

THE BEHAVIOUR OF GEOSYNTHETIC REINFORCED SOIL SYSTEMS IN VARIOUS GEOTECHNICAL APPLICATIONS

A McGOWN

University of Strathclyde, United Kingdom

ABSTRACT: Geosynthetic Reinforced Soil Systems were introduced during the 1970's. Since then a great deal of research and a large number of field observations have been undertaken. These show that there are many technical and economic advantages to be gained from the use of such systems, providing they are carefully designed. To date, Limit Equilibrium design methods have been employed, but recently the Limit State Approach has been applied to the design of Geosynthetic Reinforced Soil Systems. Limit Equilibrium methods are generally considered to be very conservative. They are based on the peak strength of the materials, essentially without regard to the strains. In addition they generally treat all types of loads as pseudo-static loads. In contrast, the Limit State Approach requires that the load-strain characteristics of the materials are fully taken into account and that different types of loads are dealt with in different manners. In so doing it provides an opportunity to be less conservative and to maximise the technical and economic benefits to be gained from the use of such systems. In order to gain the benefits available from the Limit State Approach a fuller understanding of the behaviour of the soils and geosynthetics employed is required together with a better appreciation of the operational behaviour of the various types of structure under different loading regimes. In this Paper, the composition, construction and operational behaviour of the various types of Geosynthetic Reinforced Soil Structures are identified. The application of the Limit State Approach to the design of Geosynthetic Reinforced Soil Structures is discussed. The characteristics of the geotechnical and geosynthetic materials comprising the structures are reassessed and their behaviours under different loading regimes analysed. The significance of all of this on material specifications, design codes / methods and costs is highlighted.

Keywords: Isochronous Strain Energy, Reinforcement, Design method, Traffic effects, Seismic analysis

1. INTRODUCTION

Undoubtedly the 20th Century was one of the most remarkable in the history of mankind. The quality of life of hundreds of millions of people was dramatically improved as a result of great advances in medicine, science and technology. Due to the variety and extent of these advances, the growth and technical developments within the construction industry have been largely overlooked, yet they are no less remarkable and beneficial.

In previous centuries a relatively small number of notable structures were built, each a masterpiece easily distinguishable from the mass of poor quality buildings and other structures of the same period. However in the 20th Century a vast number of highly complex and innovative buildings and structures have been built alongside generally high quality utilitarian construction works. This was all made possible by many innovations in materials engineering and construction technology. One notable innovation from the second half of the 20th Century was the introduction of Geosynthetic Reinforced Soil Structures, including walls, bridge abutments, steep slopes, embankments, load supporting pads, foundations, railroads, airfields and roads. The first prototype trials of these forms of construction only took place in the late 1960's and early 1970's, yet today they are accepted for use Worldwide.

An important feature of the rapid growth in the use of Geosynthetic Reinforced Soil Structures, was that they were first successfully applied in practice and then later researched and standardised within design codes / methods. As a result of this, over the last thirty years or so, the need to maintain the confidence of the construction industry practitioners has been of primary concern with the risk of failures always minimised. As a consequence designs have been kept very

conservative, which has been very effectively demonstrated by Berg et al (1998) and Greenway et al (1999). They undertook historical reviews of the material assessment techniques and Limit Equilibrium design methods adopted in North America for Geosynthetic Reinforced Soil Structures since the late 1970's. They showed that outcome designs from the latest design methods / codes do not differ greatly from outcome designs based on the recommendations made in the 1970's.

Over the last twenty years a great deal of research into the properties of geosynthetics and geotechnical materials has been undertaken. In addition, a number of well-documented case histories involving extensive back-analysis and monitoring have been published. Thus there is now a much greater understanding of the operational behaviour of the materials and the overall structures. By utilising this knowledge and by adopting the Limit State Approach for the design of Geosynthetic Reinforced Soil Structures, it is suggested that more economic and technically efficient structures can be built in the future.

It is the objective of this paper to identify the areas of materials assessment and design methodology, most in need of change and to propose appropriate changes. In order to do so, the composition, construction and operational behaviour of the various types of Geosynthetic Reinforced Soil Structures will be identified. The application of the Limit State Approach to the design of Geosynthetic Reinforced Soil Structures will be discussed. The characteristics of the geotechnical and geosynthetic materials comprising the structures will be reassessed and their behaviours under different loading regimes analysed. The significance of all of this on material specifications, design codes / methods and costs will be evaluated.

2. TYPES, COMPOSITION, CONSTRUCTION AND OPERATIONAL BEHAVIOUR OF GEOSYNTHETIC REINFORCED SOIL STRUCTURES

2.1 Types of Structures

Geosynthetic Reinforced Soil Structures represent a wide range of construction forms. Bonaparte et al (1985) subdivided them into two broad categories, viz. Earth Structures and Load Supporting Structures.

Earth Structures include walls, bridge abutments, steep slopes and embankments, which are not generally stable under their own weight and may or may not require to support significant external loads. Thus the primary design criterion is the stability of the structure under its own weight and where applicable externally applied loads. They are typically constructed with many alternating horizontal layers of compacted fill and geosynthetic reinforcements. With walls, bridge abutments and steep slopes, a facing is also employed to prevent localised surface erosion along the exposed sides of the reinforced soil mass. The facing units are linked to the reinforcements by special connections. For embankments, fewer reinforcement layers are employed and facing units are not normally required. In all cases, the geosynthetic reinforcement layers provide lateral tensile strength and increase the confining pressure acting on the soil, Jewell (1980). This permits the construction of side slopes at angles greater than the mobilised angle of friction of the fill. For walls, bridge abutments and steep slopes, the sub-soil is usually adequate to support the reinforced structures, however, for embankments, the sub-soil is often weak and highly compressible, McGown et al (1998).

Load Supporting Structures include road and airfield pavements, railroad tracks, load supporting pads and foundations. These structures are usually stable under their own weight, and the primary design criterion is the ability of the structure to support the externally applied loads, with limitations placed on the associated deformations. They are typically constructed with a single, or at most a small number of geosynthetic layers within compacted fill over very weak and possibly highly compressible sub-soil layers. Generally they do not have any facing units. The reinforcement function of the geosynthetic is to provide lateral tensile strength and to increase confining pressure within the fill. This increases the ability of the system to carry additional externally applied loading or to carry loads with less deformation.

2.2 Composition of Structures

In general terms Geosynthetic Reinforced Soil Structures consist of some or all of the following components:

- Reinforced fill
- Retained fill
- Sub-soil
- In-situ soil behind the reinforced fill
- Geosynthetics
- Facing units, and
- Connections

Most design codes specify that the reinforced fill consists of compacted granular, (frictional) soil or crushed rock. For example, BS8006 (1995) requires that for Earth Structures a cohesionless fill with no more than 15% of the material less than 63 μm is used. However, the use of other types of fills is steadily increasing with for example, cohesive fills being used in Japan, Kasahara et al (1992), and residual soils being used in many tropical regions.

The retained fill, sub-soil and in-situ soil behind the reinforced fill can be any geotechnical material.

The geosynthetic reinforcing layers may be produced from a wide range of polymeric materials, manufactured in a

variety of forms, including sheets, strips and grids. The vast majority can be termed relatively extensible, i.e., they have larger strains to rupture than the maximum tensile strain in the soil without reinforcement, under the same operational conditions, McGown et al (1978). Their load-strain properties are time and temperature dependent and the load transfer mechanism between the soil and the geosynthetics is dependent on surface friction, bearing stresses or possibly a combination of these.

Many types of facing units may be used in reinforced soil walls, bridge abutments and steep slopes, Jones (1993). Some of the most commonly used facings are:

- Full height panels
- Sectional panels
- Segmental panels, and
- Geosynthetic wrap-around facings.

These facings are connected in a variety of ways to the reinforcement layers, including vertically sliding, loose fitting, rigid and tightened connections, McGown et al (1993).

2.3 Construction Procedures

The most common construction procedure associated with Geosynthetic Reinforced Soil Structures is to place successive layers of Geosynthetic reinforcements between compacted layers of fill. Unfaced slopes, represent the condition of no restraint on the lateral soil boundary. However, depending on the type of facing unit and connections used in walls, bridge abutments and steep slopes, the lateral soil boundary conditions may vary according to four factors, viz., axial compressibility, lateral compressibility, flexural rigidity and frictional characteristics of the rear surface of the facing units. Temporary propping during construction together with the order in which such props are removed at the end of construction, are also important factors influencing the behaviour of the lateral soil boundary, Yagarajah (1993).

All of these lateral soil boundary conditions influence the horizontal pressures on the facing units that develop due to compaction effects during construction. McGown et al (1998) suggest that the mathematical models proposed by Broms (1971) and Ingold (1979) can be used to calculate these compaction-induced lateral boundary stresses.

There is a further compaction effect that can be evidenced in Geosynthetic Reinforced Soil Structures particularly when the geosynthetic reinforcement is a grid with integral junctions. This effect has been termed Dynamic Interlock, McGown et al (1990) and McGown et al (1994) and it develops in the following way:

- (i) During compaction of the fill the grid is stretched and soil particles are forced into the apertures.
- (ii) When the compaction load is released, the grid attempts to return to its initial condition, but is resisted from doing so by the presence of the soil particles in the apertures. This develops locked-in strains and so locked-in stresses in the grid.
- (iii) The locked-in strains have a similar effect to a confining stress on the soil and therefore increase the strength of the soil.

2.4 Operational Behaviour of Structures

The operational behaviours of Geosynthetic Reinforced Soil Structures may be characterised at three levels. The first is their behaviour under working loads and imposed deformations, so-called Working Conditions. Second is their behaviour at limiting deformation conditions, the so-

called Serviceability Limit State. Third is at collapse, the so-called Ultimate Limit State. McGown et al (1998) suggest that some or all of the following aspects of their operational behaviours require to be considered:

- Local instability of the lateral face of the reinforced fill
- Settlement of the reinforced fill, other fills and subsoil
- Rupture and deformation of the facing units, connections and reinforcements
- Pull-out and slippage of the reinforcements
- Sliding of the reinforced mass along its base
- Overturning of the reinforced mass
- Bearing failure or deformation of the subsoil and
- Overall stability of the structure.

