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The durability of geosynthetics for retaining walls and slopes for long term performance

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ABSTRACT: Geosynthetics are an indispensable part of the civil engineering industry, and their use can offer construction alternatives that are substantially cheaper than more traditional techniques. Evidence from samples recovered after several years burial have shown that these materials are inherently stable. However, there is a dearth of knowledge of the long term behaviour of geosynthetics in extreme conditions, and predictions of performance can not be made with confidence.

The paper provides an understanding of the mechanisms that reduce the durability of geosynthetics, methods used to assess the effects of different degrading mechanisms, how degradation might be negated or reduced, and provides an overview of the methods for predicting the performance of a geosynthetic. Data from three in-service structures and, tests on plastics naturally aged for thirty years, are presented. Current design procedures are discussed and methods for developing more cost-effective designs are suggested.

1 INTRODUCTION

The primary engineering functions of a geosynthetic are often summarised as liner, separator, filter, drainage and reinforcement. The required properties for each function are different, but in all applications durability is essential.

This paper considers aspects of durability required for geosynthetic soil reinforcement for permanent walls and slopes; many of these aspects are relevant to other functions. The aim of this paper is to provide an understanding of the mechanisms that reduce the durability of geosynthetics, methods used to assess the effects of different degrading mechanisms, how degradation might be negated or reduced, and provide an overview of the methods for predicting the performance of a geosynthetic. Data obtained from three in-service structures and, plastics naturally aged for 30 years, are presented, and a possible route to increasing the efficiency of design procedures is discussed.

1.1 *Background*

The rapid increase in the usage of geosynthetics within the civil engineering industry over the last thirty years is a success story that is familiar to us all. For example, in the 1970s the North American geosynthetic market grew from about 2 to 90Mm². In 1984 the world market was estimated to be about 200Mm² and subsequently usage has increased at a fairly constant rate of about 63Mm² per annum to about 1,100Mm² in 2000; a graph showing the world wide usage of geosynthetics over the last 30 years is shown in Figure 1. We should remember that it was not civil engineers but textile manufacturers who introduced geosynthetics to the world and demonstrated their effectiveness and versatility.

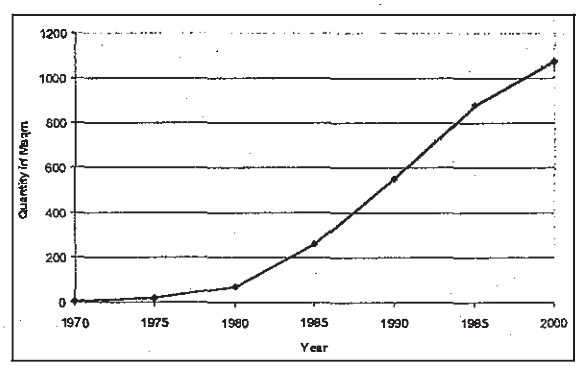


Figure 1 World-wide growth in the usage of geosynthetics

The historical development of geosynthetics has been described by Giroud (1986), who identified some of the reasons behind the rapid acceptance of these new materials. The primary reason stemmed from construction expediency:

i) incorporating geosynthetics into a soil structure can significantly reduce the volume of soil required and may permit the use of a poor quality fill,
ii) the presence of a geosynthetic can mitigate any localised abnormalities in the soil properties, and
iii) the layered construction methods normally used with soils are ideally suited to the installation of geosynthetics which are essentially a two dimensional product.

Soil reinforcement.
The concept of reinforced soil is simple and has been used by man for many years and in a variety of different forms (Jones, 1996). Possibly the earliest known usage was in the Agar-Quf ziggurat believed to be over 3000 years old (Bagir, 1944); more recently, in 1822, a comprehensive series of trials were undertaken by the British Army to investigate the benefits of reinforced soil (Pasley, 1822). The reinforcements used in both of these examples were derived from naturally occurring materials which, though adequate for long term applications in a hot dry climate, or short term applications in a more temperate climate would have suffered severe problems of durability if used in other instances. In the 1960s the Reinforced Earth system was developed which utilised durable metallic reinforcements (Vidal, 1966). Since then other reinforced soil systems have been developed, most of which use geosynthetic reinforcements

The performance of a reinforced soil structure is critically dependent on the ability of the reinforcement to support the design load throughout its service life. The tensile strength of geosynthetic reinforcement may decrease with time as a result of various degenerative agencies, and careful consideration of the durability of the reinforcement is an essential design requirement. The factors that affect the durability of soil reinforcements are presented in Table 1.

Table 1 Factors affecting the durability and performance of buried soil reinforcement materials (reproduced from BS 8006, 1995)

Material	General factors	Special factors
Metallic	Loading Water Damage	Bacterial/electrochemical composition of soil Corrosive fluids
Polymeric	Loading Damage UV exposure Temperature	Chemical/bacterial composition of soil Aggressive fluids

To fully understand the degradation mechanisms of geosynthetics, it is necessary to consider the manufacture and make up of these polymeric materials.

2 MANUFACTURE OF POLYMERS

2.1 *Introduction*

About 80 per cent of the world's output of organic chemicals is used to make polymers. The manufacture and use of synthetic polymers (as opposed to naturally occurring polymers such as proteins, cellulose and rubber) in any significant quantity started during the 1940s, and since 1945 their use has increased rapidly. Today, synthetic polymers are often used in preference to other materials because chemical engineers are able to produce polymers cheaply that have specific properties appropriate to particular applications.

Polymers are long chain molecules, with a backbone of carbon atoms comprising many small molecules called monomers. The physical properties of a polymer are principally dependent on the molecular weight (chain length), inter molecular bonding, and degree of crystallinity. The properties of a polymer can be substantially modified by additives incorporated into the polymer mix during manufacture. The physical properties of different polymers, such as strength and flexibility, can vary widely.

2.2 *Polymerisation*

The manufacture of a polymer from the constituent basic chemicals is termed polymerisation; there are two types of polymerisation reaction.

Addition polymerisation
As the name implies, the process involves an addition reaction between two monomers, usually of the same type, under fixed conditions of temperature and pressure and in the presence of a catalyst. An example of addition polymerisation is the manufacture of polyethylene (PE), which was first manufactured in 1933 by the then Imperial Chemicals Industry (ICI). The reaction occurred at a high temperature and pressure in the presence of oxygen. The resulting polymer had many branches on the chain and thus the molecules couldn't pack closely together, and was termed low density polythene (LDPE) which melts at about 105°C. A second method of manufacturing PE was discovered by Ziegler in the 1950s. Polymerisation occurred at a lower temperature and pressure than the older

process, and utilised catalysts. The resulting polymer had fewer branches on the carbon chain allowing a closer packing of the molecules, and was termed high density polythene (HDPE), this is less ductile than LDPE and melts at about 135°C.

Condensation polymerisation

This process usually involves two different monomers joining together to form the carbon chain. During the reaction a small molecule such as water or ammonia is eliminated from between the two larger molecules (hence the term condensation reaction). An example of condensation polymerisation is the manufacture of polyethylene terephthalate (PET), commonly known as polyester, from ethylene glycol and terephthalic acid. PET combines a high strength and chemical stability, and thus it is often used as a substitute for natural fibres.

2.3 *Additives*

Additives are used to ease the manufacturing processes and to improve the performance of the end product; they are normally added for one or more of the following reasons:

i) To increase lubricity during manufacture, i.e. increase the workability of the polymer thereby reducing the amount of energy required and mechanical degradation of the polymer during processing, and also the wear and tear on the machinery.
ii) To stabilise the chemical structure of the polymer, by wholly or partially combating any degradation that might result from the high temperatures used during manufacture, and through the in-service environment, e.g. anti-oxidants.
iii) To act as a filler, i.e. a relatively inert compound which improves the mechanical or thermal properties of the polymer, and/or a low cost material used to increase the volume of the polymer. Carbon black is the most widely used additive with geosynthetics, as it combines several of the above properties.

Additives may be introduced during polymerisation. More usually, additives in the form of a powder or as granules are mixed with the polymerised material, immediately prior to the extrusion process. A homogeneous mix, essential for a consistent predictable performance of the end product, may be achieved by the raw materials being ground together. This mixing process, often referred to as compounding, takes place at a temperature high enough to soften the different components, but below the melting point of the polymer, to avoid agglomeration and a non-uniform mix. The content, volume and compounding of the additive package is carefully controlled. Typical problems include: (i) incomplete mixing, which may result in the formation of flecks (small hard particles) in the polymer, (ii) insufficient additives, which when processing at high temperatures prevent heat degradation that may result in discoloured areas where the polymer is unstabilised, and (iii) degradation of the additives themselves, causing minute bubbles to form in the polymer.

2.4 *Processing*

The extrusion process primarily consists of a heated single or twin screw extruder that takes the raw materials in the form of pellets or powder and forces the heated mixture through a die at a continuous rate; the extrusion product may be drawn before being cooled and stored for later use. A sectional drawing of an extruder is shown in Figure 2. The process of drawing stretches the polymer reducing its cross sectional area, and creates a more aligned molecular structure thereby increasing the strength and modulus of the polymer. The amount a polymer is extended is defined as the draw ratio, that is the ratio of the thickness of the die aperture to the thickness of the product. The drawing process is critical; it substantially affects the physical characteristics of the end product. Drawing takes place while the material is still hot; typically draw ratios of up to 5 are used for sheet products but ratios as high as 70 might be used for high tenacity filaments.

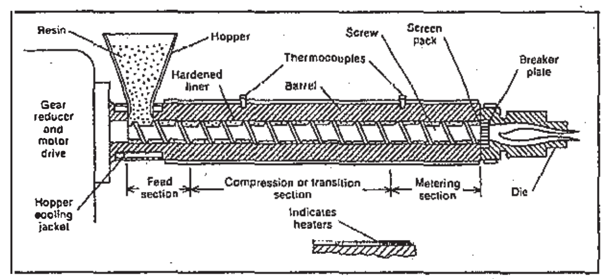


Figure 2 A typical screw extruder

The extrusion product is the basic material from which the geosynthetic is produced. A description of the processing of geosynthetics is provided by Van Zanten (1986). Detailed descriptions of the make up and processing of polymeric materials are provided by Miller (1997).

