Chapter 3

POLYMERIC MATERIALS

3.1 INTRODUCTION

At present time, the development of polymer reinforcements, which are essentially two-dimensional (geotextiles, geogrids, geomembranes) could lead to a modification of this classification, and to a distinction between linear and two-dimensional reinforcement.

For Reinforced Earth strips, various materials were initially used (Schlosser, 1977): fiberglass in a polyester resin, Tergal, or passivable metals such as aluminum or stainless steel.

In the early seventies, the use of polymers started to develop, mainly through geotextiles. Progressively, designers were attracted by the great variety of available products. They also considered polymers as an alternative, with respect to metal corrosion problems.

Numerous papers have been published on polymer reinforcement in different geotechnical journals and conference proceedings. Among the conferences partially or totally devoted to soil improvement, those related to the use of polymers in reinforced soil retaining structures are listed below:

1) Soils and fabrics, Paris, 1977.
2) Reinforced Earth and other composite soil techniques, Edinburgh 1977.
3) Earth Reinforcement, ASCE, Pittsburgh 1978.
4) International Conference on Soil Reinforcement, Paris, 1977.
5) 2nd International Conference on Geotextiles, Las Vegas, 1982.
6) 8th ECSMFE, Helsinki, 1983. General Report on Soil Reinforcement.
7) Symposium on Polymer Grid Reinforcement in Civil Engineering, London, 1984.
8) 11th ICSMFE, San Francisco, 1985: General Report on Geotechnical Construction.
9) 3rd International Conference on Geotextiles, Vienna 1986.

A journal exclusively dealing with the subject, entitled "Geotextiles and Geomembranes", is now regularly published.

The history and development of soil reinforcement systems are first presented after which the main types of existing polymer reinforced soil walls are described. The properties of polymers in relation to the behavior and the design of reinforced soil walls are briefly considered.

3.2 HISTORY AND DEVELOPMENTS

3.2.1 Coyne's Ladder Wall

In 1929, Andre Coyne patented in Paris a multi-anchorage system to be used for the construction of retaining walls and especially quay walls, dykes and so on. The idea of using such a system was the principle of constructing a wall by successive horizontal elements, formed by a light-facing element linked to continuous or discrete anchors with ties. In this system, the ratio between the total height of the wall, and the length of the anchor ties was approximately 2.5. Figure 2a shows a schematic view of such a structure as specified in the patent, and Figure 2b a cross section of an actual quay wall, 200 m long, built in Brest harbor in 1928, which was the first major application of the system. Despite some studies performed on reduced scale models (Coyne, 1945), the real mechanism of the ladder wall was not fully explained by Coyne. He indicated that the structure formed by the facing, the tie rods, the back-fill material located between this facing and the anchorages may apparently behave as a solid, sustaining small deformations, as compared to the displacements of the wall. Coyne (1945) describes some applications of this patent: the work presented in Figure 2b is a dyke submitted to tides, which may even be submerged by big waves; internal seepage may then occur, with 1.50 or 2 m hydraulic gradients,

And the structure behaves as a dam; good rocky material was used as a fill material between the ties; this structure supported 0, 50 m settlements without any problems, because of good articulations between the facings. Coyne also mentions a 200 m long quay wall in Brest, and the construction of 10 to 20m high retaining structures, such as the side walls of an overflow in Mareges, a 10 m high cofferdam at the same location, and a 14 m high dam on the river Laurenti (Pyrenees); in the case of dams, the

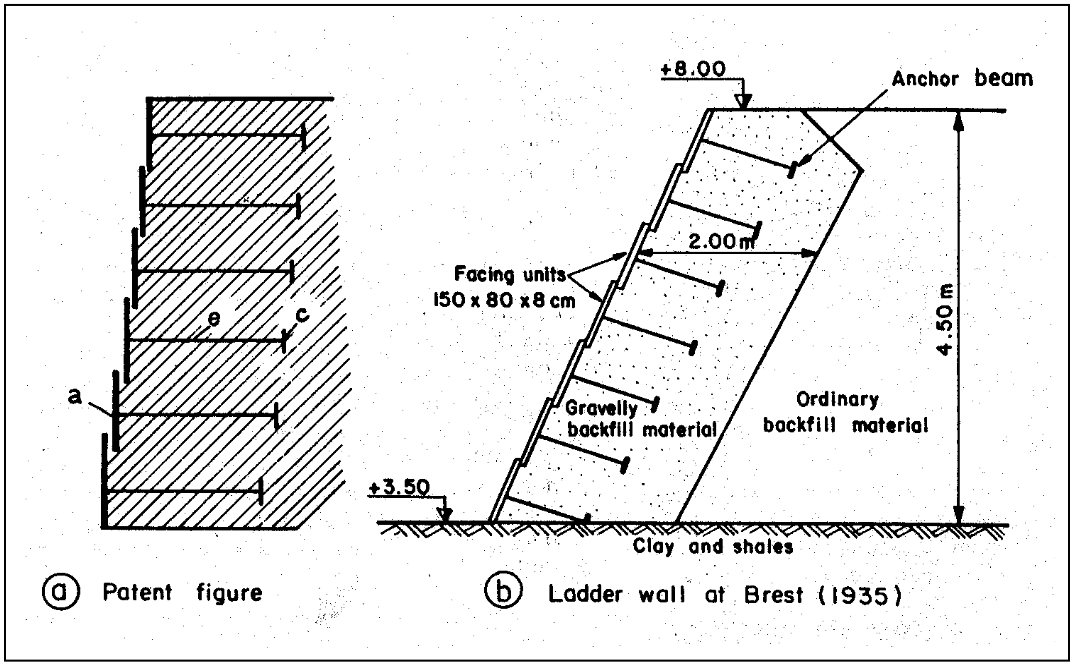


FIG. 3.1. Ladder Wall System Invented by Coyne (1929)

ladder wall may constitute either the upstream or the down-stream facing. In all cases, good rocky material is placed in contact with the ties.

Apparently, the first developments of this technique were stopped during World War II, and, in spite of Coyne's wishes, it was not sufficiently used later during the reconstruction of the country.

After a long period of time during which the Ladder Wall system was not used, several systems related to this type of reinforcement have been proposed since the starting point of Reinforced Earth development. They differ from each other with respect to the type of anchorage and facing used. Anchored Earth (Murray and Irwin, 1981) and Micro-anchorages (Costa Nunes, 1978) use frictional anchorages.

It is interesting to mention a mixed system developed by Fukuoka et al. (1982), in which the facing used is made of fabric attached to vertical columns and the anchors are concrete plates. The type of behavioral mechanism involved in this system was demonstrated with a full-scale experiment: the displacement (rotation) of the rigid columns is sufficient to reach the value of the active earth pressure on the facing, whereas the pressures on the anchor plates remain equal to the K_o state of stress.

Chabar et al. (1983) described the construction of a 21 m high dam, built according to the classical Coyne's ladder wall system.

More recently a multi-anchorage system, called Actimur (1984) has been proposed in France. It combines a vertical sheet-pile facing and horizontal tie-rods with vertical metallic anchor discs.

3.2.2 Reinforced Earth

The invention of Reinforced Earth by Henri Vidal in 1963 and the rapid development of this new technique at the end of the 60's has been the starting point of reinforcement systems, especially dealing with soil retaining techniques where the soil reinforcement is periodical and where the soil-reinforcement interaction acts all along the reinforcement. These systems have been denominated reinforced soils by Schlosser et al. (1983).

Henri Vidal considered Reinforced Earth as a new composite material and consequently introduced the very interesting concept of reinforced soil material, which has proved to be general, realistic and efficient.

It must be noticed that, in his first paper (1966), H. Vidal developed a large theory, presenting the different manners of producing a cohesive material using independent grains and reinforcements. As indicated in, Figure 3.2 he dealt first with the texture of reinforcements made with fibers (non-woven, woven, etc.) and explained the behavior of several materials (wood, paper, clay, concrete and finally human body materials) by associations of "grains" and reinforcements interacting through frictional forces.

An experimental study of the behavior of the Reinforced Earth material was performed at the Laboratoire Central des Ponts et Chaussees (Schlosser and Long, 1972) by testing samples of sand reinforced with horizontal and regularly spaced aluminum foil discs in the triaxial apparatus. It was shown that two failure modes can develop in such reinforced sand samples: failure by slippage of the reinforcement, and failure by reinforcement breakage. The yield line in the (σ_1, σ_3) principal stresses axis is presented in Figure 3.3: at low confining pressures, failure occurs by slippage, leading to a curved yield line passing through the origin; at higher confining pressure, this failure line is a straight line which proves that the reinforced sand behaves as a cohesive material having the same friction angle as the original sand and an anisotropic pseudo-cohesion due to the reinforcements. This pseudo-cohesion is very rapidly mobilized at low axial deformations, since the reinforcements behave in a rigid way compared to the relatively deformable sand. Tensile stress measurements using strain gauges show that the maximum stress value in the discs is obtained at points located at a distance from the center approximately equal to two-third of the radius. The inclined failure plane developing when the sample fails by “reinforcement breakage", indicates that a bifurcation phenomenon occurs in the development of the failure surfaces.