2.5 Actions

All of these operational behaviours may be directly related to the nature of the Actions resisted by the structure. These may be categorised as Direct Actions, which are loads or forces applied to the structure, and Indirect Actions, which are imposed or constrained deformations, Fig. 1. Within these two categories there are many types of Actions, including:

- Permanent Actions, (likely to act throughout a given design situation);
- Variable Actions, (likely to vary but with a mean value of significance);
- Accidental Actions, (likely to be of short duration and unexpected but of sufficient magnitude to cause severe consequences);
- Fixed Actions, (likely to be of known magnitude and direction with a fixed distribution over the structure);
- Free Actions, (likely to be of known magnitude and direction but a variable distribution over the structure);
- Static Actions, (likely to be stable and not cause significant acceleration of the structure or any of its components);

Classification Eurocode 7 (1995)

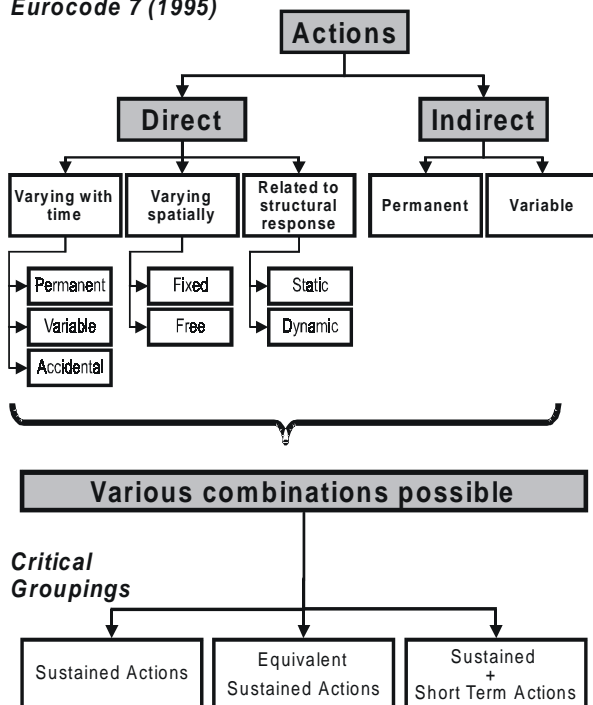


Figure 1 Classification and Grouping of Actions

Quasi-static Actions, (likely to be essentially static but have some dynamic effects), and
Dynamic Actions (likely to cause acceleration of the structure or of its components).

In view of the wide range of actions that can affect Geosynthetic Reinforced Soil Structures, their performance can be very difficult to assess and it should be recognised that it may vary considerably with time. Thus for simplicity in design it is suggested that the various types and combinations of actions should be split into three general categories, viz. *Sustained Actions*, *Equivalent Sustained Actions* and *Sustained plus Short-term Actions*. Sustained Actions and Equivalent Sustained Actions represent all types and combinations of actions that can be reasonably represented as long term sustained loads or deformations and are termed *Single-Stage Actions*. Sustained plus Short-term Actions are those actions which must be treated in design as a series of loads or deformations acting for different periods of time, either combined or acting separately, and are termed *Multi-Stage Actions*.

3 LIMIT STATE APPROACH TO DESIGN

To date most Geosynthetic Reinforced Soil Structures have been designed using Limit Equilibrium methods of analysis. These methods generally produce safe but very conservative outcome designs. They are based on the peak strength of the materials, without regard to the strains. In addition they essentially treat all types of loads as pseudo-static loads. In order to increase the technical efficiency and cost effectiveness of the use of Geosynthetic Reinforced Soil Structures, it is suggested that the Limit State Approach to design is adopted in the future. This will allow the load-strain characteristics of the materials to be fully taken into account, different types of loads to be dealt with in different manners, deformation and strain criteria to be introduced and both collapse and operational conditions to be assessed, as is explained in the following sections.

3.1 Limit State Conditions

Whenever a structure or a part of a structure operates at a level equal to any of the performance criteria, it is said to have reached a Limit State.

Limit States are divided into two categories; Serviceability Limit State (SLS) and Ultimate Limit States (ULS). Serviceability Limit States are conditions, or performance criteria, beyond which the functional or aesthetic utility of a component or the entire structure is lost. Loss of serviceability may be due to deformations in the ground or deformations in the structure itself. Ultimate Limit States are concerned with the safety, loss of static equilibrium or rupture of either a critical component or the entire structure.

Two aspects of the Limit State Approach which are of particular significance to Geosynthetic Reinforced Soil Structures are:

- (i) Consideration of the strain compatibility of the various materials involved at the Limit States, and
- (ii) The assessment of the influence of internal and external environmental conditions on the durability of the materials used.

Designs based on the Limit State Approach require the identification of suitable calculation models, properties of materials, actions, geometrical data and limiting values of deformations. The calculation models must be appropriate and based on valid behavioural mechanisms. Design values of the properties of materials must be assessed directly for particular site conditions or derived from characteristic

values based on field and laboratory test data. Actions are all loads, forces and deformations identified to be contributing to any specific Limit State condition. However, not all loads, forces or deformations are treated as Actions in all Limit State Designs. Further the duration of Actions must be considered, including any changes in Actions resulting from changes in the properties of materials with time. Geometrical data must include all level, slope and other dimensional data important to the design and allowances made for variations in these.

3.2 Partial Factors

The Limit State Approach does not use global Factors of Safety, instead Partial Factors are applied to the limiting values of the variables for each of the Limit State conditions. The use of Partial Factors aims to distribute margins of safety to the places in the calculation where there are uncertainties. Different Partial Factors are generally applied in the cases of favourable and unfavourable actions. In the calculation of Serviceability Limit States, Partial Factors of unity are frequently employed.

3.3 Design Criteria

Limit State Approach design criteria are based on equating the effects of the de-stabilising actions and the stabilising actions.

For Ultimate Limit States, both external and internal conditions are analysed and must not be exceeded. Thus when considering external Ultimate Limit States, the equilibrium or gross displacement of a structure are considered assuming the structure to be a rigid body. It is then verified that:

$$E_{d,dst} \leq E_{d,stb}$$

where, $E_{d,dst}$ = Design value for the effect of the destabilising actions (direct, overturning or sliding), and
 $E_{d,stb}$ = Design value for the effect of the stabilising actions (resisting).

When considering internal Ultimate Limit States, rupture or excessive deformation of sections, members and connections are considered. It is then verified that:

$$E_d \leq R_d$$

where, E_d = Design value of the effect of actions (internal forces, moments or vectors of several internal forces or moments), and
 R_d = Design value of the corresponding resistance (obtained from the design values of properties).

Serviceability Limit States are treated in a similar manner. For each condition it is verified that:

$$E_d \leq C_d$$

where, E_d = Design value of the effect of actions, (displacements or accelerations), and
 C_d = A nominal value or a function of certain design properties of materials related to the design effects of actions considered.

3.4 Material Properties

In Geosynthetic Reinforced Soil Structures, four groups of material properties require to be identified, as follows:

3.4.1 Soils

Ultimate Limit State analysis is concerned with collapse conditions, thus the ultimate large strain constant volume strength, that is the large strain Constant Volume Angle of Friction, $[\phi'_{cv}]$, should generally be employed. For granular soils, the constant volume shear strength is the lowest value

achievable, therefore no Partial Factor is required for this parameter when it is used in designs.

For Serviceability Limit States, no fixed value for the angle of friction can be suggested, rather various values must be used, depending on the Serviceability Limit State being considered. These values can be determined from the relationship between the mobilised angle of friction and the lateral tensile strains in the soil. For most structures, the soil is acting under plane strain conditions, hence the plane strain apparatus should be used to determine this relationship although this is rarely the done. More usually triaxial or shear box testing is used to determine the peak angle of friction, $[\phi'_p]$, and constant volume angle of friction, $[\phi'_{cv}]$. However, various researchers have carried out experiments to investigate the relationship between the mobilised angle of friction and the lateral tensile strain in the soil, $[\epsilon_t]$, Cornforth (1964), Barden et al (1969), Al-Hasani (1978) and Bolton (1986). According to their findings, the peak angle of friction, $[\phi'_p]$, for granular soils occurs at lateral tensile strains in the range 3 to 6% and the constant volume angle of friction, $[\phi'_{cv}]$, occurs at tensile strains in the range 6 to 12%, McGown et al (1993). Thus the relationship between the mobilised angle of friction and the lateral tensile strains can be defined and design values identified, Fig.2.

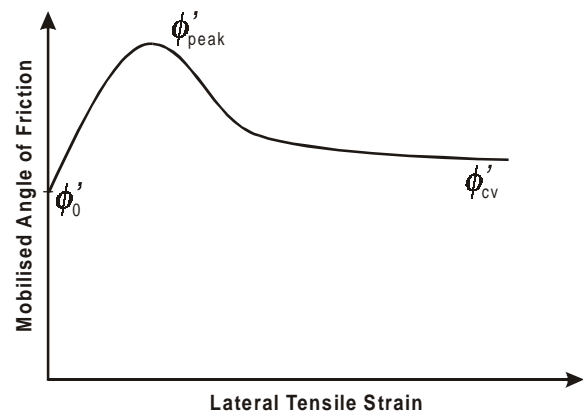


Figure 2 Relationship between the mobilised angle of friction and lateral tensile strain in cohesionless soil

At the time of design the source of the soil to be used in the construction works may not be known and the quality of the compaction control may not be guaranteed. In such cases appropriate design values for the soil at the Ultimate and Serviceability Limit States must be assumed based on the specified nature of the soil, the compaction methodology to be used and the actions to be resisted by the soil. These assumed design values should be realistic but currently tend to be extremely conservative. Khan (1999) undertook a review of the design values used to represent compacted granular reinforced fill subject to Single-Stage Actions in case histories reported in the 5th and 6th International Conferences on Geosynthetics. He found that peak angles of friction values were in the range 25 to 45° with an average value of 35°. Large strain constant volume angles of friction values were in the range 28 to 33° with an average value of 30°. These values are some 10° less than both his experimental and previously published shear box and triaxial test values for such soil types. Thus much more realistic design values for the reinforced soil strength should be used, however, the use of a small Partial Factor may be appropriate in such cases to allow for compaction deficiencies.

It should be noted at this point, that although compacted granular soils have very similar behaviours when subject to either Single-Stage or Multi-Stage Loading, other soil types may not. For these other soil types even greater care requires to be taken to choose appropriate design parameters.

3.4.2 Geosynthetics

The choice of the design parameters for geosynthetic soil reinforcements to be used in Ultimate Limit State analysis has not yet been widely agreed. Some design codes / methods presently recommend the use of factored short term constant rate of strain (CRS) tensile test data, while others recommend the use of factored sustained load (creep) test data. A more consistent approach is required and this matter is given detailed consideration in Sections 4 and 5.

3.4.3 Soil Reinforcement Interaction

The choice of the design parameters for soil reinforcement interaction in Ultimate and Serviceability Limit State analysis has not yet been widely agreed. Some design codes / methods presently recommend the use of direct shear test data and others use pull-out test data.

The direct shear test can be applied to the determination of the soil reinforcement Coefficient of Direct Sliding and is applicable to sheet and strip reinforcements, Jewell (1996). For geogrid reinforcements, however, the mode of interaction is quite different. Dyer (1985) and Milligan et al. (1990) showed that it is partly developed through the concentration of bearing stresses against the transverse members of the grid and partly developed by direct sliding. Thus for geogrids it is the Coefficient of Bond that should be measured and this can be achieved by pull-out testing, Jewell (1996).

