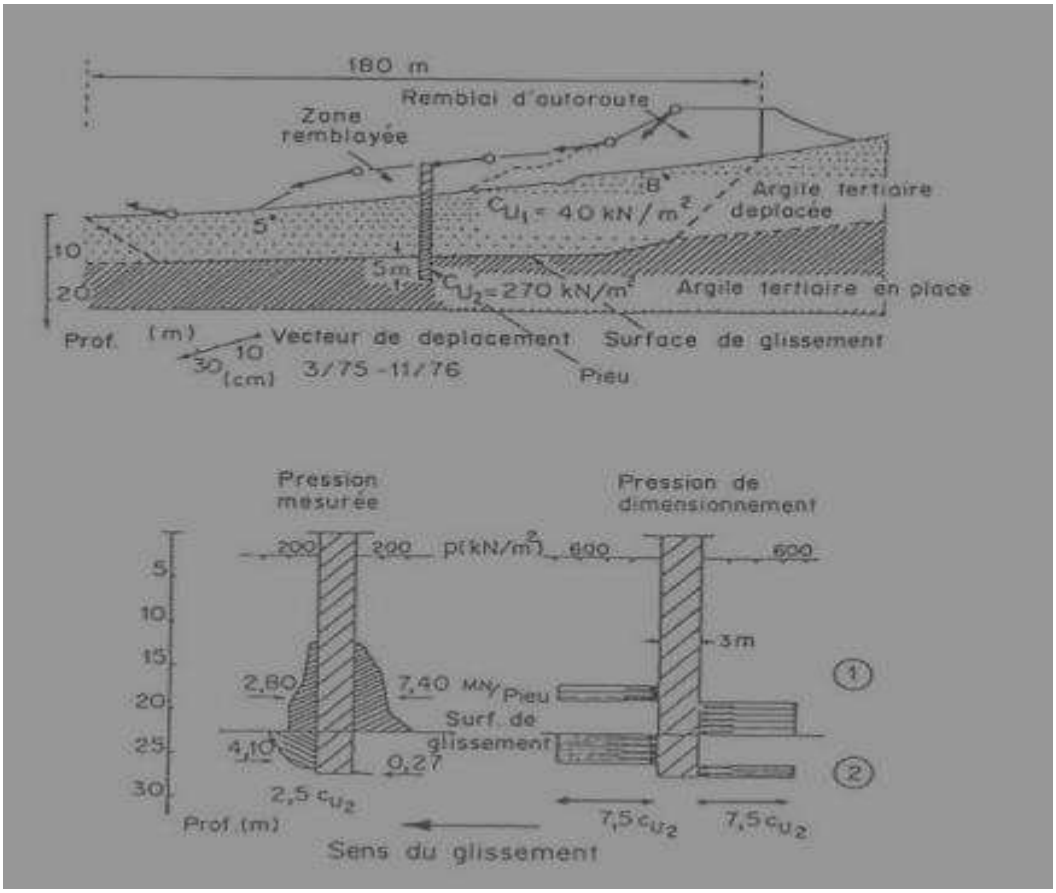


Figure 1.18. Stabilization of a slope slide (Sommer, 1979)

CARTIER and GIGAN (1933) used, to stabilize an unstable slope under a railway track, more flexible inclusions consisting of three rows of bored concrete piles (40 cm in diameter) reinforced by H200 metal profiles. The slope initially slid at a speed of 10 cm per year .

The pile displacement measurements made it possible to calculate the shear forces and bending moments and showed that an increase of about 7% in the safety factor was sufficient to reduce the sliding speed to 2.5 mm per year.

1.5.3. Reinforced shallow foundations :

Six papers presented at this session concern the influence of horizontal soil reinforcement on the bearing capacity of surface foundations. Many articles have dealt with this problem in recent years. It would be possible to find a general trend in the behavior of such a foundation by comparing all these articles, but it is likely that such a procedure would prove difficult due to the lack of real data on the research reported. For example, data on the depth of the first layer of reinforcement or sand density may be missing; or, the angle of friction measured at

The triaxial apparatus may be given without any information on the average stress level, which differs greatly in triadal or reduced model tests; or gives no indication of the limited bearing capacity of the same reduced, but not reinforced, foundation model. Another difficulty arises from the different failure criteria that can be used, so some criteria are relative to maximum loads while others consider loads corresponding to well-defined settlement values. The following is a synthesis based on articles with sufficient data to be properly analysed

1.5.3.1The bearing capacity of foundations on reinforced sand:

Particular interest was given to tests on reduced models of reinforced sand foundations. These models use one to six layers of reinforcement. The field of displacements has in general been very carefully studied by photogrammetric or stereo photogrammetric techniques. The test tank is in this case provided with a thick green wall which allows photography or else a two-dimensional model is used more simply with the Schneebeli roller technique. The displacement field is used to determine the zero extension lines or the main extension lines. The latter are very important since it is according to their direction that the inclusions are the most effective (ANDRAWES et al., 1978).

The field of movement depends among other things on the matter of inclusions. With synthetic materials that normally have a lower ground-inclusion friction than the internal ground friction, displacements tend to follow the surface of inclusions. A steel wire mesh may have a ground-inclusion friction equal to the internal ground friction (ANDRAWES and âL., 1978) and, in this case, the direction of displacement cuts the inclusions. When flexible inclusions (e.g., geotextiles) are used, the ground may rupture before inclusions are solicited.

In this case, the lines of zero extension are almost confused with the characteristic lines of the deformations as they are considered in the theory of plasticity. The fields of displacements observed are in general in good agreement with the classic diagram of rupture relating to the bearing capacity (figures 18 and 19). The plasticity theory therefore seems to constitute a good starting point for studying the bearing capacity of a reinforced soil, at least when it comes to reinforcements by flexible inclusions. In the absence of a better theory, it can also be used for rigid reinforcements. However, the observation of

displacements in the part located above the inclusions (ANDRAWES et al., 1978) seems to indicate that the upper part of the soil should be considered as a special layer.

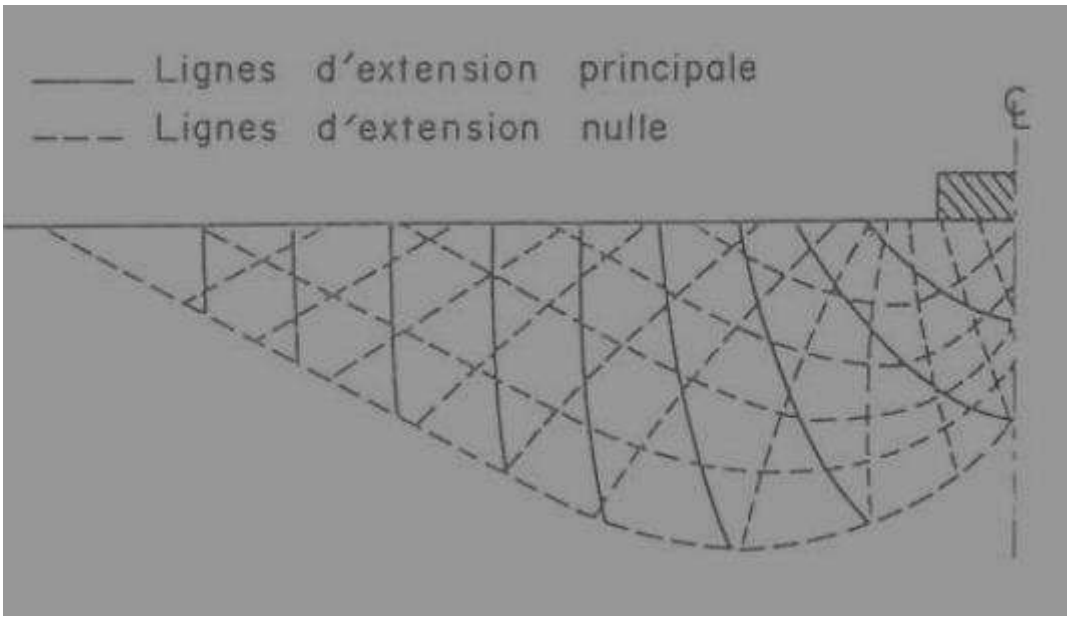


Figure .1.19. Diagram of observed trajectories, main extension lines and zero extension lines for a foundation on sand (Andrawes et al., 1983).

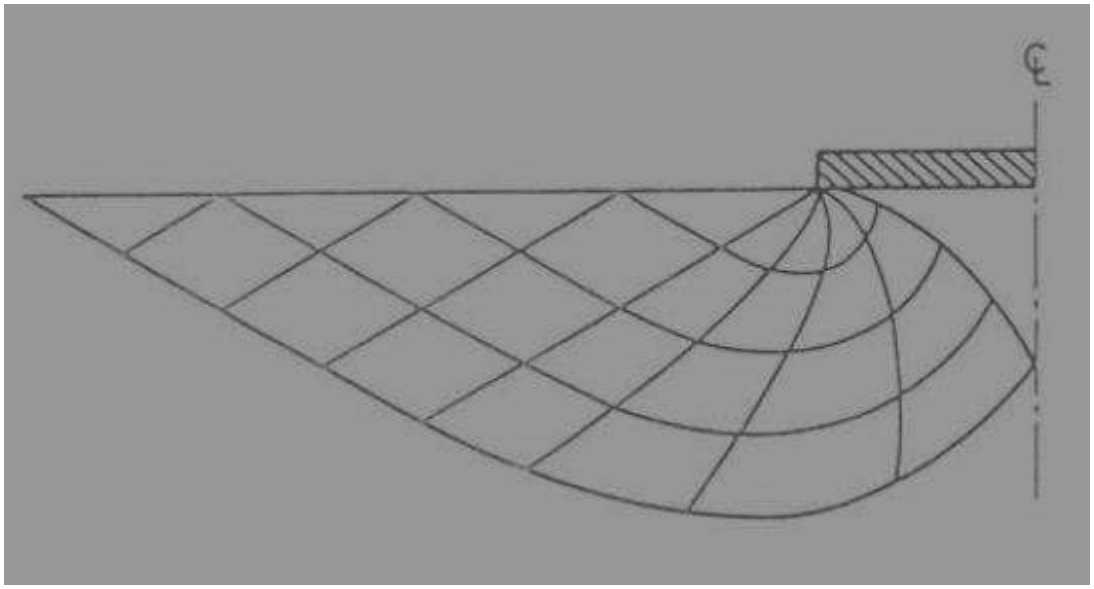


Figure1.20. Friction scheme for the bearing capacity of a foundation on sand (according to Lundgren and Mortensen, 1953).

1.5.3.2. Reinforced foundation on soft ground :

Five communications submitted to this conference concerning the reinforcement by a layer of geotextile, placed on the surface of a soft soil under an embankment or under layers of pavement.

KEBS et al (1983) carried out centrifuge tests and BOUTROUP et al. (1983) performed a finite element analysis. These two studies provide similar qualitative results which explain the mechanism: the geotextile restricts the lateral movements of the subsoil, which leads to a practically uniform settlement of the loaded surface. The amplitude of the total settlement is only slightly reduced. However, as shown by JEWELL (1982), the use of a geotextile extending down to the side benches has the effect of further restricting lateral movements, and then leads to a reduction in total settlement. BOUTROUP et al. showed that in undrained conditions ($v = 0.5$) the tensile forces that develop in the geotextile are greater than those obtained in drained conditions ($v = 0.33$).

GOURC et al. (Helsinki Congress) experimentally studied the reinforcement by a geotextile layer resting on a clay subsoil. The geotextile had been placed at the interface between the sand and the clay.

Among other things, they showed that under static loading the geotextile changes the displacement field in the clay subsoil, which has the consequence of reducing total settlement and increasing bearing capacity.

The most commonly used design method is based on a circular sliding stability analysis taking into account the tensile forces mobilized in the inclusions (JEWELL, 1982). QUAST et al. (Helsinki conference) consider that, when a relatively deformable geotextile is used (admissible deformation of 5%), this geotextile follows the movements of the soil, in particular along the potential failure surface. As a result, the tensile forces are then directed in the direction of the fracture surface.

1.6 . Reinforcement :

1.6 .1. Type of reinforcement:

There are various types of geosynthetics available to be used as reinforcement in retaining systems : rigid and flexible geogrids , woven , nonwoven and reinforced geotextiles ; geobars , strips and the use of diffuse fibres (micro-reinforcement) .

To date , geogrids and geotextiles have a more significant role as reinforcement for the construction of walls and reinforcement of steep slopes , which is why discussion about the type of the reinforcement will be focused on these geosynthetics.

Besides the sreucture itself , there are several types of polymers available , and the most frequently used are : polyester , polypropylene and hight density polyethylene . Recently , fiber polyaramid has been used for special reinforcements that require particularly high tenacity (Ehrlich ,Azambuja , 2003).

The choice of a specific type of reinforcement for a reinforced soil retaining system design requires an economic performance analysis , since any product can be used , provided that the mechanical and rheological properties are known.

However , there are some conditions or structural features of the environment that favor certain sets of geosynthetics and therefore should be observed when choosing the type of product or polymer to be used.

1.6.1.1 Height of wall :

The greater the height of the wall , the greater the strength required from reinforcements . In general , geogrids are more favourable for walls higher than 4 meters ; while geotextiles often present economic advantages for smaller walls (Ehrlich , azambuja 2003).

1.6.1.2 Strain/Deformations Restrictions :

In walls where the dimensional tolerances are important constraints or post-construction strains are undersirable , geogrids are most appropriate , followed by reinforced geotextiles in polyolefin reinforcements (high-density polyethylene and polypropylene) and impose restrictions on products with such polymers when post-construction strains (long term) are critical constraints . In such cases , the use of polyester or polyaramid reinforcements usually presents economic advantages.

1.6.1.3 Severity of the environment :

The chemical aggressiveness of the soil ot the soil or of the environment where the retaining system shall be deployed is another important factor to consider for product choice . In chemically adverse environments , geogrids are less sensitive than geotextiles This is due to greater thickness of its elements , which reduces the surface area of exposure , or to the protective coating that some products have . certain acidic environments , foe example ; are restrictions on the use of polymers by hydrolysis (Ehrlish , Azambuja , 2003).

In addition to the chemical aggression , we must consider the serverity of the environment towards the mechanical damages of installation . Coarse soils , with angular grains damage woven geotextiles more profoundly than nonwoven ones. Rigid geogrides present lesser damages than flexible geogrids . These sensitivities should be considered in determining permissible resistances of reinforcements or in the prescription of the soil type for each specific project situation ,

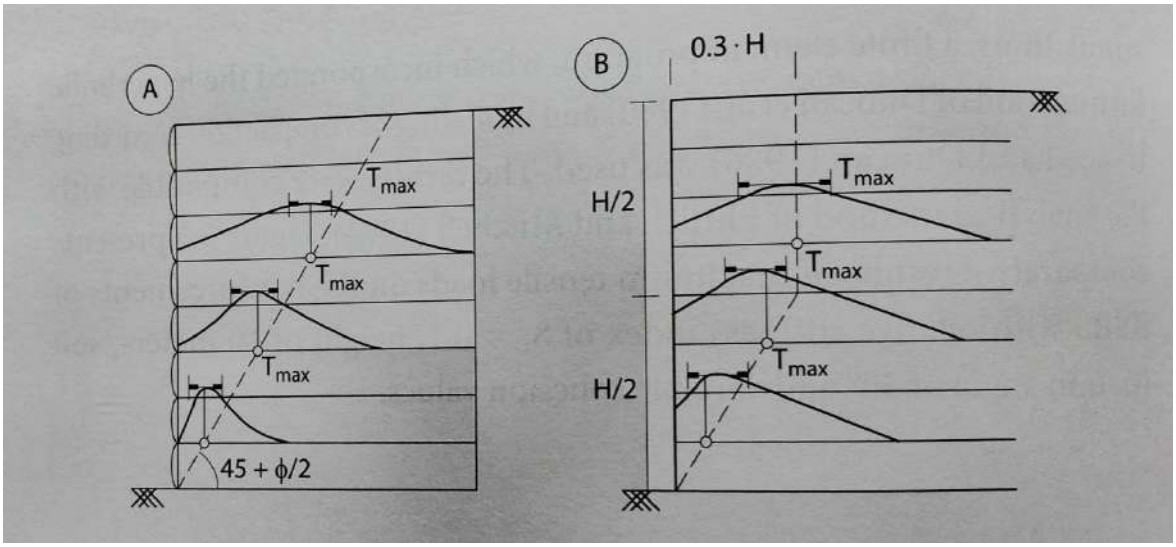


Figure 1.21 T_{max} location for reinforced soil walls (Christopher et al., 1990)

extensible reinforcement , (B) non extensible reinforcement

1.6.2 Type of facing and construction procedure :

In the first reinforced soil structures with geosynthetics , a constructive technique called wrapped facing was used . Later , constructive systems evolved to use other forms of facing . The types of systems most commonly used in practice (Ehrlich , Azambuja ; 2003) will be discussed below .

1.6.2.1 Wrapped facing systems :

The technique is comprised of the conformation of system where the geosynthetic confines the soil laterally between two layers of reinforcement , by means of its fold and anchoring inside inside the wall . In most cases , the wall or reinforcement of steep slopes is erected with the help of lateral and removable light elements , and then a permanent , and then permanent face protection system is built . Self-enveloping systems can be built with the anchoring at the base (Figure 4.1) or at the top layer (Figure 4.2) . Tapically , these anchors have a minimum length of 1 meter , but may be longer if the design requires it .

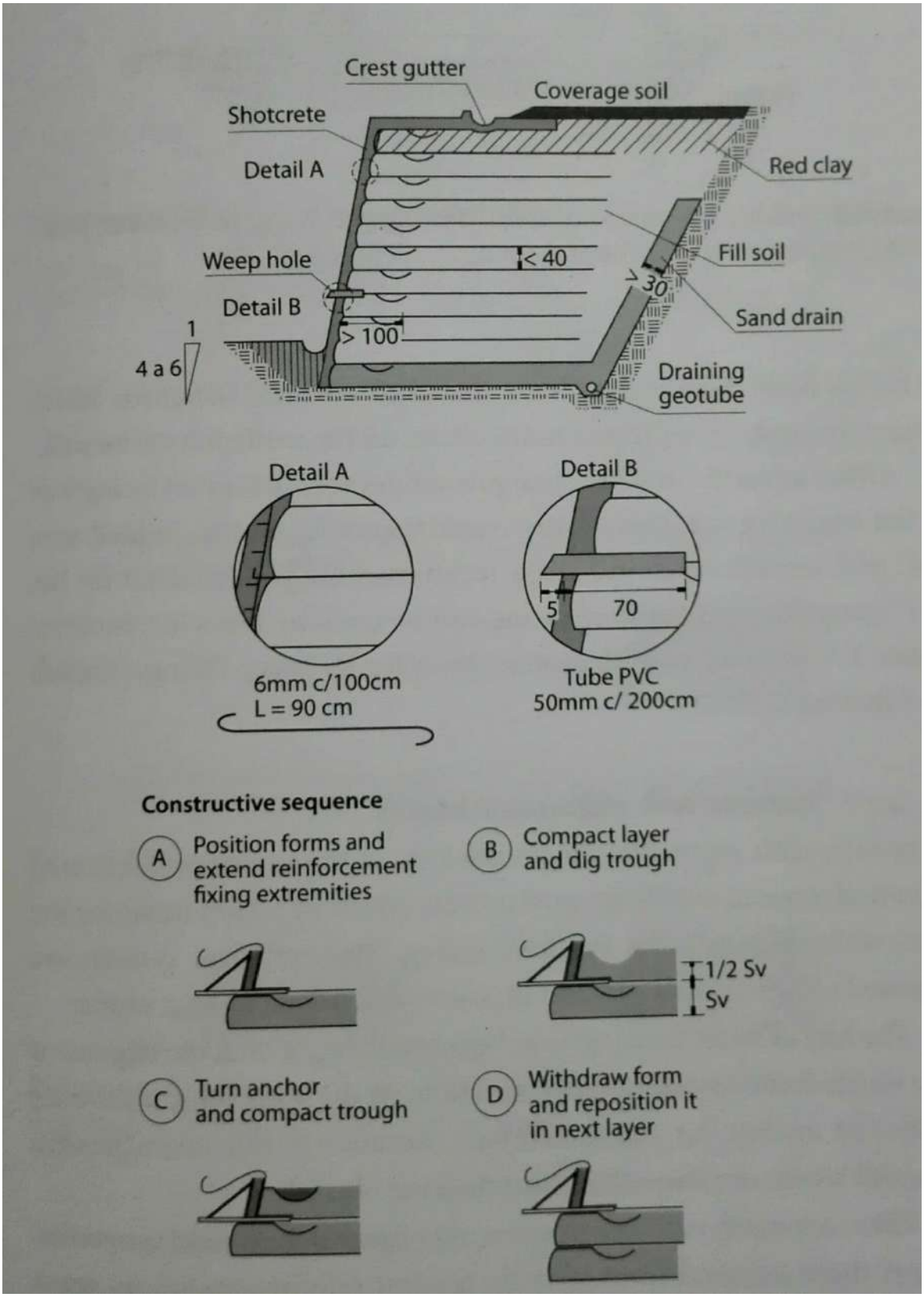


Figure 1.22 Typical cross section of wall with wrapped facing systems with top anchor (adapted from Ehrlich and Azambuja, 2003)

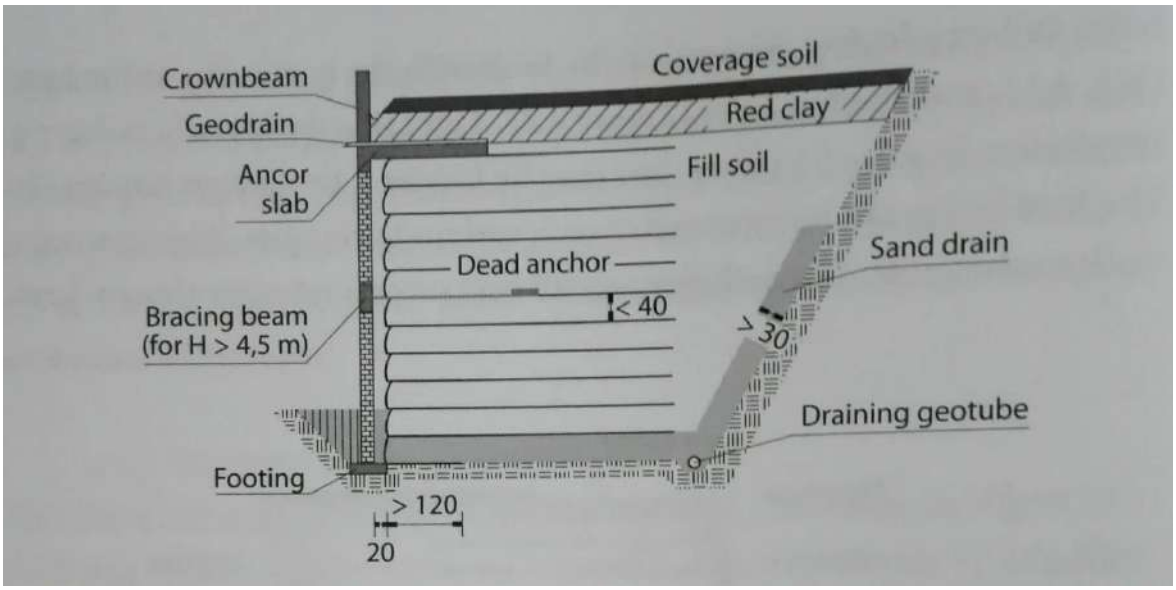


Figure 1.23 Typical cross section of wall with wrapped facing systems with lower anchor (adapted from Ehrlich and Azambuja,2003).