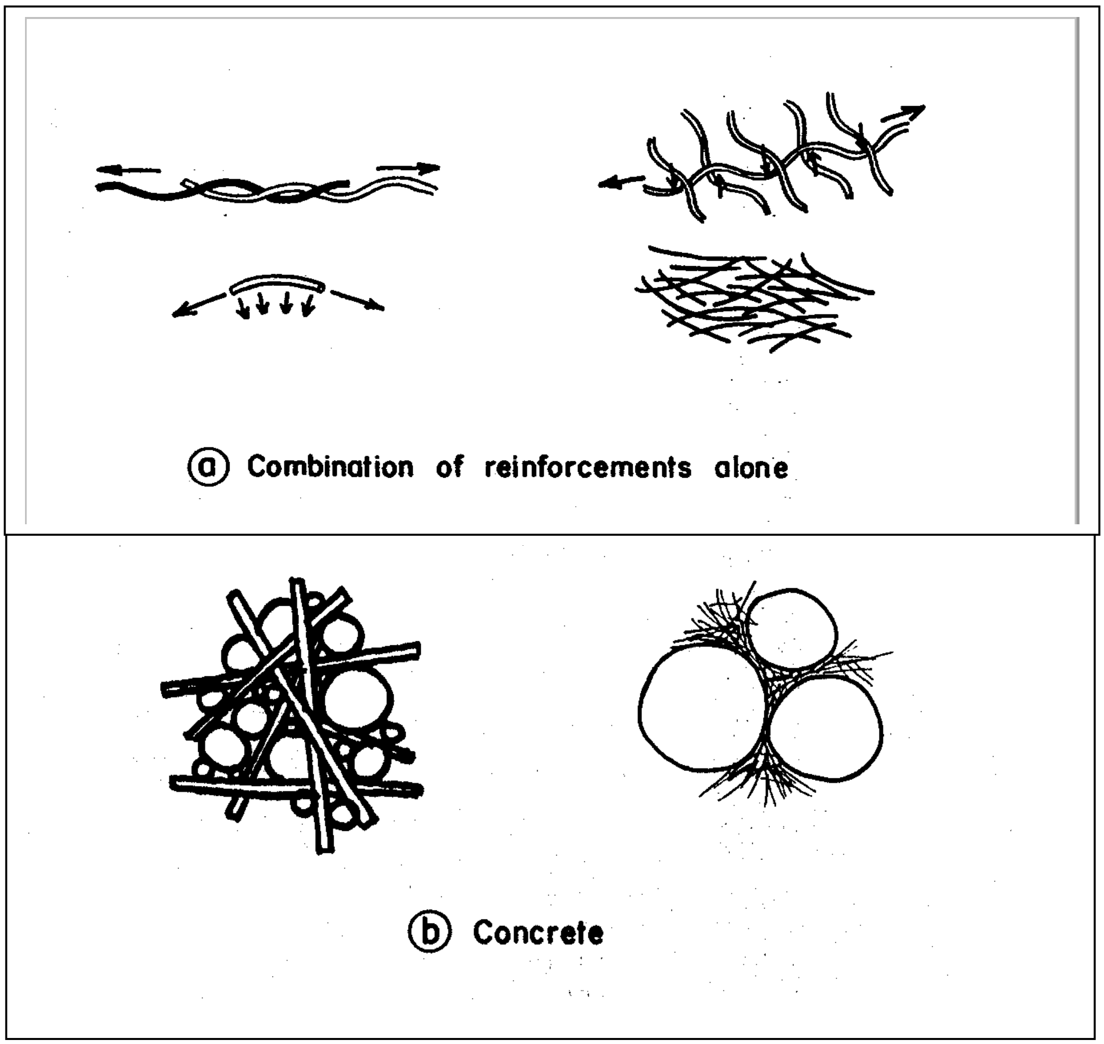


FIG. 3.2. Cohesive Materials as Combinations of “Grains” and Reinforcement”

A very interesting theoretical contribution to the subject is due to Bassett and Last (1978). These authors considered that the mechanism of tensile reinforcement involves anisotropic restraint of the soil deformations in the direction of the reinforcements. Then they used Roscoe's failure criteria for sands, based on zero extension concepts, to demonstrate that the presence of the reinforcements leads to a rotation of the principal directions of the deformations tensor. They showed (Figure 3.4) that since in a Reinforced Earth wall, the direction of the reinforcement must be aligned with the zero extension direction, the failure surface must be vertical to comply with the assumption of suppressed dilation rate (it is assumed that the Reinforced Earth material exhibits a zero dilation angle). In other words, it can be said that, due to the soil-reinforcement interaction, the presence of the reinforcements in a soil mass greatly modify the strain and stress patterns. Moreover this is consistent with the development of cracks along a cylindrical surface in reinforced sand samples in the triaxial apparatus.

Based on the above principles, Reinforced Earth consists essentially of the following components: 1) a granular backfill material, 2) linear reinforcements, generally strips 3) a facing made of pre-cast elements attached to the strips. The two major components are the granular backfill and the strips; the purpose of the facing is only to retain locally the backfill between two horizontal reinforcement layers

There has been a great improvement in the technological development of the Reinforced Earth technique, with respect to all three components, i.e. the facing, the strips and the backfill material. Initially the facing was made of U-shaped elements, 33 cm high, the weight of each being light enough to enable an easy handling. The strips were completely smooth, generally 60 mm wide and 3 mm thick. The backfill was a good granular material with less than 15 % in weight of grains smaller than 80 μm (no. 200 sieve).

Vidal first planned to use plastic strips and plastic facing elements in order to avoid corrosion problems, as indicated in his first paper (1966). He was rapidly able to produce industrially in 1967 facing elements and strips made of fiberglass coated with polyester resin. It will be seen further why the use of such a material was stopped and replaced by metals.

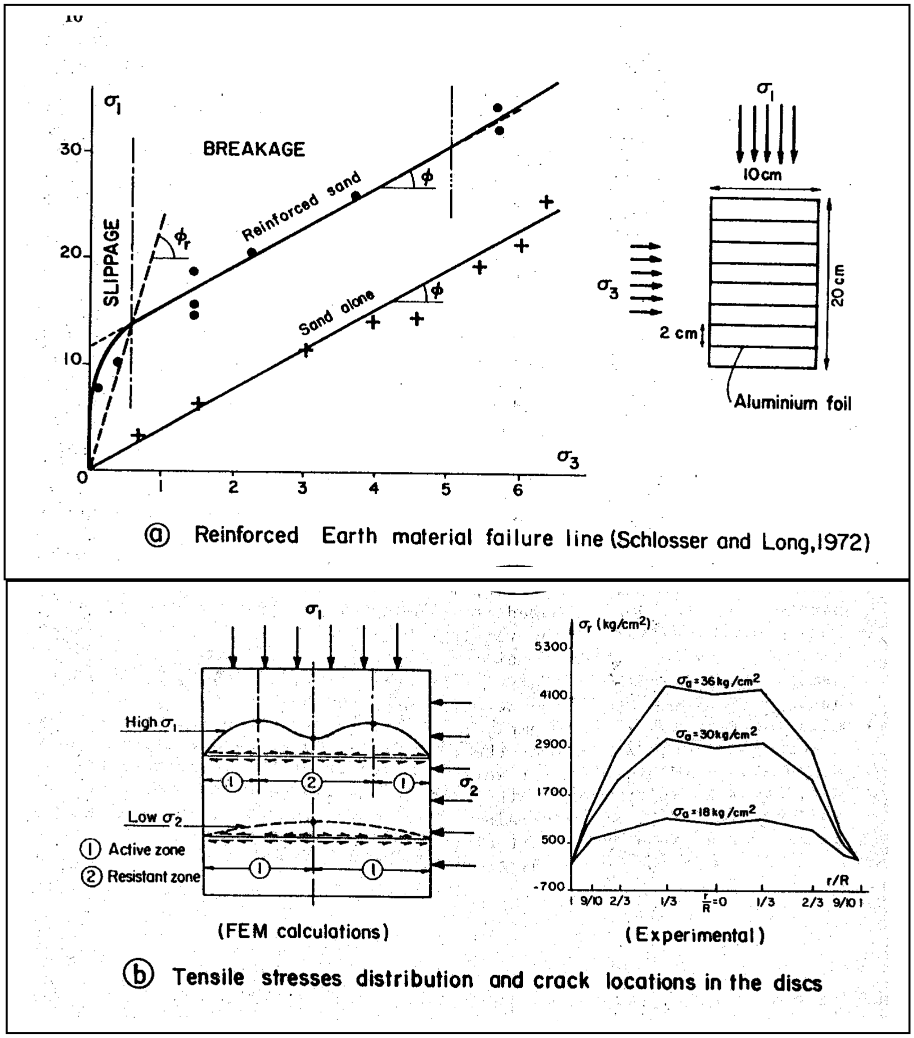


FIG. 3.3. Behavior of Reinforced Earth Material at the Triaxial Apparatus

Three events have marked the Reinforced Earth technological development. Firstly, the choice of galvanized steel for strips and facing, after a first tentative with polyester coated fiberglass and stainless steel and aluminum used for some years in France. These two metals were at that time (and even now) considered to be particularly efficient against corrosion, even when embedded in soils. They are theoretically protected by a thin layer of indestructible oxide on their surface. However, it must be now accepted 15 years later that this is absolutely not true and that these metals may be corroded in some cases more drastically and rapidly than galvanized steel, due to a special phenomenon involving an accelerated corrosion rate.

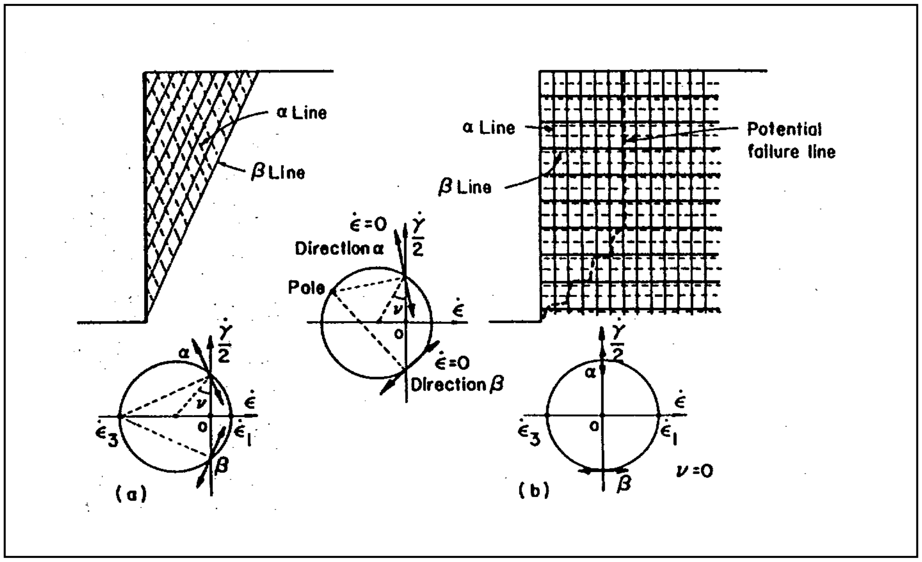


FIG. 3.4. Influence of the Reinforcement on the Potential Failure Lines

The second event has been the development in 1971 of a typical cruciform panel for the facing. This type of facing enables architectural possibilities, curved facings and it is now worldwide representative of the Reinforced Earth development.