McGown et al (1984b) suggested the possibility of applying a Partial Factor to soil-reinforcement interaction coefficients. At present no Limit State design code / method specifies any such Partial Factor. However, it may be suggested that only a small Partial Factor, equal to or slightly greater than unity, is required for cases where the direct shear test results are used, but where interaction coefficients are obtained using pull-out test results, a higher Partial Factor may be required. It should be noted that the Coefficient of Direct Sliding and the Coefficient of Bond are both functions of the angle of friction of the soil, therefore care should be taken when determining these values. If in the calculation of the coefficients, a factored angle of friction for the soil has been used, the application of a Partial Factor to the soil-reinforcement interaction coefficients could produce an extremely conservative design value.

3.4.4 Other Materials

For the Ultimate and Serviceability Limit State design of facings, in concrete, metal or other materials, connections and other components, there are existing, relevant Limit State codes of practice which can be followed.

4. GEOSYNTHETIC REINFORCEMENT PROPERTIES

4.1 The Isochronous Strain Energy [ISE] Approach

Geosynthetics are elasto-visco-plastic materials and their typical behaviour is shown in Fig. 3. A wide range of Single-Stage Loading tests for geosynthetics have been specially developed over the last 25 years, for example, constant rate of strain (CRS), sustained load (creep), stress relaxation and cyclic tests, however to date, few Multi-Stage Loading tests

are known to have been undertaken. Therefore, the design parameters for Single-Stage and Multi-Stage Actions are generally determined by factoring data obtained from the Single-Stage Loading tests. This factoring for Multi-Stage Actions has not been scientifically proven and can be considered to be empirically based.

To overcome the lack of comparisons and obtain correlations between test data acquired from different testing methodologies, to better identify the factors to be applied in the determination of Design Strengths and to allow for the variety of actions influencing the behaviour of geosynthetics, a new more fundamental approach is required. It is suggested that these objectives may be achieved by using the Isochronous Strain Energy [ISE] Approach, as set out in the following sections.

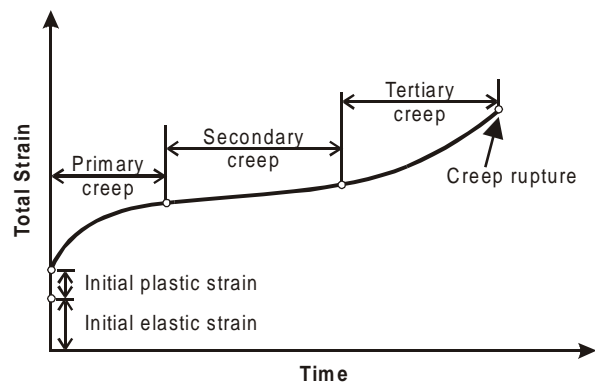


Figure 3 Idealised sustained load (creep) curve at constant temperature

For all Single-Stage Loading regimes under isothermal conditions, e.g. constant temperature CRS, creep or cyclic testing, the external work done per unit width of a geosynthetic may be taken to be equal to the “Absorbed Strain Energy” at any time [t]. All Single-Stage Loading test data can be represented by Isochronous Load-Strain curves and the areas under the curves represent for any specific time, the “Isochronous Strain Energy”, Fig. 4. The unit of Isochronous Strain Energy [ISE] for geosynthetics is:

$$\text{Force per unit width times unit strain} = (\text{kN/m}) \times (\text{m/m}) = \text{kN/m}$$

This is not the usual unit of strain energy in materials engineering. Additionally, it may be confused with the unit of strength for geosynthetics. Thus it is suggested that a new term should be adopted for the unit of Isochronous Strain Energy [ISE] for geosynthetics.

Thus at any temperature [T] and time [t] after the application of a particular loading regime, there will be a finite amount of work done per unit width, which can be represented as the “Absorbed ISE” [A]_t. The amount of ISE to develop a limiting strain or rupture at that temperature for a particular Single-Stage Loading regime is termed the “ISE Capacity” [C]_t of the geosynthetic at the specified time [t]. A feature of this property of the material is that data obtained at the same temperature from different load-strain paths, i.e. different test methods, may be plotted on the same ISE – Time plot, Fig. 5.

4.2 Identification of the Components of ISE

Figure 6 represents the load-strain-time behaviour of a geosynthetic in terms of the Absorbed ISE [A]_t. Upon application of load [P]₁ at time [t]₀, as shown in Fig. 6 (a),

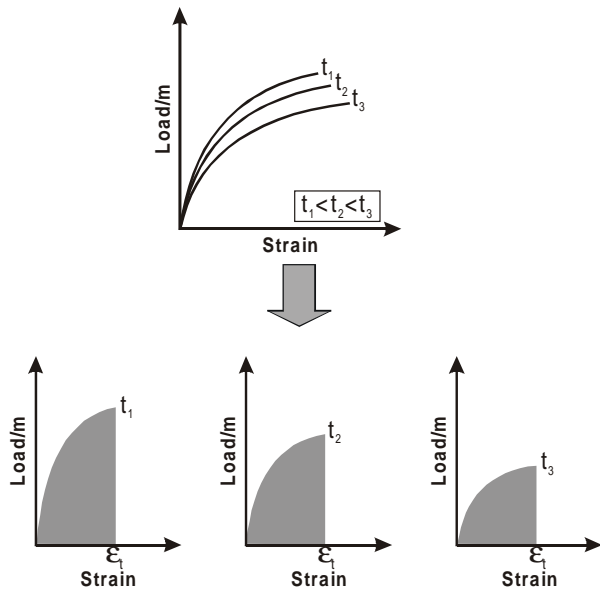


Figure 4 Calculation of Isochronous Strain Energy

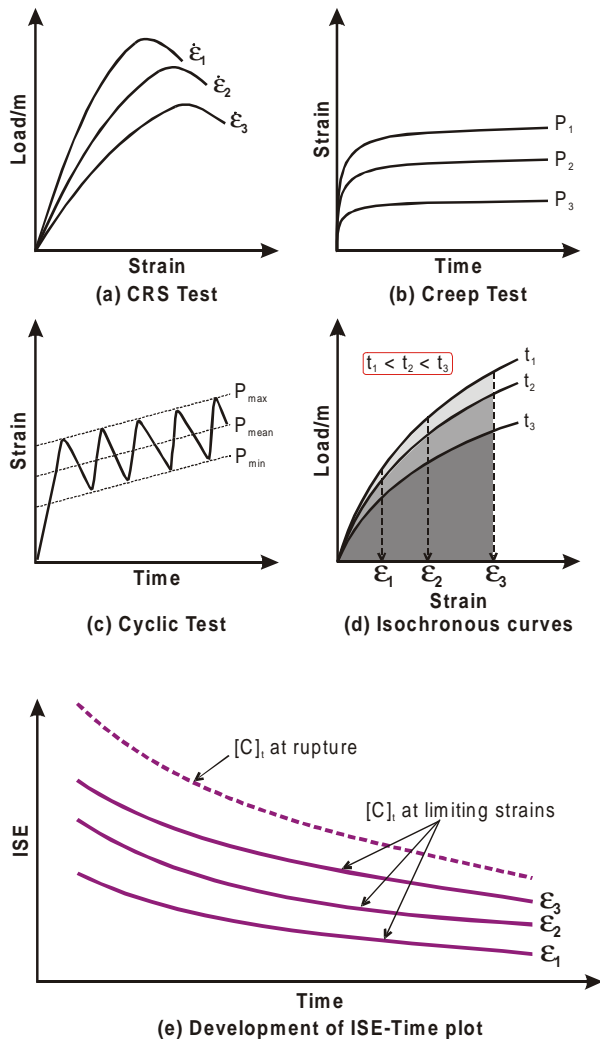


Figure 5 Development of ISE-Time plot from various tests

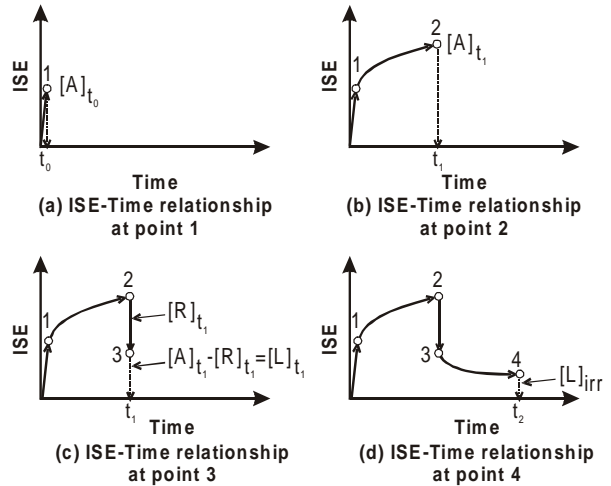


Figure 6 The isothermal load-strain-time behaviour of a geosynthetic in terms of Absorbed ISE

there will be an Absorbed ISE $[A]_{t_0}$ (point 1) within the geosynthetic. Thereafter, over a period of time between $[t_0]$ and $[t_1]$ more ISE will be absorbed by the geosynthetic, i.e. the Absorbed ISE will increase to $[A]_{t_1}$, (point 2). At time $[t_1]$ when the load is removed, a part of Absorbed ISE $[A]_{t_1}$ will be recovered immediately and this is termed the “Immediately Recoverable” ISE $[R]_{t_1}$, (point 2 to 3). At this point in time, $[t_1]$, the Absorbed ISE remaining in the geosynthetic is termed the “Locked-in” ISE $[L]_{t_1}$. If no further load is applied to the geosynthetic, then with time part of this Locked-in ISE will be recovered due to viscous rebound but part will never be recovered, the “Irrecoverable Locked-in” ISE $[L]_{irr}$.

Thus at any time, $[t_1]$, the Absorbed ISE comprises two components, which are the Immediately Recoverable ISE $[R]_t$ and the Locked-in ISE $[L]_t$. These components are likely to vary with time for any limiting strain condition or rupture. Calculation of the ISE components requires derivation of Isochronous Load - Recoverable Strain and Isochronous Load - Locked-in Strain curves. In so doing, Isochronous Load - Total Strain and Isochronous Load - Recoverable Strain curves are first developed from loading and unloading tests respectively, as shown in Fig. 7. Then since the Total Strain is equal to the summation of Immediately Recoverable Strain and Locked-in Strain, the Isochronous Load - Locked-in Strain curves can be obtained, Fig. 8. The isothermal ISE Capacity and its components for any Single-Stage Loading regime, i.e. $[C]_t$, $[R]_t$ and $[L]_t$, can then be calculated as the areas under the respective isochronous load-strain curves for different times and strains, Fig. 9, to produce the plots shown in Fig. 10. It should be noted that the ISE Components $[R]_t$ and $[L]_t$, of the ISE Capacity $[C]_t$ at any limiting strain level or rupture, are likely to be intrinsic properties controlled by the polymeric composition, micro and macro structure of the geosynthetic.