The final facing can be constructed with various techniques , from masonry walls to shotcrete . The techniques with best success rates are those where the wall is built slightly away from the reinforced soil structure , which reduces the post-construction strain effects on the aesthetics of the wall .

Other attractive construction procedures for a wrapped facing wall are the ones that use a lost-form , consisting of an electro welded wire mesh and of geogrids , and the final facing can be constructed with shotcrete (figure 4.3) for walls or with ground cover for less steep facings (Ehrlich and Azambuja 2003).

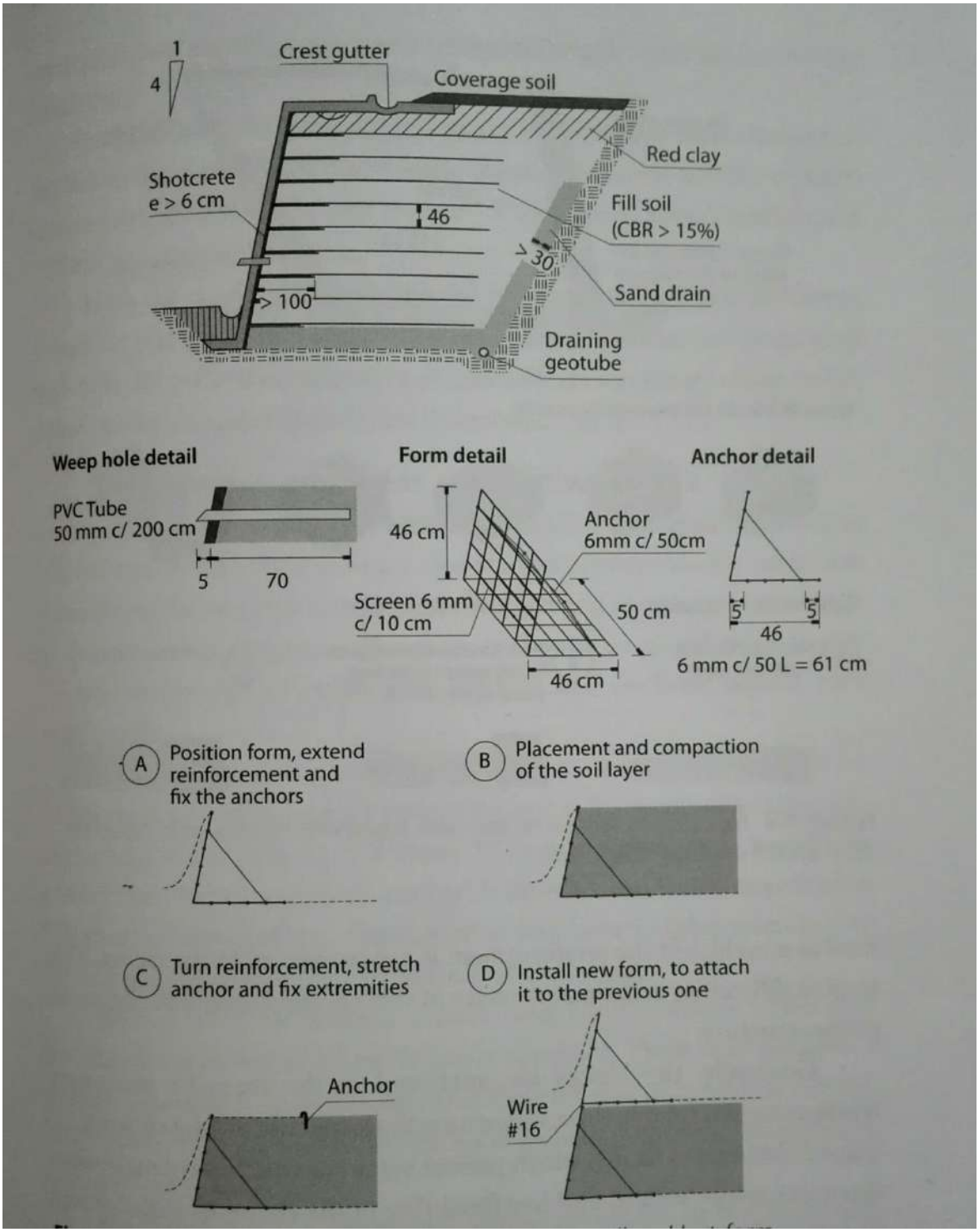


Figure 1.24 typical cross section of wrapped facing wall and lost-form construction procedure (Ehrlich and Azambuja, 2003)

1.6.2.2 Systems with segmental blocks :

The system with segmental blocks consists of the use of prefabricated concrete elements , which are used as a leteral compaction mold for the layers , while they are also the final facing . Most of these systems are composed of light blocks that can be assembled manually by a worker .

The blocks have fitting devices between them , so that the alignment of the wall is facilitated during construction . At the same time , it provides an efficient an efficient anchor for reinforcement . Because of this characteristic , segmental blocks are also called "interloking blocks" .

There are many such constructive systems with industrial properties , some of them adapted for a specific type of reinforcement – in most cases , geogrids . An example of segmental block system is presented in **figure 1.25**.

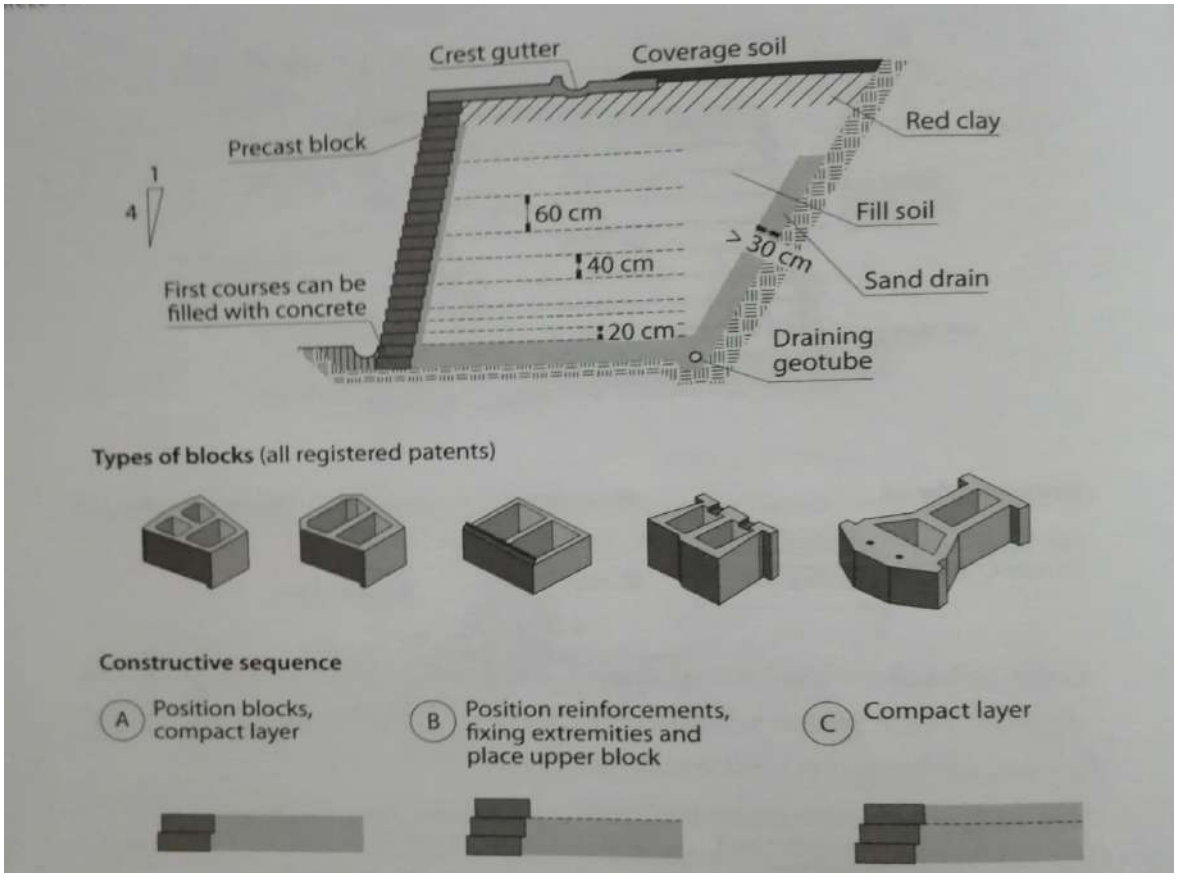


Figure 1.25 typical cross section of wall with segmental block system (adapted from Ehrlich and Azambuja , 2003) .

1.6.2.3 Hybrid systems :

Some system with segmental blocks are associated with wrapped facing techniques , constituting a hybrid method , mostly used for reinforcements with geotextiles. There are techniques in which the final facing is also used as a mold , but the reinforcement is not connected to the facing . This implies difficulties in the maintenance of vertical and horizontal alignment of the structure .

Generally , the blocks are anchored to the mass by secondary reinforcements . Among these systems are those that use precast concrete “L” shaped elements as facing , which present some constructive entanglements due to excessive weight and less flexibility of elements (Ehrlich and Azambuja , 2003) .

1.6.2.4 Systems with modular panels and integral panels

The construction processes of reinforced soil with modular panels was disseminated with the wide employment of reinforced earth. In such cases , the facing also acts as a mold ; however , the connection of the panels with the reinforcements and between the panels is more complex . Although relatively slim, modular panels are heavy and require mechanization for handling .

Integral walls , in turn , are tall structures in which each element of the face is the total height of the wall . These systems are applied to walls that require special aesthetical features of the facing , since there is great difficulty to manage the parts .

Both the modular panels and the integral walls are systems better suited for less extensible reinforcements , such as geobars or straps with hight tenacity polymers , since they do not tolerate significant constructive deformation / strain .

1.6.2.5 Comparison between constructive systems

Among the various constructive systems available , the designer or developer will have to choose the technique that is most appropriate for each case , considering the cost , the mechanical function and architectural function of the retaining structure . However , it should be noted that despite the effectiveness of all techniques , the cost and aesthetic aspects vary significantly.

Wrap around walls are usally built with geotextiles , but geogrids can also be used , provided there is a system to prevent the escape of soil through the facing system (lost – form system) . With this system , one can achieve a lower cost , but dimensional control is difficult and facing appearance falls short of expectations . These systems also have a higher tolerance to settlements , because of the low stiffness of the facing .

Segmental block systems usually have good aesthetic finish and are often used in works where facing is apparent . These systems have a reasonable tolerance to settlements and allow for easy dimensional control , which is why they are best suited for buiders with no previous experience in reinforced soil works.

Modular panel or integral wall systems should not be built with low stiffness reinforcement . The facing appearance is good , as is dimensional control , however , costs tend to be higher , and the tolerance to settlements is lower .

1.6.3 Arrangement of Reinforcements :

Reinforcements may be arranged with uniform or variable vertical spacing . In wrapped facing systems , uniform spaces are recommended due to constructive issues , especially for control of constructive strains , although such practice leads to higher consumption of geosythetics . With segmental block systems , variable spacing between layers allows a better rationalization of the system , but of course , the maximum spacing between reinforcement depends on the capacity of blocks to absorb the stresses generated by the compaction without rupturing a portion of the face .

Instability problems on the face due to large spacing can be solved with the use of short secondary reinforcements between the layers of the main reinforcements , in any constructive system .

In general , limiting the vertical spacing ($S\nu$) to 0.80 meter is recommended . For segmental blocks system , it is recommended that the spacing not be greater than the depth of the blocks.

1.6.4 Drainage System :

Draining is one of the most important aspects of building reinforced soil wall . In all currently used design methods , it is assumed that pore pressure is null in the reinforced soil mass .

Tropical soils , usually compacted near the optimum moisture content ; are partly saturated and usually have considerable amounts of suction (negative pore pressure due to partial saturation) . Suction is difficult to estimate and it is not considered in the calculations, but has the effect of increasing stability .

The presence of water level within the reinforced soil is doubly undersirable because is leads to positive pore pressure , invalidating the hypothesis of null pore pressure and it eliminates suction . In both cases , the effect is reduced structure safety .

To prevent this from happening , the reinforced soil must be provided with edequate draining systems .

It is strongly recommend to employ a draining blanket with thinckness between 20 and 50 cm under the reinforced soil mass and between the mass and non-reinforced area .

Figure 1.26 shows a typical cross section of a reinforced soil wall with segmental block facing with a draining blanket between the reinforced soil mass and the natural soil . It is also evident that another layer of draining material is placed near the face and blocks are filled with gravel . The top of the structure is covered by a layer of compacted clay material to prevent seeping of rainwater , and it is advisable to install drain gutters .

The draining material may be composed of clean sand or pebbles . To play the role of a filter and prevent the escape of fines from the reinforced and non-reinforced soil , the draining material can be encased in geotextiles or grain size of the material can be selected so that the granulometric curve is not discontinuous and it meets the following filtration criteria :

$$5.D_{15\ soil} < D_{15\ drain} < D_{85\ soil}$$

Where D_{15soil} is the diameter below which are 15% of soil weight ; $D_{15\ drain}$ is the diameter below which are 15% of draining material , and D_{85} soil is the diameter below which are 85% of soil weight .

Besides the drainage of groundwater , there should be a superficial draining/waterproofing system to prevent erosion and infiltration of rainwater into the reinforced soil mass , which usually consists of longitudinal and transverse gutters (stairways) , catch basins , storm drains , etc.

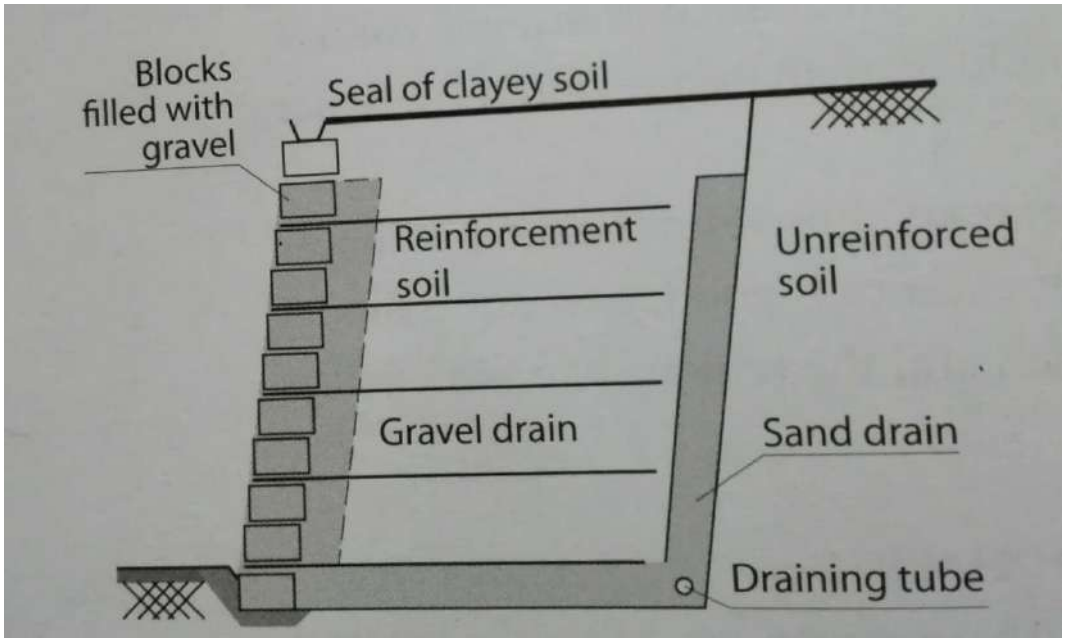


Figure 1.26 Draining blanket system

1.6.5 Backfill material

Because of transport costs , one can say that , except for useless materials , the best soil quarry is the one that is nearest . It is recommented that the first soil search for the work be done through be the observation of cut slopes or excavations near the work site and auger surveys . The soil thus obtained should be classified by visual-tactile analysis carried out by experienced staff .

Obviously useless soils should be discared immediately , such as peat , organic clays , soft clays , granular soils rich in mica etc.

After verifying that the chosen soil is enough in quantity for the requirements of the work , a representative sample of about 20 kg and 40 kg , in case of fine or coarse soil , respectively , should be sent to the laboratory for testing as listed in Table 4.1 .

The characterization test results can be used to classify the soil according , for example , to the Unified Soil classification System described in Table 2.2. However , it is know that this classification is inappropriate for tropical soils , wherther lateritic or saprolite , since these soils are product of very different evolutionary processes from those in temperate climates , and they behave differently . For example , high percentages of fine particles in soils of colder climates usually lead to poor mechanic performance , which does not occur in tropical soils .

1.6.6 Quality control and constructive tolerances :

There are three aspects are important in quality control of reinforced soil retaining walls : nominal resistance of the reinforcements , mechanical damage control and control of strains/deformations during construction . It is recommended that the reinforcements be tested per batch and each 1,000 m^2 of reinforcements .

The minimum recommended tests for reinforcement control are wide width strength and puncture , and they should provide result compatible with the nominal strength , for a 95% confidence level .

Constructive strains/deformations should be controlled at each layer .

The distortions of the face (ratio between horizontal displacements at the crest and the height of the wall) should be less than 1% for integral walls and panels , 2% for segmental blocks and 5% for wrap around systems (before the final face).

1.6.7 guardrails :

The designer should be careful with vehicle impact problems at the top of the reinforced soil walls , where they usually hold roadbeds . Guardrails should not be directly attached to the facing , but to an independent retaining structure .

1.7 .Analysis and design:

A reinforced soil structure must meet internal and external stability conditions . Its design specifications consist of several steps for defining dimensions , materials and verifications of safety factors , as will be described bellow .