In 1975, the Reinforced Earth Company patented the ribbed strip. This new technological aspect was directly issued from research on the soil-reinforcement frictional interaction. As indicated later by Schlosser et Elias (1978), the main phenomenon in this 3-dimensional friction mechanism is the restrained dilatancy effect. The consequently apparent friction coefficient is much influenced by the volume of the sheared soil zone around the strip.

After 20 years, the Reinforced Earth major development appears to be related to following features: 1) R.E. behaves satisfactorily even in various critical situations (large differential settlements, movements in the foundation soil, seismic event, etc.). 2) R.E. cost is competitive-and generally low compared to other solutions. 3) R.E. wall facings are attractive and aesthetic.

For the time being, the only problem is related to the special corrosion of the stainless steel and aluminum strips embedded in walls built in France 10 to 15 years ago.

3.2.3 Geotextiles

The use of geotextiles in earthworks for reinforcement and separation at the base of an embankment on soft soil started approximately at the same period as the Reinforced Earth early development. In fact, the first paper dealing with such an application has been published in 1969 (Vautran and Puig).

Since this time the application of geotextiles to roadways embankments and slopes has intensively increased. According to Giroud and Carroll (1983), the largest quantity of geotextiles is now utilized for roadway construction, principally temporary and construction roads.

The first application of geotextiles to multi-layered soil-fabric retaining systems was done in 1971 (Puig et al, 1977). It was an experimental wall using a non-woven fabric (Bidim) and a very poor backfill material (wet clayey and sensitive soil). The wall was 4 m high and was founded on a very compressible soil (peat layer, 3 m thick).

Since this first application, geotextiles have been used for retaining walls, and for earth dams (Kern, 1977). They present interesting features: low cost, drainage, possibility of using poor backfill material. However their utilization has been rather limited until now probably because of their deformability (particularly in the case of unwoven geotextiles) and to the relatively unaesthetic appearance of the facing.

3.2.4 Grids

The first reinforced soil retaining structure using grids a, reinforcements was constructed in 1974 on Interstate Route 5, near Dunsmuir, California (Forsyth, 1978). One year before in 1973, the California Transportation Laboratory developed a large direct shear device in order to test the pull out resistance of different reinforcement systems (smooth strips, ribbed strips, bars, bar mats). The purpose was to find a reinforcement system, which could enable the use of granular backfill material containing a large percentage of fine-grained material. It was found that the best system was the grid or bar mat, which provides a relatively linear reinforcement, withstanding large pull-out forces, thanks to the passive thrust mobilized against the transversal bars. However, compared with the pure frictional interaction reinforcements, i.e. strips, these bar mats require large displacements, 5 cm and more, to fully mobilize the pullout resistance (Chang et al., 1977).

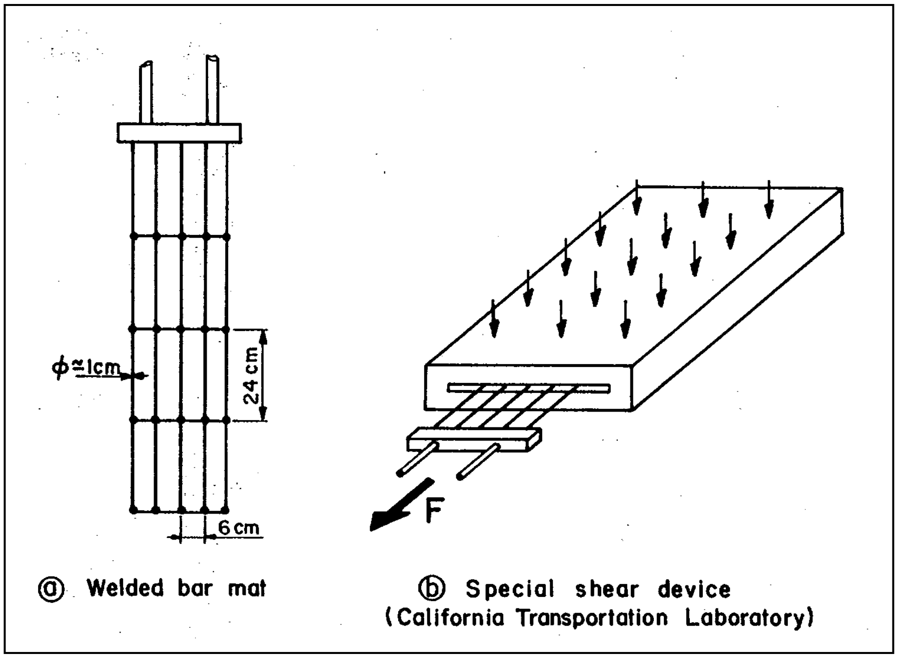
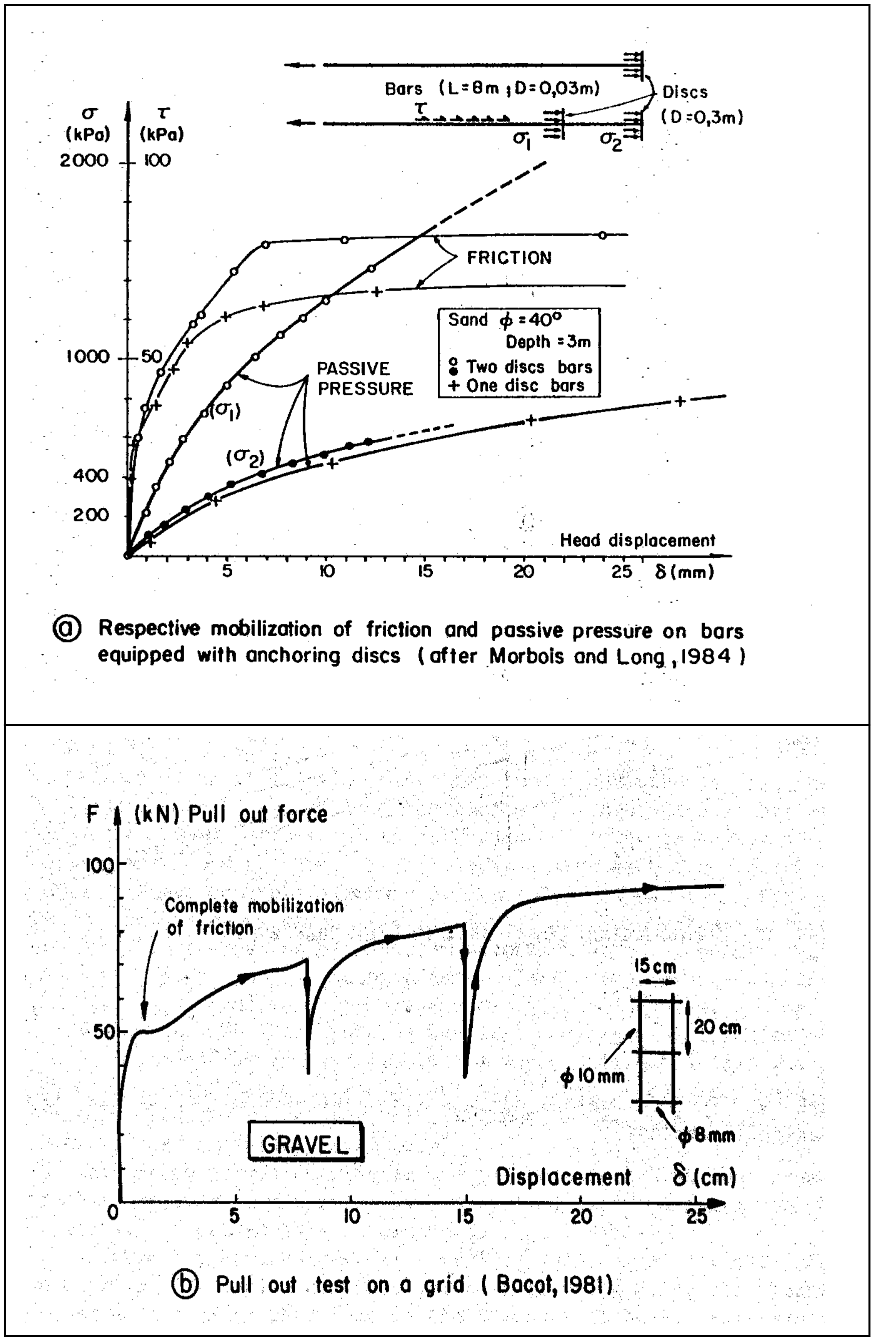


FIG. 3.5. Type of Welded Bar Mat and Panel Used by California Transportation

Figure 3.5 a shows the welded bar mat tested and used by Caltrans in the construction of the first "mechanically stabilized embankment" at Dunsmuir. This structure was approximately 120 m long and consisted of two walls, 6 m high. The vertical facings were made from rectangular and long precast concrete elements 3.75 m X 0.6 m X 0.2 m in size. The bar mats, which were 1.2 m wide and 3 to 4.5 m in length, were attached to the facing elements by inversion of the two-bar yoke through precast holes in the facing panels. These two pre-threaded bars were bolted into position, inducing some interesting pre-stressing of the reinforcement. The construction was very similar to Reinforced Earth.

As indicated by Forsyth (1978), it was "anticipated that the bar-mat mode of reinforcement would have significant economic advantage in certain areas of the state of California where high quality backfill material is not readily available".

This argument was considered and put forward by VSL when promoting in 1980 an equivalent retaining system, called Retained Earth, in which the same type of welded bar-mat was used.



ⓐ Respective mobilization of friction and passive pressure on bars equipped with anchoring discs (after Morbois and Long, 1984)

ⓑ Pull out test on a grid (Bacot, 1981)

FIG. 3.6. Mechanism of Bar Pullout Resistance

As shown by Schlosser et al. (1983, 1985), the bar-mat interaction mechanism is complex and involves both friction along the longitudinal bars and passive thrust again the transversal bars. For small soil-reinforcement displacements (< 0.5 cm) there is initially a mobilization of the friction along the longitudinal bars. For larger displacements there is a mobilization of the passive pressure on the transverse bars and the stress-displacement curve keeps increasing even for displacements greater than 10 cm. Figure 3.6 shows two typical results about this phenomenon.