2.5 *Polymer type*

Polymers may be separated into two groups, i.e. thermoplastic and thermoset polymers.

- Thermoplastic polymers may be repeatedly softened and hardened by increasing and decreasing their temperature. The polymers can be shaped by extrusion and drawing, or moulded into a specific shape. All geosynthetic products are manufactured from thermoplastic polymers.

- Thermoset polymers have a high level of cross-linking between molecules. Once cured, by means of heat or chemical reaction, thermoset polymers are non ductile and can not be re-moulded.

Polymers contain both crystalline regions, where the molecules are regularly arranged and closely packed, and amorphous regions where there is no ordering and the molecules are randomly oriented. By and large the behaviour of the molecules in the amorphous regions dictate the physical properties of a polymer. Thermoplastic polymers are less crystalline than thermoset polymers. A schematic representation of the crystalline and amorphous regions within a polymer is shown in Figure 3. In partially crystalline polymers, such as thermoplastic materials, the majority of the chains link one crystal to another, and these tie molecules carry the load supported by the reinforcement.

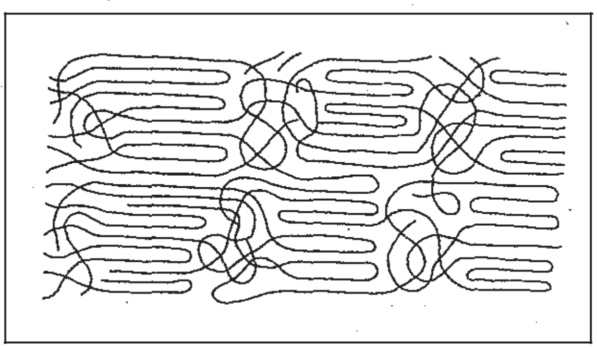


Figure 3 Amorphous and crystalline regions within a thermo plastic polymer

2.6 *Geosynthetics*

The polymers most widely used for the manufacture of geosynthetics, density (ρ), glass transition temperature (T_g) range, and the chemical formulae of the repeat units from which the polymer chain is formed, are listed in Table 2.

2.7 *Terminology*

Glass transition temperature. The midpoint of the temperature range at which a reversible change takes place in the amorphous regions within a polymer, from a relatively hard and brittle state to a viscous or rubbery state.

Cross-links. Linking between polymer chains that increases the stiffness and reduces the flexibility of the end product.

Branches. Side chains attached to the main polymer chain. Highly branched polymers can not pack closely together, and therefore tend to have low densities, strengths and melting points.

Chain length. The length of the linear polymer molecule, normally expressed as the number of repeated units in the chain. The strength, modulus, durability and melting point of a polymer increase with chain length. Individual molecules of the same polymer can vary in chain length, thus it is usual to talk about an average chain length.

Molecular weight. The sum of the atomic weights of all the atoms in a molecule. For a polymer this is a measure of the chain length. The molecular weight of a polymer is normally determined from an estimation of the number of carboxyl end groups (CEG).

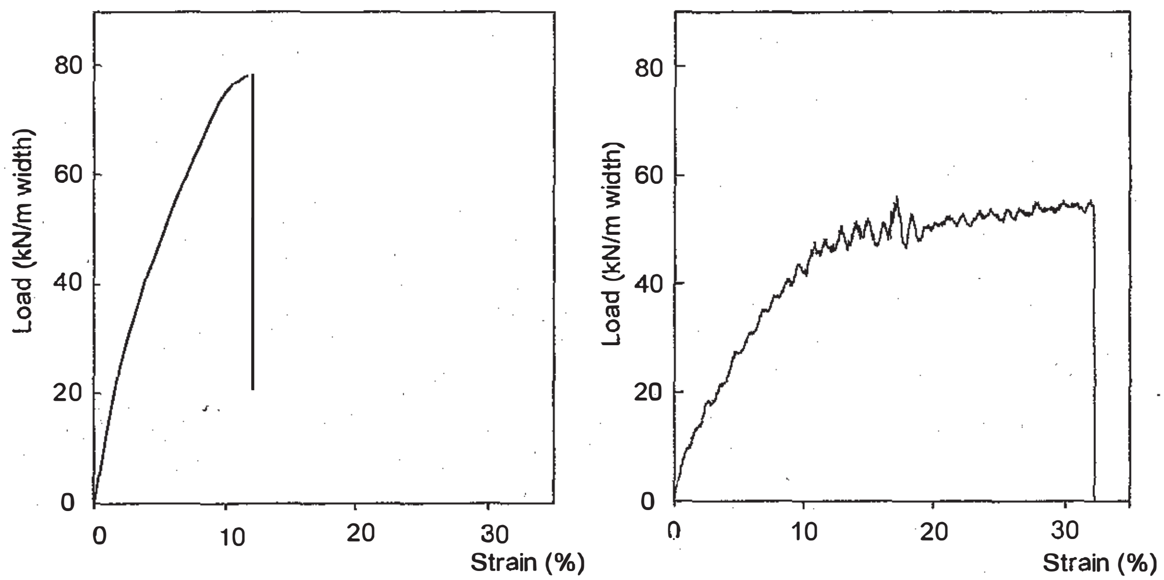
3 DURABILITY

A material that does not change in appearance or undergo any change to its properties with time may be said to be durable for the given conditions; however, materials do not last forever and such changes will inevitably occur. Thus a time period is normally implied or defined when considering the durability of a material. The magnitude of the change in a time interval is a measure of the material's durability, and the magnitude depends on the ambient conditions and the form of the material itself. Durability is a function of fitness for purpose. A reduction in fitness for purpose is caused by degradation.

Durability may be defined as the ability to successfully perform a design function for a specified minimum period of time; for soil reinforcement the function is to support a tensile load. A designer must be satisfied that a reinforcement is fit for purpose, this necessitates an assessment of the in-service conditions and the performance of the geosynthetic. Unfortunately, the degradation mechanisms involved are complex and do not allow the effect on geosynthetics to be readily predicted; however, an excellent summary of the factors affecting durability and how they might be assessed is provided in CR ISO 13434 (1998).

Table 2 Polymers most widely used for the manufacture of geosynthetics

Polymer	ρ (g/cm^3)	T_g (°C)	Repeat unit
Polyethylene (PE)	0.95	-100 to −70	$-CH_2.CH_2-$
Polypropylene (PP)	0.91	-20 to−12	$-CH_2.CHCH_3-$
Polyethylene terephthalate (PET)	1.38	220 to 250	$-CH_2.CH_2.O.CO.\circledcirc.CO.O-$
Polyamide (PA) 6	1.13	40 to 60	$-NH(CH_2)_5.CO.NH.(CH_2)_5.CO.NH-$
Polyamide (PA) 6.6	1.13	40 to 60	$-NH.(CH_2)_6.NH.CO.(CH_2)_4.CO-$
Polyvinyl chloride (PVC)	1.40	-25 to 100	$-CH_2.CHCl-$



(a) 200mm wide specimens, tested at 10% per minute

(b) 200mm wide specimens, tested at 2.5% per hour

Figure 4 The effect of the rate of test on the measured tensile properties of an HDPE geogrid

4 DEGRADATION

Degradation may occur at a microscopic level (e.g. biodegradation and chemical attack), or at macroscopic scale (e.g. mechanical damage). Different deleterious agencies dominate the degradation of a geosynthetic at different stages of its life

- Above ground
  weathering: exposure to ultra violet (UV) light, extremes of temperature and humidity
- Installation
  mechanical damage: cuts, nicks, abrasion, breaks, tears
- Below ground (in-service)
  environmental degradation: load related effects, high/low pH, contaminants in the soil resulting in oxidation, hydrolysis, temperature effects

However, in practice, the degradation processes can not be clearly separated as indicated above. A consideration of the different degradation mechanisms, in no particular order, is presented in the remainder of this Section.

4.1 *Load-related effects*

In partially crystalline polymers, such as thermoplastic materials, the majority of the chains link one crystallite to another, and these *tie* molecules carry the load supported by the reinforcement. In time these molecules unravel and align themselves in the direction of the load. This time dependent extension is termed creep.

The rate of deformation of a visco-elastic material under an applied load, i.e. creep, increases with ambient temperature and the magnitude of the load. The importance of the visco-elastic properties to design, was recognised early in the development of geosynthetic, and led to the development of tests to assess the relation of tensile load and extension with time, and how the relation varies with temperature.

The relation between stress (σ) and strain (ε) for an elastic body follows the simple relation:

$$\sigma = E.\varepsilon$$

where: E is the modulus of elasticity

However, the response of a visco-elastic material to an applied stress is a function (f) of both strain and time (t), and because the behaviour of polymer materials are dependent on temperature (T):

$$\sigma = f(\varepsilon,t,T)$$

The time dependent behaviour of a geosynthetic is illustrated by the results of tensile tests undertaken at different rates of strain on an HDPE grid shown graphically in Figure 4 (Watts and Brady, 1994). The tensile strength of the grid, measured at the slower rate of strain, was slightly lower than when measured at the higher strain rate; however the strain at rupture was about twice that measured in the faster test. The high rupture strain resulted from the test specimen continuing to strain under a constant stress approximately equal to the strength of the grid; in effect the grid was undergoing cold drawing.

In the context of durability, creep is viewed as having a deleterious action on the long term performance of the reinforcement. However, in certain instances, creep can be of benefit to the engineer. For example: after the normal post construction movements associated with a reinforced soil wall have ceased, any actions that reduce the stability of the wall are likely to result in further creep of the reinforcement and further movement of the wall. The plasticity of the geosynthetic provides a level of tolerance to disturbing actions, allowing the wall to move and signalling that something may be wrong.

4.1.1 *Creep strain and creep rupture*
The creep performance of a geosynthetic reinforcement should be determined in accordance with the standard method of test EN ISO 13431 (1999).

Measurement of creep strain. Essentially the extension of a test specimen, under a constant tensile load, is measured with time. The test is repeated with a range of applied loads, up to 60 per cent of the ultimate tensile load (determined in accordance with EN ISO 10319, 1996).