1.7.1: External Stability :

The analysis of external stability can be conducted considering the reinforced soil mass as a conventional gravity wall . Thid "wall" guarantees the stability of the no reinforced soil zone . Under the earth pressure of the non-reinforced mass (E) , wall are designed in order to ensure stability to avoid sliding and overturning , beyond the load capacity of the foundations , and prevent global instability (**Figure 6.1)** .

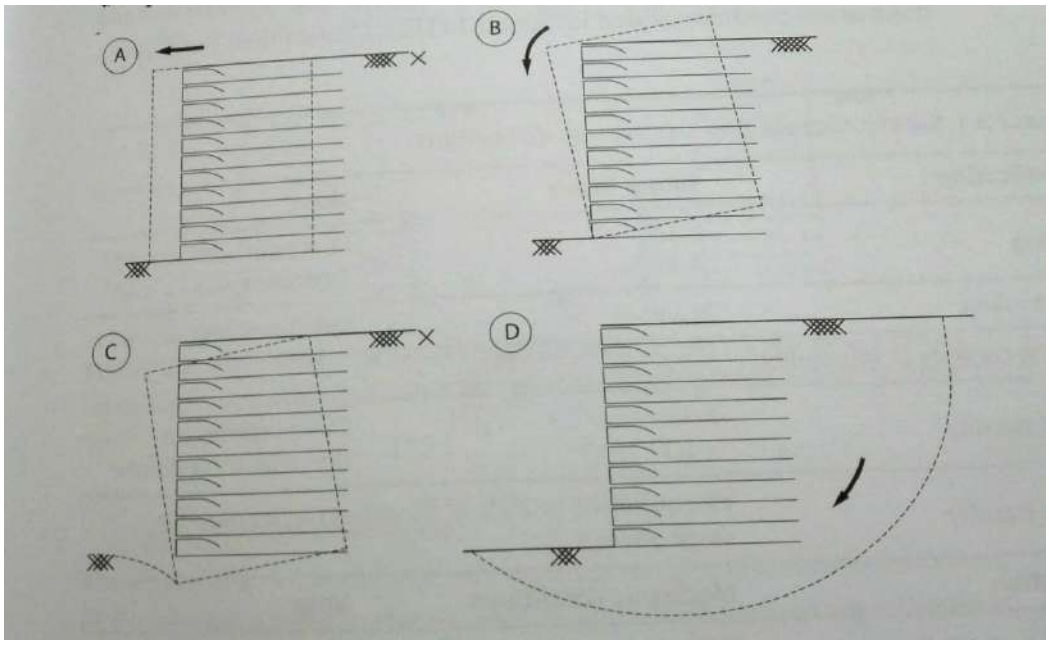


Figure 1.27 mechanisms of external instability: (A) sliding,(B) overturning , (C) load capacity of foundation ; (D) global instability

To determine the ground pressure the non-reinforced soil mass imposes on the reinforced mass (E) , it is possible to adopt the classical theories based on limit equilibrium . Many authors recommend using the Coulomb formulation , assuming the friction between the wall (reinforced area) and the ground (non-reinforced area) are equivalent to the angle of internal friction of the soil at critical state ($\delta=\Phi_{cv)}$ .however , some authors consider the mobilisation of friction unlikely , since the reinforced area does not behave as a stiff block

. Thus , the formation of Rankine would be more appropriate and the active pressures woild be parallel to the ground surface (δ=0) .

If there in intention to apply external overloads on the embankment of the retaining structure, the resulting stress increase **(Section 6.9)**must be considered in the external stability analysis .

Just as the stability analysis of conventional walls , the verification of external stability can be achieved through the adoption of global safety factors (deterministic approach) , taking into account the slope of the structure face (ω) and the material of the foundation or by using weighting factors (deterministic approach) . In the event that deterministic criteria are adopted , the external stability takes into account the safety factors and mechanical condition lised in **table 1.5**

Verification	Safety factor	Note
Sliding	$\geqslant 1.5$ $\geqslant 1.3$	Structures with $\omega > 65°$ Structures with $\omega < 65°$
Overturning	$\geqslant 2.0$	Structures with $\omega > 65°$
Bearing capacity of foundation	$\geqslant 2.5$	
Global stability	$\geqslant 1.5$ $\geqslant 1.3$	Critical conditions Non critical conditions
Seismic stability	FS equivalent to 75% of FS values can be used for static analysis	
Verification	**Mechanic conditions**	**Note**
Eccentricity	$e \leqslant B/6$ $e \leqslant B/4$	soil supported structures rock supported structures

table 1.4: safety factors and mechanical conditions

1.7.3 Internal stability:

The particular aspect in design of a reinforced soil wall is the internal stability . The internal design of a RSRS must ensure safety against rupture and pullout of the reinforcements , and avoid instability located on the face . For this purpose , the following verifications are usually performed :

1.7.3.1 Maximum tension in the reinfocements :

The determination of the maximum tension in the reinforcements (T_{max}) is the key aspect in the internal stability analysis . To avoid reinforcements failure , the T_{max} value should not be greater than the minimum expected value for the geosynthetic design resistance , considering the environmental conditions and installation , and a reference time for the project (t_d) , ensuring an adequate safety factor **(Figure 1.28)**

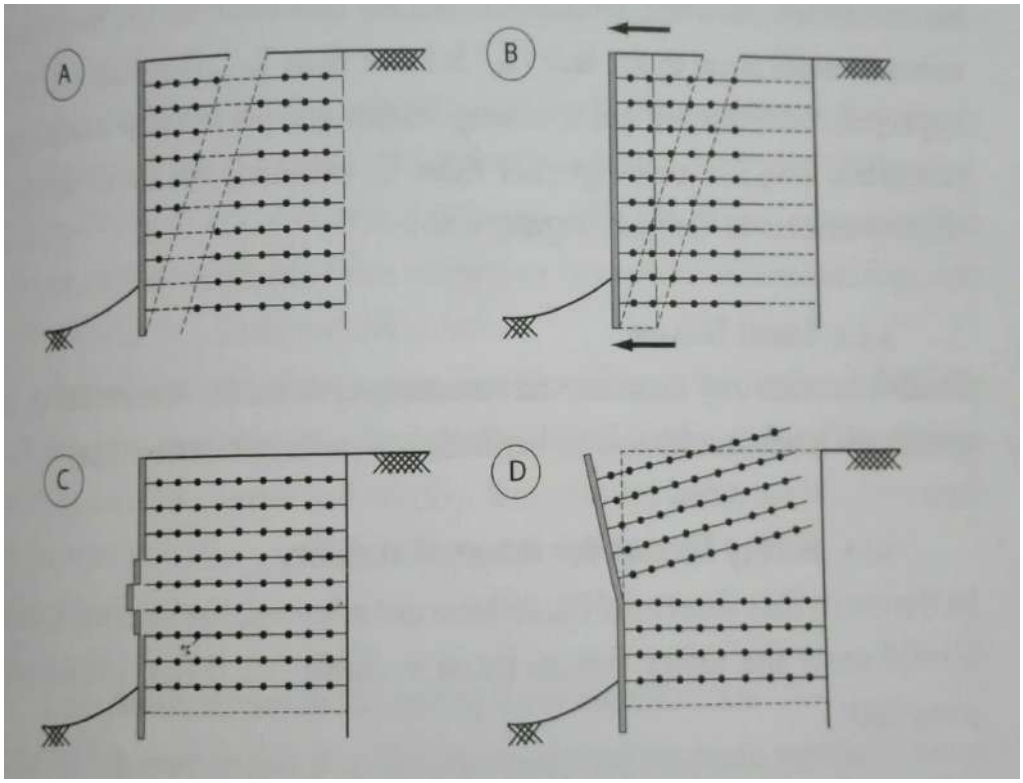


Figure 1.28 mechanism for stability analysis (A) reinforcement failure , (B) reinforcement pullout , (C) detachment of the face , (D) local instability (Ehrlich , Azambuja, 2003)

1.7.3.2 Pollout resistance :

The design of reinforced soil wall must also ensure minimum embedding of the reinforcement in the resistance zone , thus preventing its pullout **(Figure 1.28)** . Therefore , the T_{max} value should not exceed the pullout resistance value of the reinforcement embedment into the resistance zone (P_r) , in the respective layer , ensuring the safety factor . It is also necessary to know the position of the cristiral surface that defines the boundary between the resistant and active zones.

1.7.3.3 Reinforcement and facing connections :

Despite being crucial to the final aspect of the structure , the facing system does not play a relevant role in internal stability of the reinforced soil mass .

Its function , from a mechanical viewpoint , it to ensure the stability of the soil mases located between the geosynthetic layers in areas close to the face . Global equilibrium would be possible even in the absence of facing .

However , the connection between the reinforcement and the face should ensure that the remaining existing stresses in the reinforcement are transferred to the face . In general , the connections have less resistance than the reinforcements , but , to compresate , the mobilized tensile force in the reinforcement close to the face (T_0) is lesser than T_{max} . In an appropriate reinforced soil retaining system , the permissible connection resistance ($P_{r,0}$) must be greater than T_0 ,avoiding the detachment of reinforcements at the face

1.7.3.4 Local failure:

Finally , as with any incremental retaining system , the instability in any given wall level should also be verified ,

1.7.3.5 Safety factors for internal stability :

In the event that deterministic criteria are adopted , the internal stability should meet the safety factors listed factors listed in

Condition	Verification	Safety factor	Note
Reinforcement failure	$T_d \geqslant T_{max} \cdot FS$	$\geqslant 1.50$	permanent and critical works
		$\geqslant 1.15$	temporary and non critical works
Pullout	$P_r \geqslant T_{max} \cdot FS$	$\geqslant 1.50$	
Connection stability	$P_{r,0} \geqslant T_0 \cdot FS$	$\geqslant 1.50$	

table 1.5 safety factors and mechanical conditions

1.7.4 Detarmination Of Maximum Reinforcement Tension

The determination of the maximum tension (T_{max}) that acts on each level of the reinforcement is one of the most important aspects to be considered in a reinforced soil structure project , and the usual procedures for obtaining it are based on limit equilibrium methods . The T_{max} value is determined by considering the nessary streangh of the reinforcement and soil shear resistance .

This type of procedure is limited , as it does not consider the reinforcement stiffness and compaction effects in the analysis . Despite the advantages of its simplicity , limit equilibrium methods are not reliable for estimating the magnitude and distribution of stresses in the reinforcements under working conditions .

1.8. Reinforecement Tension:

1.8.1 Influence Factors On Reinforecement Tension:

Before addressing the determination of stresses that occur in geosynthetics in an RSRS , it is convenient to study the factors that influence it .

The interaction between soil and geosynthetic depends on the resistance and deformability properties of the soil , geometry , resistance and stiffness of geonthetic and boundary conditions .

1.8.1.1 Soil-reinforcement interaction mechanisms :

The basic soil-reinforcement interaction mechanisms provides by geogrids are different than those presented by other reinforcement such as geotextiles , for example . In geotextiles and geobars , stress transfers occur only due to friction , because of its format .

Because geogrids are hollow reinforcing elements , the soil penetrates the voids , and the interaction between soil and geogrids occurs through shear at the interfaces and passive resistance against the transverse elements .

It is generally accepted that shear develops in both longitudinal and transverse elements .

And there are three interaction mechanism : lateral friction in longitudinal elements , interlocking of the soil trapped between the voids and passive resistance against the transverse elements **(Figure 1.29)**

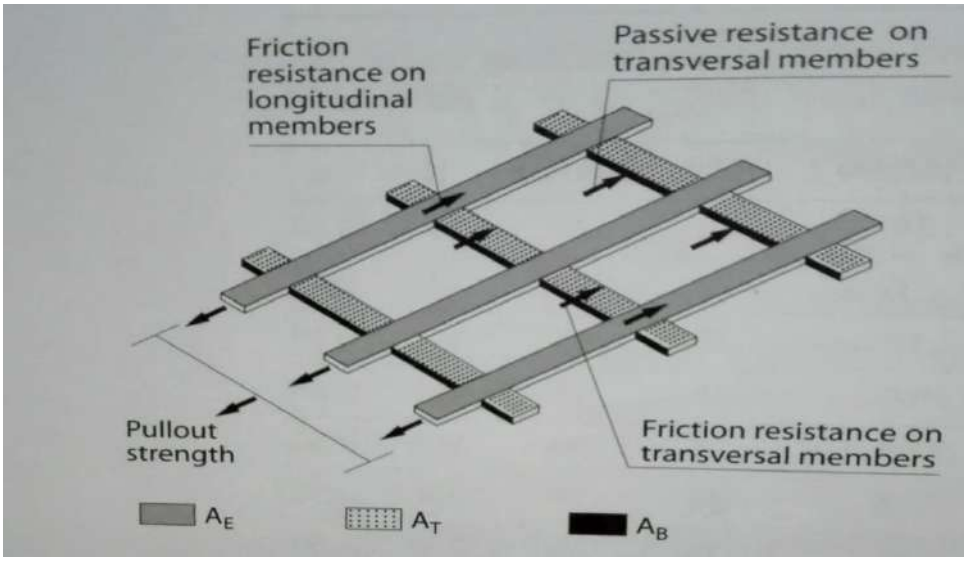


Figure 1.29soil-geogrid intraction mechanisms (adapted from Wilson-Fahmy and Koerner,1993)

It is hard to estimate the portions that belong the each of mechanisms . The larger the area of longitudinal members , the higher the interface shear resistance portion will be . And in case of geogrids with thin strips , the portion granted to shear may represent about 10% of the mobilized resistance .

To study the soil-geogrids interaction mechanisms , interface shear or pullout tests are primarily used , there are possible mechanism for internal collapse in reinforced soil structure**(Figure1.30)** .

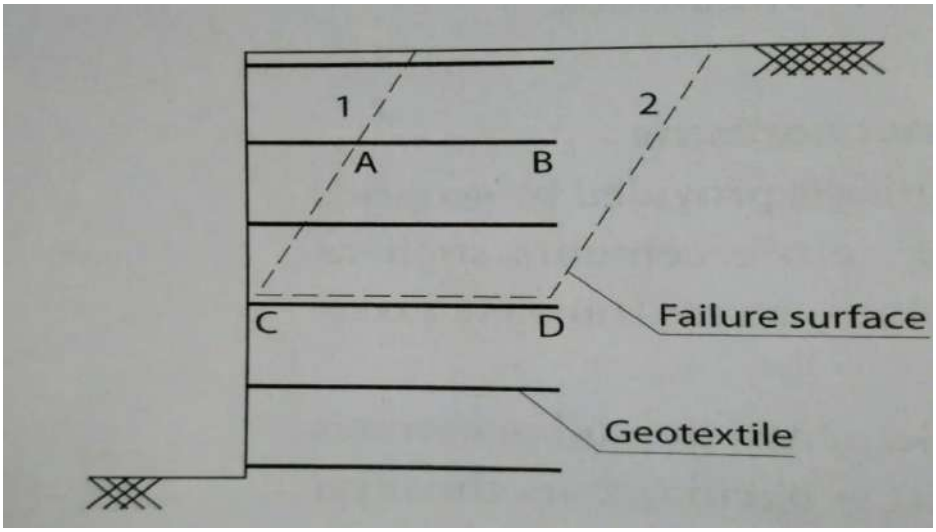


Figure 1.30: failure mechanisms in a reinforced soil structure (Miligan, Palm , 1987)

As first mechanism , the failure surface 1 crosses and eventually failure does not occur at point A . If reinforcement failure does not occur at point A , pullout may occur along AB . It is also possible that sliding would occur at the soil-reinforcement interface along CD , according to surface .

In **figure 1.31** , three tests are schematically presented . These tests are capable of inducing , in the reinforcement sample , efforts similar to those that occur in the field : direct shear , direct shear with sloped reinforcement and pullout , which aim to determine the resistance parameters of the soil-geosynthetic interface .

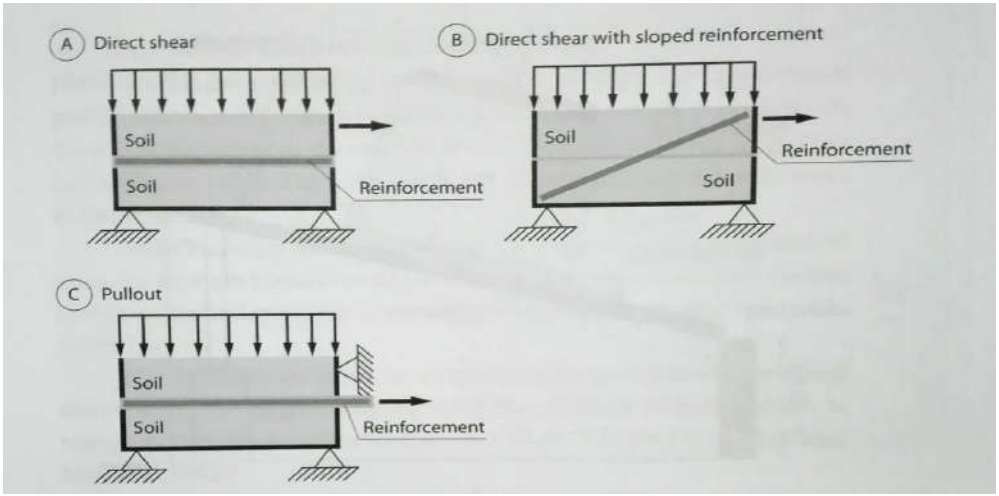


Figure 1.31 tests for: (A) direct shear of soil-reinforcement interface , (B) direct shear with inclined reinforcement , (C) pullout of reinforcement

The shear tests consist mostly of adaptations of the traditional direct soil shear test . The shear plane of the boxes should be adjusted to pass exactly at the interface between soil and geosynthetic . It is also possible to place the geosynthetic at an shear plane surface .

In the pullout tests , the reinforcement is confined by soil in the upper and lower faces . One end is buried in the ground , while the other is attached to a clamp , where increasing tensile strength is applied for pullout .It should be noted , however , that the boundary conditions vary significantly from test and therefore the obtained parameters for interface resistance may vary greatly.

Besides the direct shear and pollout tests , shear tests on an inclined plane or ramp tests can also be performed to determine resistance properties of soil-reinforcement interfaces . The technique has been used by sereval authours for some decades . **Figure 1.31** shows an arrangement recently by Becker (2001) .

Tests should be performed for different vertical stress , in order to define the shear resistance envelope . The goal is to determine the envelope and its corresponding parameters .

In the active zone , the direction of soil-reinforcement relative movement (and the mobilized stress) is apposite to that found in the resistant area . Thus , the point at which we observe the maximum value of tensile strength in the reinforcement (T_{max}) should occur in potential failure surface that separate these two zones **(Figure 1.31)** .

The maximum tensile strength value of the reinforcement depends on several factors , particularly the reinforcement and the soil stiffness properties and the induced stresses in the soil due to compaction . Based on the work of Ehrlich and Mitchell (1994) , a discussion of these factors and how they can be evaluated in a reinforced soil wall designis presented as follows .

1.8.1.2 Relative soil-reinforcement stiffness

Under working conditions , it is reasonable to consider the hypothesis of perfect interface adherence between the soil and the reinforcement , i.e. , there is no slip between the soil and the reinforcement . Thus , the soil and the reinforcement strain are the same at this interface .

Under such conditions , the behavior of reinforced soil is inderpendent from the strength transfer mechanism of the soil-reinforcement interface and thus also independent of the type of reinforcement used (geogrids ,geotextiles , etc).

The equilibrium stress or strain between geosynthetics and soil depends on the relationship between the stiffness of both , which is represented by the relative stiffness index (Si), as defined by Ehrlich and Mitchell (1994).

$$Si = \frac{\mathrm{Jr}}{\mathrm{K.\,Pa.\,Sv}}$$

where K is the hyperbolic modulus number of the soil (Duncan et al., 1980), Pa is the atmospheric pressure, and Sv is the vertical spacing of the reinforcements.

The reinforcement tensile stiffness modulus, Jr, is determined through the tensile strength per width unit of the geosynthetic (Tr), determined in wide-width strip tests (NBR 12824/93), and the maximum elongation at burst (Ɛr), as follows:

$$Jr = \frac{Tr}{\mathcal{E}r}$$

Figure 1.32 illustrates a simple model for the mechanism of stress mobilization in a mass of reinforced soil.Two curves are shown representing a soil (a) where the stress-strain curve shows no peak resistance and another (b) with a peak resistance.There are also curves of two reinforcements with different stiffness , $(Si)_1$ and $(Si)_2$ .Hypothetically , under zero horizontal strain , the soil would be in the state of stress corresponding to at rest conditions and there would be no tension load in the reinforcements. With increasing horizontal strain, the horizontal soil stresses ($\sigma_{s,x}$) decrease, approaching the active condition. Simultaneously, the stresses in the reinforcements increase until the equilibrium of the reinforced soil mass is satisfied. This equilibrium can be achieved with relatively small strains, when the reinforcement is stiffer (points A_2 and B_2). When the reinforcement has lower stiffness, the strains required to reach equilibrium are higher (point A1), soil failure may occur when the soil stress-strain curve shows a post-peak strain softening (point B_1).