Because of this mechanism, bar-mats are more resistant in pullout than frictional reinforcements (bars, strips) only for large displacements (5 to 10 cm). If such lateral displacements values are allowable for the structure, it appears that bars-mat reinforcements permit to use poor quality backfill material with a large percentage of fine-grained soil in a retaining system. However, further research in this field still required specifying suitable soils and what may be the best m geometry.

In the early 80's, Netlon manufactured and developed a plastic grid product called Tensar. This material consists of a high strength; oriented polymer grid structure obtained from punched and stretched polymer sheets. The rapid development of this product, used in a variety of so reinforcement applications (embankment reinforcement, retaining wall rafts, repairs of slope failure, gabions), led to a new type of tri-dimensional reinforcements called geogrids. A geogrid has a small opening size (about a few centimeters) compared with a bar-mat (10 and more), but compared with non-woven geotextiles; it exhibits a large deformation modulus and tensile resistance.

Properties and applications of geogrids will be further discussed with respect to soil-grid friction, there is some similarity with the soil-bar mat interaction. However, the mechanism involves a new phenomenon in coarse granular soils, resulting from interlocked soil partial within the grid apertures, which act as an anchor for the transverse ribs of the grid. Forsyth and Bieber (1984) have compared a plastic grid (Tensar) (mesh size of a few centimeters) with a metallic bar-mat (20 opening size), both having identical surface areas. The force-displacement curves obtained in pull-out tests for a normal stress equal to 34 kPa are proportional with a ratio of about 3, because the passive pressure effect observed on transverse elements is lower for the Tensar (Figure 3.7). The type of soil used was decomposed granite (ϕ '= 35^o).

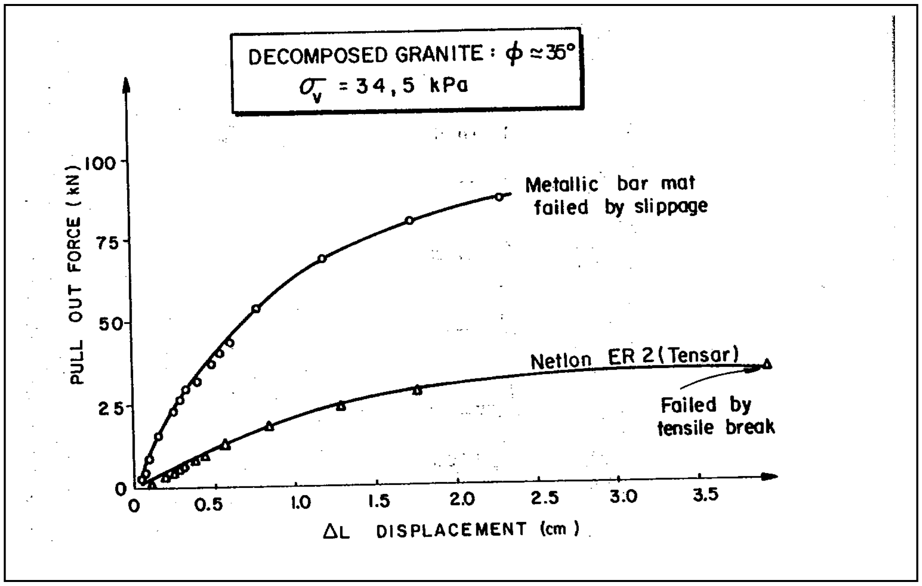


FIG. 3.7. Pullout Force/Displacement Curves for a Metallic Bar Mat and a Plastic Grid

However, comparing rapid direct shear tests on clay samples transversally reinforced by a metallic or plastic grid, Jewell and Jones (1981), and Ingold (1983) found no difference between plastic and metallic grids. The reinforcement effect on the undrained shear resistance was the following: 12 % increase with smooth steel sheet, 37 % increase with corrugated steel sheet, 42 % increase with steel grid and 44% increase with the geogrid.

It must be noted that there is a great difference between the two types of tests: pullout test and Jewell's direct shear on transversally reinforced samples. The latest is more representative of the friction phenomenon close to the potential failure surface, for instance in a reinforced soil retaining wall. However when using long reinforcements beyond this surface, as generally recommended for a design, the first type of test appears more adequate, according to the authors opinion, Figure 3.8.

According to Bonaparte et al. (1984) high-density polyethylene or polypropylene are suitable for soil reinforcement because of their in- ground durability and resistance to chemical, as well as micro-organisms attack. Generally speaking, durability is one of the most important problems, because reinforced soil structures are alternatives to classical reinforced concrete structures and they must therefore present an equivalent service life.

At present time, geogrids have largely been used in embankment reinforcements; rafts, gabions and corrections of landslides, but only a small amount of retaining walls have been built. It seems that like for geotextiles, the problem related to the facing still need to be solved: in-aesthetic aspect and erection difficulties in geogrid facings, geogrids attachment to the panels in prefabricated facings.

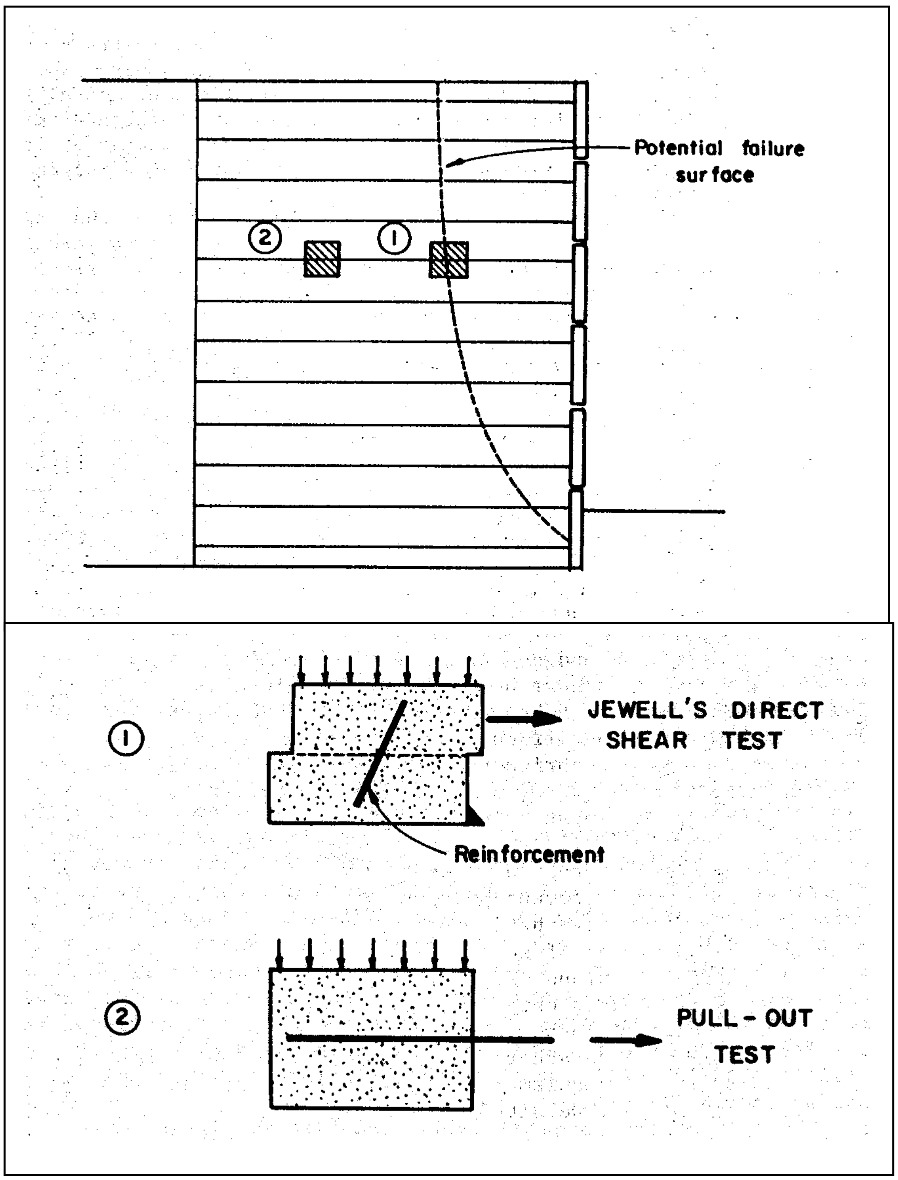


FIG. 3.8. Difference Between Jewell's Direct Shear Test and Pullout Test

3.3 MAIN TYPES OF POLYMER SOIL RETAINING WALLS

3.3.1 Types of Polymer Used as Reinforcement

Since the beginning of Reinforced Earth development, tentative efforts have been made for using polymeric reinforcements instead of metallic ones. Compared to metals, polymeric materials have large ranges of deformation modulus and tensile strength, and the following polymer products have been used as reinforcements: geotextiles sheets, geogrids sheets, woven geotextile strips, coated fiber strips, rigid plastic strips. Figure 3.9 shows for instance some mechanical properties of geotextiles and geogrids. Generally speaking, polymeric materials are more deformable and less resistant than metals. Moreover, they exhibit creep behavior; nevertheless it is possible to adapt in each retaining system the type of polymeric reinforcement used to according the allowable deformation.