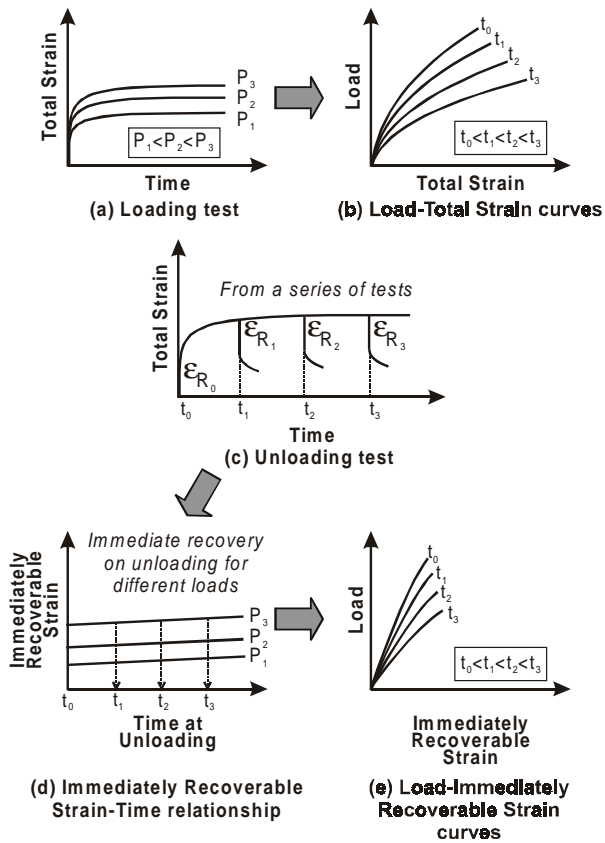


Figure 7 Development of isochronous Load-Total Strain and Load-Immediately Recoverable Strain curves from sustained loading/unloading tests

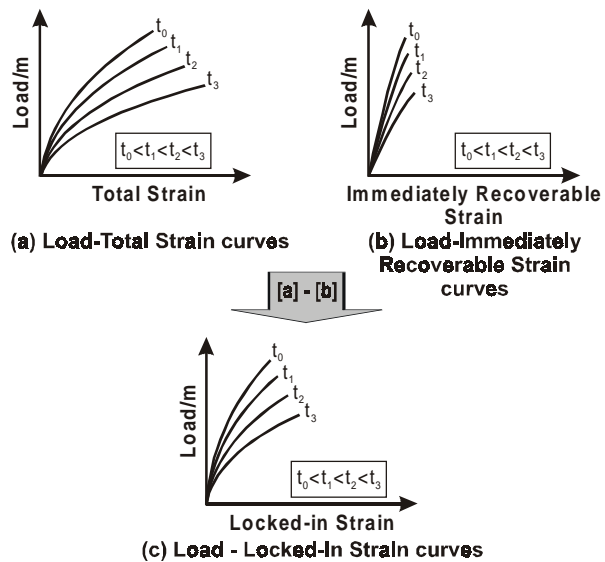


Figure 8 Development of Isochronous Load - Locked-in Strain curves

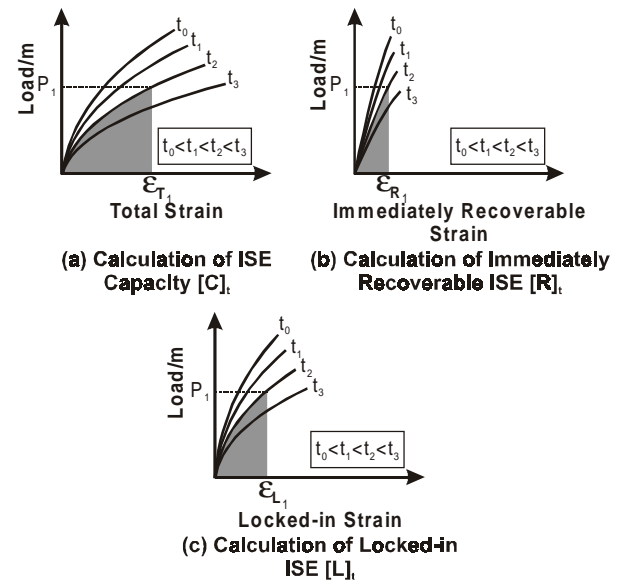


Figure 9 Calculation of the ISE Components / Strains

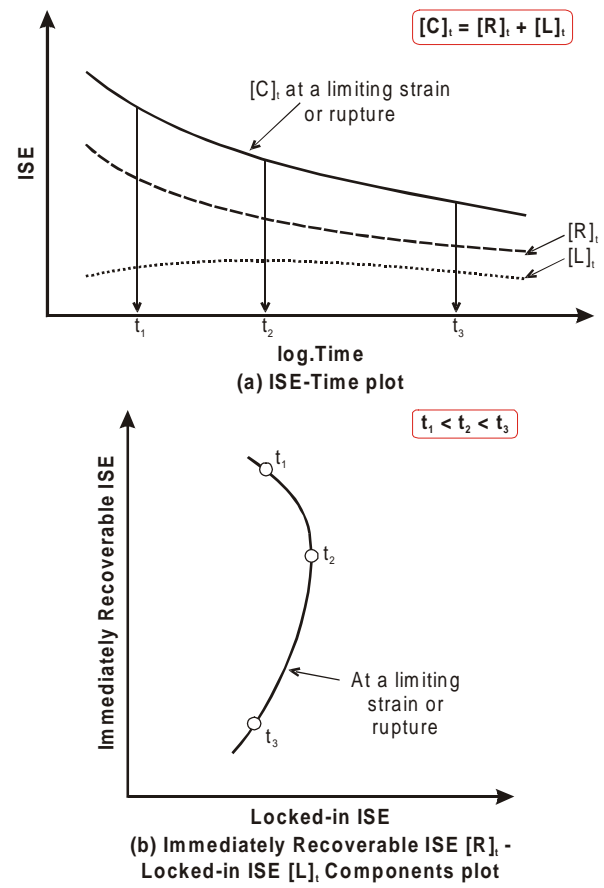


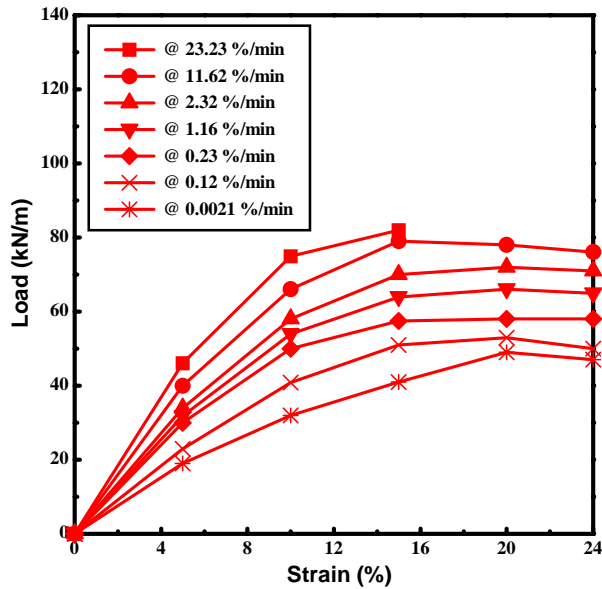
Figure 10 Possible variation of the ISE Components with time

4.3 Application of the ISE Approach to Test Data

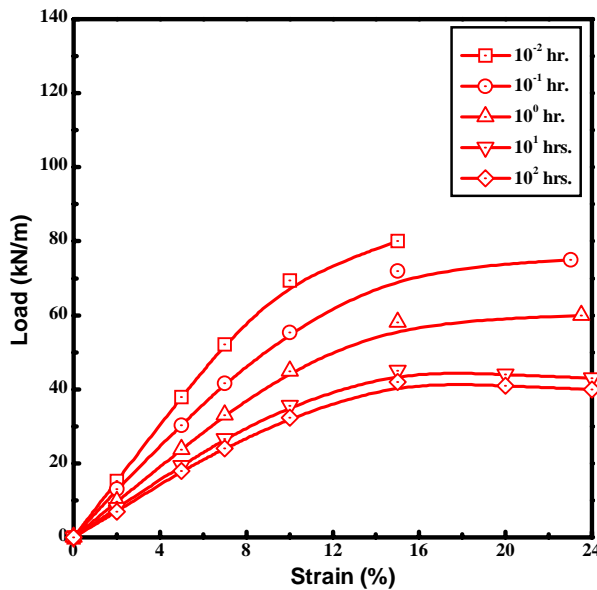
Five types of geosynthetics produced from different polymers by different manufacturing processes, were tested using different test methodologies by Kabir (1984) and Yeo (1985). Khan (1999) applied the ISE Approach to these data and proved it to be applicable to all of them. To illustrate this work, the test data for a uniaxial geogrid manufactured from stretched punched sheets of high density polyethylene are presented below.

4.3.1 Determination of the ISE Capacity

CRS testing was carried out on the uniaxial geogrid at different rates of strain and at a constant temperature of 20°C, Fig. 11(a), and the Isochronous Load-Strain curves drawn from the test data are shown in Fig. 11(b).

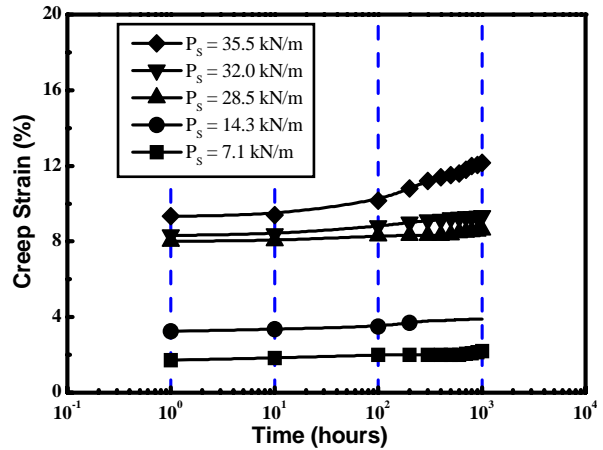


(a) Load-Total Strain curves

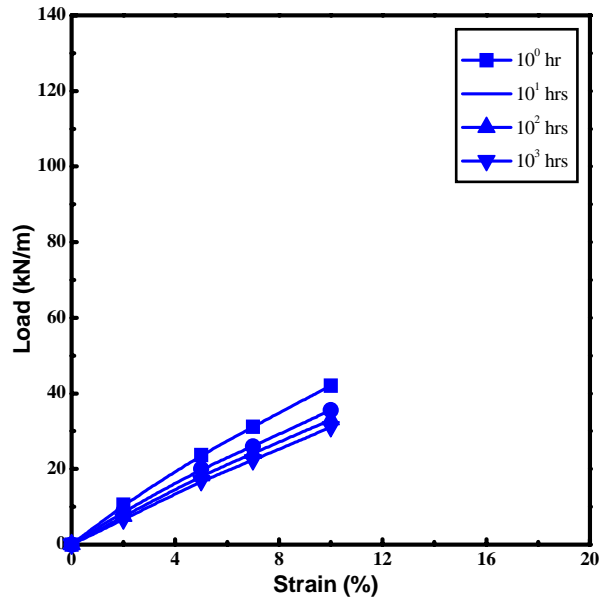


(b) Isochronous Load-Strain curves

Figure 11 Load-Strain and Isochronous Load-Strain curves for the uniaxial geogrid at 20°C from CRS tests (after Yeo, 1985)