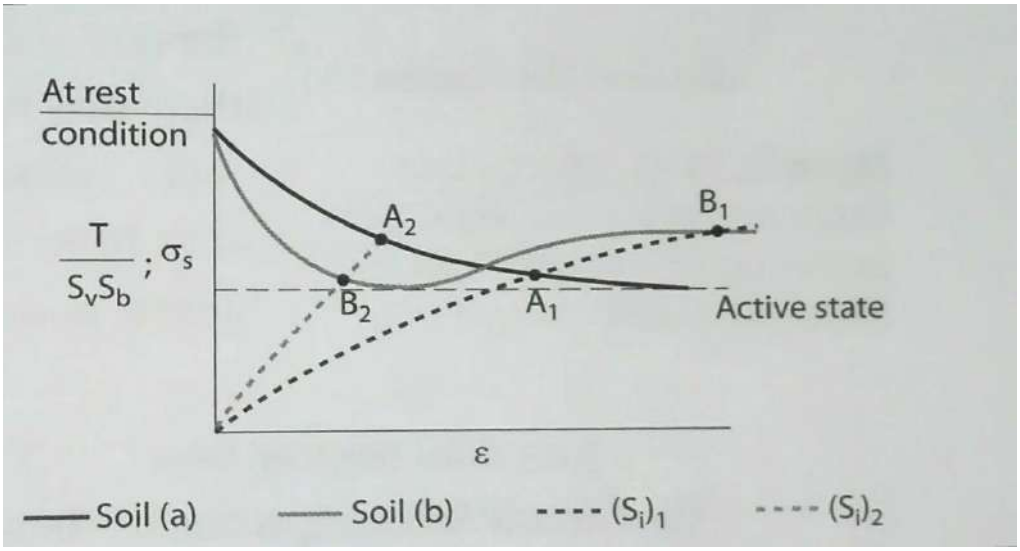


Figure 1.32: stress mobilisation in a mass of reinforced wall

Figure 1.33 shows a typical test result for a polyester geogrid, from which it is possible to determine a value for tangent stiffness modulus at failure equal to 1270 kN/m, undermining the initial curvature of the graph.

Table 1.5 shows S values for typical reinforcement conditions, considering soil with K = 450 kPa and vertical spacing of 0.5 meter.

Soils with different stiffness or other spacing may affect the value of Si .

It is worth mentioning that the larger the value of Si , the stiffer the reinforcement and , in general , higher tensions are mobilized by it and lower strains are suffered by the reinforced soil structure . Thus , structures reinforced with PVA geogrids , for example , tend to have higher tensile strengths and lower and lower strain compared to those observed in structures reinforced with nonwoven geotextiles . However , it should be noted that in the case of soil with post-peak strain softening , there may be a different situation . Higher stress mobilization may be required in reinforcements with lower stiffness to reach equilibrium (soil type b , point B1 , **Figure 7.8**).

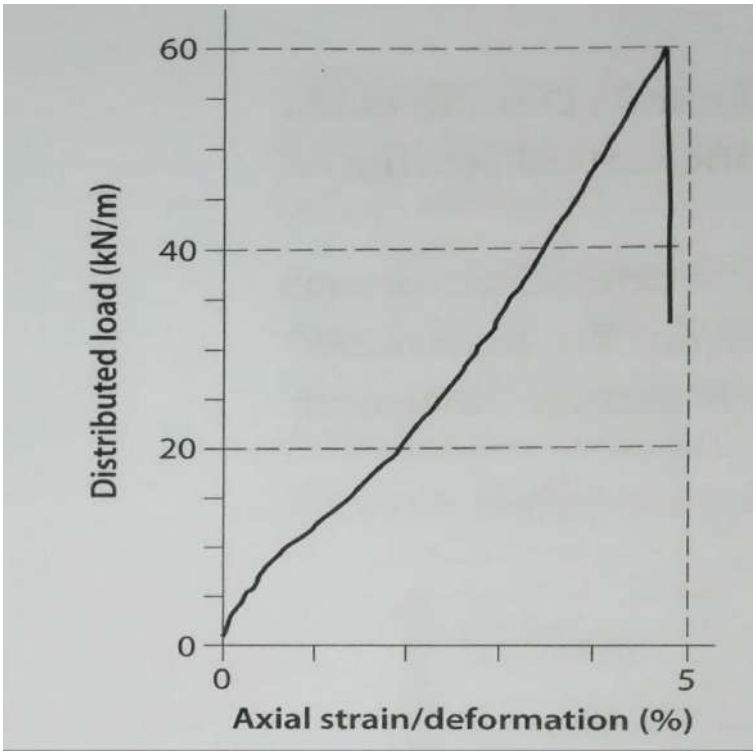


Figure 1.33: result of a wide band tensile test for a Fortrac 55/25-20/30 geogrid

Type of geosynthetic	S_i
PVA geogrids (1)	0.020 a 0.200
PET geogrids (1)	0.010 a 0.100
PP geogrids (1)	0.015 a 0.150
Woven geotextiles (1)	0.010 a 0.100
Nonwoven geotextiles in general (2)	0.001 a 0.003

(1): tensile strength between 20 and 200 kN/m
(2): tensile strength between 20 and 50 kN/m

table 1.6 : typical valus for the relative stiffness index

1.8.1.3 Soil compaction :

The effect of soil compaction on the behavior of reinforced masses must be considered , as compaction can significantly affect the internal stress of these structure .

Duncan and seed (1986) indicate that the compaction operation may be modelled by load and unload cycles that would induce high horizontal residual stresses in the soil . Depending on compaction , the horizontal residual stresses can be much greater than those from geostatic origin , which significantly increases the tensile mobilized load in the reinforcements. That does not mean that compaction in harmful, because of that the structure becomes less sensitive to surcharge loading applications , which induce lesser stress than those resulting from compaction . The final effect of the process can be understood as a kind of over-consolidation of the soil.

Chapter 2
Geological and geotechnical context of the project

2.1. Introduction

This chapter presents the geological and geotechnical study of the Project embankment of the highway section between the CHIFFA and BERROUAGHIA on 53KM (RemblaiArmé PK39+542).its purpose is to give the elements necessary for the analysis of the stability of an embankment.

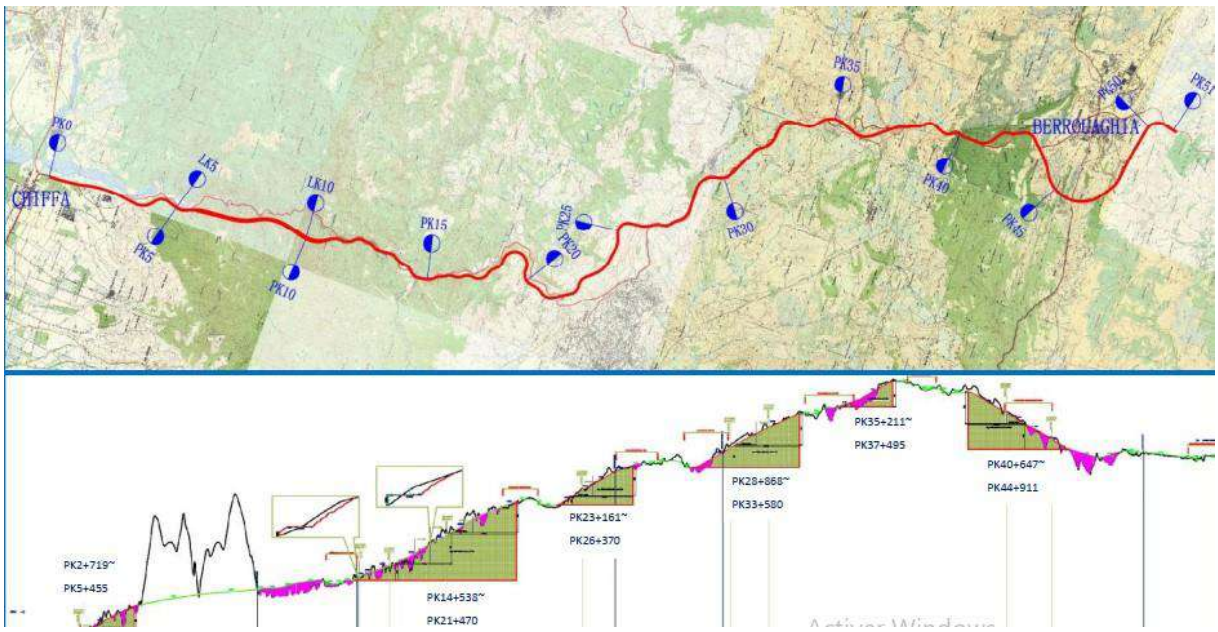


Figure 2.1 site presentation

2.2Site presentation:

The project for the development of the RN1 decoupling between CHIFFA and BERROUAGHIA over 53KM is located in the Wilayas of Medea and Blida, over a total length of 53km. The work passing through the Atlas Mountains and the Chiffa Gorge is recognized by its highest level of environmental protection requirements, the most complex geological conditions and the most significant difficulty in the implementation of the highway in Algeria. The project consists mainly of 10 km of tunnel and 16 km of works of art, as well as road works.



Figure 2.2 CHIFFA-BERROUGHIA road.

2.3.Topography and Geomorphology :

The terrain is hilly with hills and valleys, with developed ravines and a natural slope of 20° to 40°.The minimum altitude on the axis is 1174m, and a maximum altitude of 1217m, the road section is located in a geomorphological unit that is not mountainous (Figure 2.2).



figure 2.3 Shallow Mountain Landscape.

2.4.Lithological Training

There is the outcrop of the miocene of marne and sandstone, the direction and the dip is 5° ∠ 18°, the surface cover composed of quaternary clays.We have not found the break structure.

N⁰ couche	nom géotechnique	Symbole Lithologique	épaisseur (m)	Couleur	statut	Description
①	Remblai artificiel	TR	0.0-1.0	Bistre	—	Ce sont des remblais de plate-forme de RN01, les composants principales sont des graviers argileuses.
②	Argiles peu plastiques	Ap	3.1-6.5	Jaune -brun	très dur	bonne homogénéité
③	Marne	RM	non exposé	gris, bistre	mou	sous forme de colonne,molle, c'est facile de casser les roches à la main et ainsi l'apparition des fissures après séchage.
④	Grès fortement altérée	RG	non exposé	Jaune brunâtre	mou	altération forte, sous forme de caillou et sable,molle,fragile

Table 2.1 of lithology of major formations

The thickness of the platform embankment of RN01 is 2 to 3 m, the surface covering has a thickness of 2 to 3 m, it is mainly composed of the clay underlying the sandstone, the latter rests on a highly impaired swamp. The overall direction and hanging of the known forces are 5°∠18°.



Figure 2.4 .Completion of S39-04 on the embankment



figure2.5.S39-05 Marne



Figure 2.6 banc de Grès

2.5 Site type and seismic zone :

According to the 'PARASISMIC RULES APPLICABLE TO ART WORKS' (RPOA2008), the project area is classified as zone II a, the route of this study area corresponds to Group 2, Taking Seismic Acceleration A=0.20g.

2.6.Surface and underground water:

Through our investigation we found:

Surface water: in the study area, the ravines are developed, there's usually no surface water, but there's runoff during the rainy season, so it's easy to find temporary water collection on the low surfaces after the rain.

The soil on the surface has a low infiltration, a poor permeability, due to clay formation, the water inflates through the cracks, which leads to an alteration of the underlying marbling rock, and a decrease in its mechanical strength.

Groundwater: no surface water or natural water resources were discovered in all the depths reached in the reconnaissance wells.

2.7 Relief and geomorpholigia :

The area where the reinforced embankment is installed is the hilly geological configuration of hills and valleys, the relief is relatively hilly, the dimension of the axis of the reinforced embankment is 1172.5 -1181.5.On the RN01, PK39+558~PK39+578 D 10.0m, there are small slides locally. According to a geotechnical recognition, the body of the slides consists

N° Sondage	PK	Profondeur d'échantillonnage (m)	Symbole Lithologique	Pourcentage de masse tamisée (%) (mm)					
				20.0	2.0	0.2	0.08	0.02	0.002
S39-06	PK39+645D32m	1.20~1.40	Ap		100.0	94.1	75.9	33.8	19.5
		8.00~8.20	RM			100.0	98.7	98.6	42.3

essentially of an artificial embankment composed essentially of stones and bass, its thickness is 2.8m, overlying a very thin layer of 0.08m thick plastic clay, very soft, which represents a breaking surface between the embankment and the marl completely altered, saturated,plastic and soft.

2.8. Geotechnical Characteristics Analysis:

2.8.1 Granular Analysis:

The results of the granulometric test of the surface collection samples (according to NF P94-041/NF P94-056/ NF P94-057) are as follows:

2.8.2 Atterberg Limits :

The samples were taken to determine the Atterberg limits, see below the result:

N° Sondage	PK	Profondeur (m)	Symbole Lithologique	e_0	W (%)	Teneur en eau limite				GTR
						WL (%)	WP (%)	IP (%)	IC	
S39-06	PK39+645D32m	1.20~1.40	Ap	0.676	20.7	46.7	21.3	25.4	1.02	A3m
		8.00~8.20	RM	0.641	21.1	53.6	25.0	28.5	1.14	A3m

2.8.3. Mechanical tests:

In order to determine the mechanical characteristics of the soil foundation, soil samples were compacted (reference specification: NF P94-090-1/NF P94-071-1).

Compressibility tests

N° Sondage	PK	Profondeur (m)	Symbole Lithologique	ρd (g/cm^3)	Pc (KPa)	Cc	Cg
S39-06	PK39+645D32m	1.20~1.40	Ap	1.64	220	0.146	0.017

②very plastic clay Cc / (1 + e0) = 0.087mean compression

Shear tests

N° Sondage	PK	Profondeur (m)	Symbole Lithologique	Ccd (KPa)	Φcd (°)
S39-06	PK39+645D32m	1.20~1.40	Ap	71	18.4

2.8.4. Heavy dynamic penetration test :

The test consists in driving a string of rods with a metal tip into the ground by driving.

The dynamic penetrometer type «TYPE B»according to the reference: NF P94-115.

The diameter of the tip being greater than that of the rods, the resistance to penetration is due to the tip alone.

The test is interpreted using the following Dutch formula:

$$Qd = \left[\frac{M^2 g\mathrm{h}}{e(M+T)S}N\right]$$

With:

Qd=Peak Dynamic Resistance(bar)

M=Mass of sheep (63.5kg)

T=Stem train(6.28kg/m)

h=Drop height(0.50m)

e=Reference recess(0.20m)

N=Number of hits

S=Area of the right section of the tip(20×10-4m2)

g=Weight acceleration (9.81m/s2).

The platform project is located east of the city of BL. The results are represented in the form of a graph, the resistance to dynamic penetration Rd(kg/cm2) is shown on the abscissa and the depth in meters is ordered.

The results obtained for this test have made it possible to plot the dynamic resistance variation diagram as a function of depth.

The results of the registrations are annexed to this document.

The penetrator ranges obtained indicate a variation in resistance which increases with depth.

Rd = dynamic resistance

N°	Epaisseur	Désignation	Rd min（bars）	Rd max（bars）	Rd moy（bars）
①	0.0-1.0	Remblai artificiel	14	19	14
②	3.1-6.5	Argile peu plastique	20	62	38

2.9 Geotechnical soil characteristics :

According to the geotechnical results, a slope slip site has been found on the RN1, mainly in the platform embankments, taking into account the small volume and light thickness of landslides, there is no need for special geotechnical recognition, it is recommended to clear the landslides before execution.

The surface waters are not recorded from the site. During the exploration period we did not detect groundwater.

The proposed soil support and foundation design parameters listed in the following table:

N° couche	Type du sol	γ(g/cm^3)	C(kPa)	φ(°)	classe	
①	TR	2.20	5	25	I	déblai meuble
②	Ap	2.0	40	18	I	déblai meuble
③	RM	2.10	60	22	I	déblai meuble

Site seismic zone —zone II a, type of land S2

The physical and mechanical nature of the support soil is favorable, with good geotechnical condition, it is proposed to take the 3rd layer of marl as the seat.

In the case of observation of a variation in geological/geotechnical conditions during the execution of the work, a skilled technician must be involved to take the appropriate precautions.

2.10 Conclusion:

The analysis of geotechnical recognition results allows conclusions to be drawn

following:

- The clays we encounter over a depth of 8m are generally very plastic;

- Particle size analysis shows that it is fine soil;

- soil compressibility tests put us in the presence of very compressible soil and swelling;

- The consolidation pressure obtained shows that the soil is normally consolidated;

- Based on the values obtained from the pressure test (limit pressures), it has been found that the laying of the embankments will take place in the substratum (sands, clays). compact marl).

Chapter3

Stability and numerical study

3.1. Presentation of software « PLAXIS »:

The analysis and calculation of geotechnical structures is feasible through many finite element codes and programs. However, the transition from reality to the calculation model reveals some ambiguities and difficulties.

Designed by numerical geotechnicians, the finite element code « PLAXIS » represents an optimal version for 2 D geotechnical analysis on 02 planes: scientific and practical.

Scientifically, it is a non-linear analysis tool in elasto-plasticity, with consideration of interstitial pressures (and even linear consolidation), with robust, proven methods of resolution and algorithms, as well as procedures for automatic choice even before the operator is little informed. Although highly numerically reliable, the analysis code uses high precision elements (15-knot triangles), as well as recent resolution control processes (arc length method).

From a practical point of view, the on-screen menu system makes the use of the menu flexible and enjoyable, as the operator does not clutter his mind excessively. The set of default options (boundary condition) makes data easy and fast. Finally, the simplified options (stress initiation, pore pressures) make it possible to go straight to Aubut (predict the behavior of a work), even if it means carrying out a refined calculation later, with the same code and the same data.

3.2. 1 Default Options and Approached Solutions :

The system of default options and specific approximate solutions, which is one of the spearheads of the geotechnical project tool, is intended to save the operator time.

avoid having to make unnecessary choices, and finally improve the usability of the software. This system is inseparable from processing from a tree menu.Each branch of the menu is obviously fixed, because it performs a precise, well-defined task, but the diversity of the branches makes it an extremely flexible tool overall:

- The default options start from the mesh.
- In the same way as regards the boundary conditions in displacements.
- Pore pressures have been the subject of special care in PLAXIS.
- The conduct of nonlinear calculations is another example of flexibility of use

provided by this software.

- A computation by finite element provides an imposing mass of results.

3.2. 2 The behavior models used in PLAXIS :

The models of behavior of soils are very numerous: from the elastoplastic model of Mohr-Coulomb to the most sophisticated constitutive laws making it possible to describe almost all the aspects of the elasto-visco-plastic behavior, as well under monotonic stress as cyclic. These models were developed with the aim of being integrated into finite element calculations.

In this dispute, finite element modeling makes it possible to solve the problem at the limits by taking into account, by an appropriate law of behavior, the actual behavior of the soil. Two major difficulties prevented the complete realization of this scheme :on the one hand, behavioral laws that describe soil behavior are complex and demand-driven , for the determination of the parameters contained in them, heavy specific studies outside the framework of even complex engineering projects.The validation of behavioral laws was carried out , in the 80s of several workshops to compare the responses of different models on different solicitation paths.The second difficulty was the integration of these behavioral laws into finite, bi or three-dimensional codes. Few codes are currently operational, with sophisticated laws. The cost of these calculations is generally significant.

3.2.2.1 Linear elastic model :

The linear elastic model used in « PLAXIS » is conventional. The data entry tables require the shear modulus « G » and the Poisson coefficient «υ ». The advantage of « G » is that it is independent of the material drainage conditions (G_U = G), which is not the case for Young's modules: the undrained Young's modulus is greater than the drained Young's modulus.

The relationship between the Young's modulus « E » is the other modules are given by the equations:

$$K=E/ [3(1+v)] ; \quad (2.1)$$

$$G= [(1-v)E]/[1-2v(1+v)] ; \quad (2.2)$$

$$E = [(1-v) E]/[1-2v(1+v)] ; \quad (2.3)$$

The linear elastic model of PLAXIS can be used mainly to model concrete or metal structural elements interacting with the ground. It may also be interesting for some problems of rock mechanics.

3.2.2.2 Mohr-Coulomb model :

The Mohr-Coulomb behavior has a perfectly plastic elastic, non-clutter-free behavior.It has a great use in geoengineering given the results obtained in calculations. In Mohr's plan, the intrinsic line is represented by:

$$\tau=\sigma_n \tan\varphi + c\ ;$$

Where σ_n and **t** are the normal and tangential shear stresses respectively, and c and Θ are the material cohesiveness and friction angle respectively (Fig.II.1).