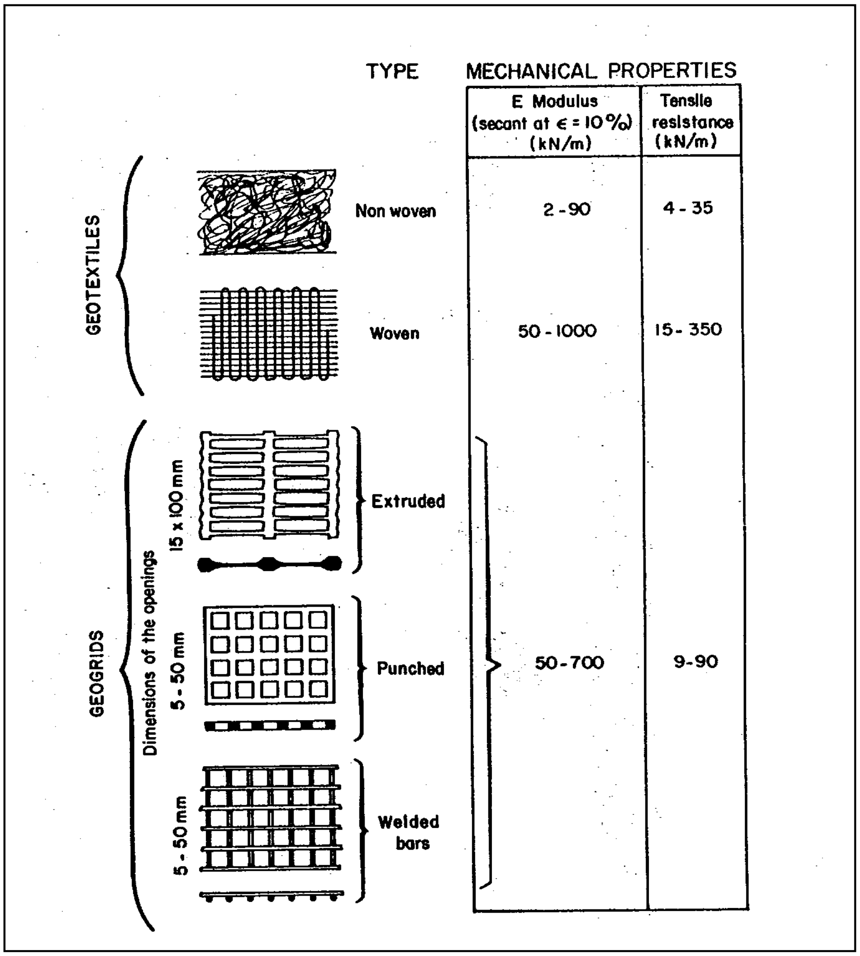


FIG. 3.9. Types and Mechanical Properties of Geotextiles and Geogrids

Inclusion extensibility greatly influences reinforced soil behavior. This has been very clearly shown by McGown et al. (1978), who have considered extensible and inextensible inclusions. Besides increasing strength, the principal action of extensible inclusions is to increase soil ductility and decrease or even cancel the softening observed in dense sand behavior. Inversely, inextensible inclusion mainly Increase soil strength and deformation modulus, but they cause the deformation soil modulus to be more brittle. These features are presented in Figure 3.10 and allow the following distinctions to be made:

1- Reinforcement with ideally inextensible inclusions, mainly represented by Reinforced Earth, for which the reinforcements are generally linear and metallic.

2 - Reinforcement with ideally extensible inclusions, represented by “ply-soil" or "multi-layer soil" (McGown, 1978), for which the reinforcements are generally plane and made of synthetic materials (geotextiles, etc.).

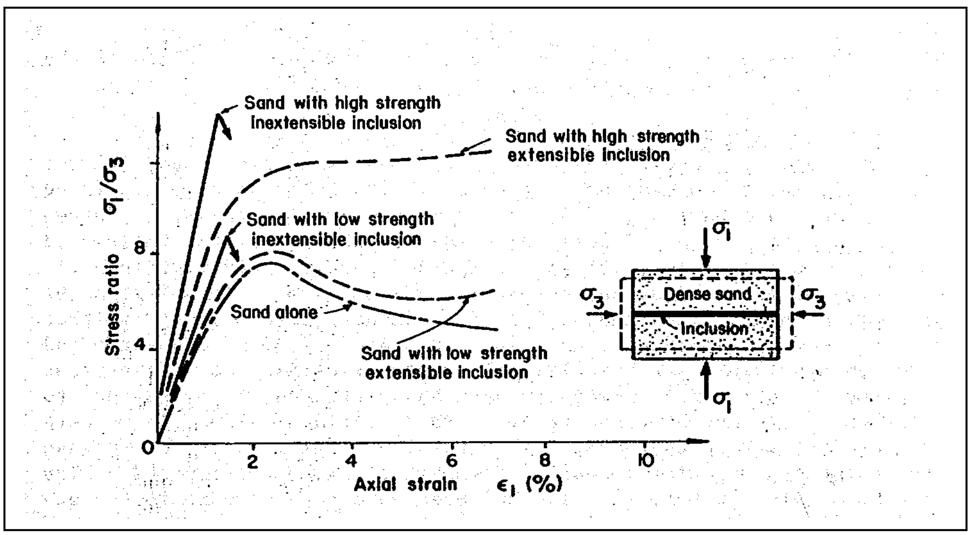


FIG. 3.10. Deformability and Strength Inclusion Influence on Reinforced Dense Sand Behavior

3.3.2 Polymer Uses in Reinforced Earth Technique

Considering the whole research performed on Reinforced Earth particularly at the beginning in the 60's, it could appear surprising that nothing would have been done in order to use polymers as reinforcements. In fact, Vidal (1966) planned to use at first polymer materials: nylon strips, tergal strips and particularly rigid plastic constituted of fiber glass coated with polyester resin. This last material was chosen in 1965 and an important investment was made at that time in order to produce industrially U shaped facing elements and reinforcement strips. More specifically, this material was a fiberglass reinforced plastic, in which strength and stiffness were imparted to easily molded resins by glass fibers. The individual glass fibers were elastic and as strong as the strongest tensile steels, so that they gave to the composite material a small deformability without creep and a high strength. In 1965, this material had been used for 10 years in under ground pipelines and tanks and had proven to behave satisfactory. However, as indicated by Mallinder et al. (1977), some degradation was observed when the material was maintained in wet conditions for Ion periods of time.

In 1966, a first experimental Reinforced Earth wall using fiberglass reinforced plastic strips and facing units was built, in order to test the construction process and the mechanical behavior of the wall. Unfortunately, the plastic material was attacked by bacteria and the Reinforced Earth wall was destroyed within 10 months. No biological test was performed on the backfill material after failure and practically no reliable information and expertise about the type of bacteria and the degradability process is now available.

After Mallinder et al. (1977), this failure might have been accidental, since these authors gave biological test results on fiberglass reinforced plastic, indicating that this type of material was no degraded by bacteria (immersion time was however limited to 6 months).

Nevertheless this failure has been the turning point for the use o plastic materials in Reinforced Earth: at the beginning of 1967. Vidal decided to develop Reinforced Earth with metallic reinforcements.

However, the use of plastics in Reinforced Earth was not yet completely abandoned, since another very interesting attempt was done in 1971 with the construction of the Poitiers wall using Tergal strips. This wall was a temporary structure, 5 m high and 40 m long. The tergal strips were attached to cruciform concrete panels. The calculation of the wall took into account tergal creep and during its, service life, the wall behaved satisfactory. However, it appeared during construction that tergal strips had to be slightly prestressed in order to prevent excessive lateral displacements of the facing.

In 1981, ten years after its construction, the wall was dismantled and interesting durability tests were performed on the strip material. As presented further, it was shown that the plastic fibers had been degraded and that the mechanical properties of the plastic strips had decreased.

3.4 PROPERTIES OF POLYMERS WITH RESPECT TO THE BEHAVIOUR OF REINFORCED SOIL WALLS

When metallic reinforcements are inserted into the soil, they may be considered as rigid with respect to the soil deformability, and to the magnitude of the induced stresses. On the contrary, polymeric inclusions are characterized by weaker mechanical properties, i.e. high extensibility, low-tensile strength, associated with long-term creep. Furthermore, although not sensitive to electrochemical corrosion, polymers may also suffer some degradation under soil physico-chemical environment, and durability problems are to be considered. Several studies have been conducted on strength-strain properties of geotextiles since they have been used in various applications of geotechnical engineering. It has been observed that among all those applications, reinforced soil retaining walls constitute a case in which the magnitude of stresses applied by the, soil to the geotextile is the higher. In fact, it is quite difficult to fully understand the load transfer mechanism that occurs between the soil and the textile, and to know the exact stress field the textile is submitted to.

3.4.1 Extensibility

a. Polymers Properties

It should be noticed that, for each polymer, both bulk material and filament properties tested. In fact, filaments are produced by extrusion of the heated polymer mass, and high tenacity yarns, and are obtained by controlling the cooling process of the filament, and by stretching it during extrusion. In such a way, polymeric chains have a preferential orientation and an increased anisotropy, thus resulting in much higher (up to ten times) mechanical properties of the material, such as tensile strength or elastic modulus. The main difference existing between metallic and polymeric materials corresponds to high extensibility of polymers. This is illustrated by the values of elastic moduli, which are, for current polymers, from 10 to 30 times lower than for metals; the tensile strength is also lower (2.5 to 5 times), whereas the deformation at failure is much higher (20-30 % instead of, 3 %). When comparing current polymers, it may be seen that polyester and polyamide, which have higher densities have higher stiffness and tensile strength than polypropylene and polyethylene high density (0.91 and 0.95 respectively).