(a) Creep Strain-Time curves

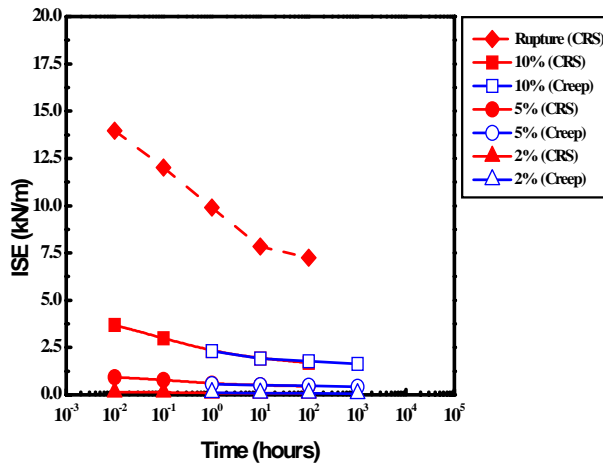


(b) Isochronous Load-Strain curves

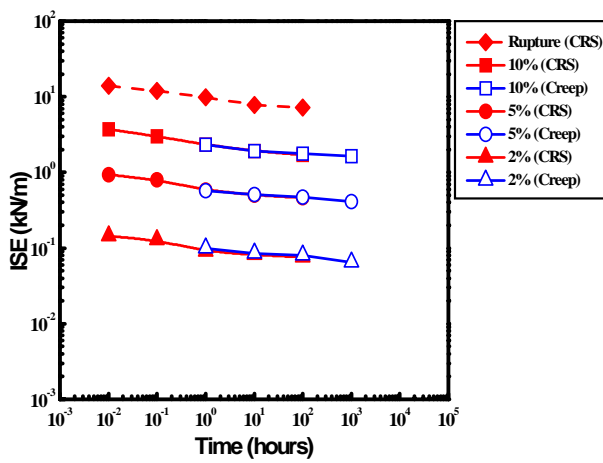
Figure 12 Creep and Isochronous Load-Strain curves for the uniaxial geogrid at 20°C (after Yeo, 1985)

Sustained load (creep) tests were also carried out at 20°C at different load levels. The test data were again plotted as Isochronous Load-Strain curves, Fig. 12 (a) and (b). The areas under these Isochronous curves were calculated at 2, 5 and 10 per cent strains, and plotted in an ISE-log.Time plot, as shown in Fig. 13 (a). These data were re-plotted to a log-log scale and the best-fit curves were drawn through the data at limiting strains, Fig. 13(b) and (c) respectively. It can be seen that the data from the CRS and creep tests lie close to the same best-fit curves. The best-fit curves represent the ISE Capacity $[C]_l$ of the uniaxial geogrid at each strain level for this material.

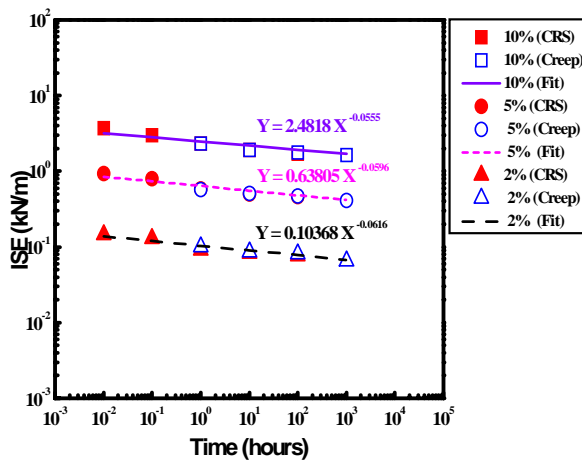
Strictly speaking, for a particular strain at a specific time, the load required to achieve this strain in a CRS test should be higher than the load required in a sustained load (creep) test. Therefore theoretically, the ISE Capacity $[C]_l$ at a particular limiting strain $[\epsilon]_l$ and at a specific time from CRS tests should always be higher than data from sustained load (creep) tests. However, for relatively short-term tests the response of many geosynthetics to loading is dominated by their initial elastic and plastic strains and a limited amount of rapidly developed primary creep, as previously indicated in Fig. 3.



(a) ISE-Time relationships



(b) ISE-Time relationships



(c) Best fit curves for ISE-Time relationships at a limiting strain

Figure 13 ISE - time plots for the uniaxial geogrid at 20°C

The result is that for all practical purposes, the ISE Capacity $[C]_t$ determined from different short-term test methods will be very similar and can be represented by the same best-fit curves. Therefore, such curves can be used to compare and correlate the data obtained from CRS and other short-term tests, including short-term sustained load (creep) tests. For very long periods of time, it is only practical to obtain data from long term sustained load (creep) tests, hence correlation between different test methods is not relevant over long time periods.

4.3.2 Determination of the ISE Components

The ISE components were derived from sustained load (creep) test data involving strain measurements during both loading and unloading, as was shown in Fig.8. The ISE capacity and its components for any Single-Stage Loading regime, i.e. $[C]_t$, $[R]_t$ and $[L]_t$, are then determined from the areas under the respective Isochronous Load - Strain curves for different times and strain levels or rupture, as was shown in Fig. 9. The test data from the uniaxial geogrid analysed in this way, show that the Isochronous Load - Immediately Recoverable Strain relationship for this geosynthetic is essentially time independent over the test duration of 10,000 hours, Fig.14.

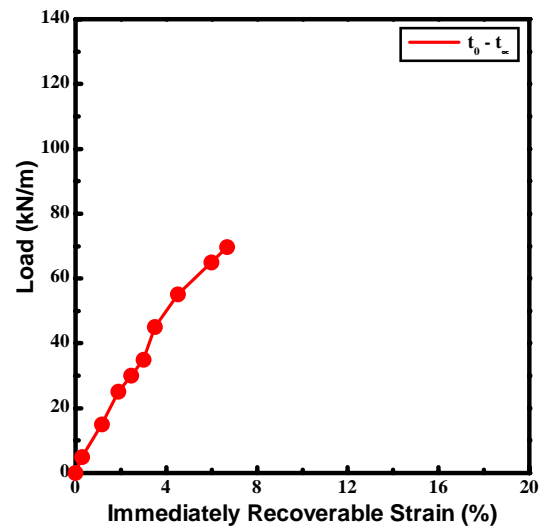
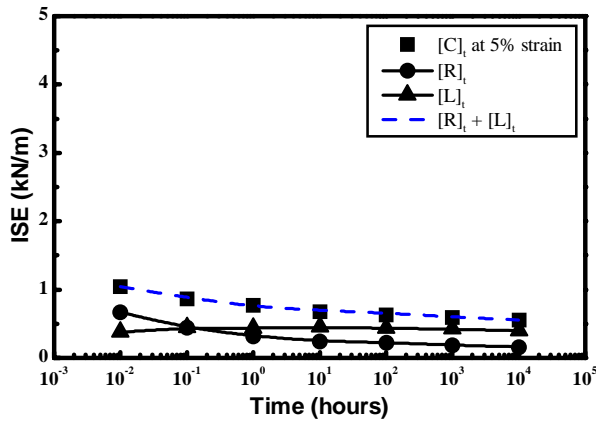


Figure 14 Isochronous Load - Immediately Recoverable Strain curve for the uniaxial geogrid at 20°C

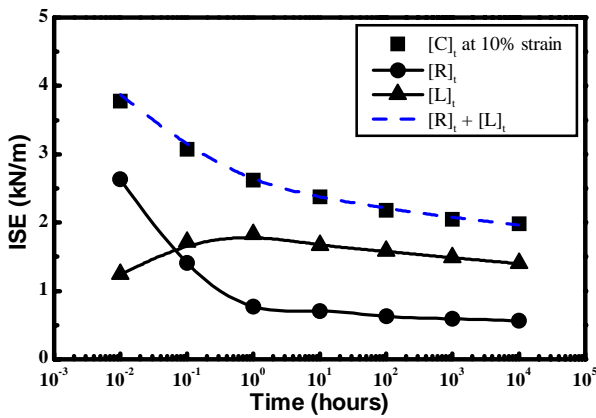
Figure 15 shows the variation of each of the ISE components with time at 5 and 10% limiting strains. It may also be noted that the summation of the Immediately Recoverable ISE $[R]_t$ and Locked-in ISE $[L]_t$ is equal to the ISE Capacity $[C]_t$ for any particular limiting strain. Further, this figure shows that if a limiting strain (say 10%) is reached in 0.1 hours, the amount of Immediately Recoverable ISE in the geosynthetic is likely to be higher than the Locked-in ISE. In contrast, if the same limiting strain is reached in 1000 hours, the amount of Locked-in ISE in the geosynthetic will be greater and may be higher than the Immediately Recoverable ISE. This means that totally different values of ISE components are measured in short-term tests than in long-term tests.

The combinations of ISE components, i.e. Immediately Recoverable and Locked-in ISE, are shown in Fig. 16 for the uniaxial geogrid B at 5% and 10% limiting strains, as determined by sustained load (creep) test data. This figure shows that the combinations of Immediately Recoverable and Locked-in ISE at a particular limiting strain, establish a unique boundary for a particular geosynthetic. Thus the material cannot reach a particular limiting strain in a specific time without a certain combination of Immediately Recoverable and Locked-in ISE under a Single-Stage Loading regime.

Although test data have not been presented for rupture conditions, the same pattern of behaviour is expected to be exhibited.