A schematic representation of the total strain response for a typical polymer is presented in Figure 5. The curve comprises four regions that may be arbitrarily defined:

- Initial creep strain: includes elastic recoverable strain and plastic irrecoverable strain.
- Primary creep strain: some strain would be recovered if the specimen was allowed to relax.
- Secondary creep strain: constant rate of increase of strain with the logarithm of time.
- Tertiary creep strain: leading to rupture of the test specimen

Boundaries between the different regions on a creep curve can not be defined, but serve to help engineers and scientists compare and discuss the characteristics of polymer products

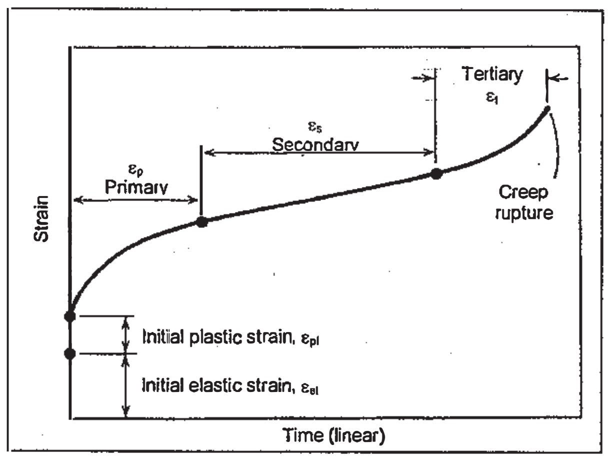


Figure 5 The total strain response of a polymeric material

Measurement of creep rupture. A constant tensile load is applied to the test specimens, and the time to rupture is recorded. Tests are undertaken at a range of loads from 50 to 90 per cent of the ultimate tensile strength of the material.

There is a plethora of published information on phenomenon of the creep of geosynthetics, how it is measured and how the data can be presented; e.g. Watts et al (1998) which also demonstrates how the data may be usefully presented, and seeks to identify many of the pitfalls associated with undertaking these apparently simple tests.

4.2 *Environmental and microbiological degradation*

4.2.1 *Weathering*

Polymeric materials are susceptible to degradation from ultra violet light. Approximately 6 per cent of the light reaching earth from the sun is in the ultra violet (UV) range of the spectrum. Degradation occurs when energy levels of the photons (dependent on the wavelength of the incident light) correspond to a particular bond energy within the molecular structure resulting in the rupture of those bonds. The rate of degradation is dependent on the intensity of the light and may be increased in the presence of moisture that absorbs thermal energy from UV light. Different chemical bonds are susceptible to different wavelengths; and hence the degradation varies with the wavelength of the incident light. Degradation is initiated by the action of the UV photons and proceeds by reaction with oxygen in exactly the same way as for oxidation described in the next Section. The reaction is self propagating:

Initiation (stage 1)

$$R–R + h\nu \rightarrow R^{o} + R^{o}$$

Peroxide formation (stage 2)

$$R^{o} + O_2 \rightarrow R.OO^{o}$$

Propagation, and formation of hydroxy peroxide (stage 3)

$$R.OO^{o} + R–H \rightarrow R^{o} + R.OOH$$

where: R-R represents two adjacent molecules in the polymer chain
$h\nu$ is the energy of the UV light photons
^{o} indicates a free radical

Resistance to UV degradation is provided by additives that prevent light penetrating the material and act as light stabilisers, and additives such as carbon black which also act as lubricants, pigments and fillers. Stabilisers are also used to prevent the formation of free radicals resulting from the reaction with oxygen, as described in the next Section.

The resistance to weathering should be determined in accordance with EN 12224 (2000). In the test, the test specimen is exposed to artificial UV light for a fixed period, at elevated temperature, with wet and dry cycles.

4.2.2 *Oxidation*

All polymers are susceptible to degradation by reacting with the oxygen in the atmosphere; the reaction rate increases with temperature. Oxygen reacts with free radicals on or at the end of a polymer chain. There are three stages to the self propagating degradation reaction:

Initiation (stage 1)

$$R–H \rightarrow R^{o} + H^{o}$$

Peroxide formation (stage 2)

$$R^{o} + O_2 \rightarrow R.OO^{o}$$

Propagation, and formation of hydroxy peroxide (stage 3)

$$R.OO^{o} + R–H \rightarrow R^{o} + R.OOH$$

The hydroxy peroxide may decompose to form further free radicals

where: R-H represents a polymer molecule
^{o} indicates a free radical

The rate of degradation depends on temperature, in-service environment and polymer type. The effects of oxidation are:

- Discolouration (may be obscured by the presence of carbon black filler), shiny surfaces may be dulled
- Cross linking of the molecules leading to an increase in surface hardness
- Coating of the oxidation product on the surface of the material
- Break up of the polymer chains leading to a reduction in molecular weight

Of the polymers commonly used to manufacture geosynthetics, PE and PP are the most susceptible to oxidation. The rate of reaction is dependent on temperature and the availability of oxygen, but can be increased by the presence of transition metal ions such as Fe^{3+} and Cu^{2+} which act as catalysts to the reaction. The tendency of the polymer to oxidise may be reduced where limited oxygen is available, e.g. during in-service conditions.

Other polymers, such as PET and PA, are much less susceptible to oxidation than PE and PP, but the rate of reaction may increase in the presence of heavy metals and in acidic conditions. Additives can reduce the level of degradation over the service life to an acceptable level. However, anti-oxidant stabilisers within a polymer product, may themselves degrade and/or leach out over a period of years, leaving the polymer susceptible to oxidation..

Anti-oxidant stabilisers used to prevent thermal degradation work in two different ways: (i) by providing hydrogen atoms, that reacts with the peroxide before propagation can take place, and (ii) by reacting with hydroxy peroxide to prevent their decomposition which generates further free radicals.

The resistance to oxidation should be determined in accordance with ENV ISO 13438 (1999). Test specimens are exposed to oxygen at high temperature in an oven. A thorough appreciation of oxidation of geosynthetics is provided by Elias et al (1999).

4.2.3 *Hydrolysis*
PET is susceptible to hydrolysis, and PA to a lesser extent. The action of hydrolysis ruptures the ester bond in the long chain polymer molecules, resulting in a loss in strength and strain at rupture. As explained in Section 2, esters are the product of a condensation reaction between an alcohol and a carboxylic acid, water is emitted as a by product; i.e.:

Alcohol + Carboxylic acid ↔ Ester + Water

Esters are stable in dry conditions, but in a warm humid environment the reaction may be reversed (i.e. hydrolysis); PET is susceptible to degradation in these conditions, which leads to break up of the polymer chain and reduction in molecular weight. The potential for hydrolysis to occur in PET geosynthetics may be reduced by increasing the draw ratio during processing.

Hydrolysis can take two different forms. Internal hydrolysis attacks the whole of the fibre and occurs in aqueous solution or conditions of high humidity, at all values of pH. External hydrolysis which has the appearance of surface etching occurs in alkaline conditions at pH>10 particularly in the presence of Ca^{2+}. Both forms of hydrolysis are very slow at low temperatures, and except in extreme conditions may be discounted as a significant source of degradation in Northern Europe; however, hydrolysis can be a source of degradation in warm humid climates.

The resistance to hydrolysis should be determined in accordance with ENV 12447 (1997). Test specimens are immersed in liquid for periods of up to 90 days. A thorough appreciation of the hydrolysis of geosynthetics is provided by Elias et al (1999).

4.2.4 *Chemically aggressive liquids*
The two forms of chemical attack, oxidation and hydrolysis, have been discussed above. However, degradation in an aqueous solution of other aggressive chemicals is also possible. A standard test has been developed for conditions of acidic or alkaline contamination, or where dissolved oxygen might be present. The test procedure could also be used as a protocol for testing any aggressive liquid.

The resistance to degradation by liquids should be determined in accordance with EN 12960 (2000). Test specimens are immersed in liquid at an elevated temperature for three days.

4.2.5 *Microbiological*
Microbes, bacteria and fungi live in soils and may attack a geosynthetic in aerobic or anaerobic conditions. However to date there has been no reported attack on in-service material. Nevertheless the possibility of microbial action should not be neglected.

The resistance to microbial degradation should be determined in accordance with EN 12225 (2000). Test specimens are buried in a biologically active soil for a given time period.

4.3 *Mechanical*
In most instances installation damage is likely to be the most severe source of degradation (Watts and Brady, 1990 and 1994). The severity of the damage increases with the particle size and angularity of the soil and with the applied compactive effort. The severity of damage is also related to the form, surface texture and thickness of the geosynthetic. An example of severe installation damage is shown in Figure 6.

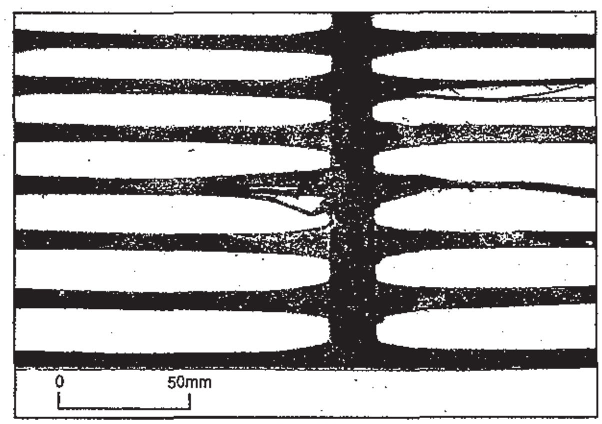


Figure 6 Severe installation damage to a geogrid

Because installation damage is heavily dependent on the geosynthetic, soil and construction conditions it is strongly recommended that the resistance to mechanical damage is determined from the results of site specific tests. These should be undertaken in accordance with Annex D to BS 8006 (1995) or Watts and Brady (1990).

The index test ENV ISO 10722-1 (1998) should not be used for determining the resistance to installation damage, but it can be used to compare the performance of similar materials.