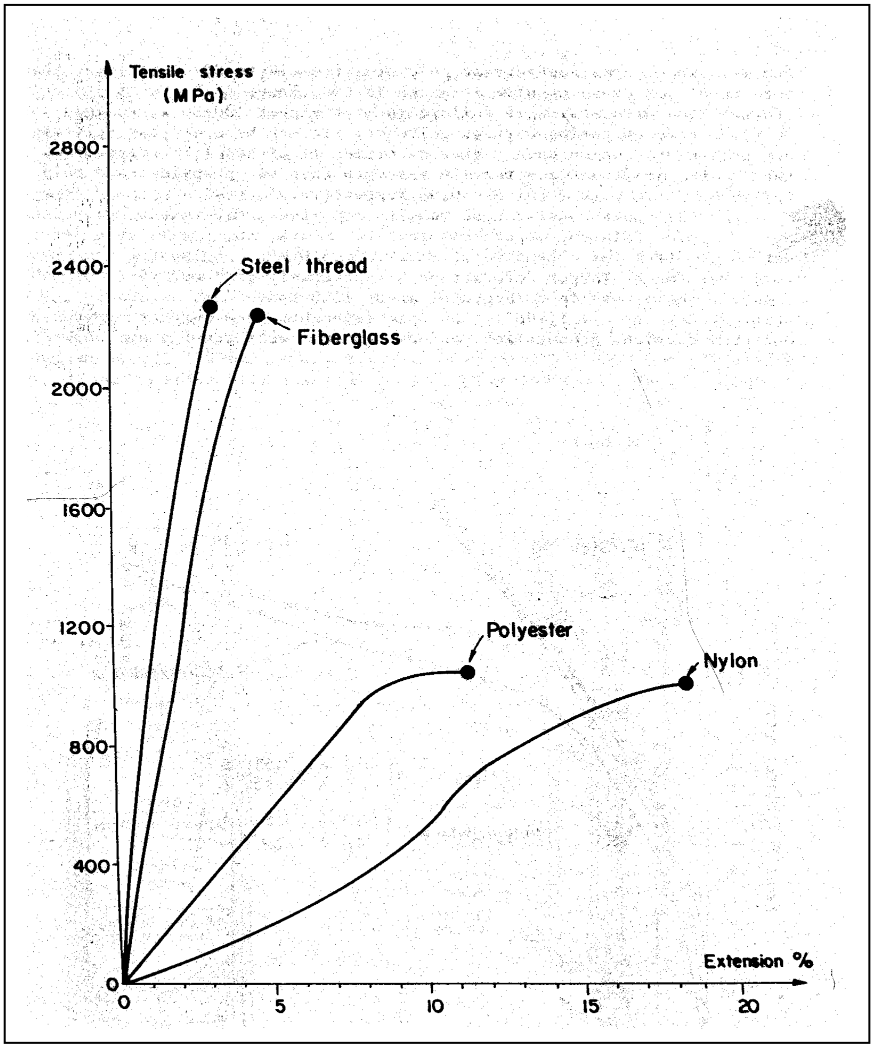


FIG. 3.11. Stress-Strain Behavior of Different Material Fibers

Figure 3.11 shows tensile test results for steel fiberglass, polyester and polyamide filaments. Whereas failure occurs, for steel, at a deformation of 3.2% for a tensile strength of 2340 MPa, polyester and nylon fail for much larger deformation (respectively 11 % and 19 % on the Figure), and a tensile strength of about 1000 MPa.

Baudonnel et al. (1982) took some filaments from current polyester and polypropylene geotextiles, and elongation tests results are shown in Figure 3.12.

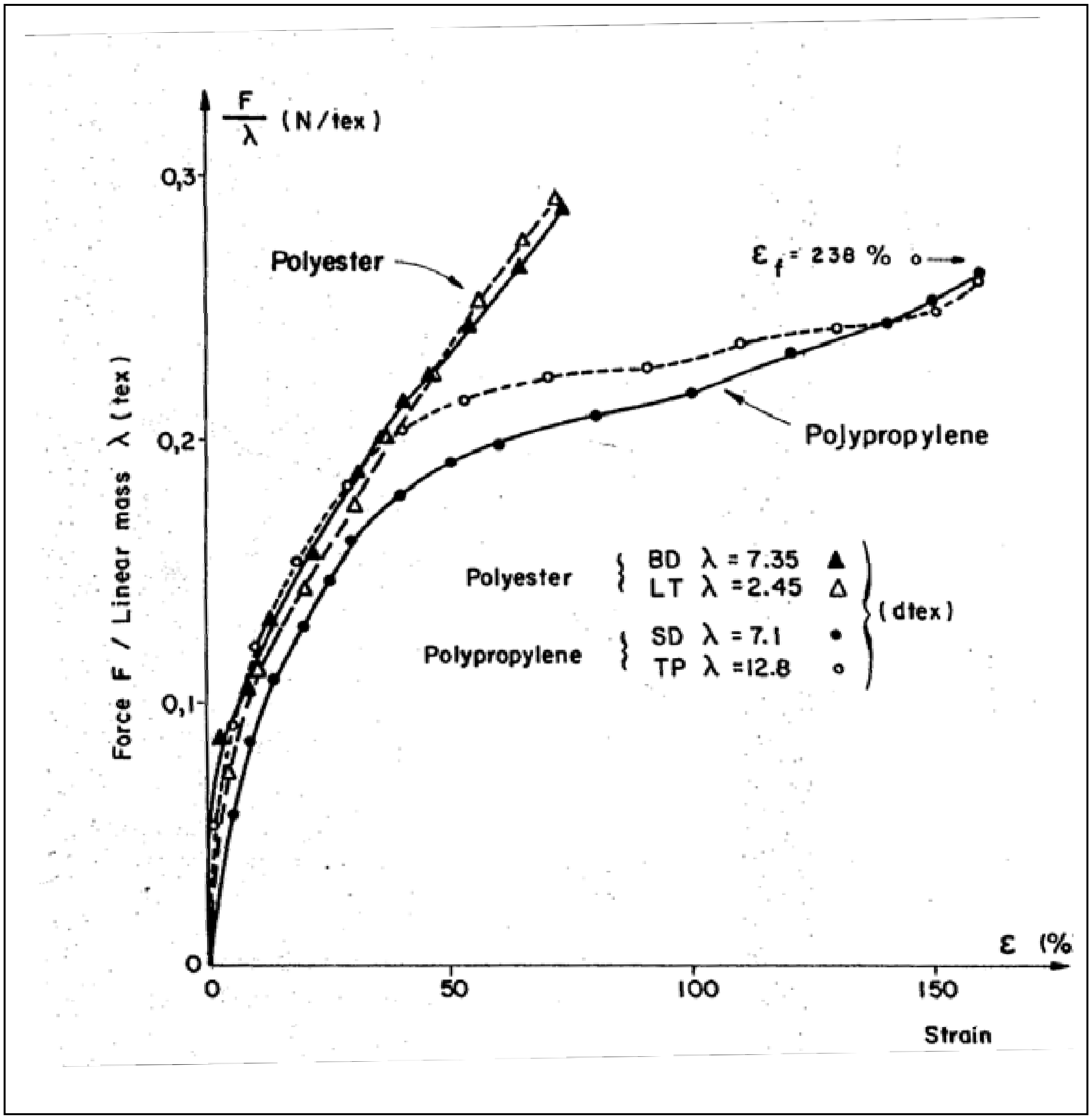


FIG. 3.12. Elongation Test on Filament Extracted from Non-Woven Fabrics

The strength value is expressed in terms of tenacity; say the force divided by the linear mass of the yarn, expressed in N/tex. In the case of non-woven fabric extracted filaments, the deformation at failure is in both cases quite higher (71-78 % for polyester, and 155-239 % for polypropylene). The polypropylene curve is composed of two parts having different slopes, the first one being identical to the polyester one. These results illustrate the decrease of the mechanical properties that occur when fibers are processed for fabric construction.

The previous data were related to solids and fibers, but the great variety of polymeric inclusions used in geotechnical engineering may exhibit various properties, which do not only depend on the nature of the polymer, but also on the structure of the inclusion, and on the influence of the soil confinement.

The influence of those parameters has been explicitly evidenced by McGown et al. (1982), who performed load-extension tests on geotextiles confined in soil, with the help of the apparatus.

In this apparatus, a geotextile is included between two layers of soil (Leighton Buzzard sand, D_{50} =0.85 mm), and pressure is applied on each side of the fabric by two air-activated rubber pressure bellows. McGown et al. tested four different fabrics: woven, non-woven melt bonded, non- woven needle punched, composite woven and needle punched. The elongation tests were performed at 20^oC and at a constant rate of strain of 2% per minute. It may be seen that for the woven polypropylene fabric (Lotrak 16/15, 120 g/m2), the in-soil confinement has a very little influence. On the contrary, in the case of a polyester non-woven needle punched fabric (Bidim U24, 210 g/m^2), the effect of in-soil confinement is quite important, and a 100 kPa confinement pressure results in a strengthening of the fabric, which corresponds to a higher stiffness, as illustrated by a higher slope of the elongation curve. This phenomenon may be interpreted in the following manner: the fabric elongation does not involve individual filaments, but rather induces a rearrangement of the needle punched filaments, which affects the bonds between the filaments. When soil is in contact with the fabric under a given pressure, it contributes to the stability of the bonds between the filaments, and provides a higher strength to the fabric.

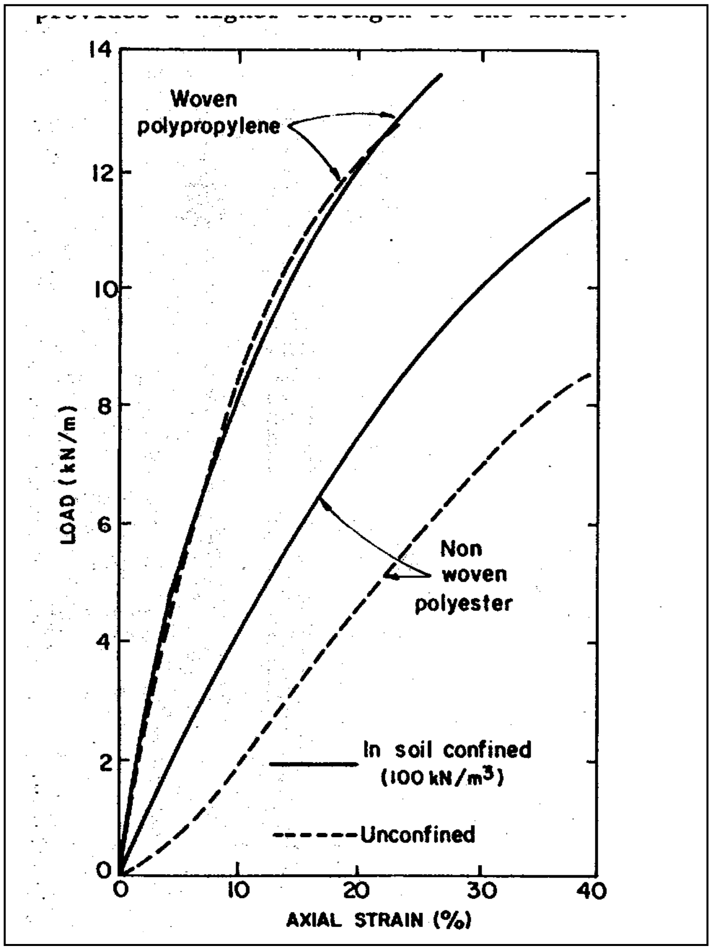


FIG. 3.13. Influence of the In-Soil Confinement and of the Fabric Structure on Elongation Properties.

It is interesting to notice, on Figure 3.13, the predominant influence of the structure of the fabric as compared to the polymer type, since a 120 g/m^2 polypropylene woven fabric is stiffer than a confined 210 g/in^2- non-woven needle punched polyester fabric. For the woven fabric, failure occurs at 23-28 % deformation, whereas for the non-woven material, lower levels of load induce deformations as high as 40 %, without occasioning rupture.