(a) ISE-log.Time plot at 5% limiting strain



(b) ISE-log.Time plot at 10% limiting strain

Figure 15 Variation of ISE Capacity and its Components with time for the uniaxial geogrid at 20°C

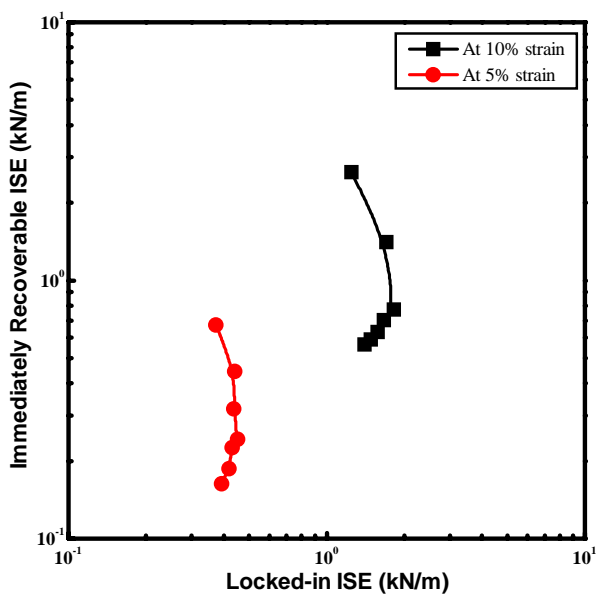
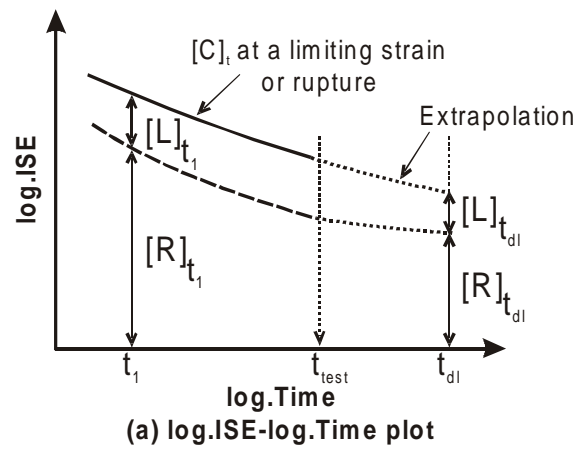


Figure 16 Combination of $[R]_t$ and $[L]_t$ ISE at different limiting strains for the uniaxial geogrid at 20°C

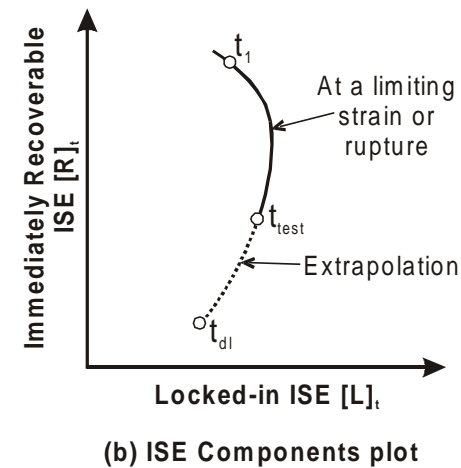
4.4 Extrapolation of Test Data Using the ISE Approach

Geosynthetic Reinforced Soil Structures are often designed for a specific operational period of time, which is greater than the duration of long term (creep) tests. It is therefore necessary to extrapolate test data to the required design lifetime $[t_{dl}]$ of the structure.

The extrapolation of ISE capacity $[C]_t$ and Immediately Recoverable ISE $[R]_t$ can be obtained by determining the best fit curves for these two parameters derived from the test data, Fig. 17(a). The difference between $[C]_t$ and $[R]_t$ then gives the extrapolated value of Locked-in ISE $[L]_t$ for the required design lifetime. This allows $[R]_t$ and $[L]_t$ values to be plotted beyond the test duration, Fig. 17(b), for either limiting strains or rupture conditions.



(a) log.ISE-log.Time plot



(b) ISE Components plot

Figure 17 Extrapolation of data beyond test duration on the basis of the ISE Approach

This procedure is illustrated in Fig. 18 for the uniaxial geogrid at limiting strains of 5 and 10%, extrapolated up to 2 log cycles, i.e. up to 1,000,000 hours. This extrapolation confirms the data obtained from rheological models used by Kabir (1984) and Yeo (1985).

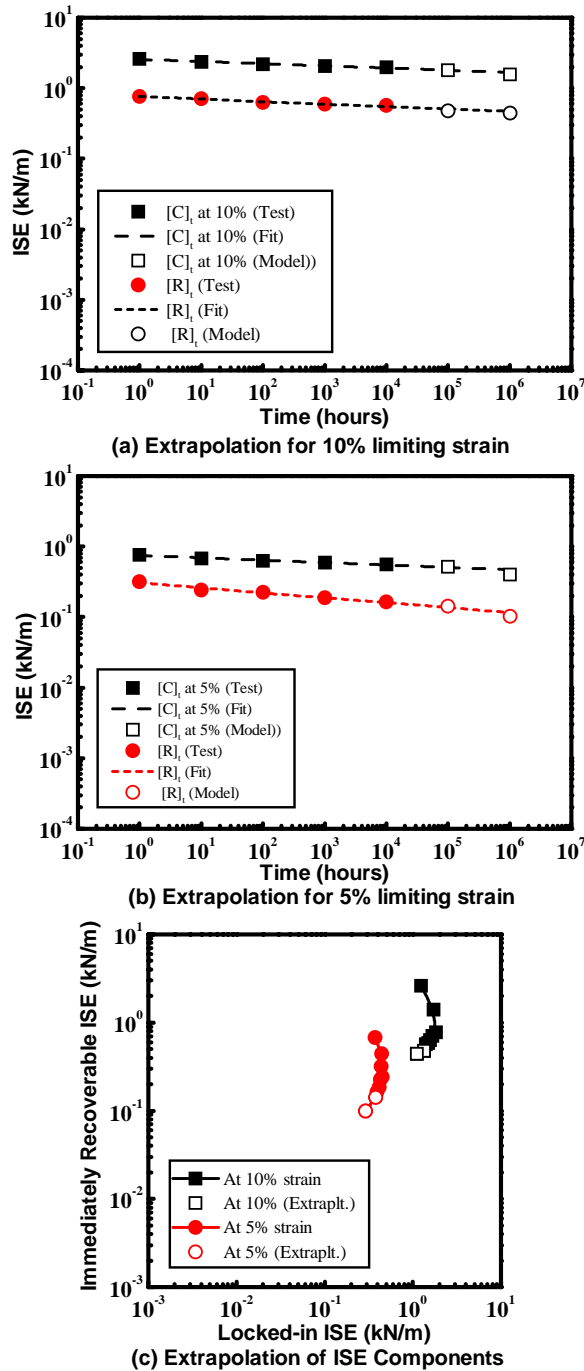


Figure 18 Extrapolation of test data for the uniaxial geogrid at 20°C

5 CHOICE OF DESIGN PARAMETERS FOR GEOSYNTHETICS USING THE ISE APPROACH

The design input parameters for geosynthetics for Single-Stage and Multi-Stage Actions may be identified using the ISE approach in the following ways.

5.1 Reference Strength for Single-Stage Actions

It was shown in Section 4, that for isothermal conditions at a particular time and strain level up to rupture, there is a single value of Absorbed ISE due to Single-Stage Loading. Thus it was shown that the isothermal ISE Capacity $[C]_t$ of 'Ex-work' geosynthetics is time and strain level dependent.

Further, it was shown that there is an unique combination of the Immediately Recoverable ISE $[R]_t$ and the Locked-in ISE $[L]_t$ for a given time and strain level or rupture.

Providing that test data extend over the design life of the structure, or more likely that test data have been extrapolated to this time, then the Design ISE Capacity and the ISE components can be identified from the plot of ISE Capacity against time, such as indicated in Fig. 17(a). The plot of Isochronous Loads against ISE Component Strains, Fig. 17(b), then allows the Immediately Recoverable and Locked-in Strains to be identified. For Single-Stage Actions these data are not significant, although as will be shown later, they are important factors in determining the response of geosynthetics to Multi-Stage Actions. Thus when using the ISE Approach to determine the Reference Strength for Single-Stage Loading the procedure is simply that both the design lifetime and design strain levels are specified and the Reference Load is taken directly from the Isochronous Load-Strain curves from which the ISE Capacity was derived.

5.2 Partial Factors for Single-Stage Actions

The amount of Absorbed ISE to reach a limiting strain or rupture at a specific time is dependent on the polymeric composition, micro and macro structures of the geosynthetic. Therefore, if a geosynthetic is physically or chemically altered, then the amount of ISE required to reach a limiting strain or rupture under a Single-Stage Loading regime, will also change. Such changes in ISE

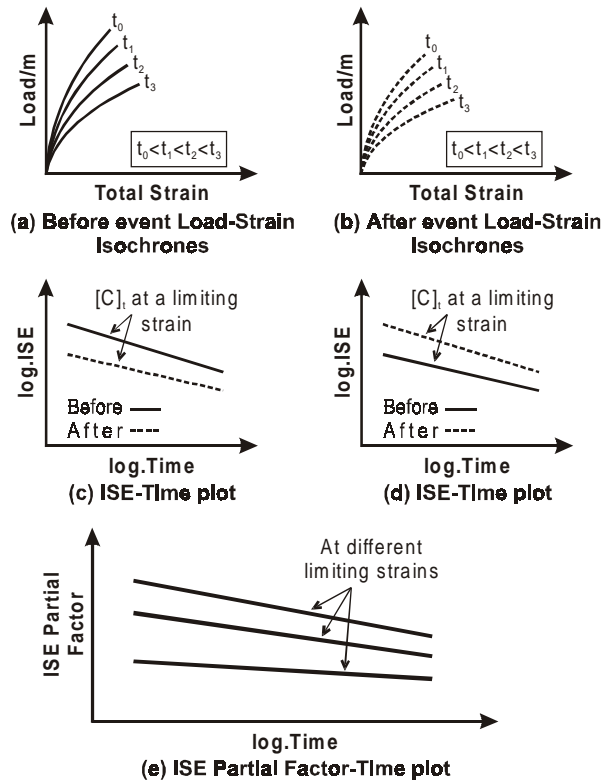


Figure 19 Determination of Partial Factors on the basis of the ISE Approach

may then be used to identify appropriate values of Partial Factors and eliminate the difficulties of interpreting data obtained from different test methodologies. Further, the ISE approach can be used to identify different effects of construction damage and environmental degradation on the ISE components, i.e. the Immediately Recoverable and Locked-in ISE components.

Thus Partial Factors can be redefined as the ratios of the ISE Capacity of the geosynthetics at a limiting strain or rupture 'before' events $[C]_{t(\text{before})}$ to the ISE Capacity of the geosynthetics at a limiting strain or rupture 'after' events $[C]_{t(\text{after})}$, i.e. 'before' and 'after' construction damage or environmental degradation.

The procedure for determining Partial Factors on the basis of the ISE Approach is shown in Fig. 19. The test data should be first converted to Isochronous Load-Strain curves for the geosynthetic 'before' and 'after' the events, Fig. 19(a) and (b). The ISE Capacities 'before' the event $[C]_{t(\text{before})}$ and 'after' the event $[C]_{t(\text{after})}$ are then calculated from the areas under the Isochronous Load-Strain curves for limiting strains at various times. These data can be shown as ISE-Time plots, examples of which are Figs. 19(c) and (d). The ISE Partial Factors are calculated by dividing the $[C]_{t(\text{before})}$ with the $[C]_{t(\text{after})}$ and the variations of these Partial Factors are shown in Fig. 19(e).

Al-Mudhaf (1993) and Esteves (1996) have studied the effects of construction damage and environmental degradation for a wide range of geosynthetics. CRS and sustained load (creep) tests were carried out on the geosynthetics 'before' and 'after' they were subjected to the site damage or long-term exposure. The specimens 'before' and 'after' these activities were termed 'Ex-works', 'Damaged' and 'Exposed' specimens respectively. Khan (1999) has analysed these test data using the ISE Approach. To illustrate this work, Isochronous Load-Strain curves for the 'Ex-works' and 'Damaged' specimens of the uniaxial geogrid considered previously, and a biaxial polypropylene geogrid, are plotted from the test data, Fig. 20 (a) and (b). The areas under these Isochronous Load-Strain curves were calculated for 2, 5 and 10 percent strains, and plotted in an ISE-Time plot, as shown in Fig. 21. The Damage Factor was then calculated according to the definition of ISE Partial Factors given previously. The variations of Damage Factors for the uniaxial geogrid and biaxial geogrid with time and strain levels are shown in Fig. 22. Using CRS and sustained load (creep) tests carried out 'before' and 'after' 12 months of exposure, the same procedure was used for the determination of Environmental Factors. The variation of Environmental Factors for the uniaxial geogrid and biaxial geogrid are given in Fig. 23.