Other tests. It should be noted that there are additional tests, some of which are intended to model specific site conditions, though how the test results relate to on-site degradation is unclear; the most well known of these tests are:

Static (CBR) Puncture	EN ISO 12236 (1996)
Cone drop	EN 918 (1995)
Resistance to abrasion	EN ISO 13427 (1998)

4.4 *Synergy*

The effect of degradation on the load bearing behaviour of a reinforcement cannot be satisfactorily quantified. In all probability mechanical damage will affect creep behaviour but there is insufficient published information to quantify its effect.

Environmental stress cracking (ESC), that is the embrittlement of the geosynthetic caused by effects of environmental degradation and tensile load, will reduce the stress rupture lifetime but has little effect on the measurement of creep strain. (CR ISO 13434, 1998). Effects are more apparent in amorphous polymers (e.g. PVC) than semi-crystalline ones (e.g. PE and PP). Though the latter are not immune, modern manufacturing and processing techniques have made these polymers resistant to ESC.

5 COMBATING DEGRADATION

Though modern polymers are inherently stable, the manufacturer, supplier and end user all have a responsibility to ensure the longevity of a geosynthetic. Degradation can not be avoided, but efforts can be made to minimise its effects.

The battle against degradation starts during manufacture when the additive package is introduced into the polymer to optimise performance for the intended end use; the precise composition of the package is crucial to the performance of the product. Following production, the geosynthetic will be stored before delivery to site. On site, inappropriate storage conditions, abuse and mis-handling can all contribute to degradation of the geosynthetic. The product should be stored off the ground in cool dry conditions, away from direct sunlight, and where it will not be disturbed until required for construction. Manufacturers provide guidelines for the handling and storage of their products; their instructions should be adhered to at all times.

The correct procedures for installing geosynthetic reinforcements can not be described herein. Reference should be made to standard installation procedures issued by the manufacturer/supplier and the specification for the works in hand. However, it may be apposite to list the principal items, observance of which can save effort, time and money.

i) care should be taken avoid damage when moving or lifting the material,
ii) the material should be laid flat on a prepared surface,
iii) the specification for placement and compaction of the fill should be adhered to,
iv) site traffic should be routed around the region of reinforced soil,
v) the geosynthetic should be covered with soil as soon as possible to minimise UV degradation.

It is recommended that site operatives should be made aware of the importance of adhering to the correct handling and installation procedures, and operations should be supervised.

6 ASSESSMENT OF DEGRADATION AND PREDICTION OF PERFORMANCE

The standard tests discussed in Section 4.2 for climatic and environmental degradation were developed to simulate the level of deterioration that might be encountered in a service life of 25 years. For service lives in excess of this period the test conditions may be made more severe by increasing the concentration of the degrading specie and/or the duration of the exposure of the specimen. Alternatively a prediction of performance may be made by using the technique described in Section 6.2.3 for determining the rate of degradation reaction.

Most of the screening tests used to assess the resistance to degradation depend on an increase in the concentration of the aggressive medium and accelerating the reaction by testing at elevated temperatures. Such techniques are well established but there are pitfalls. There are two principal rules that must not be transgressed without proof that it is reasonable to do so: (i) ensure that the degradation mechanism is the same throughout all the tests, and (ii) ensure the range of test temperatures does not span a temperature at which a change occurs to the material, such as the glass transition temperature. Further advice and guidance is available in the literature, e.g. CR ISO 13434 (1998).

(It must be remembered that performance and propensity for damage of a geosynthetic are dependent on its form and structure. Therefore, when performing any test on a composite product it is strongly advised that the material is tested as a whole.)

6.1 *Assessment of degradation*

Loading (visco-elastic) effects

Extrapolation of the results of both creep and creep rupture tests, will enable a prediction to be made of the maximum sustained load that will lead to rupture at the end of the service life and the strain at rupture. Such predictions may be made using the techniques outlined in Sections 6.2.

Environmental tests

The loss in durability of a geosynthetic specimen should be evaluated in accordance with EN 12226 (2000). The effects of degradation are evaluated by visual inspection (microscopic inspection if required) and the percentage retained strength and elongation at rupture.

Mechanical damage

The loss in durability should be expressed as the percentage retained tensile strength. The strength of the damaged and control specimens should be determined in accordance with EN ISO 10319 (1996).

Installation damage and long term creep are the prime cause of degradation, except for highly aggressive in-service conditions the effects of all other degrading mechanisms may be ignored. (Note the index test, ENV ISO 10722-1 (1998) should not be used to estimate the retained strength of a reinforcement installed on site, because the test conditions are not representative of site conditions.)

6.2 *Predicting performance*

The designer of a reinforced soil structure needs to evaluate the properties of the geosynthetic at the end of the design life. For permanent works, the design life of the structure is far in excess of the duration of available test data, thus the behaviour must be predicted by extrapolation. But extrapolation is fraught with pitfalls and can lead to inaccurate assessments, thus the longer the time period for which data are available the better.

Accelerated tests, or rapid ageing tests, undertaken at temperatures substantially above the service temperature permit the results of long term tests at low temperatures to be predicted from the results of short term tests at high temperatures.

6.2.1 *Time-temperature superposition.*

Tests to determine the long term creep performance of geosynthetics, or the effects of a degrading mechanism at or near service temperature, require an impracticably long test duration. The results of such tests may be extrapolated by eye, but considerably more confidence may be placed in curve fitting and mathematical techniques. It is important to check that the predicted loads and strains are attainable, i.e. they are below the values at which the geosynthetic would rupture.

A popular method for predicting long term performance is to utilise the principle of time-temperature superposition. This technique is widely accepted and used throughout the geosynthetic industry for predicting long term performance. Essentially a series of short term tests are conducted at a selection of temperatures, higher than the anticipated in-service temperature. The graphical results may be *shifted* in relation to a reference temperature, normally ambient or in-service temperature, and used to generate a master curve that extends beyond the test period.

One of the problems encountered with time-temperature superposition (TTS) of creep results for geosynthetic materials is the uncertainty in the magnitude of the shift factors. The uncertainty can be reduced by testing multiple samples.

6.2.2 *Stepped isothermal method*

An approach that has been shown to overcome this problem is called the Stepped Isothermal Method (SIM) (Thornton et al, 1998[a]) and involves generating a master creep curve from a series of sequential tests, at increasing temperatures, on a single specimen. To date this method has been used for geosynthetics manufactured from polyester; an assessment of the applicability of the SIM for use with other polymers is underway. A comparison of conventional predictive methods and the SIM, from tests on 3 different (polyester) geosynthetics, has been reported by Thornton et al (1998[b]).

6.2.3 *Rate of reaction*

Many of the degradation mechanisms occur through chemical reactions between the geosynthetic and the surrounding environment. The rate of reaction may be determined, and from this the level of degradation that will occur by the end of the service life. A full description of this complex topic can not be given here but some of the concepts are discussed below.

The rate of a chemical reaction is proportional to temperature and the concentration of the reactants. The rate of reaction between two or more chemically reactive compounds is defined as the rate at which a product is formed, or the rate at which a reactant is consumed. It is important to specify which species is used to measure the rate because it may differ for the different substances involved in the reaction. Thus:

$$R = k.Q^n \qquad (1)$$

where R is rate of reaction
Q is the rate of change of concentration of a reactant
n is the order of the reaction
k is the rate constant

The rate constant for a reaction is dependent on temperature. In 1889 the Swedish chemist Svante Arrhenius defined the relation that has come to bear his name.

$$k = A.\exp^{-Ea/RT} \qquad (2)$$

where A is a constant for the reaction
E_a is the activation energy, constant for the reaction
R is the gas constant
T is the temperature in degrees Kelvin

A more useful form of the relation is obtained by taking the logarithm of both sides of the equation, i.e.

$$\log_e(k) = \log_e(A) - E_a/RT \qquad (3)$$

The Arrhenius equation is widely used to predict long term performance at low temperature from the results of short term tests at higher temperatures. Thus, for geosynthetics, it is used to determine the in-service degradation from tests undertaken at elevated temperatures (ageing tests). However, for the predictions to be valid the following conditions must apply: (i) the mechanism of degradation must be the same at all temperatures, and (ii) the physical form of the polymer should be the same at all temperatures.

Correct use of the Arrhenius equation is essential if predictions are to be reliable. A useful description of the methodology and pitfalls for predicting polymer degradation is provided by Koerner et al (1992).

The following is a brief description of how the Arrhenius equation might be used to provide information for an engineer intending to use a geosynthetic in aggressive environmental conditions.

The engineer will require to know either (a) what percentage of the material's strength will remain at the end of the service life, or (b) after what time the strength will reduce to the minimum acceptable value. Both calculations require the use of Equation (1) and thus the value of the rate constant (k) must be determined.

The constants in Equation (3), may be determined from the results of a series of laboratory tests. Specimens of the geosynthetic would be aged in a solution equivalent to, or of a stronger concentration than, the aggressive specie; at intervals, specimens would be retrieved and tested. The time required for the strength of the geosynthetic to reduce to the minimum permitted strength would be recorded and the rate constant would be determined. The test would be repeated at a range of temperatures. The results of the tests would be graphically presented by plotting $\log_e(k)$ against 1/T. The value of the constant A is determined from the intercept on the axis ($\log_e(k)$) and the slope of the graph is equal to $-E_a/R$, as shown in Figure 7. Thus the rate constant for a given temperature can be calculated using Equation (3).

In practice an engineer should consult the manufacturer/supplier of the reinforcement for advice.

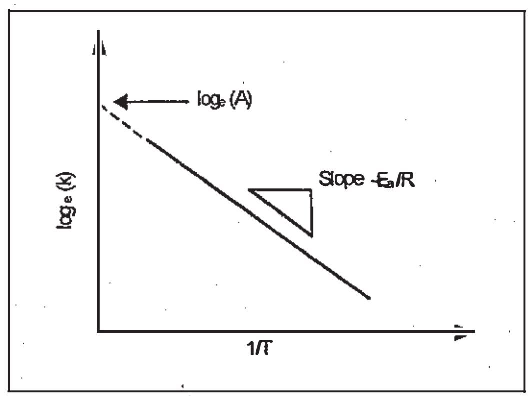


Figure 7 Determination of the constants in the Arrhenius equation

7 REDUCTION FACTORS

The philosophy used by most current design codes is limit state design based on the assessment of ultimate and serviceability limit states.