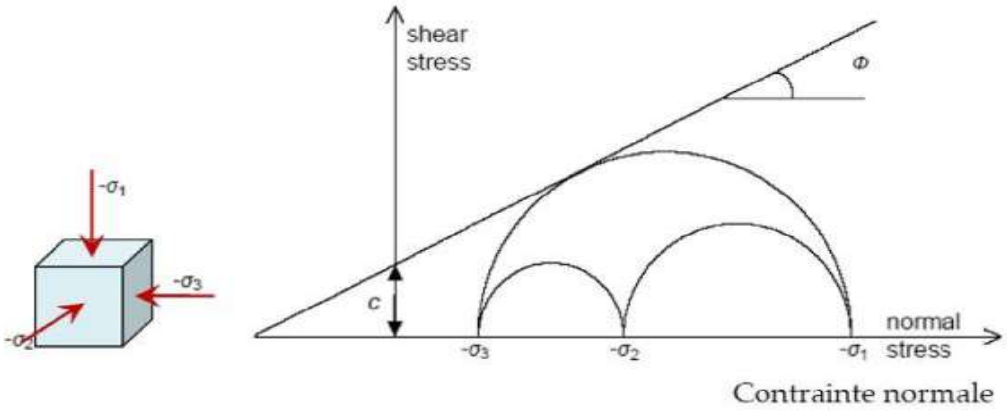


Figure3. 1: Mohr-Coulomb Model Intrinsic Curve

The three-dimensional Mohr-Coulomb criterion assumes that the intermediate stress does not occur.

The shape of the criterion is that of an irregular pyramid built around the trisector (Figure V.2) on the irregular Mohr-Coulomb hexagon.

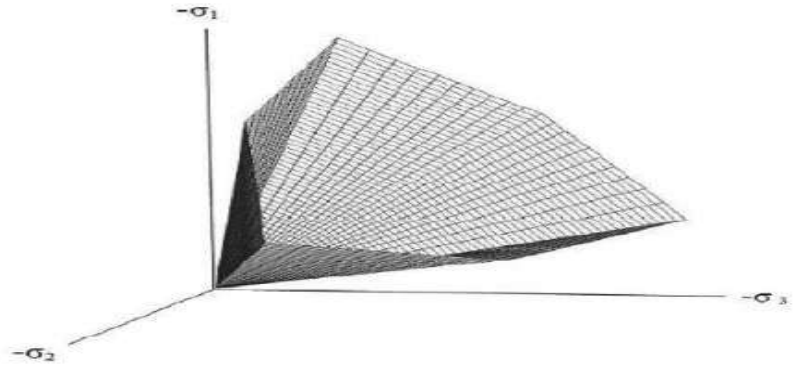


Figure3 .2: Mohr-Coulomb pyramid plotted for c=0.

The model requires the determination of five parameters called conventional geotechnical parameters:

Young's modulus (E), Poisson's coefficient ($_{\nu}$), cohesion (c), angle of friction ($_{\varphi}$) and angle of expansion (ψ).

These parameters are provided by laboratory tests, which are necessary for calculations for stability and geo-technical resistance.

(a) <u>Young's module :</u>

In the Mohr-Coulomb model, the module is constant. It seems unrealistic to consider a module tangent to the origin (which corresponded to the G_{max} measured in dynamic tests or very low deformations). This module requires special tests. It is advisable to take an average module, for example, that corresponding to a level of 50% of the rupture deflector (FIG. II.3).

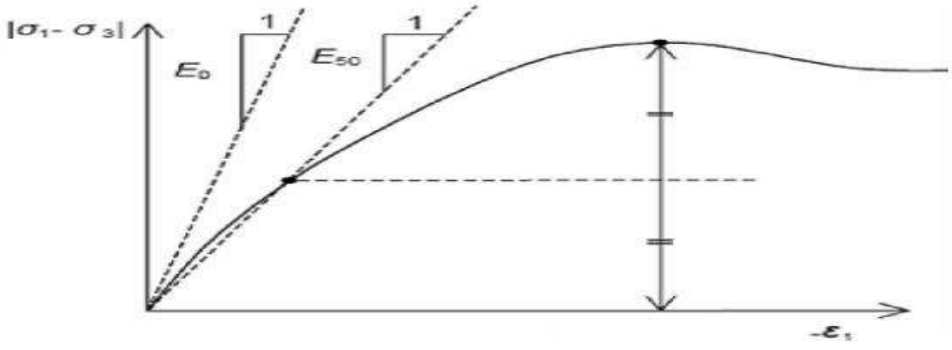


Figure 3. 3: Definition of modulus at 50% of failure.

The user must remain aware of the importance of the choice of module that he will take into account.

In the advanced parameters dialog, it is also possible to enter a gradient giving the variation of the module with the depth.

(b) Poisson Coefficient:

A value of 0.2 to 0.4 is recommended for the Poisson's ratio. This is realistic for the application of self-weight (k_0 procedure or gravitational loading). For certain problems, in particular in discharge, lower values may be used. For incompressible soils, the Poisson's ratio approaches 0.5 without this value being usable.

(c) Angle of friction:

« PLAXIS » does not take into account a variation of the angle of friction with the mean stress. The friction angle to be introduced is either the peak friction angle or the bearing friction angle.

Attention is drawn to the fact that angles of friction greater than 35° may considerably lengthen calculation times. It may be advisable to start calculations with reasonable friction angle values, even if this means increasing them later.

This value of 35 ° is compatible with the friction angles φ cv (at constant volume, at the level).the friction angle can be determined from the intrinsic curve of the Mohr-Coulomb model.

(d) Cohesion:

It may be useful to attribute, even to purely rubbing materials, a very low cohesion (0.2 to 1 kPa) for numerical questions.

For undrainedanalyzes with φ_u =0, PLAXIS offers the option to vary theundrained cohesion with depth: this corresponds to the linear growth of the cohesion as a function of the depth observed in profiles at the scissometer or at peak penetrometer resistance.

(e) Angle of expansion:

The last parameter is the angle of expansion noted; this is the least common parameter. However, it can be easily evaluated by the following (crude) rule:

$_{\psi}$=φ- 30° for φ> 30°.

$_{\psi}$ = 0° for φ < 30°.

The case $_{\psi}$<0° corresponds to very loose sands.

The ψ value $_{\psi}$= 0° corresponds to a perfectly plastic elastic material, or there is therefore no expansion when the material reaches plastic; This is often the case for clays or for sands of low or medium density under fairly strong constraints.

3.3. Stability and numerical Study :

A. Unreinforced soil:

3. 3.1. Initial Stability Study :

The purpose of this study is to determine a reference geotechnical model for our project. For this purpose, a modeling of the illustration example of this chapter will be carried out. In this example, it is proposed to study stability without comfort.

- From the profiles extracted along the zone of the slide, the most critical profile was chosen to check its stability. The lithological section of the profile treated with the calculation software is carried out on the basis of the cored bores and the penetrometric tests (PDL) carried out during the geotechnical study.
- The soil was considered in its short-term state (in total stresses); with a layer level on the surface.
- The geotechnical characteristics taken as initial parameters in the calculations are taken from the laboratory tests and are as follows:

	Name	γ	Ɵ	c	Δc
1	Embankment	20.00	35.00	5.00	0.00
2	Bêche	22.00	38.00	2.00	0.00
3	RM	21.00	22.00	60.00	0.00
4	AP	20.00	18.00	40.00	0.00

Table 3. 1: Geotechnical characteristics of materials.

Such as :

AP : Low plastic clays **RM :** Marne

3 .3.2. Site type and seismic zone :

According to « PARASISMIC RULES APPLICABLE TO THE FIELD OF WORKS OF ART»(OAPR2008), the project area is classified in zone IIa, the road of this study area corresponds to Group 2, Taking the seismic acceleration A=0.20g.

3.3.3.Surface and groundwater :

Surface waters are not recognized in the project area during recognition.No groundwater was found at the survey depth .

3.3.4. Calculation :



Figure 3.4: First general setting window

Once we introduce these data we begin the modeling of lithological sections, separate the layers. All of this is done through the useful(geometry line). Well, we set the limit conditions by the useful (standard fixities).

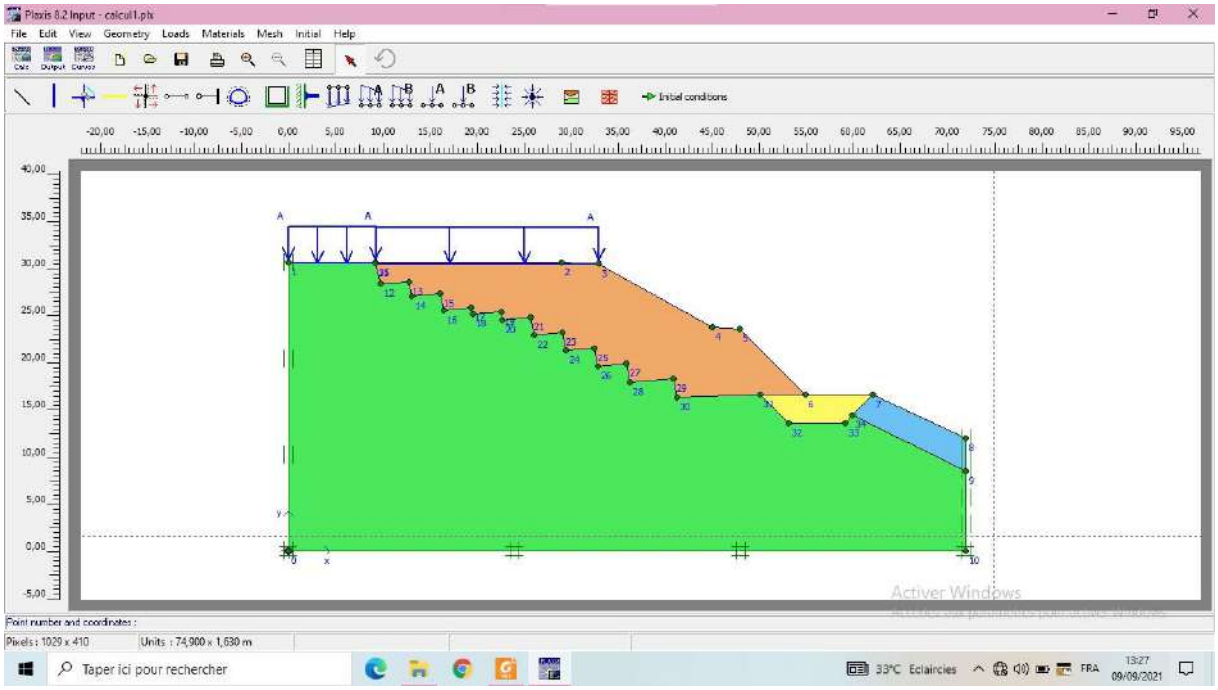


Figure 3.5: Profile representation.

The next step is to define each layer of soil by the useful material sets. You can choose the type from a library on software that you can modify the characteristics. Knowing that there was already a series of **AP** in the upstream side of the slope as mentioned in **the figure. 3.9** these characteristics are as shown in the following figure:

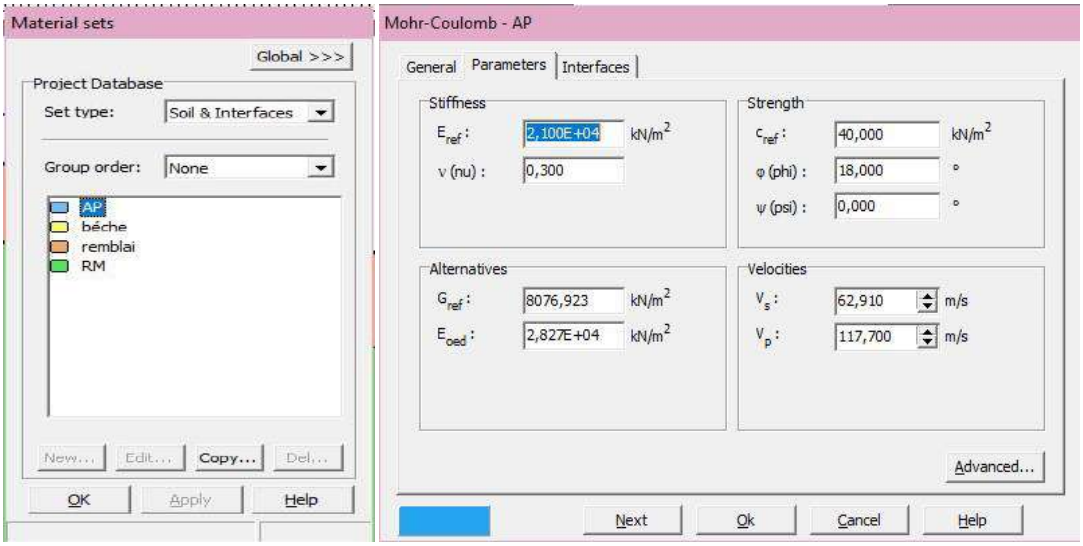


Figure 3.6: window for setting the material characteristics

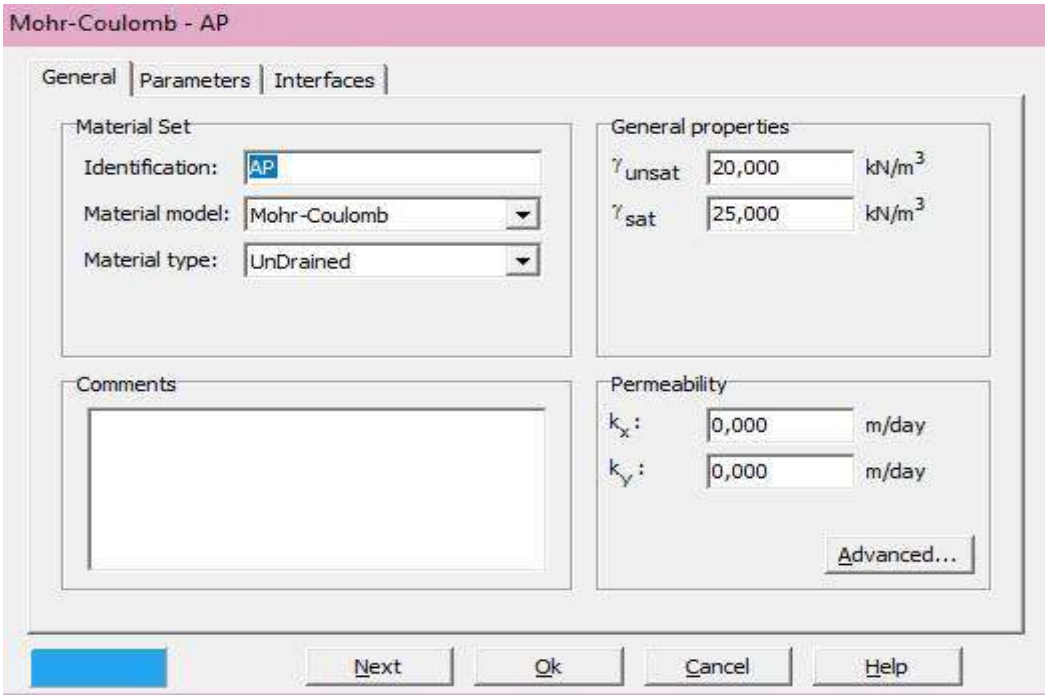


Figure 3.7: material characteristics window

In this case we have Artificial Trim, Clays, Marne and heavily weathered sandstone.

The traffic load value is returned as follows:

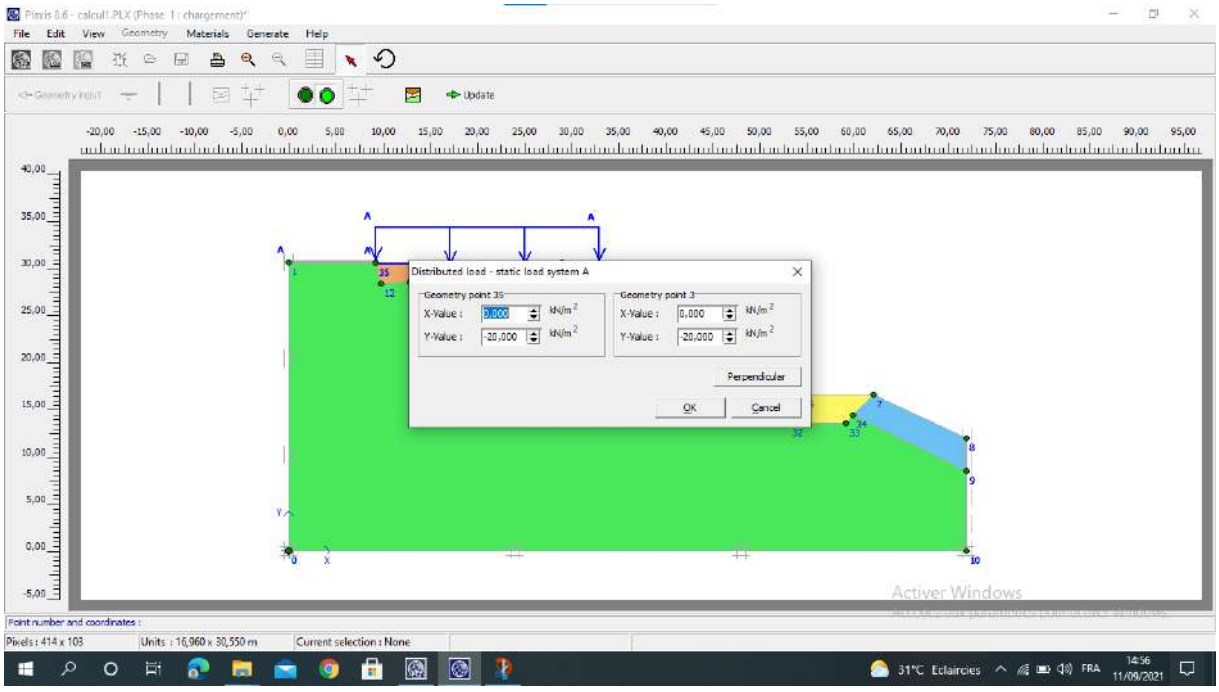


Figure 3.8: Loadwindow.

The mesh generation is set to the very fine element distribution and this gives us the mesh shown in the following figure:

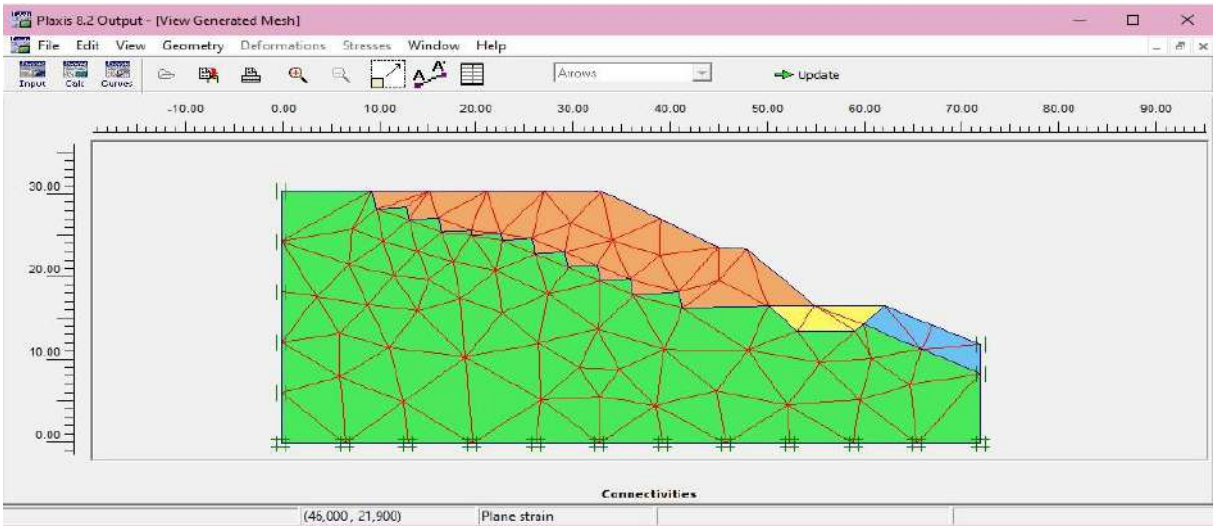


Figure 3.9: Profile mesh.

The mesh is satisfactory, we click on « update » for the done.And with its going to the initial condition to determine the water table and generate the water pressure.