The magnitude of the improvement or degradation of the properties of a geosynthetic identified in the manner described may be shown to be dependent on how the Immediately Recoverable ISE $[R]_t$ and the Locked-in ISE $[L]_t$ components are affected. Indeed the ISE Components may be differentially affected by construction damage and environmental degradation. For some geosynthetics the effect on the Immediately Recoverable ISE $[R]_t$ may be more than on the Locked-in ISE $[L]_t$ and vice-versa. Also, the effect on each of the components may be differential with time, strain level and temperature. Thus the current practice of using Partial Factors obtained from short-term test data to obtain long-term Design Strengths is likely to be problematic.

To illustrate the possibility of differential effects on the ISE components from damage and environmental exposure,

Khan (1999) further analysed the test data for the uniaxial geogrid and a biaxial geogrid.

The Immediately Recoverable ISE $[R]_t$ and the Locked-in ISE $[L]_t$ 'before' and 'after' were calculated, which allows the Immediately Recoverable ISE $[R]_t$ -Time and Locked-in ISE $[L]_t$ -Time plots to be developed 'before' and 'after' an event, Figs 24 and 25. The effects on the Immediately Recoverable ISE $[R]_t$ and the Locked-in ISE $[L]_t$ can then be determined by comparing the amount of change of these components to their original value.

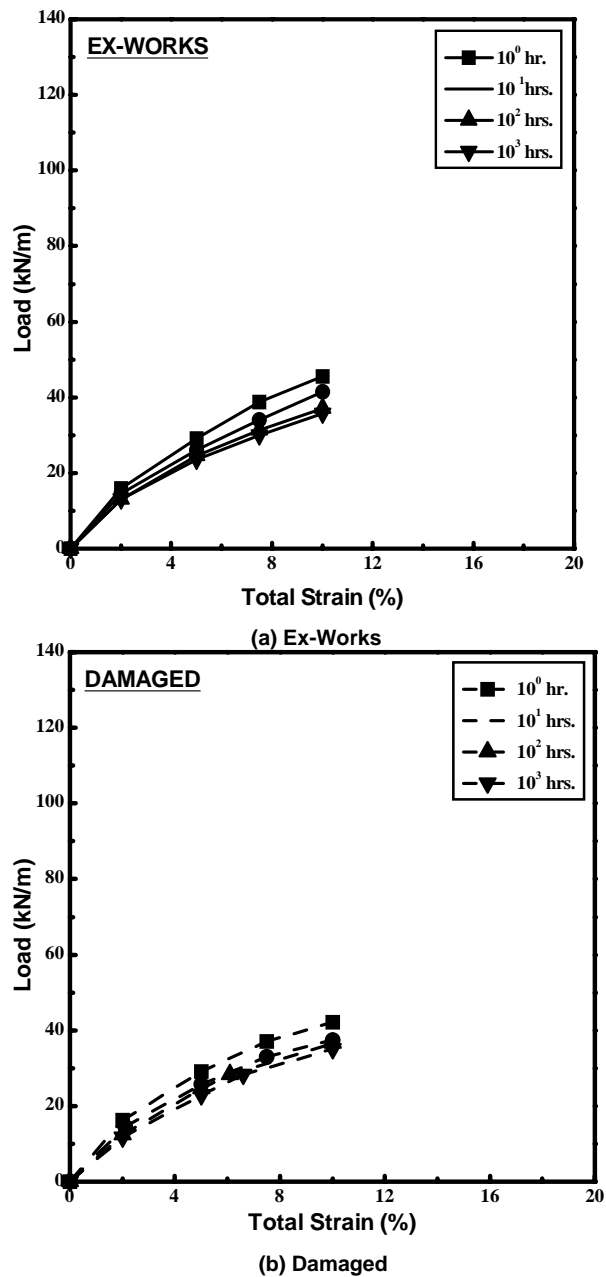


Figure 20 Isochronous Load-Strain curves for Ex-works and Damaged uniaxial geogrid at 20°C (after Esteves, 1996)

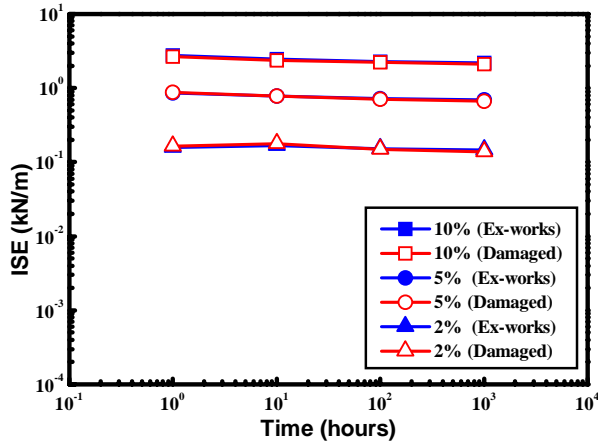


Figure 21 ISE-Time relationships for the uniaxial geogrid at 20°C

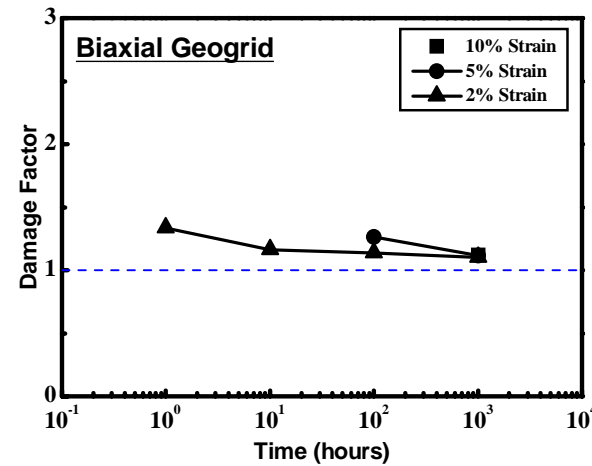
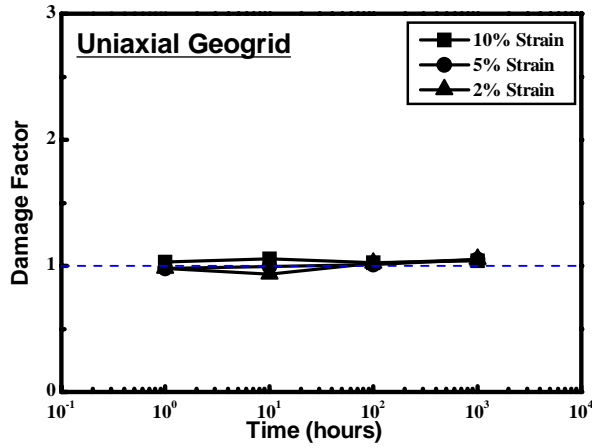


Figure 22 Damage Factor-Time relationships at 20°C

These data show that the effects of damage and exposure on both the Immediately Recoverable ISE $[R]_t$ and the Locked-in ISE $[L]_t$ may vary with time. This clearly suggests that tests carried out for a very short period of time will measure totally different amount of changes of ISE components than for tests carried out over longer periods of time.

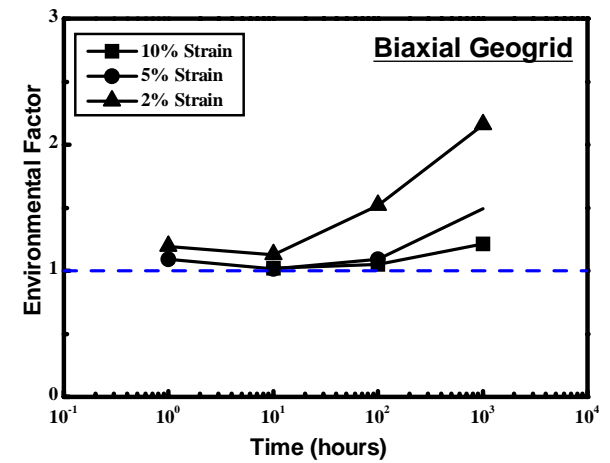
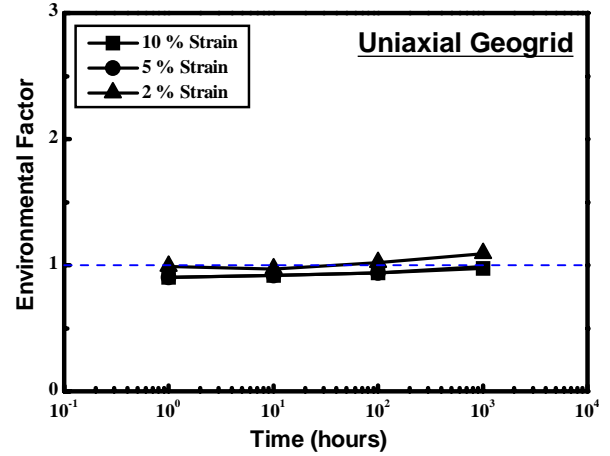