For a reinforced soil wall or earthwork the design load in a reinforcement is assumed to be constant throughout the design life, and is selected such that:

i) during the life of the structure the reinforcement shall not rupture, and

ii) at the end of the design life of the structure, strains in the reinforcement should not exceed a prescribed value.

The design load is taken to be the lesser of the extrapolated maximum creep rupture load at the end of the design life (T_{CR}) or, the extrapolated tensile load based upon the creep strain at the end of the design life (T_{CS}). (The design load for the basal reinforcement to an embankment on soft ground is taken to be the maximum load in the reinforcement, which is assumed to occur at the end of construction.)

The required design strength of a reinforcement (T_D) is equal to the design load reduced by a partial material factor (f_m):

$$T_D = \frac{T_{CR}}{f_m} \quad \text{or} \quad \frac{T_{CS}}{f_m}$$

The value of f_m is dependent on the properties of the geosynthetic and construction effects. The value is the product of component partial factors that account for the intrinsic properties of the material, and the reduction in strength due to construction effects and environmental degradation. Each of the sub-factors has numerical value equal to or greater than unity, and are numerically equal to the inverse of the fractional retained strength due to a loss in durability. A description of the component sub-factors is provided in the literature; by way of example the factors defined in BS 8006 (1995) Appendix A are summarised in Figure 8. Guidelines for determining the partial factors of safety for the durability of a geosynthetic reinforcement are provided in CR ISO 13434 (1998).

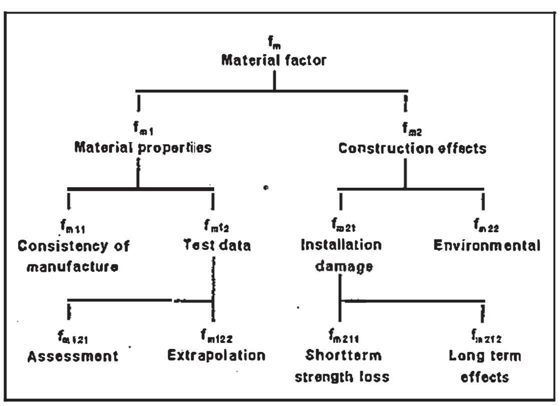


Figure 8 Derivation of f_m from the sub-factors

In most instances, the partial factors for creep and installation damage dominate the magnitude of f_m, others have negligible effect. Thus, given the inherent durability of modern geosynthetic products, unless high concentrations of aggressive specie are anticipated, design strengths may be determined on the basis of these two partial factors alone.

Note. Where predictions of strain are required the unfactored strain is multiplied by the appropriate partial factor for strain. This contrasts with predictions of strength where the unfactored strength is divided by the partial factor for strength. The partial factors for strength and strain are likely to be numerically different, and each should be determined in accordance with EN 12224 (2000).

8 REQUIREMENT FOR (INDEPENDENT) TESTING

The rapid acceptance and widespread usage of geosynthetics generated an urgent requirement for standard methods of test for determining the properties of the different products and provide engineers with the necessary knowledge to make an informed choice of a product for a specific purpose. As is usual in such circumstances, the development of the methods of assessment lagged behind the use of the products themselves. Different countries started to produce their own standard tests, many of which are still in use today despite the efforts of the organisations such as the International Standards Organisation (ISO) and the Comité Européen de Normalisation (CEN) to harmonise, and so reduce the number of similar tests. At the current time there are over 80 CEN standards relating to geosynthetics.

In-service conditions vary widely and it is impractical to develop standard performance tests to cover all situations, and therefore virtually all durability tests are index tests and not design or performance tests. It is important that a clear distinction is made between these two types of test.

Index tests
Such tests are usually inexpensive short term tests that don't require specialised equipment. The test results may be used to characterise a product or rank a selection of products, and may be used for quality control. Though some index tests may crudely relate to engineering behaviour the tests do not provide data that can be directly used in design. Index tests are often termed screening tests.

Design tests
Such tests seek to model in-service conditions and so provide data that can be used directly in a design. The tests may be of long duration, require specialised apparatus and may be expensive to undertake. Of the tests discussed in this paper only those recommended

for determining the resistance to installation damage, i.e. those described in BS 8006 (1995) and Watts and Brady (1990), are design tests. Design tests are often termed performance tests.

The most appropriate design tests are those that reproduce site conditions as closely as possible. The results of the tests allow the use of appropriate partial factors of safety resulting in a safe and yet efficient design. Site specific tests should be undertaken wherever possible, but they are all too rarely carried out for a variety of reasons, e.g. time constraints, lack of site investigation data etc. It is recommended that site specific tests for installation damage are always undertaken because of the relative importance of this factor in comparison to other sources of degradation.

Whatever tests have been completed, an awareness of the limitations of each test is crucial if appropriate use is to be made of the data. Blind acceptance of test results might lead to the selection of an inappropriate product with possible catastrophic consequences.

9 DESIGN

9.1 *Aspects of design*

The concept and construction of reinforced soil structures is simple and have been used for thousands of years. Until relatively recently designs were based principally on experience. Today, reinforced soil structures incorporating extensible (geosynthetic) reinforcement are usually designed with the tie back wedge method, which uses a mathematical model to determine the internal stability of the reinforced soil block. Currently there is no accepted method for modelling the serviceability of a structure, because the interaction mechanism between the reinforcement and the enveloping soil is not fully understood; estimates of deformation are still largely empirical.

It is a generally accepted that reinforced soil designs are overly conservative, i.e. the conditions assumed for design are substantially more onerous than measured in-service. This was demonstrated by Jenner (1990) who presented data from construction schemes in four different countries. He concluded that the reinforcement had a far greater impact on the behaviour of the retained soil than was explained by the design theory and suggested that this interaction should be investigated. (Some examples of predicted and measured data are provide in Section 10.)

Standard tests for determining tensile strength and creep behaviour do not take account of the interaction of the geosynthetic reinforcement with the soil. Research to determine the effect of soil confinement has been undertaken, e.g. Murray and McGown (1987), Chang et al (1993) and Chang et al (1996). These works have demonstrated that in-air tests do provide overly conservative results, but the degree of conservatism is not easily quantified. Problems encountered with confined tests include: strain compatibility between the soil and the reinforcement, and the boundary effects due to the practical limitations on the size of the test apparatus. To date the interaction mechanism between the reinforcement and the confining soil has not been modelled satisfactorily and, given the fundamental problems above, it is unlikely that we shall be able to do so by experimental means alone. Thus for the time being at least, structural predictions of deformation must continue to be mainly based on experience and empiricism.

Centrifuge modelling tests provide a versatile method of investigating the behaviour of reinforced soil structures, e.g. parametric studies can be undertaken and the criteria for achieving ultimate and serviceability limit states can be investigated. Test results could be used to calibrate designs and to determine realistic serviceability limit state criteria.

Current trends in design procedure are moving away from the traditional load/stress approach to a consideration of extension and strain. This move results partially from a requirement to be able to make improved predictions of construction and in-service movements and the desire to build more cost-effective structures. Such a move will necessitate modelling the soil reinforcement interaction mechanism which will require the use of numerical analysis. Any such design procedures will have to be checked against existing design methods and calibrated against the observed performance of in-service structures. It is expected that these new techniques will be developed and become accepted practice within two or three years.

9.2 *Knowledge for improved designs*

Pilot scale tests provide a wealth of information on constructibility and the performance of the structure and its integral parts. Tests are site specific; it is uneconomic to undertake a wide range of different trials, but it may be possible to determine upper and lower bounds to factors such as structural movements. Such trials may be instrumented to maximise the amount of information that may be obtained from one test, but this should be tempered with the knowledge that the inclusion of instrumentation in soil will alter the response of the soil to external load. Care must be exercised when interpreting in-soil measurements.

The instrumentation of a full scale structure, regular monitoring and the interpretation of data, is expensive. Such information is invaluable; it may provide confidence in design and lead to more cost-effective design procedures with the spin off that more reinforced soil structures may be built. Funding for such work is hard to find, but the potential benefits can outweigh the financial outlay.

It is strongly recommended that authorities responsible for the construction and/or maintenance of reinforced soil structures carry out a programme of monitoring. Regular, simple, line and level surveys obtained throughout the service life provide a useful database that can be used to confirm stability and provide an indication of the long term creep performance of a reinforcement. Such surveys should be undertaken as a part of regular maintenance inspections.

There is a dearth of information on the in service durability of polymeric materials. Valuable knowledge can be gleaned by testing samples of exhumed geosynthetics for change in strength, loss of additives, enviromnental degradation etc.; but the benefits of such research is reduced if details of the as-supplied material are not available. Engineers should be aware of the need for such information and be encouraged to inform the manufacturer, or other interested parties, of any opportunity to recover old materials. It is only by doing this that an understanding of the in-service durability of geosynthetics will be obtained.

The most farsighted and enlightened document that pursues and expands these ideas is Eurocode 7: Part 1 (1994). The code requires the preparation of formal reports that define the design strategy and follow up the design into the construction stage and beyond. The reports are:

- Ground investigation report (GIR). The aim of the report is to record all the geotechnical information from a construction site, the results of all relevant field and laboratory tests, and the evaluation of the geotechnical parameters.
- Geotechnical Design Report (GDR). The report is a comprehensive record of all details relevant to the geotechnical aspects of construction, and including as-built drawings. The report is required to include a plan for the supervision, monitoring and maintenance during and after construction. (The report also encourages designers to monitor structures, to build up databases of actual performance, check the validity of predictions of performance made during design, and ensure that the structure will continue to perform as required after completion.)

The GIR and GDR are not targeted solely at reinforced soil works. At first glance the requirements of the Eurocode seem to represent much extra work; but the effort involved could be substantially reduced by the introduction of a pre-formatted document and checklists etc. The implementation of such a scheme and collation of the reports might not be straightforward, and might best be co-ordinated at national level by professional institutions using modern electronic methods of archiving and storage. Such a resource would represent an invaluable source of practical experience to both practising engineers and researchers, while encouraging improved good practice throughout the industry.