The problem of tensile testing of fabrics is then particularly important for non-woven fabrics, since unconfined testing induce an important retraction of the strip, as shown in Figure 3.14. For this reason, an initially current textile test which consisted of stretching up to failure a 50 mm wide and 200-300 mm long strip has been replaced by some other tests, with wider strips.

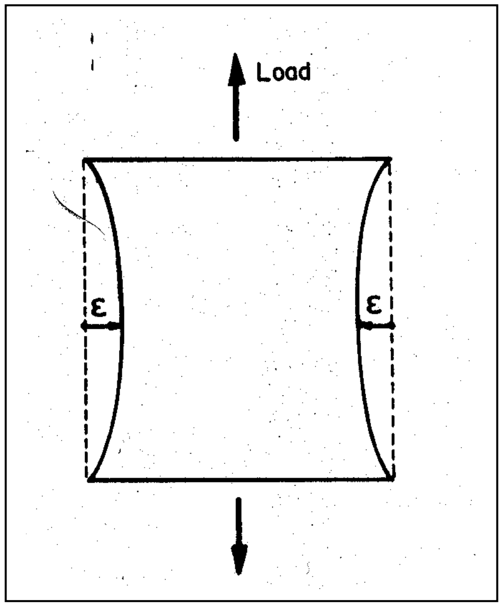


FIG. 3.14. Lateral Construction Occurring During an Elongation Test on a Non-Woven Fabric.

Presently, agreement does not exist on the best width to be selected for the strip. McCown et al. (1982) have studied the influence of the width on unconfined elongation tests and have compared their results with in-soil 200 mm wide tests results. It is observed that, due to retraction, width has a great influence on strength-strain behavior, and that strip widths of 50 or 100 mm are definitely too small. It is interesting to notice that, the 200 mm test corresponds to the 0 kPa in-soil confined test, whereas the 500 mm test corresponds to the 10 kPa in-soil confined stress.

In fact, present discussions on strip width are related to those two values of 200 and 500 mm, for a length of 100 mm, and argumentation elements have been obtained from special tests, where lateral retraction was avoided. Such special tests include a hydraulic tensile test (Raumann, 1979), biaxial tensile tests, (Viergever et al., 1979), cylindrical sleeve test (Paute et Segouin, 1977), and a special test in which lateral restraint is achieved by means of lightweight wooden brackets in which steel pins have been set (Sissons, 1977). The fabric is pressed on ten of these brackets regularly scattered along the length of the 200,mm wide strip. The pins cross the fabrics and avoid restraint. Shresta and Bell (1982) have used this device for testing several geotextiles.

Testing of 200 mm wide strips has been also performed by Murray et al. (1986), and Richard and Scott (1986).

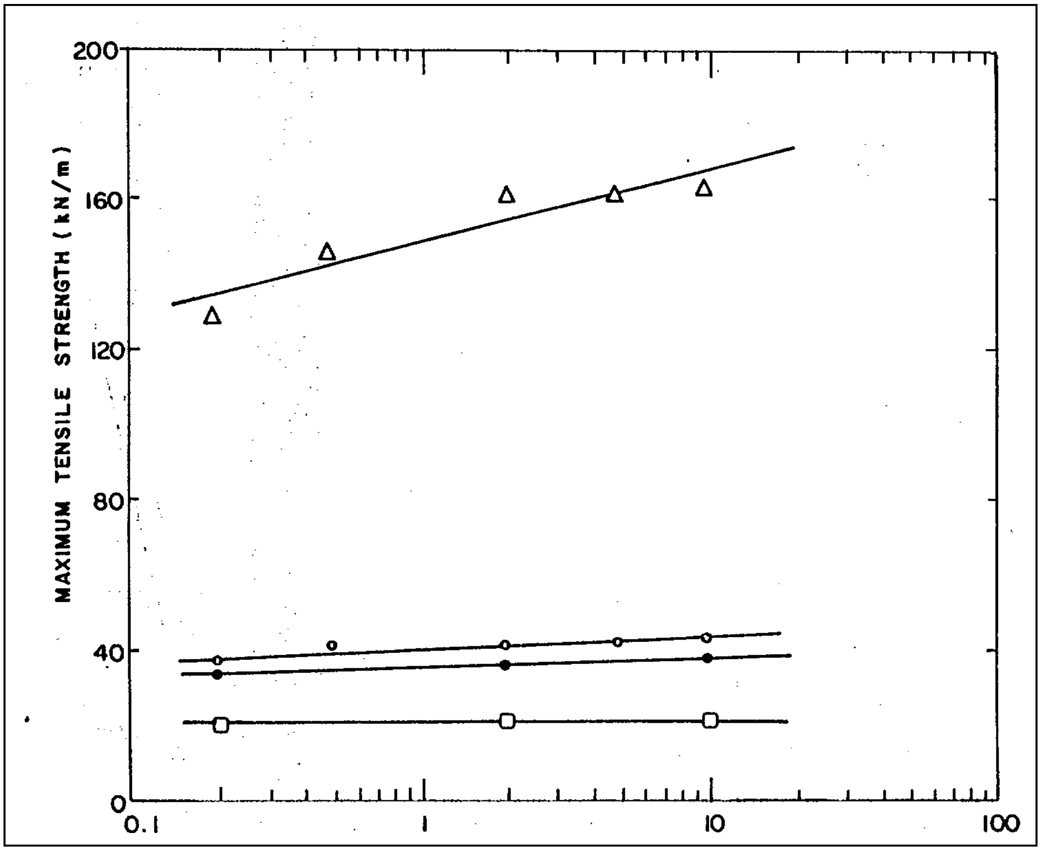


FIG. 3.15. Variation in Tensile Strength of Some Woven Fabric With Strain Rate

Leflaive et al. (1982) performed 500 mm wide tests, and proposed a correction corresponding to the lateral contraction. The corrected results they obtained compared favorably with results of cylindrical ,sleeve test, and in-soil confined tests (McCown et al., 1982). This approach has been adopted by Cazuffi et al. (1986), Leclerq and Prudon (1986). Rowe and Ho (1986) also suggest a 500 mm wide value.

Other parameters have been studied. Figure 3.15 shows the influence of the strain rate on the maximum tensile strength of some woven fabrics (Rowe and Ho, 1986). In this case, where the constitutive fibers are directly sollicitated by the tensile test, the effect of strain rate is important, and the writers suggest a 2 % rate. Several writers considered the influence of the tensile direction, as compared to warp, weft, or diagonal direction for woven fabrics, and production direction for non-woven fabrics. In the later case, Van Leeuven (1977), Leclerq and Prudon' (1986) observed no variations of the tensile strength, whereas Paute et Segouin (1977) mention some decrease in the production direction. In the former case, warp and weft directions give generally similar results, whereas a 20-40 % decrease is observed in diagonal directions. However, Rowe and Ho (1986) observed significant variations on some woven fabrics, the warp direction being sometimes stronger.

For non-woven fabrics, and for a given type of polymer, the tensile properties are dependent on the weight per unit area of the fabric, as shown on Figure 3.15 (Paute and Segouin, 1977), from results of the cylindrical sleeve. The curves show, for various non-woven polyester and polypropylene fabrics, the influence of the value of the weight per unit area, expressed in g/m2.

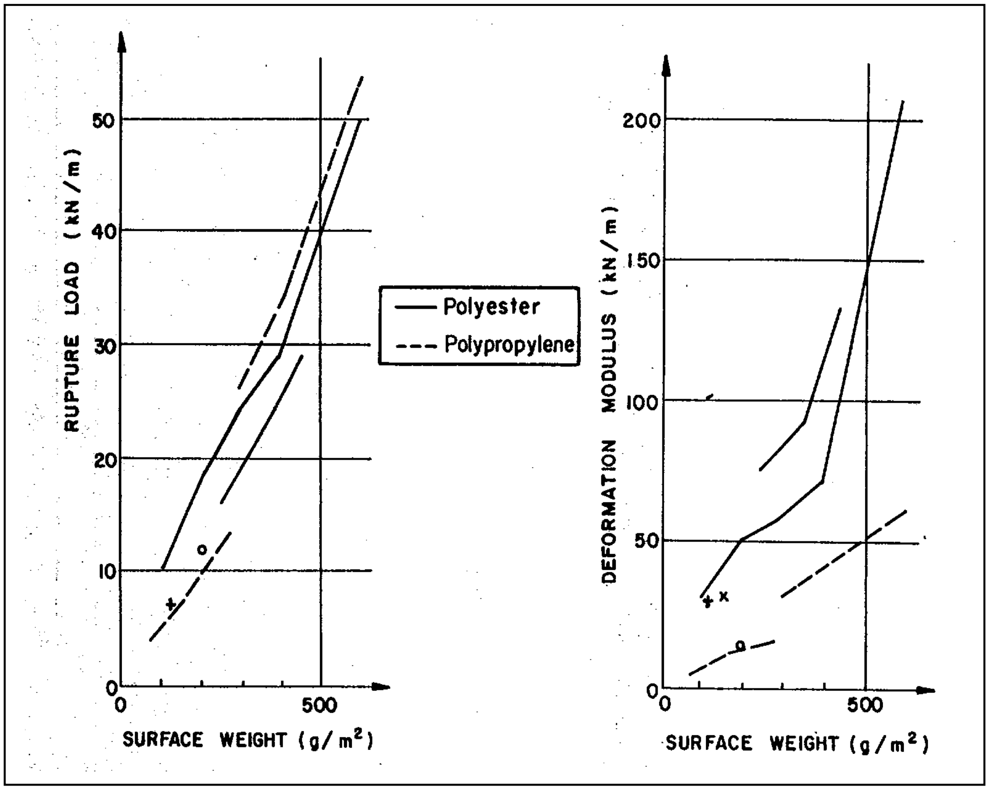


FIG. 3.16. Tensile Strength and Deformation Moduli of Polyester and Polypropylene Fabrics Listed on the Sleeve-Cylinder Apparatus (Paute and Segouin, 1977)

The mechanical data are the rupture strength (3.16a), and the deformation modulus E (3.16b). The increase at rupture strength is fairly linear for all fabrics, whereas the deformation modulus shows, in the case of polyester, a slope increase of about 400 g/m2. It is interesting to notice that the nature of the polymer has little influence on the mechanical properties of the material. These curves give an idea on non-woven rupture strength, which may vary , according to the weight per unit surface, between 10 and 50 kN/m except for weaker fabric.

b. Creep behavior of Polymers

In the case of metallic reinforcement, the level of stress induced through soil-structure interaction in retaining structures is quite low as compared to metal tensile strength, and creep of metals is not significant. On the contrary, the tensile strength of polymers is much lower, and creep behavior has to be seriously considered for assessment of long-term stability of retaining structures.