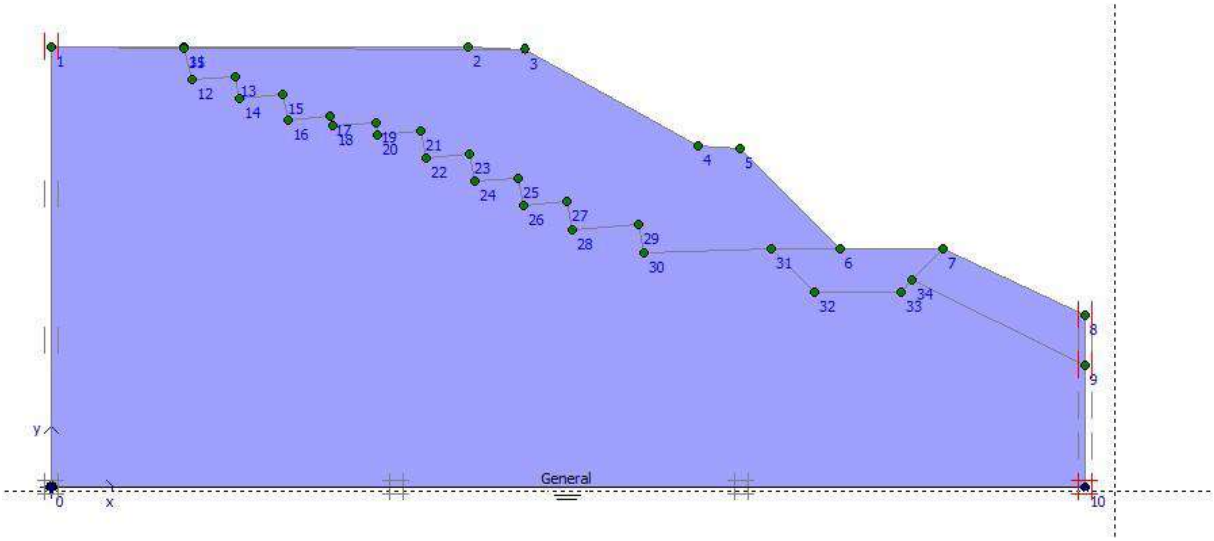


Figure3.10: profile water table determination.

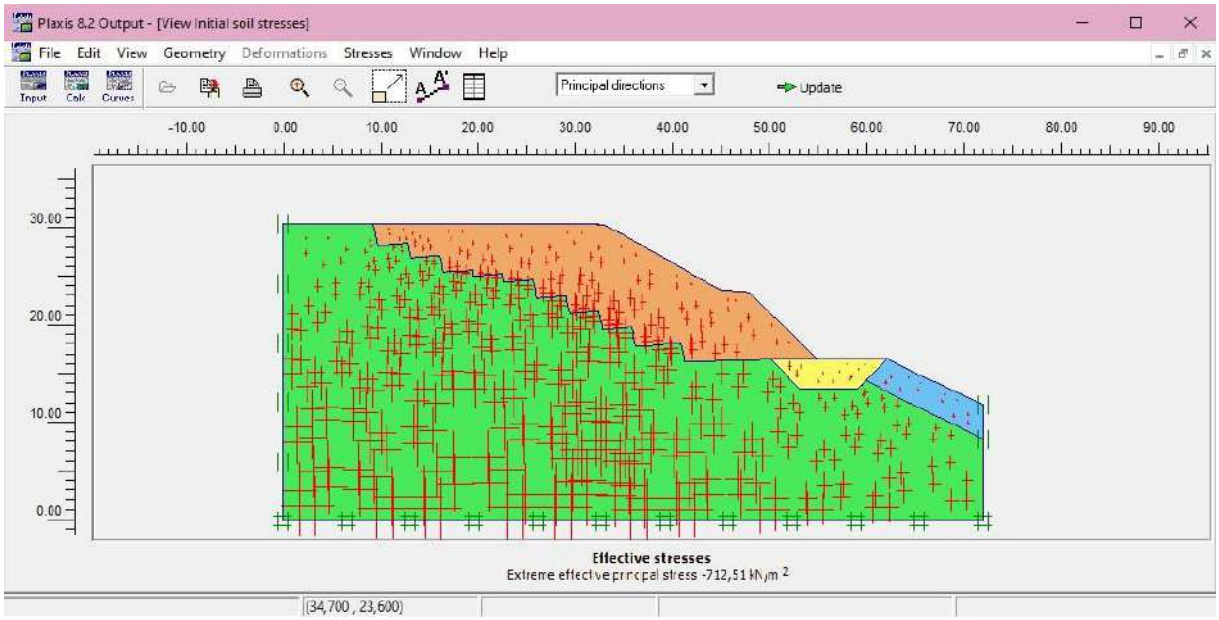


Figure 3.11: determination of effective constraints.

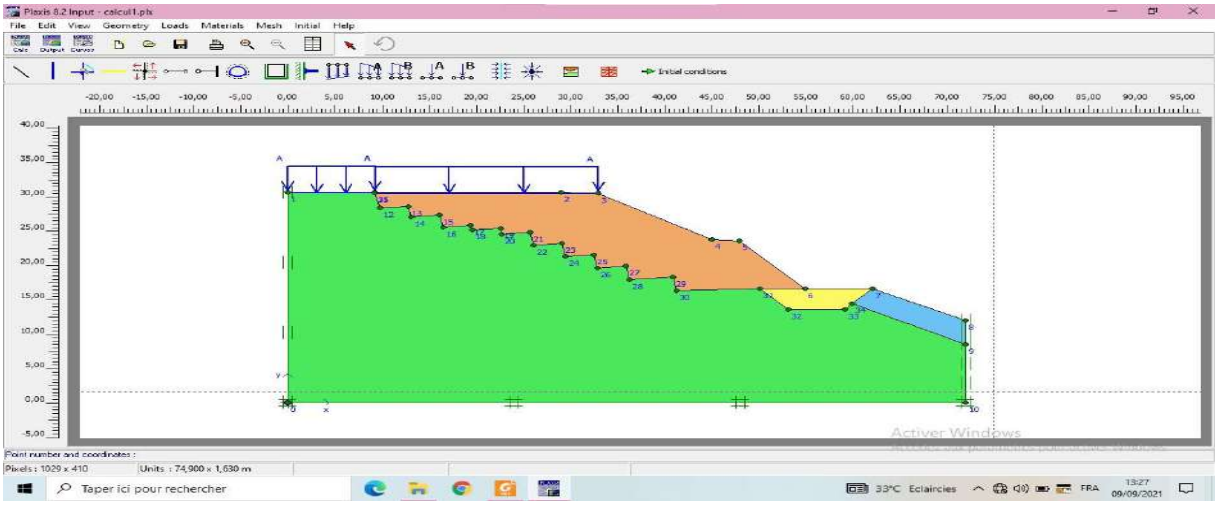
Moving on to the calculation step, two phases are used more than the initial phase :

The 1st phase is the charging phase and the second is a step of calculating the safety coefficient which are illustrated in the figure .

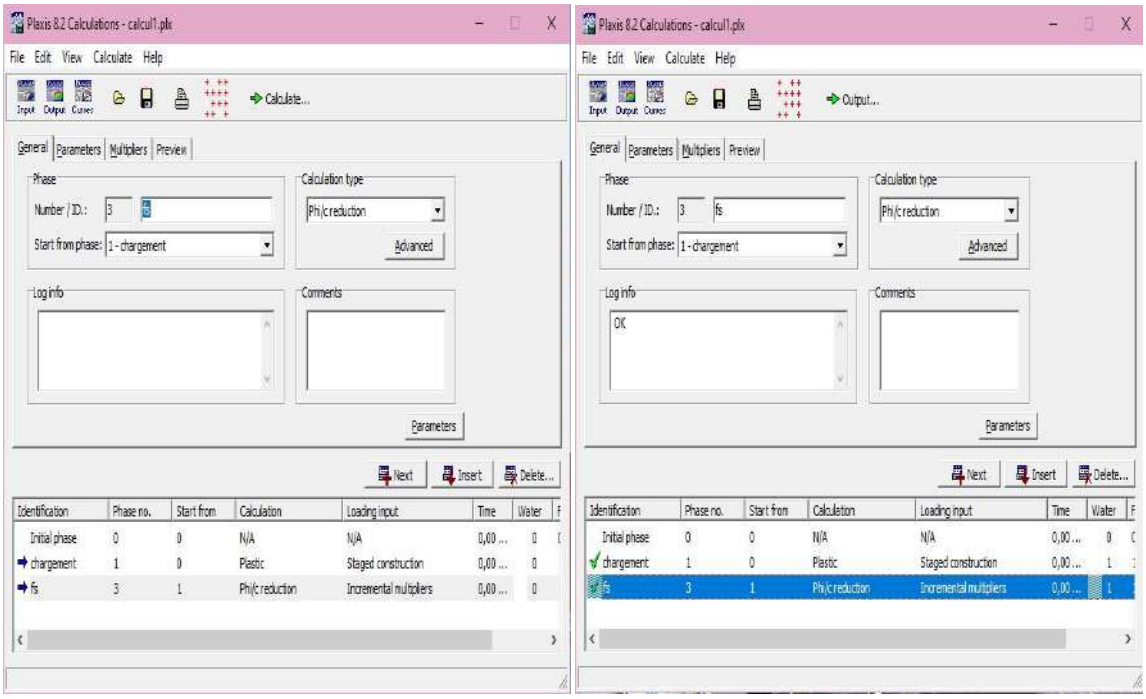
- theintial phase profiles it and under the influnce of its weight only.

- the loading phase, we add the road load .

- the FS phase is for the purpose of calculating the safety factor .



(a)



(b) **(c)**

Figure 3.12: Preparation phases for the profile. (a) the geometry of the loading of phase 1 (b) phase 2: the calculation of Fs (b) the calculation procedures.

A profile safety factor calculation as it is without any comfort gives the following results:

- A safety factor of 1.08 which means that the slope is not stable.

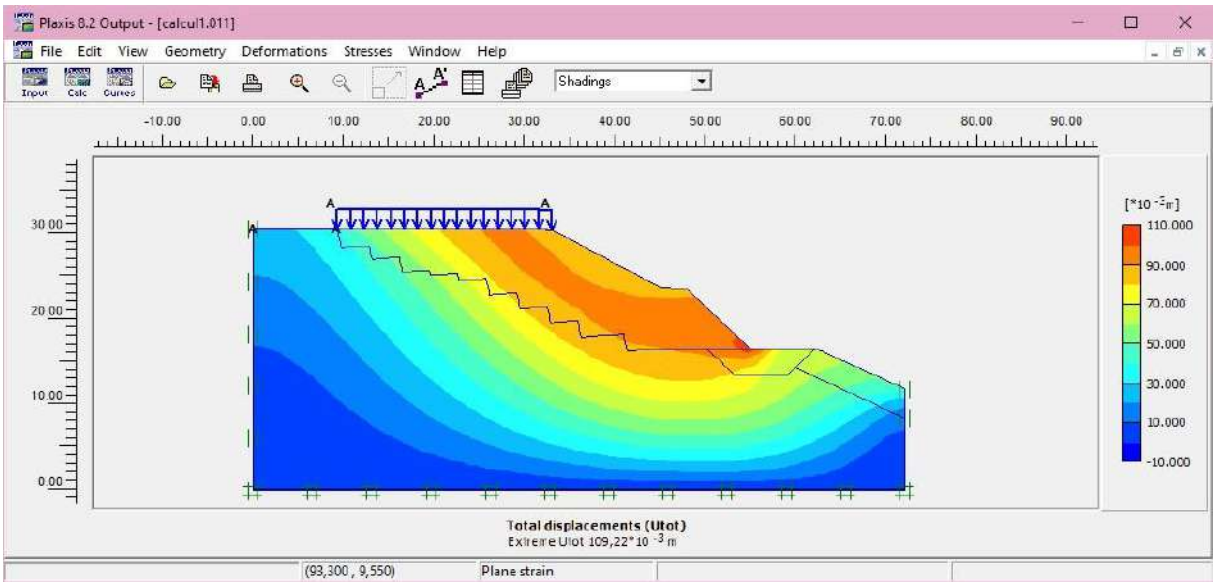


Figure 3.13: Break line.

According to the figure (III.11), the slippage circle is located under the body of the road.

The following figure shows the result of the safety factor without comfort:

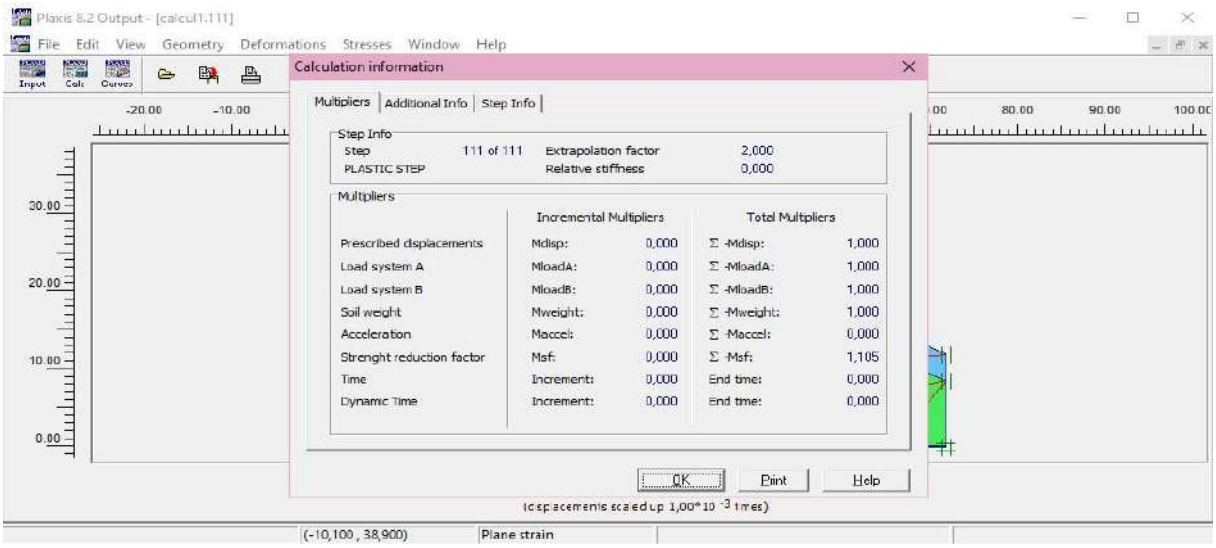


Figure 3.14: MSF safety coefficient.

- From the results obtained by the Plaxis software, it is found that the value of the safety coefficients is around 1, so the slope is in unstable equilibrium.
- The increase in the level of the ply above the breaking surface, made increase the density (driving force), and at the same time decrease the shear strength by increasing pore pressures, resulting in a decrease in the safety coefficient.
- On the basis of these results, it can be said that the presence of the water table has a direct effect on the stability of the ground (reduction of the safety factor).

Based on what has been developed in this chapter, the different limit equilibrium methods with their different assumptions have been presented. All these methods lead us to the calculation of the safety coefficient which is the most important parameter for the evaluation of the stability of the slopes, using the finite element method based on the Plaxis 2D software, it has been found that the case studied of the chosen route is unstable, therefore it is necessary to carry out comfort methods to stabilize it.

B. Reinforced soil :

This study is concerned with the study of the stability of an embankment on Compressible soil treated with ballasted columns surrounded by geosynthetics by applying: the three-dimensional numerical method with ballasted columns surrounded by geosynthetic (3D) sheets.

The finite element method is a mathematical method that allows the resolution of differential equations. It was developed in the 1960s and allows the global behavior of a complex structure to be described from simple and parameterized functions defined for each zone of the model.

A modeling was performed using the PLAXIS software based on the finite element method and used as a tool to analyze the behavior of a high-rise embankment on compressible soil PK. 39+542.

In a first part of this analysis, an evaluation study of the safety factor of a high-rise embankment reinforced by geotextile with a view to the effect of geotextile on the reduction of settlement.

The second part of thisanalysisisdevoted to the study of the stability of ageotextile-reinforcedhighfill.

All simulation results in this study are presented in graphical tables and curves and are widely discussed.

3.3.5. Numerical analysis :

The behavior of backfill and soft clay is governed by an elastic-perfectly plastic law and the Mohr-Coulomb criterion with the associated flow rule was adopted. This criterion is characterized by the density γ, the Young modulus E, Poisson's ratio ν, the cohesion c, the internal friction angle of soils φ. Figure 4.3 represents the model proposed for the numerical modeling for this study.

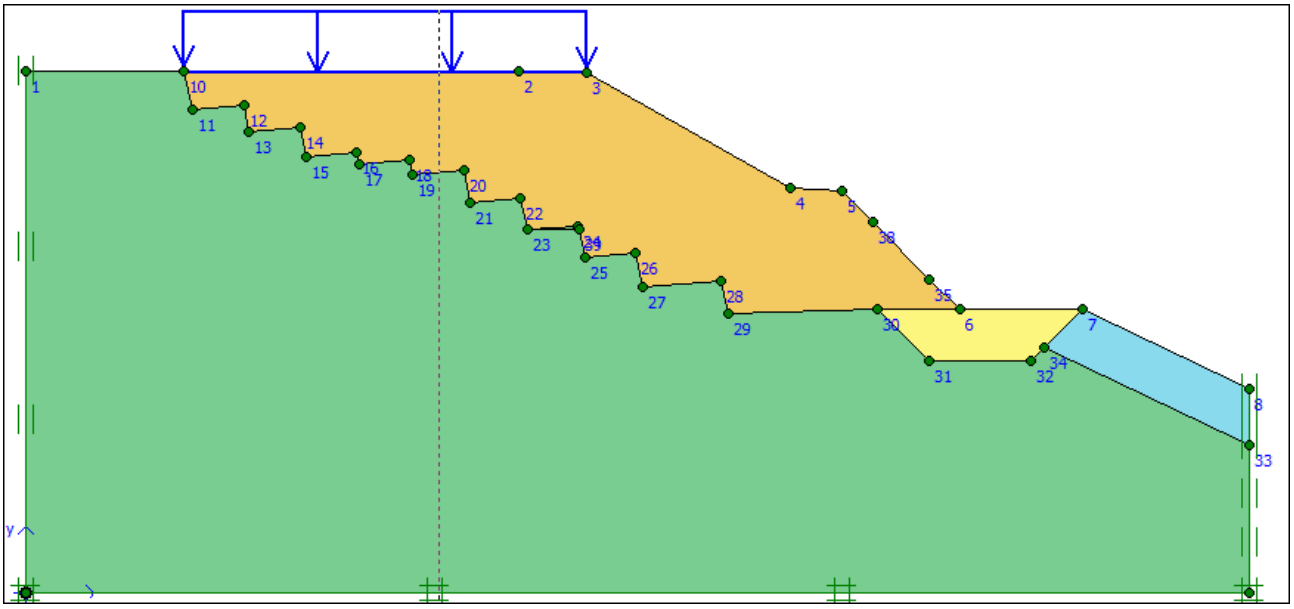


Figure3.15: geometry model.

The characteristics of the backfill and support soil are presented in the following table:

Characteristic	γ (KN/m²)	E (KN/m²)	ν	C(KN/m²)	φ (°)
Backfill	29	21000	0.35	5	35
Grés frottement altérée	21	20000	0.35	50	22
Marne	21	21000	0.35	60	22
Clay Low plastic	20	19000	0.35	40	18

Table 3.2: characteristics of the backfill and supporting soil.

The geotextiles use its non-wovengeotextiles of the type ALVEOTER® AFITEX.