10 SITE DATA

This Section briefly summarises the results from three studies of the long term behaviour of reinforced soil structures, and an investigation of the performance of 30 year old samples of polymeric materials stored in three different climatic conditions.

10.1 *Basally reinforced soil embankments*

The technique for construction requires the embankment to be built on a prepared foundation of free draining granular material containing the reinforcement; the material may be laid directly on the soft ground. The reinforcement resists the outward lateral thrust, a proportion of which is resisted through the side slopes and underlying soil. Tensile forces generated in the reinforcement resist the tendency of the embankment to spread and the inward acting shear stresses generated at the interface of the granular layer and the underlying soil. The action of the reinforcement improves the bearing capacity of the embankment; the degree of improvement increases with the ratio of the width of the embankment to the depth of soft soil, and so the effect may be substantial for embankments constructed on a thin stratum of soft soil.

In the latter half of the 1980s the new dual alignment of the A414 trunk road, some 20 miles North of London, required the construction of a reinforced soil embankments where it passed over weak compressible ground at Stanstead Abbotts and Harlow. The design, instrumentation, construction and performance of the embankments were described by Brady et al (1996). At both sites, a basal layer of Tensar SR2 geogrid (supplied by Netlon L^{td}, 1986) was used to improve the stability of the embankments during construction. Typical data from a short term tensile test and isochronous curves for Tensar SR2 are reproduced in Figure 9.

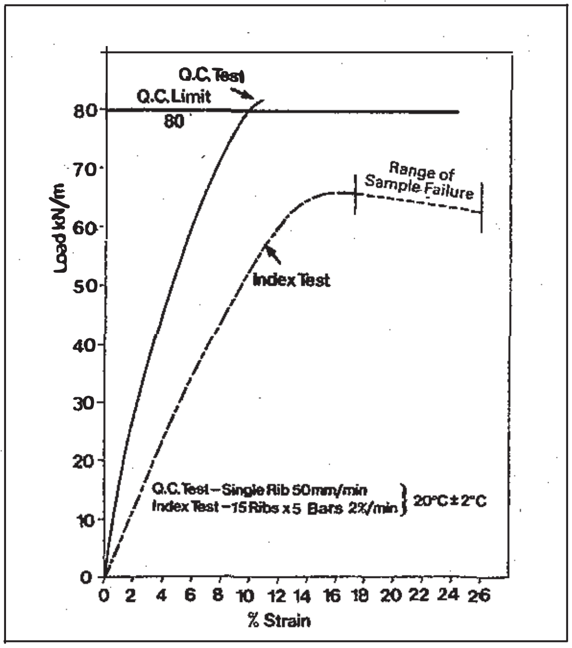


Figure 9a Typical tensile test results for Tensar SR2 (reproduced from Netlon Ltd, 1986)

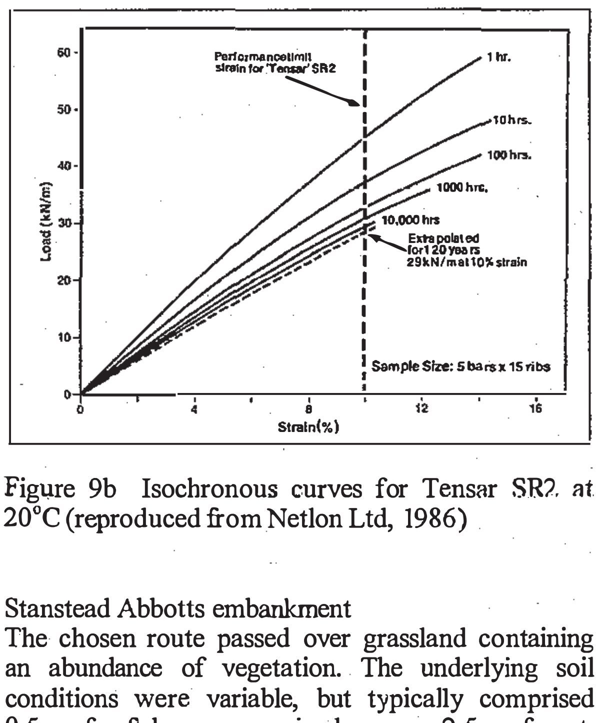


Figure 9b Isochronous curves for Tensar SR2 at 20°C (reproduced from Netlon Ltd, 1986)

Stanstead Abbotts embankment

The chosen route passed over grassland containing an abundance of vegetation. The underlying soil conditions were variable, but typically comprised 0.5m of soft brown organic clay, over 2.5m of peat, over 1.2m of grey organic silty clay, over sandy gravel with chalk bedrock at depth.

Different construction options were considered, ranging from a viaduct to a reinforced soil embankment. The out-turn figure for the embankment and the estimated costs for the other construction options are show in Table 3.

Table 3 Stanstead Abbotts: Construction options and associated cost

Construction option	Cost (£M) at 1984/85 prices
Viaduct	9.90
Excavate peat and replace with imported material and then construct an embankment	4.80
Excavate peat and replace with site won material and then construct an embankment	2.75
Reinforced soil embankment	1.11

The reinforced soil embankment option provided a saving of at least £1.5M. To assess the feasibility of the construction, a 7m high trial embankment was constructed on the top of the upper clay layer. The trial served to provide a check on the construction technique and to gain knowledge on the likely consolidation of the clay subsoils and the peat layer. There were concerns about the consolidation of the highly compressible peat layer, because though the initial permeability was high it would reduce substantially as the root tubers collapsed. To aid consolidation of the peat layer, vertical band drains were installed and a 1.5m surcharge layer was placed on the embankment. The embankment was instrumented to monitor the behaviour of the embankment, consolidation of the subsoil, dissipation of pore water pressure, and the performance of the geogrid.

During construction the load and strain in the reinforcement increased with height of the embankment. The surcharge was removed after 6 months; the measured maximum load and strain were about 15kN/m and 3.0 per cent respectively; typical load and strain data are presented in Figure 10. Throughout the trial the strain in the reinforcement was in good agreement with movements in the top of the upper clay layer obtained using a hydrostatic profile gauge, thus it may be assumed that there was no slippage between the geogrid and the clay. Inspection of the isochronous curves for Tensar SR2 given in Figure 9b, show that a strain of 3 per cent would be generated over a period of 6.5 months by a load of about 9kN/m.

The embankment trial was successful, and the trial section was incorporated into the main embankment which was opened to traffic in October 1987. The behaviour of the trial embankment was the subject of a prediction symposium reported by Bassett (1986).

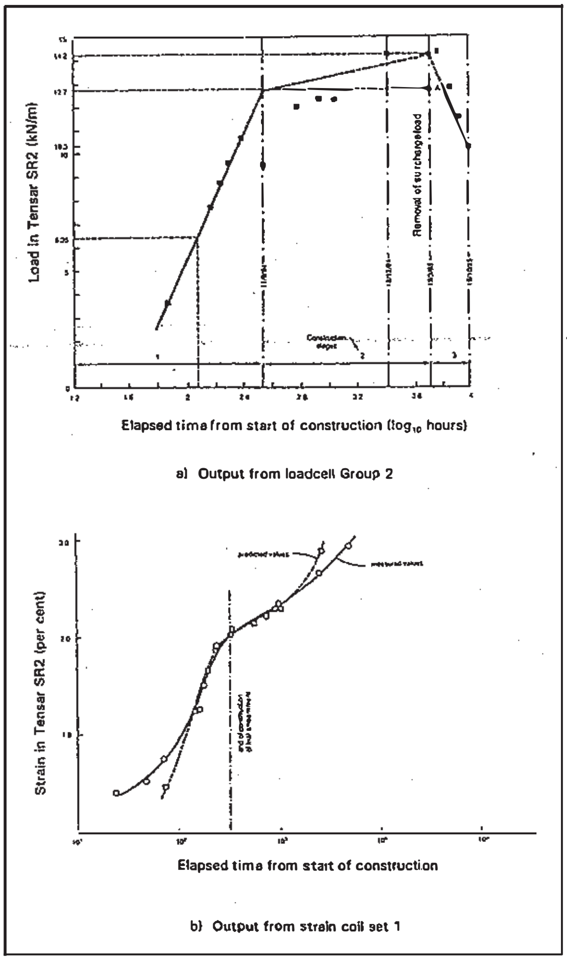


Figure 10 Typical load and strain data from the trial embankment at Stanstead Abbotts

Harlow embankment

The site was about 2km east of the Stanstead Abbotts by-pass, where new alignment crossed over an area of marshland close to the River Stort. The water table was just below ground surface and the area was prone to flooding. The subsoil comprised 3m of peat, overlying 6m of alluvial clay, over chalk bedrock.

Different construction options, similar to those for the Stanstead Abbots by-pass were considered; the different options and the estimated construction costs are given in Table 4. Again, the reinforced embankment proved the least expensive solution, offering a saving of at least £0.5M, and was an order of magnitude less than the construction of a viaduct.

The embankment, 7.5m high (including the surcharge layer) was constructed in three stages over a period of 18 months. Vertical band drains were used to accelerate the dissipation of excess water pressures within the subsoils; the embankment settled

Table 4 Harlow: Construction options and associated cost

Construction option	Cost (£M) at 1987 prices
Viaduct	6.35
Excavate the soft subsoil, replace with granular material and then construct an embankment	1.20
Reinforced soil embankment	0.72

a maximum of 2.7m. The road was opened to traffic 24 months after the start of construction.

The performance of the geogrid reinforcement was monitored from the start of construction. A summary of the results:

- The load and strain in the reinforcement increased with the height of the embankment, and were about 23kN/m and 1.4 per cent respectively, when the surcharge was placed.
- The surcharge was removed about 12 months later, during which time the load in the geogrid had remained reasonably constant but the strain increased to about 1.7 per cent, and the embankment settled a maximum of 2.7m.
- In the 4 years after the opening of the road to traffic, the values of load and strain remained reasonably constant despite the embankment settling a further 50mm.