Creep of polymers yarns is a well known phenomenon, and Figure 3.17 presents results of long term elongation tests, on yarns submitted to constant tensile loads (Greenwood and Myles, 1982), during 10 000 hours, i.e., 1 year and 50 days. For this duration, and for loads not exceeding 40 % of the rupture load, strain is a linear function of the logarithm of time, as in many other materials like soils, for example. Polyester yarns are characterized by a relatively high instantaneous strain. The values of the instantaneous strain and of the creep rates are in good agreement, for polyester, with previous results presented by Finnigan (1977). As compared to polyester, polypropylene exhibits lower instantaneous strain, but higher creep rates. This creep tendency of polypropylene is well known among textile people. At 60 % of the rupture load, there is an upturn of the curves, which may be characteristic of the rupture phenomenon initiation. In fact, long duration tests mentioned by Greenwood and Myles on other polymers yarns (parafilrape) during seven years, showed such upturns, which were initiated after more than 10 000 hours testing. Therefore, linear extrapolation of the curves for long durations may not be realistic, and it may underestimate creep strains. As mentioned by Finnigan (1977), techniques such as heat stretching of the filament (12% to 10 % at 235^{o}C for 75 seconds) may significantly reduce the creep tendency.

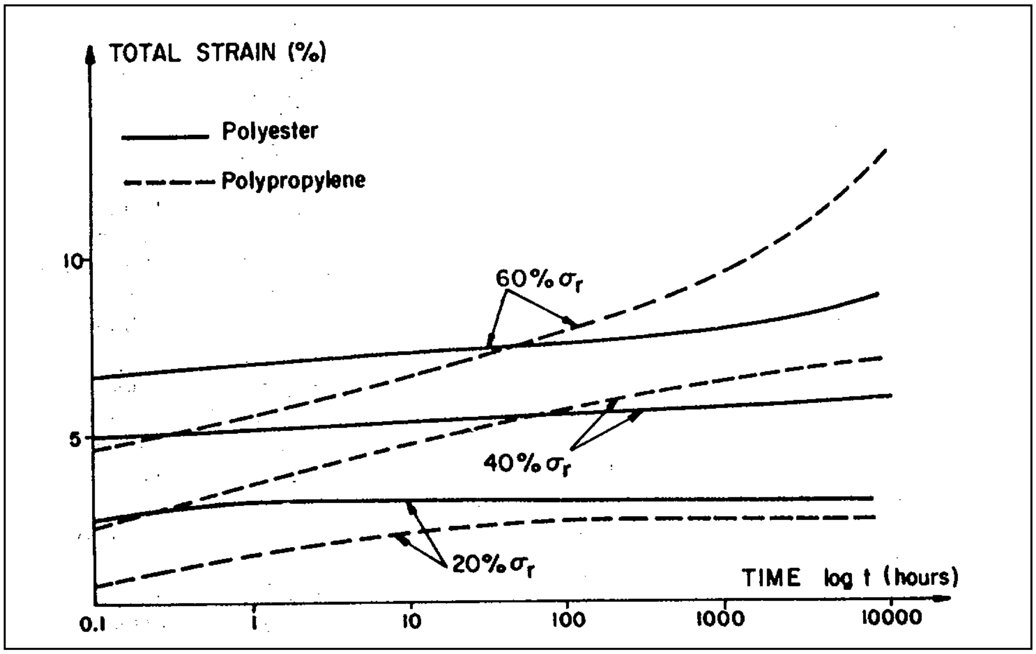


FIG. 3.17. Creep of Polyester and Polypropylene Yarns (Greenwood and Myles, 1986)

In the case of woven fabrics, data from Van Leeuwen (1977), presented in the discussion session of the Paris Conference (Vol. 111, p. 102) are shown in Figure 3.18. These data concern polyester, polyamide and polypropylene, for loads equal to 50 % of the rupture strength. As for short term elongation tests described previously, the properties of the constitutive filament have a strong influence on the behavior of the woven fabric. The best creep behavior is observed for polyester fabric. Polyamide fabric exhibits a little higher tendency to creep, whereas the creep observed for polypropylene fabric is quite important. However the author mentions that no heat treatment was performed to improve creep properties of those polymers fabrics. It may be seen that few differences exist, in terms of creep behavior, between yarns and woven fabrics. Such a statement has formerly been made by Finnigan (1977).

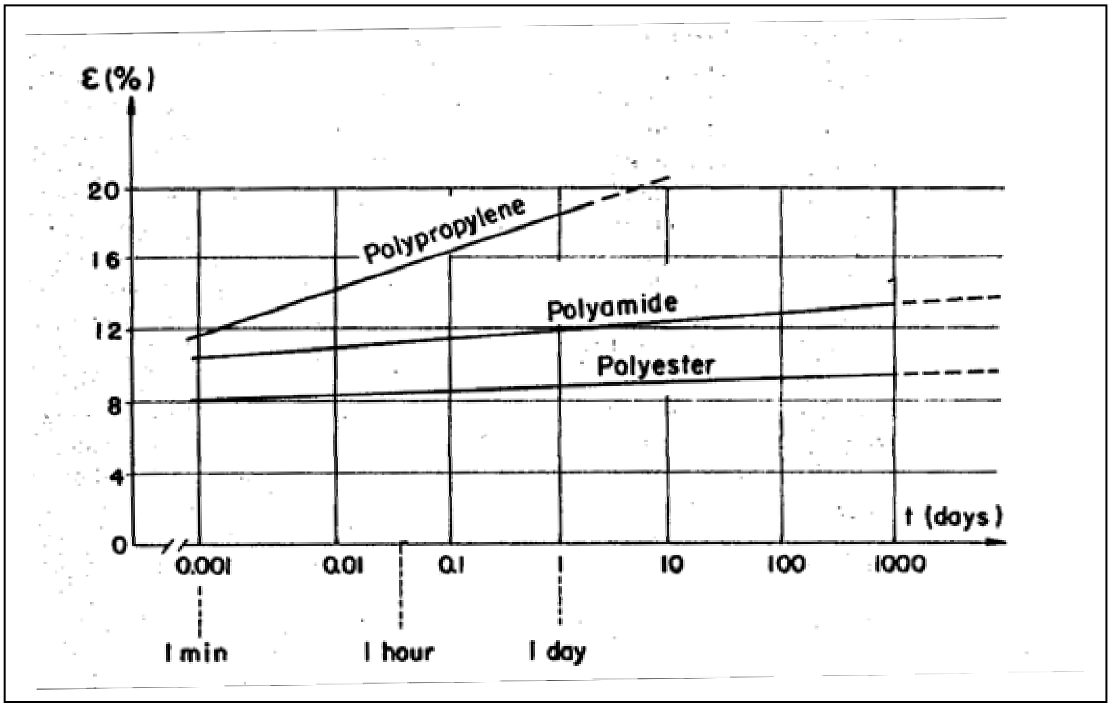


FIG. 3.18. Creep of Synthetic Woven Fabrics Under Prolonged Loading (50% of the Breaking Strength) (Van Leeuwen, 1977)

Such a comparison made for polypropylene woven fabrics shows, according to the results of Van Leeuwan, that a larger Increase in creep is observed when passing from yarns to fabric ; creep rates at 50 % of breaking load increase from approximately 1.13 (Greenwood and Styles) to 2.1 (Van Leeuwen). Bell et al. (1982) mention the possibility of improving the polypropylene yarns creep rate under 40 % breaking load, from 1.5 to 0.40 by an adequate treatment. However, for woven fabrics, they only give creep rates at 20 % breaking load (0.40 – 0.73), which does not allow a direct comparison to be made.

For the SR2 Tensar geogrids, composed of high-density polyethylene, data from the isochronous load-strain curves presented by McGown et al. (1984) are reported in strain/logarithm of time diagrams in Figure 3.19.

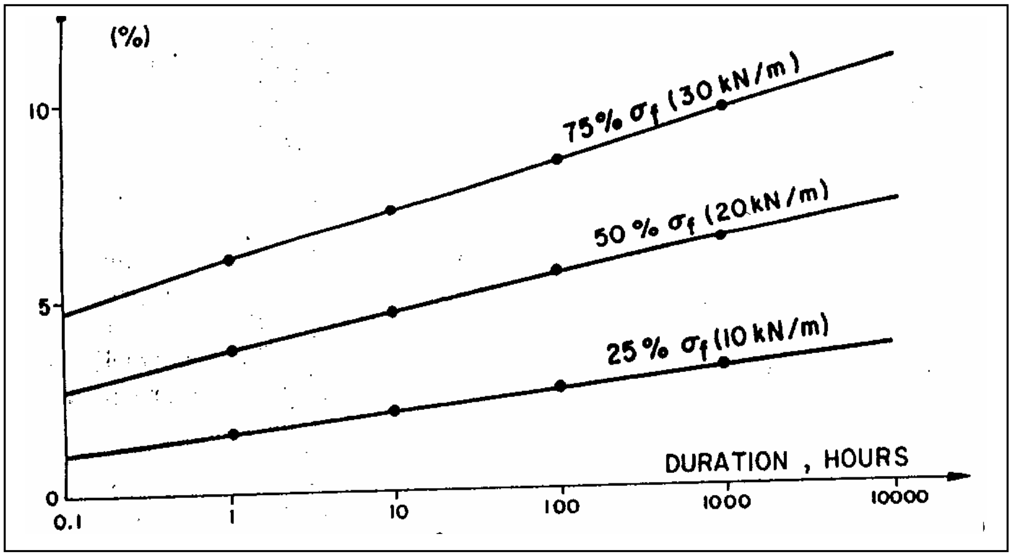


FIG. 3.19. Creep Behavior of Tensar SR2 (After Mc Gown et al., 1982)