Young's module (KN)	Tensilestrength(N)	Section (m²)	EA (KN/m)
$2 \text{ X } 10^{6}$	5000	$2 \text{ X } 10^{-4}$	400

Tableau 3.3: Propriété du géotextile

3.3.6. Mesh size:

When the geometry model is fully defined and the material properties are assigned to all clusters and structural objects, the geometry must be divided into finite elements to perform finite element calculations. A finite element composition is called a mesh. The basic type of the element in a mesh is the 15-node triangle element or the 6-node triangle element, the element chosen in this study is the 15-node triangle which provides fourth order interpolation for displacements and numerical integration that is done on twelve Gaussian points (stress points).PLAXIS enables fully automatic finite element mesh generation. The generation of the mesh is based on a robust triangulation procedure, which results in «unstructured» meshes. Ces mailles peuvent sembler désordonnées, mais la performance numérique de ces mailles est habituellement meilleure que pour les maillages réguliers (structurés).Le maillage de modèle étudié est donne sur la **figure 4.3**

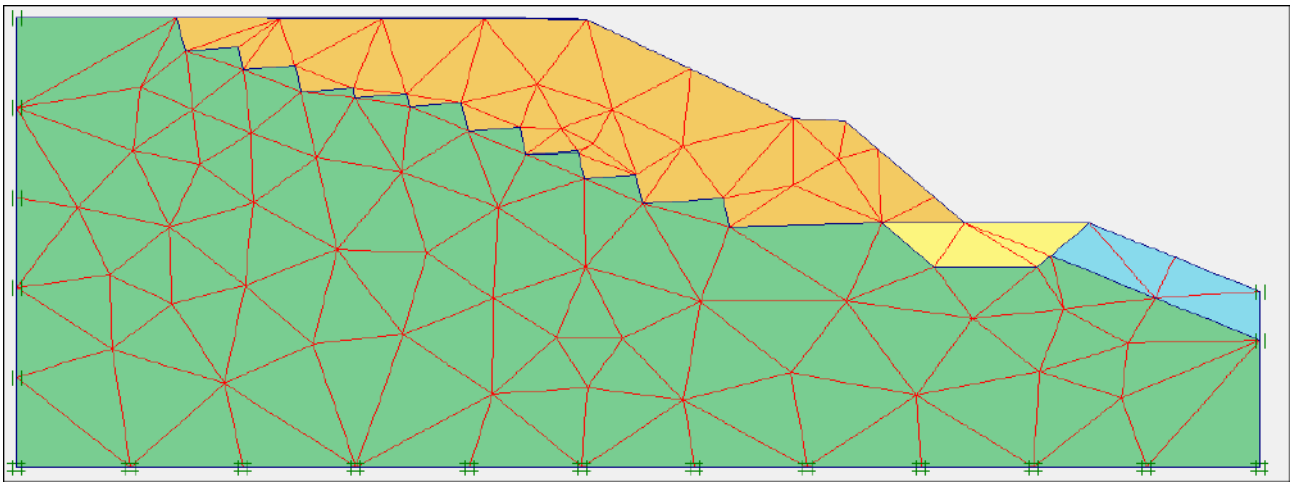


Figure 3.16: Generating mesh.

3.3.7. Initial conditions:

Initial conditions require the generation of initial stresses, for initial stress calculations the structural elements and the soil element created by default at the pavement level must be deactivated, the initial stresses are generated by taking the K0 values by default, the K0 value is automatically proposed according to the Jaky formula, the soil weight is kept at 1, corresponding to a total application of gravity.

3.3.8. Results and disincentives:

In order to study the evolution of the stress of a geotextile reinforced embankment, a series of numerical analyzes was carried out only by a vertical stress equal to 20kpa.

The various results obtained are presented in the following figures:

- **one sheet**

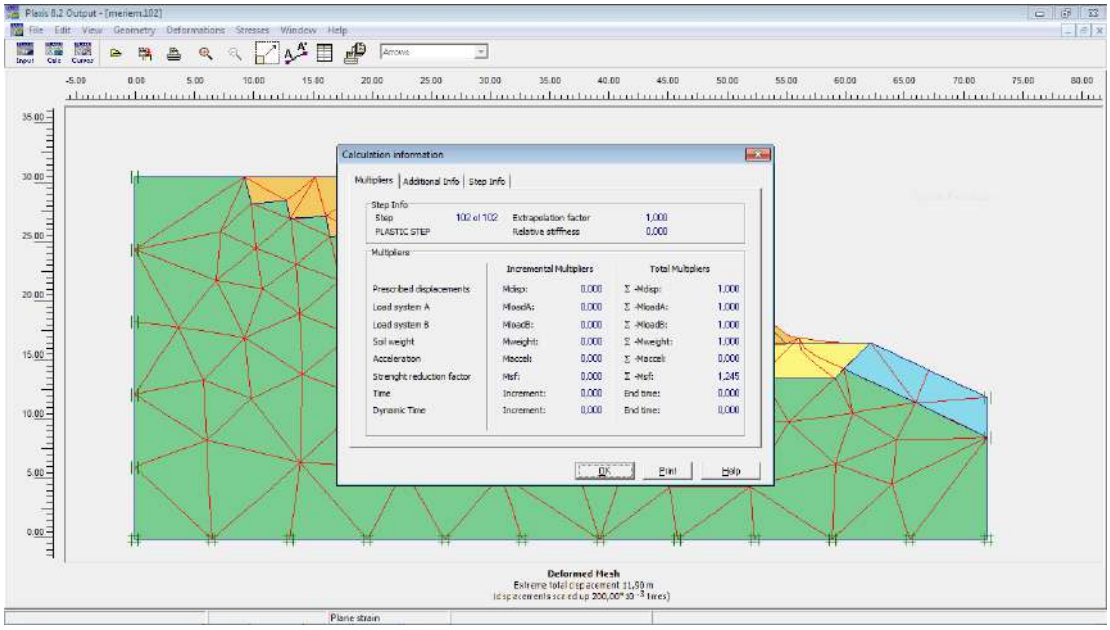


Figure 3.17: Fs with reinforcement (sheet).

- **Twosheet**

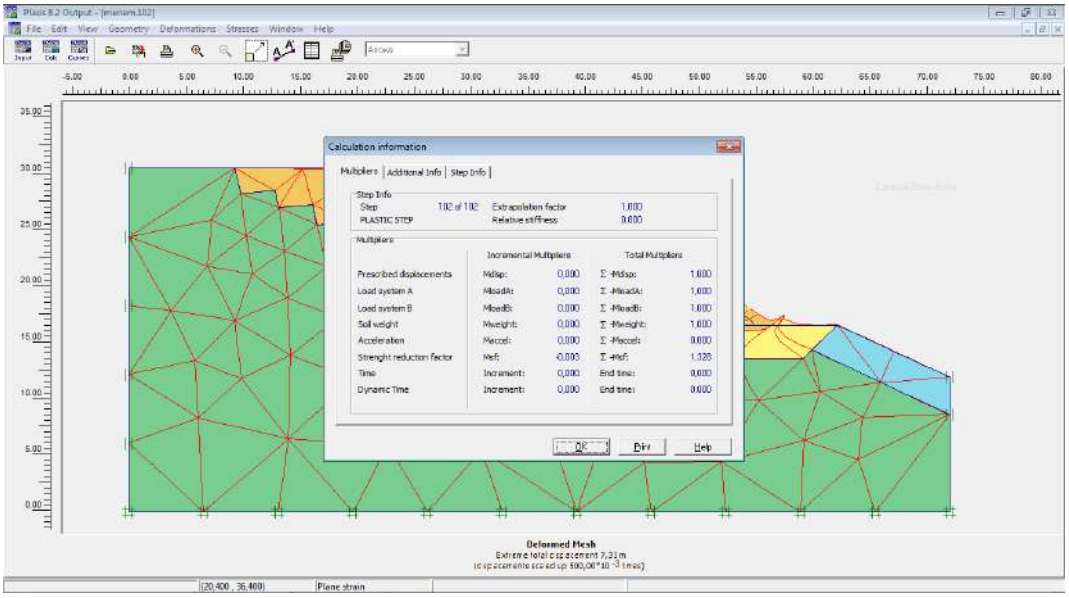


Figure 3.18: Fs with reinforcement (two sheets).

- **Three sheets :**

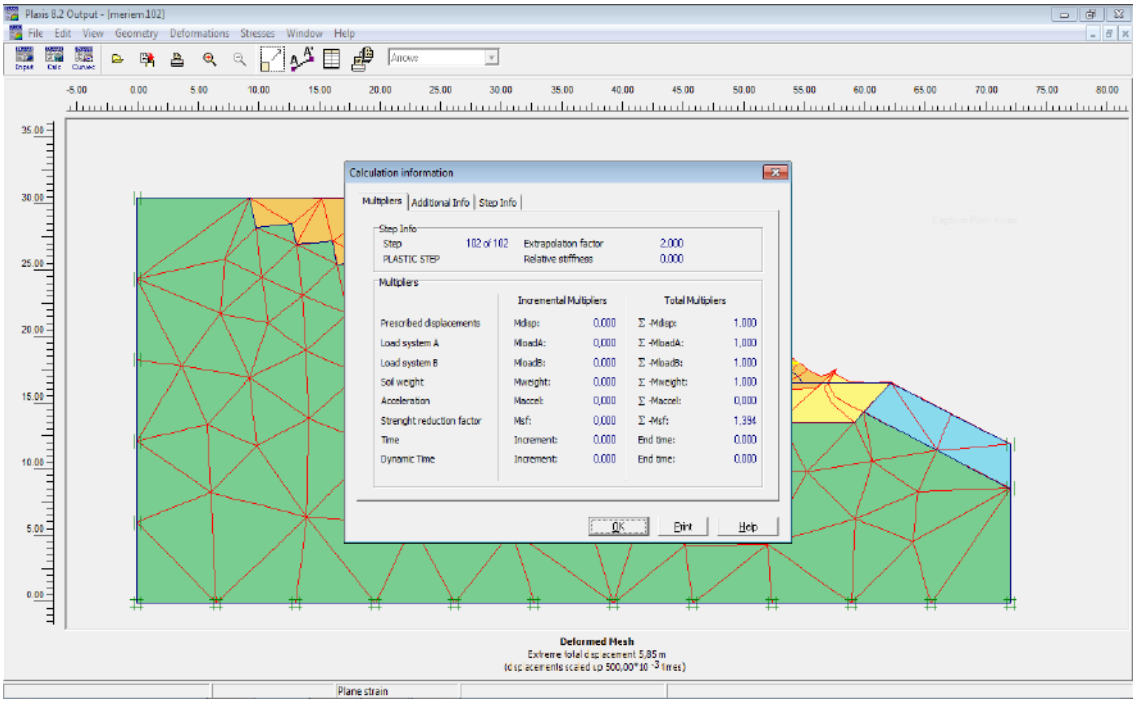


Figure 3.19: Fs with reinforcement (three sheets).

- **Four sheets**

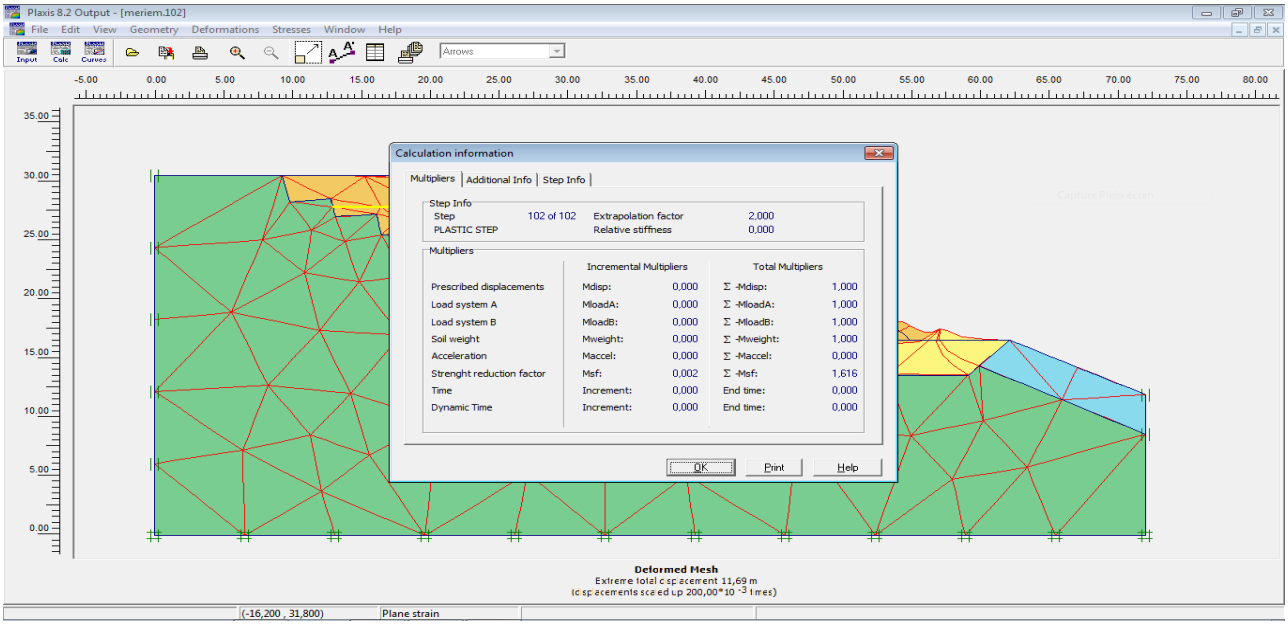


Figure 3.20: Fs with reinforcement (four sheets).

Table 4.3 and Figure 4.8 show the variation of the Fs as a function o f the number of geotextile basins

Number of sheets	Fs value
Withoutreinforcement	1.106
sheets	1.245
Twosheets	1.328
Threesheets	1.394
Four sheets	1.616

.

Table 3.4: Displacement results (without and with reinforcement).

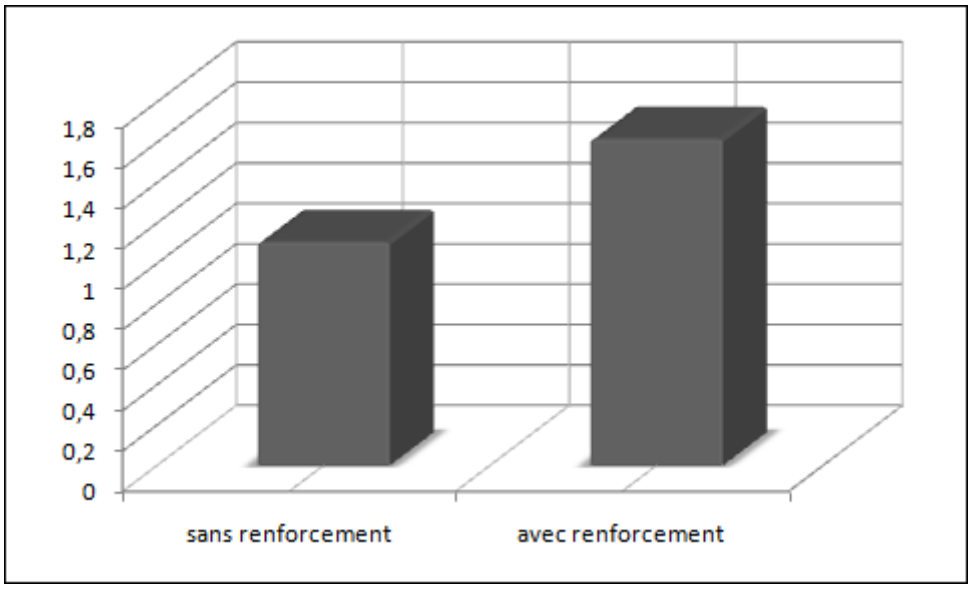


Figure 3.21: Total Fs (without and with reinforcement).

After the results obtained, it can be seen that the safety coefficient is less than 1.5, therefore the embankment is unstable. Hence the need to improve the soil to ensure the stability of the embankment.

It is also found that the geotextile recess produces an increase in safety coefficient, three geotextile layers giving a safety factor strictly greater than 1.5(1.616).

3.4. Parametric study:

In this section we will show the effect of the parameters on the evolution of the Fs of a reinforced embankment.The same embankment is used, with the same data and the parameter to be compared with the initial model is changed each time

3.4.1. Effect of the friction angle:

The safety factor is directly related to the characteristics of the embankment. It is proposed here to vary the angle of friction of the embankment (35KN/m²,45KN/m², 60kN/m², 80K/m²) to study their influence on stability.

The different srésults obtened Fs are présented in thefollowing figures :

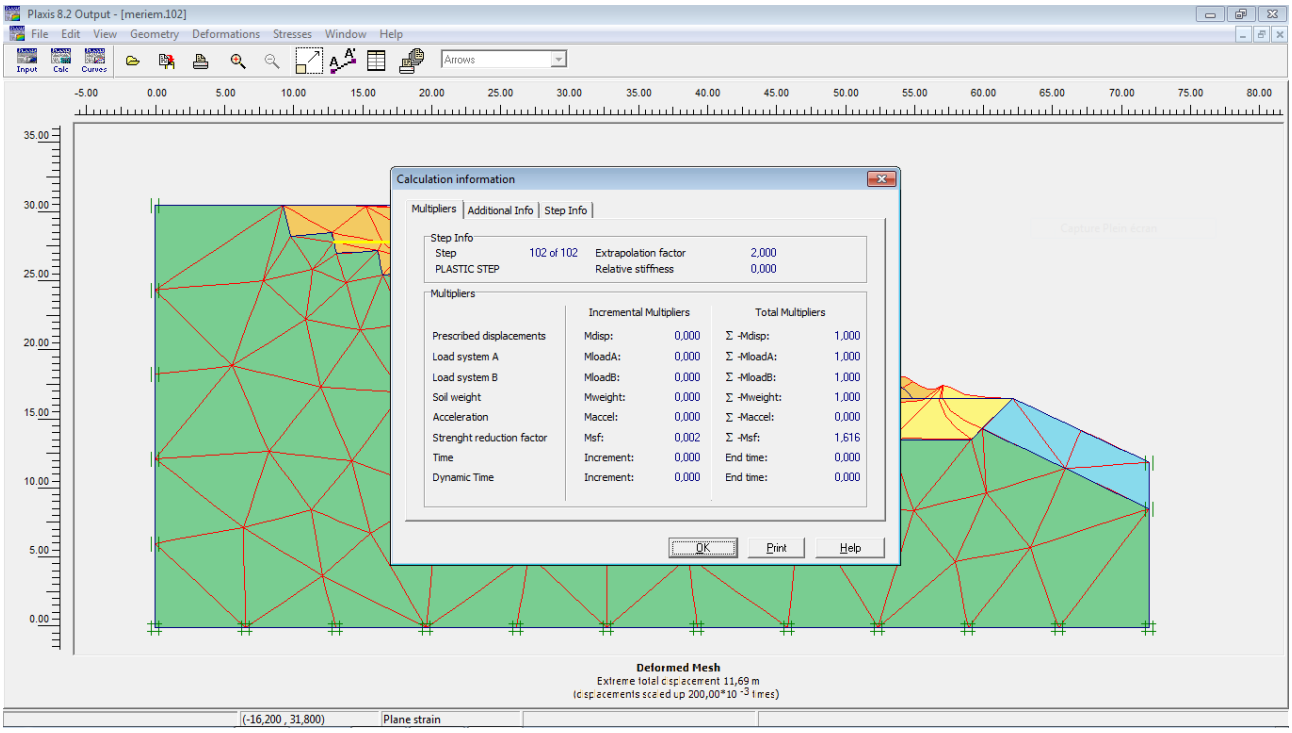


Figure 3.22 : Total displacement (settlement)(Q=35 KN/m^2).

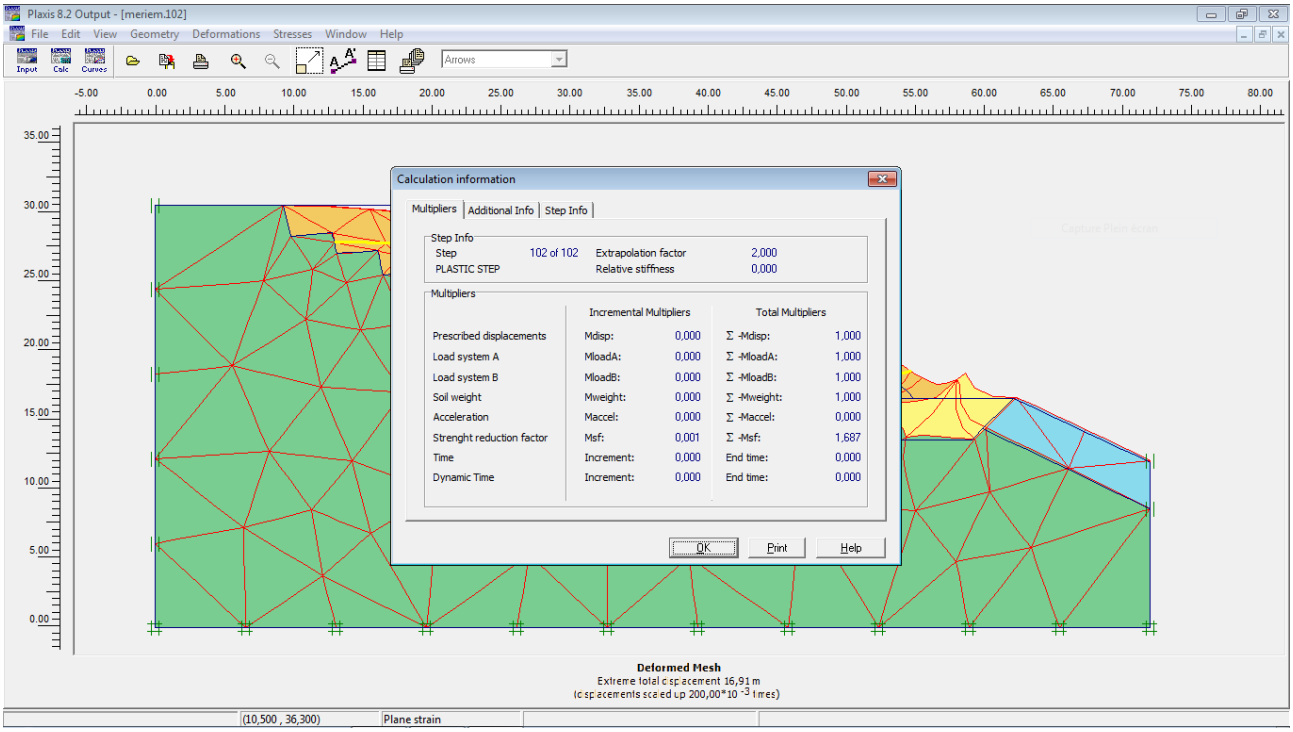


Figure 3.23 : Total displacement (settlement)(Q=45 KN/m^2).