Relations between load and strain in the reinforcement with time, from the start of construction to end of the first year in-service, are presented in Figure 11.

Throughout the monitoring period the strains in the reinforcement were in reasonable agreement with the data obtained from a hydrostatic profile gauge installed directly below the reinforcement, thus it may be assumed that there was no slippage between the geogrid and the soil.

The long term mobilised strain of 1.9 per cent was substantially less than the performance limit strain of 10 per cent for Tensar SR2 (Netlon Ltd 1986). The isochronous curves, given in Figure 9b, predict an equivalent load of about 6kN/m whereas the measured load was 23 kN/m, which is 80 per cent of the 120 year characteristic load of 29kN/m (Netlon Ltd, 1986). Thus the recorded strains do not agree well with laboratory data.

Comments

The observed behaviour of the geogrid reinforcement did not agree well with that predicted by the isochronous curves. The measured loads in the

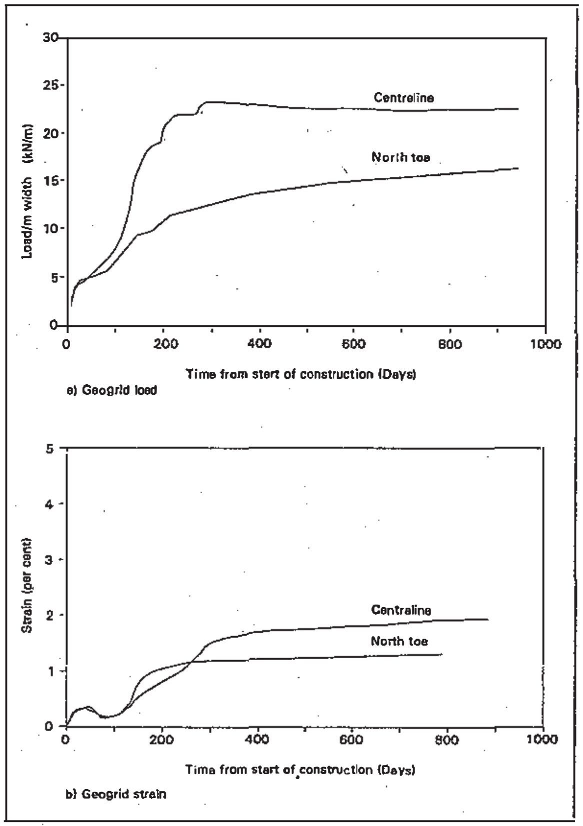


Figure 11 Load and strain data from the embankment at Harlow

reinforcement couldn't be corroborated by other means, and although data was fairly consistent the load measuring devices did not function perfectly.

The reasons why the measured loads were substantially higher than the loads derived from the isochronous curves are not fully understood. A number of contributory factors to such discrepancies include:

- The isochronous curves were derived from laboratory tests at 20°C. At low ambient temperatures, as exist in the ground, comparatively higher loads would be required to generate a given strain.
- In a laboratory test the tensile load is applied quickly and smoothly across the full width of the test specimen. In the field, the loading would of necessity be much slower and possibly not uniformly distributed.
- The frictional model of soil/reinforcement interaction is overly simplistic. This is possibly the single biggest obstruction to the development of more cost effective designs.

10.2 *Reinforced soil bridge abutment*

The construction of the Carmarthen by-pass (A48, Wales), across the alluvial flood plain of the River Tywi, required the construction of over bridges to the river and the railway. Due to the low bearing capacity and high compressibility of the subsoil the abutments were constructed using the Websol system (Agrément Board, now the British Board of Agrément, 1979). These were the first abutments on a major trunk road in the UK that incorporated polymeric soil reinforcement (Paraweb). The performance of the western abutment to the bridge over the railway was monitored by TRL. The abutment was 64m long and up to 8.5m high. Construction started in 1981 and the bridge was opened to traffic two years later. The performance of the structure 10 years after opening to traffic was reported by Brady et al (1995).

During construction, the tension in the reinforcement increased with the height of the fill, and apparently further increased as a result of settlement caused by piling operations adjacent to the abutment. The maximum tension in the reinforcement, 9kN, was reached about 2 months after the start of construction; thereafter the tension remained sensibly constant but monitoring ceased some 4 months later following the failure of the load cells due to the ingress of water. The maximum tension was less than the design value of 12.5kN. Due to the nature of the reinforcement it was not possible, at that time, to measure the strains within it.

It was evident that the magnitude and rate of settlement of the adjacent abutment had been substantially affected by the piling operations. Continued monitoring showed that that the settlement of the subsoils was virtually completed within three years of opening the bypass to traffic; a graph showing the settlement under the centreline of the abutment with time is presented in Figure 12.

Over the 10 year period there was little if any change to the vertical alignment of the abutment. It is unlikely that the abutment has moved en masse, therefore it is reasonable to assume that creep of the Paraweb reinforcement was negligible during this time. Settlement under the centreline of the abutment increased by 15mm, due to secondary consolidation of the clay subsoils, decreasing towards the ends of the abutment. As a result differential settlement along the abutment has increased from 1:110 to 1:95; no superficial damage has been caused to the facing panels but the tolerance of the Websol System to such movements is not known.

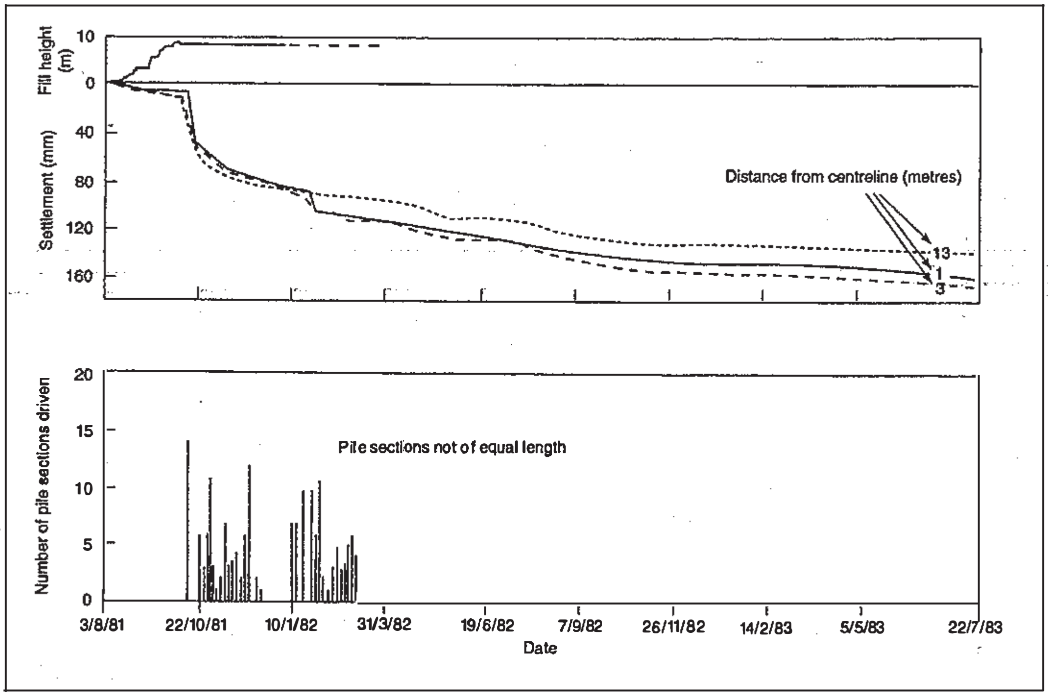


Figure 12 Settlement under the centreline of the abutment at Carmarthen (reproduced from Brady et asl, 1995)

Table 5 Climatic conditions at the storage sites

Climate	Location	Daily mean and range of temperature (°C)	Mean annual rainfall (mm)	Average daily mean relative humidity (%)
Temperate	RAPRA Laboratory, UK	22 (21-25)	-	60
Hot and dry	Cloncurry, Australia	26 (16-32)	450	40
Hot and wet	Cairns, Australia	24 (20-29)	3,350	80

Comments

The data from the reinforced soil bridge abutment at Carmarthen indicate that the Paraweb reinforcement achieved a maximum tensile load before the end of construction, and since that time there has been negligible creep of the soil reinforcement.

10.3 *Thirty year old plastics*

A programme to investigate the ageing of 6 plastics, under various climatic conditions over a period of 30 years, was undertaken by the Rubber and Plastics Research Association (now RAPRA Technology Ltd) in the 1950s. Details of the study have been reported by Brady et al (1994[a] and 1994[b]).

The materials included in this study were: PVC plasticised with dioctyl pthalate (PVCD), PVC plasticised with polypropylene adipate (PVCP), a high-impact unplasticised PVC (UPVC), a styrene butadine copolymer (SB), a low density polyethylene and a high density polethylene. Specimens of each material were stored out of direct sunlight, unstressed, in 3 different climatic conditions as reported in Table 5.

Accelerated laboratory tests at 70°C and 100°C in either a dry atmosphere or at 100 per cent humidity were also undertaken.

The tensile strength and strain at rupture were determined at intervals throughout the 30 year study. (Tests to monitor changes to the physical characteristics of the samples were also carried out, details of which are given in the references.)

A brief summary of the findings:

- The performance of the plastics was variable, but all exhibited an increase in tensile strength and a reduction in strain at rupture, with increased temperature.

- Little if any changes were recorded with the PVCD, LDPE and HDPE, which provides some assurance that these materials may be used for long term applications but an allowance might be required for loss in ductility.
- PVCP, UPVC and SB underwent a change of appearance. The properties of PVCP and UPVC changed due to a loss or a reduction in the effectiveness of additives. The strength of PVCP increased substantially with temperature; typical tensile test results are shown in Figure 13.
- The effects of natural ageing were not replicated by the accelerated tests; however, the rate of ageing could not be determined from the limited available data.

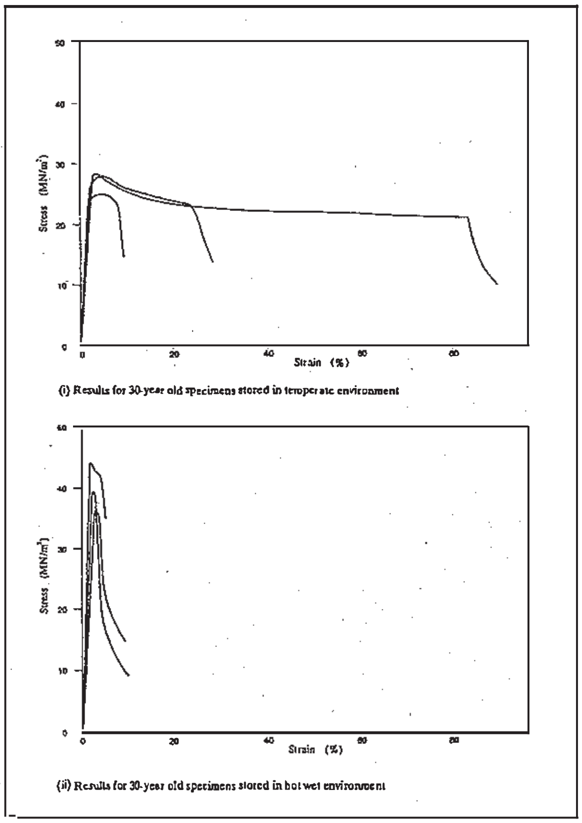


Figure 13 relations between stress and strain for PVCP stored in dufferent environments (reproduced from Brady et al, 1994)

Comments

Though the test environments are unlikely to be representative of in-service conditions, and the precise chemical make up of the plastics may not be the same as those in use today, the findings of the study give certain confidence that LDPE and HDPE are suited for long term applications. The behaviour of the different PVC materials demonstrate the importance of the use of additives, and that different additives may have substantially different effects on the long term behaviour of the product. The possible effects of different environmental conditions was illustrated by substantial change in the stress-strain relation for PVCP in the temperate and hot wet environments, as shown in Figure 13; these differences probably result from variations in the loss of plasticiser at the different sites.

There are two important conclusions to be drawn from this study that should be borne in mind by both manufactures and end users: (i) information derived from accelerated tests are product and not polymer specific, due to the use of different additive packages, and (ii) the long term performance of a geosynthetic may be dissimilar to that predicted from the results of ageing tests. This second point is of particular importance as engineers and others become familiar with the standard tests for assessing the durability of a geosynthetic. The relation between accelerated ageing tests and the natural ageing of geosynthetic reinforcements in the soil, can only be demonstrated by a programme of exhuming and testing samples over a period of many years.

11 DISCUSSION

11.1 *Durability*

Geosynthetics have become an indispensable part of the modern civil engineering industry, they are simple to install and may offer construction alternatives that are substantially cheaper than more traditional techniques. However, with typical design lives up to 120 years a sound knowledge of the durability of geosynthetics is essential.

Evidence from tests on samples of geosynthetics recovered after several years burial has shown that these materials are inherently stable in all but the most extreme conditions, and that damage inflicted during installation has been the primary cause of loss in strength. This provides confidence for the continued use of these materials provided installation operations on site are controlled.

Standard tests for determining the load bearing performance of geosynthetics have existed for some years, but it is only recently that standard tests for determining the resistance to degradation have become available. Of these screening tests, that for determining the resistance to oxidation has yet to be ratified. (There has been much discussion for the past two years or more regarding the test procedure, because it has been shown that for a specific product different test method can produce substantially different results.)

There is a dearth of knowledge of the long term behaviour of geosynthetics in extreme conditions, and predicted loss in strength can not be made with confidence. Confidence will only be obtained through comparisons of test results from accelerated tests and samples that have aged naturally in the ground. This work will take many years, and though new polymers and products will undoubtedly enter the market unless appropriate steps are undertaken now we shall be none the wiser ten or twenty years from now.

11.2 *Aspects of design*

Can structures be designed to be more cost efficient?

The method for calculating the design strength of a geosynthetic soil reinforcement is overly conservative, principally because the load bearing properties are based on the results of tests in air, but also because the partial factors of safety that combine to form the material factor of safety (f_m) are chosen to represent the worst possible combination of degradation effects.

Few tests have been developed to replicate and monitor the in-soil load bearing performance of a geosynthetic. This is because the equipment and techniques required to undertake such tests are complex and, there are both practical and technical limitations to measuring the load and strain in soil. Confidence in the interpretation of the test results has yet to be established before the results of these tests could be used to provide information for design. Therefore the strength properties of geosynthetic reinforcements continue to be determined from the results of tests in air.

Research on the strength of polyester tendons undertaken and reported by Orsat et al (1998) identified the concept of residual strength, and is illustrated in Figure 14. There is a dearth of published information on this subject, but the concept would appear to offer a new method for deriving a design strength for a geosynthetic. If the time to the onset of rupture can be determined then this time period, reduced by an appropriate partial factor of safety, would define a service life; the design strength would equal the factored residual strength. Thus this method has the potential to provide an improved efficiency in design, as the residual strength is greater than the allowable strength derived from the creep rupture curve, and less reinforcement is therefore required for stability.

The actual factor of safety for the internal stability of a reinforced soil structure is unknown. To date there have been no recorded failures of geosynthetic

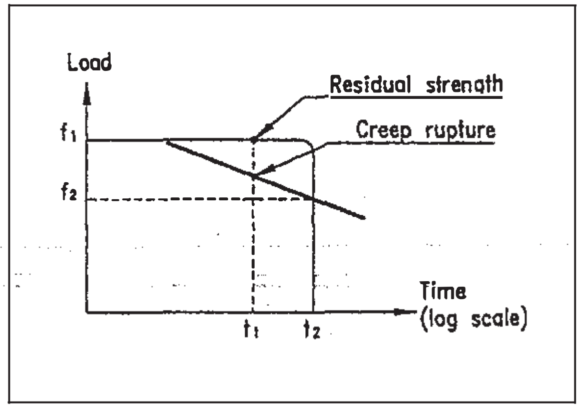


Figure 14 The concept of residual strength (reproduced from Orsat et al, 1998)

reinforced soil structures resulting from failure of the reinforcement, and though of course this is to be applauded it may be an indication that the design procedures are overly conservative. Could the amount of reinforcement be reduced without reducing the overall safety to an unacceptable level? Trial by experimentation is the only certain way; i.e. by building trial structure using normal site techniques, and then inducing failure. Such trials are complex, expensive and require careful control. But they can supply invaluable information that may lead to more cost-effective designs.

At the present time the interaction mechanism between a geosynthetic reinforcement and the surrounding soil is not fully understood; a reinforcement installed in compacted fill, will exhibit a stiffer modulus than would be measured in air. The apparent increase in the modulus is presumed to be a direct consequence of the interaction between the soil and the reinforcement. This effect has been demonstrated in the laboratory but interpretation of the tests results is not straightforward. Further research is required to determine the interaction mechanism. It likely that the resulting design model will be complex and require the use of numerical techniques to obtain a solution.

Further evidence that current design methods do not adequately model in-service performance is provided by the results of long term monitoring of in-service reinforced soil retaining structures. Data show that post construction movements do not continue indefinitely, i.e. at some point in time creep of the reinforcement ceases, and further movements will not occur except through the action of an external agency. This suggests that a state of equilibrium is reached between the disturbing forces and the restraining tensile forces in the reinforcement; at this time the loading condition of the reinforcement becomes similar to that in a stress relaxation test. Therefore it would seem inappropriate that predictions of long term deformation should be based solely on a consideration of creep strain.

The cost of developing and verifying a new design procedures will be substantial. It could be argued that the many structures that have been built world wide have performed successfully, there is no incentive to progress. But current trends in construction are towards a requirement for increasingly tighter tolerances on structural deformation. This demands a new design method for determining the serviceability limit state. Additionally, more companies are specifying their requirements for reinforced soil, but rely on the manufacturers/suppliers undertake designs on their behalf. Thus it is likely that the costs of developing new design procedures will be borne by those who stand to benefit from increased usage of geosynthetics.

11.3 *The future*

The geosynthetic reinforced soil industry, in common with others, is driven by economy. Therefore new developments must result in improved economy for construction.

New products are being developed that will have a higher strength and a lower strain at rupture than current the materials; their use may reduce in-service movements. Such products should be more resistant to degradation thus allowing their use in areas that may contain some aggressive specie.

Attempts to achieve onerous deformation criteria, such as those required by BS 8006 (1995), result in increased construction costs. Serviceability limits should be achievable and, set at levels appropriate to the structure type: e.g. a reinforced soil retaining wall close to an adjacent structure will rightly require stringent tolerances on construction and post construction movement, whereas the movement of a similar wall in another situation may be of no importance provided the structure is safe.

The design of parapet plinths for vehicular crash barriers on reinforced soil structures, are overly conservative; their cost eliminates savings accrued from using reinforced soil construction. Thus until improved designs are available reinforced soil structures are unlikely to be used where containment is necessary.

12 CONCLUSIONS

The durability of a geosynthetic reinforcement is primarily dependent on load bearing performance and damage incurred during its service life. Degradation may also result from aggressive climatic and environmental conditions, but except in the most extreme instances these effects will be negligible in comparison.

Evidence from geosynthetics recovered after several years burial have shown that these materials are inherently stable. However, there is a dearth of knowledge of the long term behaviour of geosynthetics in extreme conditions, and predicted loss in strength can not be made with confidence. Confidence will only be obtained through comparisons of test results from accelerated tests and samples that have aged naturally in the ground.

It is recommended that surveys of reinforced soil structures are included as a part of regular inspection procedures; such data are invaluable for the development of new designs.

Current design methods are overly conservative, and design models are not representative of in-service conditions. The precise nature of the interaction mechanism between a geosynthetic reinforcement and the surrounding soil is currently unknown. This is possibly the single biggest obstruction to the development of more cost effective designs.

New geosynthetic reinforcements are being produced, and new designs are being developed that will more accurately predict performance. Such advances will result in improved economy for construction.

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