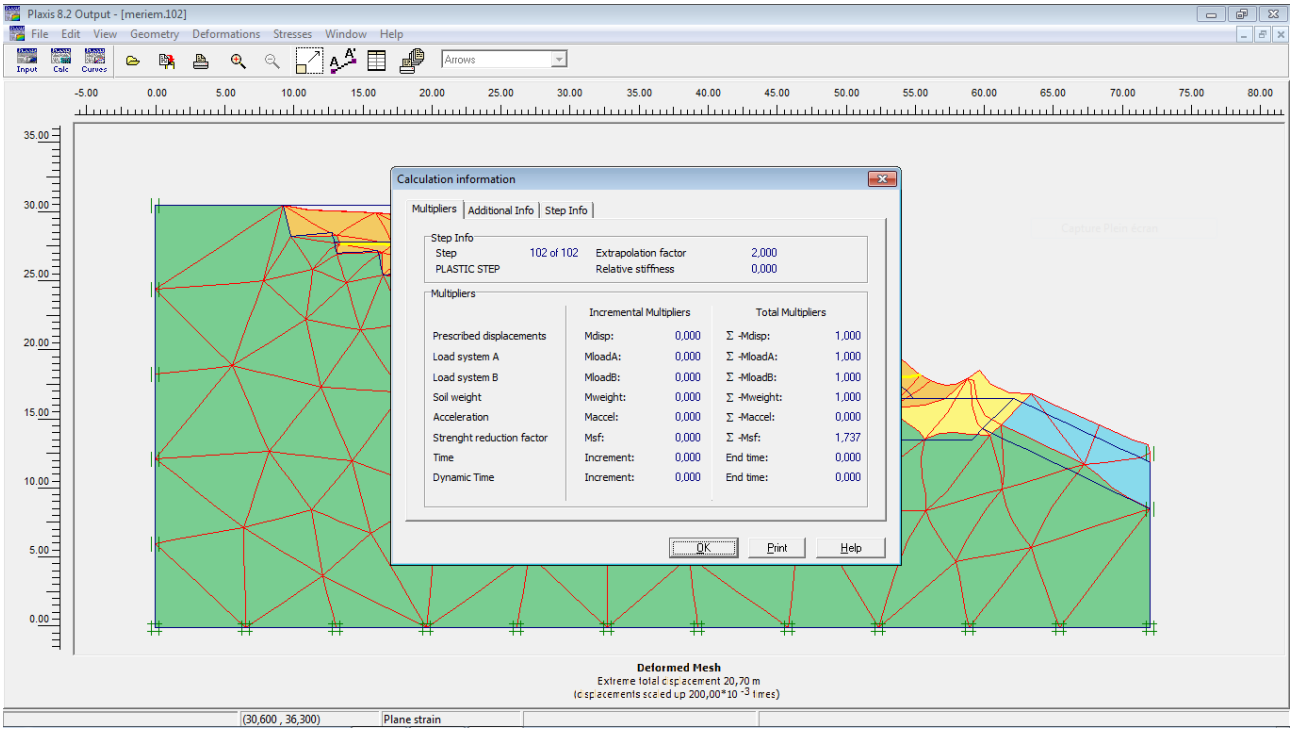


Figure 3.24: Total displacement (settlement)(Q=60 KN/m^2).

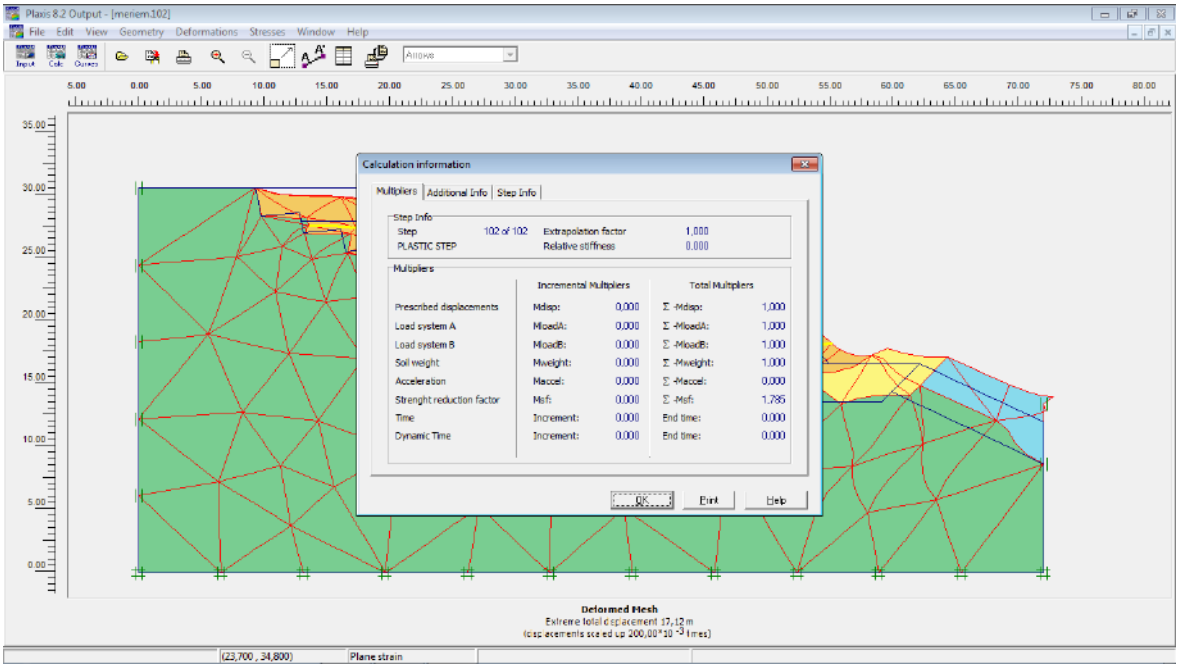


Figure 3.25: Safety coefficient (Fs) (Q=80KN/m^2).

Table 4.5 and Figure 4.14 show the evolution of the pressure for the various loads applied.

Friction angle (KN/m^2)	Fs
35	1.616
45	1.687
60	1.737
80	1.785

Table 3.5: Fs according to the friction angle.

According to the table, there is a complementary relationship between the friction angle and the safety coefficient, i.e. the increase in the friction angle produces an increase of Fs.

For a friction angle of 35 kN/m² m the Fs is 1,616, and for a friction angle of 80 kN/m² the settlement is 1,785.

3.4.2. Effect of cohesion variation:

In order to see the cohesion effect on the variation of Fs, a series of numerical analyzes has been carried out, and each time the cohesion value is increasingly varied.

The various results obtained from deformed mesh are presented in the following figures:

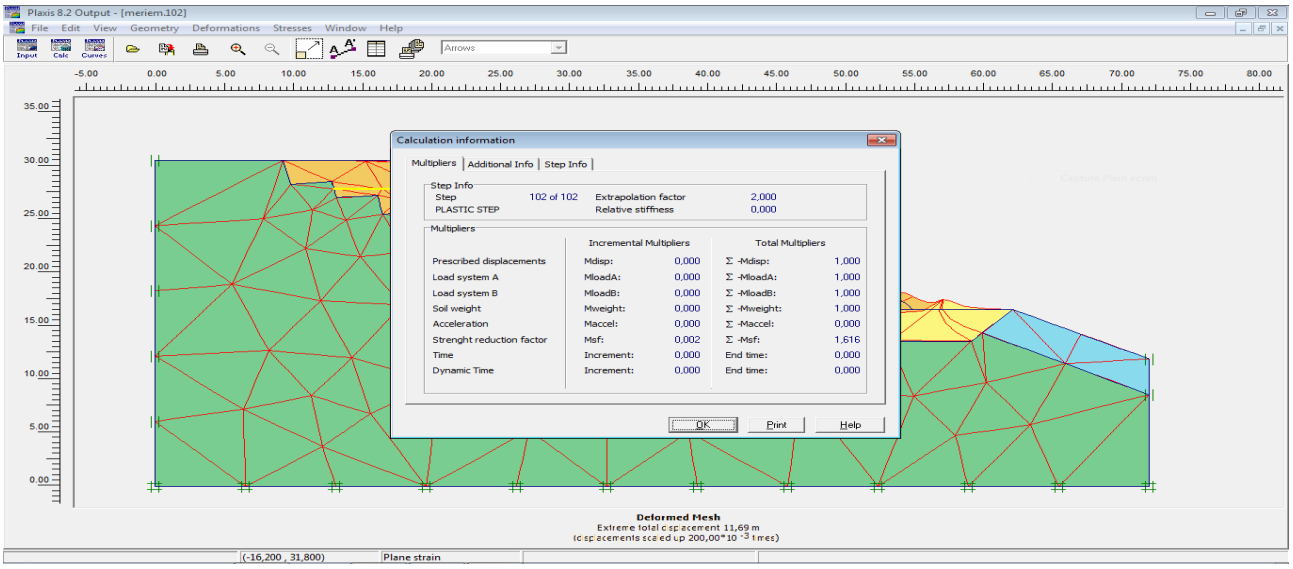


Figure 3.26: safety coefficient (Fs) (C=5 KN/m^2).

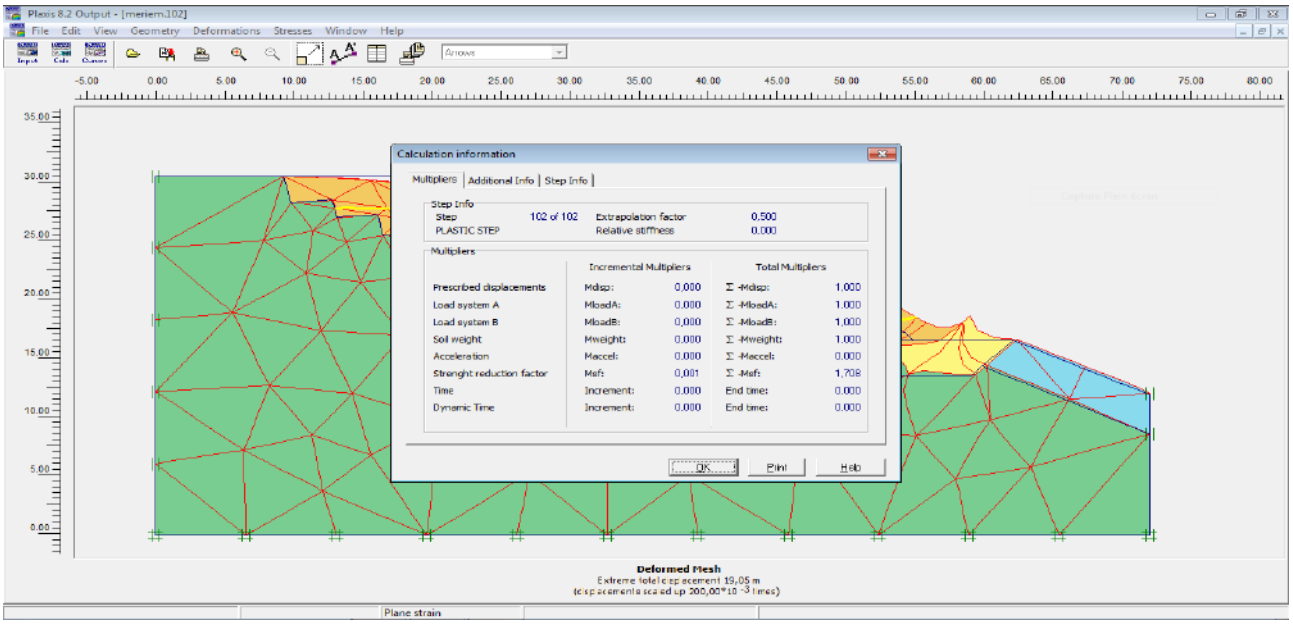


Figure 3.27: safety coefficient (Fs) (C=15 KN/m^2).

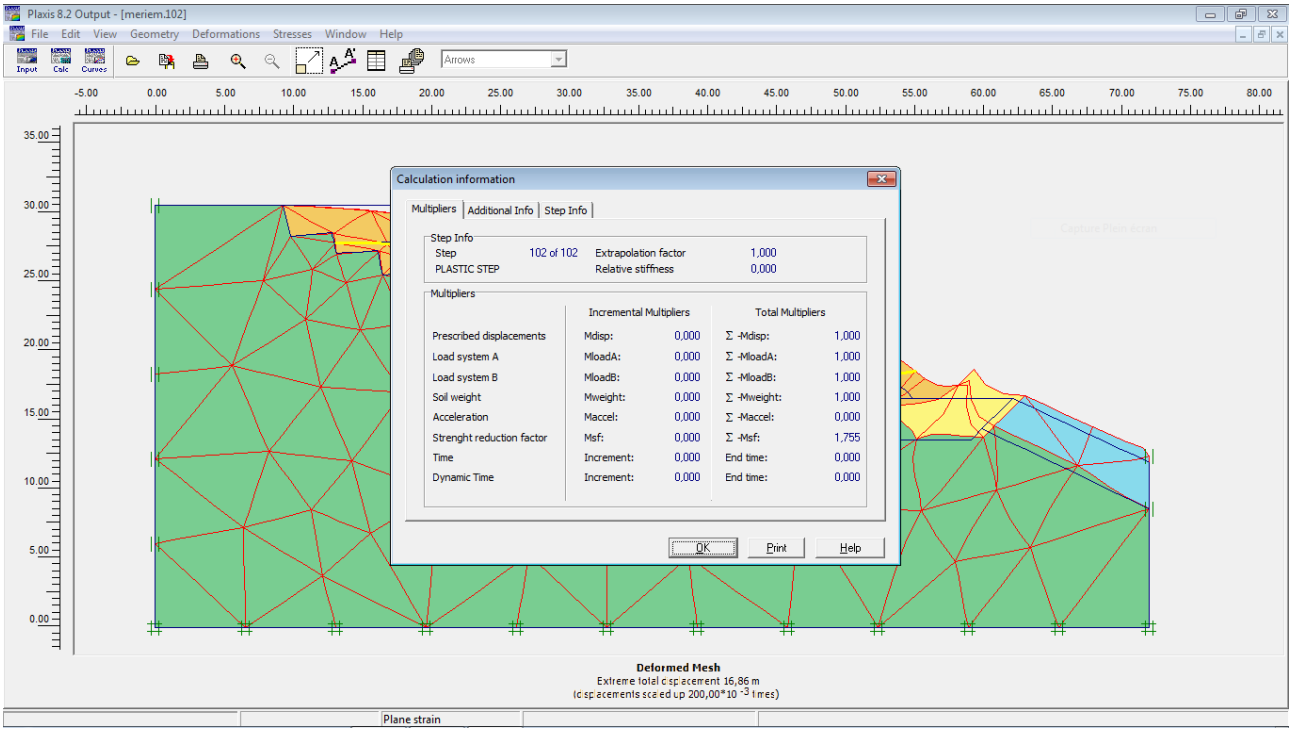


Figure 3.28: Safety coefficient (Fs) (C=30KN/m^2).

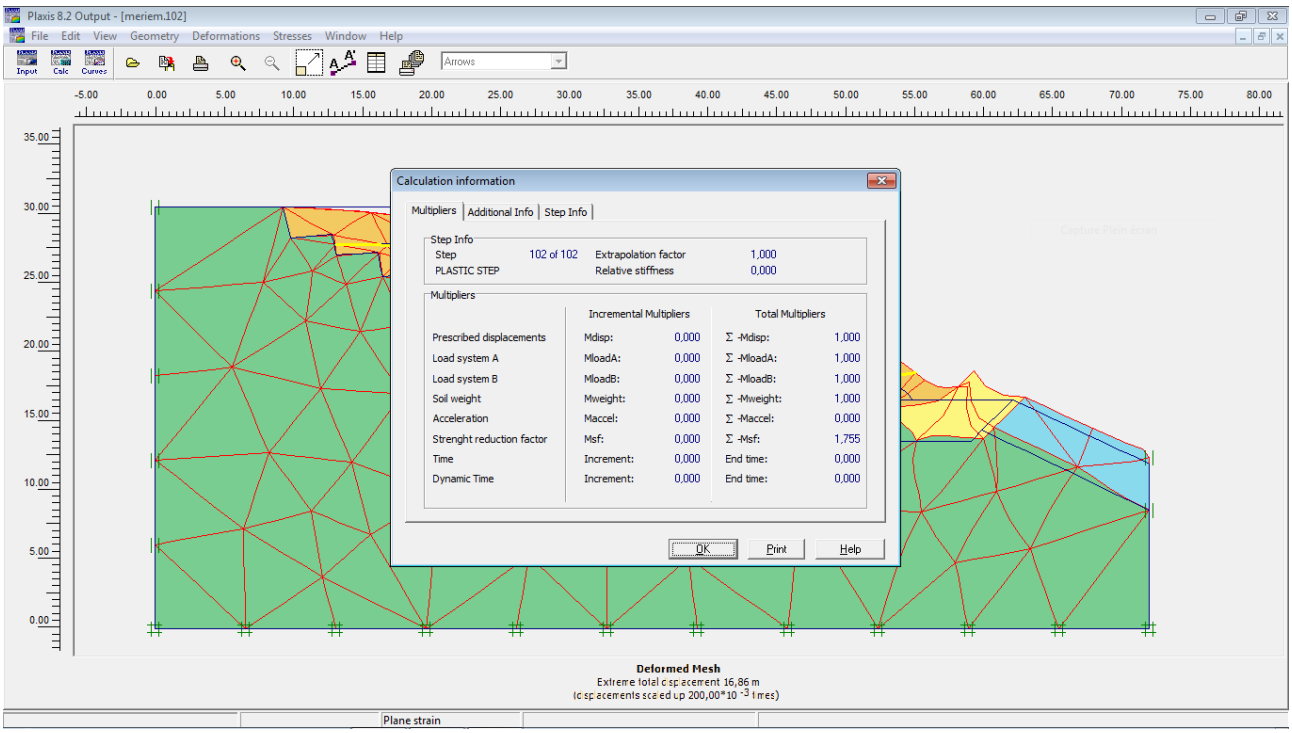


Figure 3.29: safety coefficient (Fs) (C=50 KN/m^2).

The set of results of variation in cohesion and fish coefficient as a function of Fs are shown in Tables 4.8, 4.9 and in the figures

cohésion(KN/m^2)	Fs
5	1.616
15	1.708
30	1.755
50	1.755

Table 3.6: Cohesion influence on the Fs.

From these results of the various calculations carried out, it is concluded that cohesion has a significant influence on displacement.

3.5. Conclusion :

This study led us to some interesting conclusions about the stability of the geotextile-reinforced high embankment. On the basis of the results obtained from numerical analysis, it is found that:

- Geotextile fill reinforcement increases the safety factor;
- Increasing the angle of friction results in an increase in the Fs;
- A considerable increase in the security factor as a function of the increase in cohesion;
- The increase in geotextile sheets results in an increase in the safety coefficient of the embankment.
- The change in the characteristic of the embankment increases stability assurance.

CONCLUSION.

Geosynthetically reinforced embankment applications demonstrate the undeniable value of this reinforcement solution for low-lift compressible soils.The mechanical behavior of these structures is however complex since it involves mechanisms that are not well known, such as the membrane behavior of the geosynthetic layer and the action of the compressible soil.

The effect of geosynthetically compressible backfill reinforcement is a complex phenomenon limiting the use of analytical calculations. Numerical modeling of this effect

requires professional tools (such as PLAXIS, ABACUS, FLAC,...) involving reinforcement elements, interface elements and Mohr-coulomb behavior models with finite element and deference calculations.

The purpose of this brief was to develop an understanding of the behavior of the embankments reinforced by geosynthetic reinforcements, particularly in terms of vertical deformation and stability.

So at the end of this work it can be said that the reinforcement of the embankments by geosynthetic means has several advantages such as:

- Save resources ;
- Increase performance ;
- Extend the service life;
- Reduce completion times;

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