Figure 23 Environmental Factor-Time relationships at 20°C

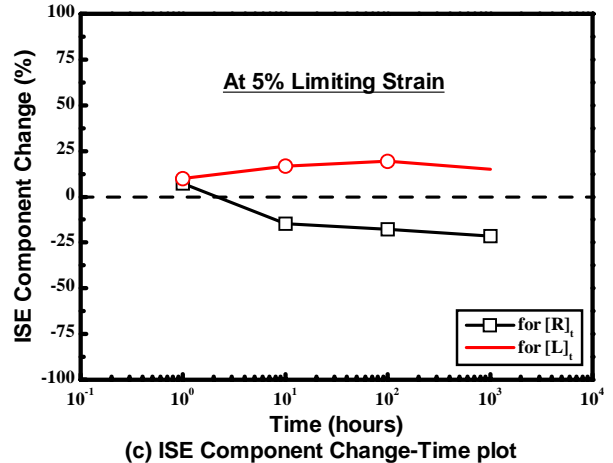
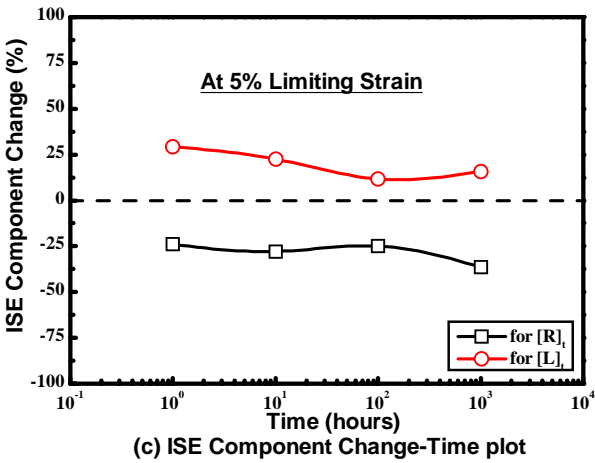
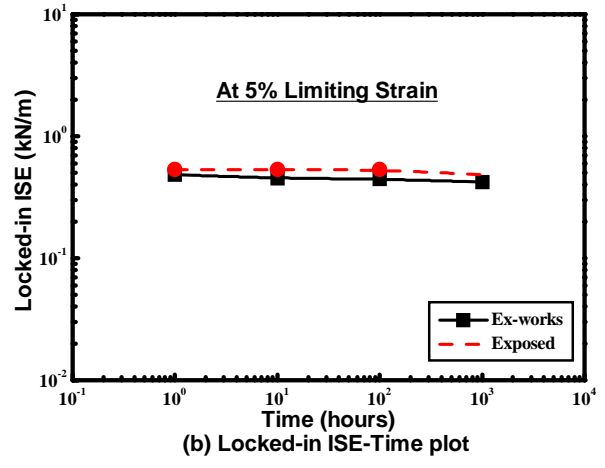
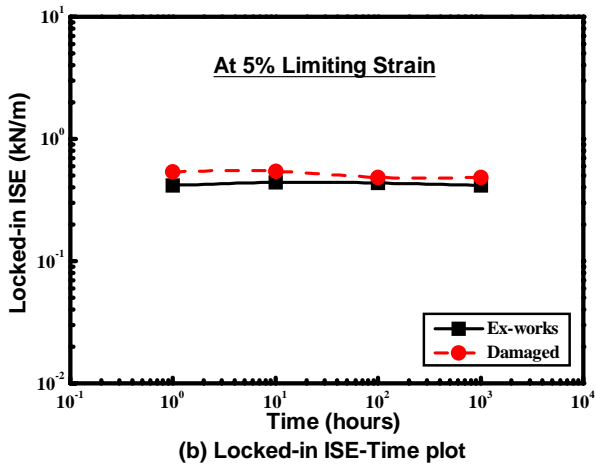
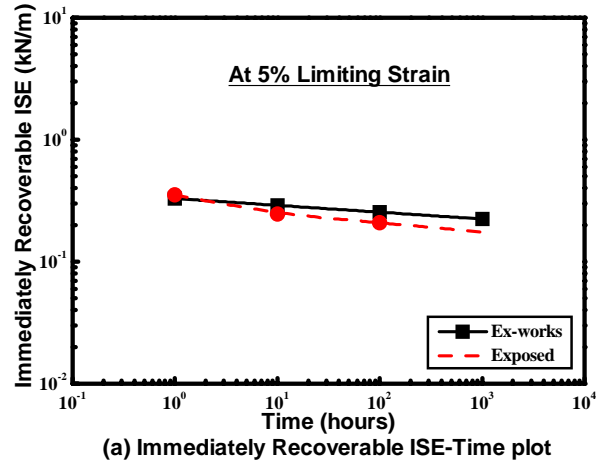
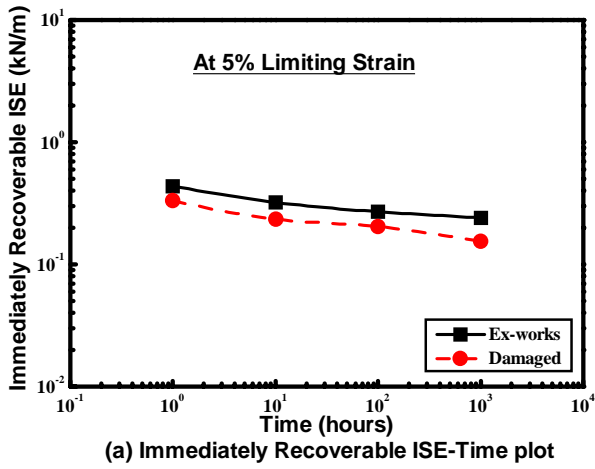


Figure 24 Effects of damage on the ISE Components of the uniaxial geogrid at 20°C

Figure 25 Effects of environmental exposure on the ISE components of the uniaxial geogrid at 20°C

5.3 Design Strengths for Single-Stage Actions

To obtain the Design Strength of geosynthetics using the ISE approach, the procedure to be followed is similar to that used to determine the Reference Strength. First, the variation of the Design ISE Capacity with time needs to be established. This is found by modifying the Ex-works ISE Capacity using the Partial Factors derived from the ISE Approach, as shown in Fig.26. Next the ISE components and the Isochronous Load-Strain curves may be identified and the Design Strength determined for any time and strain level or rupture. However, the Design Isochronous Load-Strain curves will be very similar in shape to the Ex-works Isochronous Load-Strain curves, hence the areas under these curves, that is the Absorbed ISE for any strain level or rupture, will be proportional to the ordinate, i.e. the Strength. Thus the Design Strength may be directly obtained as follows:

$$\text{Design Strength} = \frac{\text{Reference Strength}}{\text{ISE Partial Factors}}$$

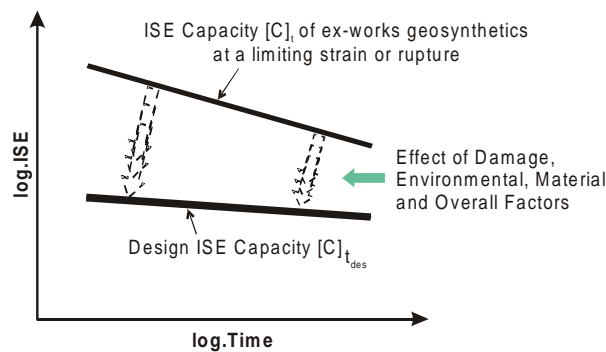


Figure 26 Derivation of Design ISE Capacity

5.4 Design Strengths for Multi-Stage Actions

It was shown in Section 4.2 that at any time, the Absorbed ISE in a geosynthetic comprises two components, the Immediately Recoverable ISE $[R]_i$ and the Locked-in ISE $[L]_i$. Further, it was suggested that for ISE Capacities at limiting strain levels or rupture, the ISE Components were intrinsic properties controlled by the polymeric composition, micro and macro structures of the geosynthetic.

When a geosynthetic is subject to Multi-Stage Actions, it will develop Absorbed ISE at different strain levels at different times. This Absorbed ISE will comprise different combinations of Immediately Recoverable ISE $[R]_i$ and the Locked-in ISE $[L]_i$. If these ISE Components combine in such a way as to exceed intrinsic limits then limiting strain levels or rupture will develop, as shown in Fig. 27. Thus for the case of Multi-Stage Actions, it is the critical combinations of ISE Components that must be used to define the Reference Strength. In effect this means that a single value of Reference Strength cannot be identified for geosynthetics subject to Multi-Stage Actions. Rather the strength of the geosynthetic will vary as a function of the nature and sequence of the Actions to which it will be subjected, i.e. it is a function of its stress history and in this sense its behaviour is similar to the behaviour of many types of soil.

To deal with the difficulty of identifying the Reference Strengths of geosynthetics subject to Multi-Stage Actions, it is suggested that the approach to the design of GRSSs should be different in such situations. This new approach has been termed the *Modified Materials Approach* and is probably best described by providing the following example.

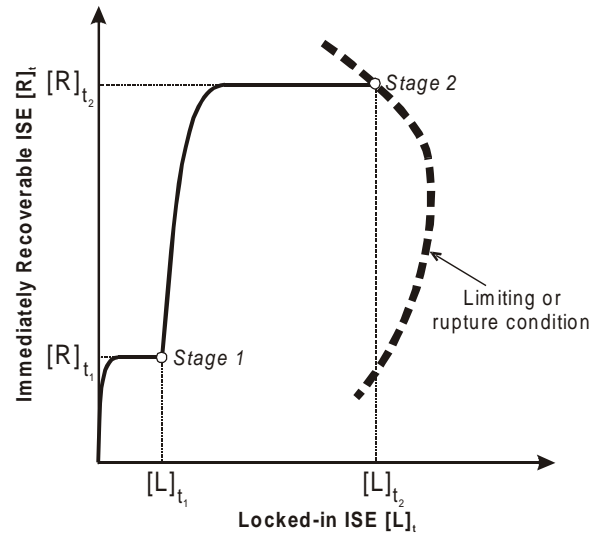


Figure 27 Immediately Recoverable and Locked-in ISE Components for a Multi-Stage Loading condition up to a limiting strain or rupture

For a GRSS subject to combined sustained loading plus a short-term loading, the design should be approached in a series of stages. First, the GRSS should be analysed for the sustained load as a Single-Stage Action using a reduced value of the Single-Stage Action - Design Strength. This reduced Design Strength is obtained by factoring the Single-Stage Action - Design Strength, which implies that the Single-Stage Reference Strength and Partial Factors otherwise used are applicable. However, the additional factor reduces the strain developed and so the strength available. It ensures that the strain level developed is less than the design limit strain or rupture, Fig. 28. The difference in the strain allows the geosynthetic to accommodate the strains induced by the Immediately Recoverable ISE $[R]_i$ and the Locked-in ISE $[L]_i$ resulting from the Short-term loading.

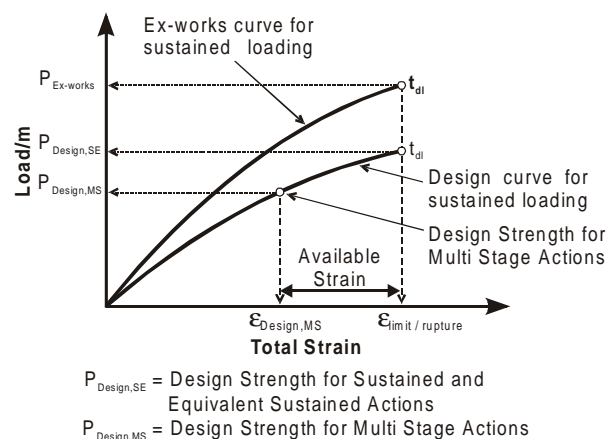
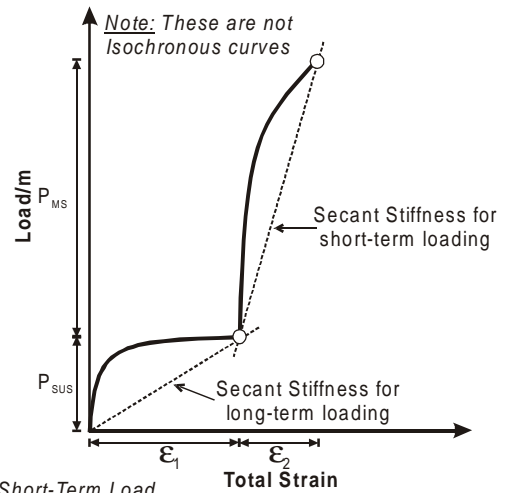
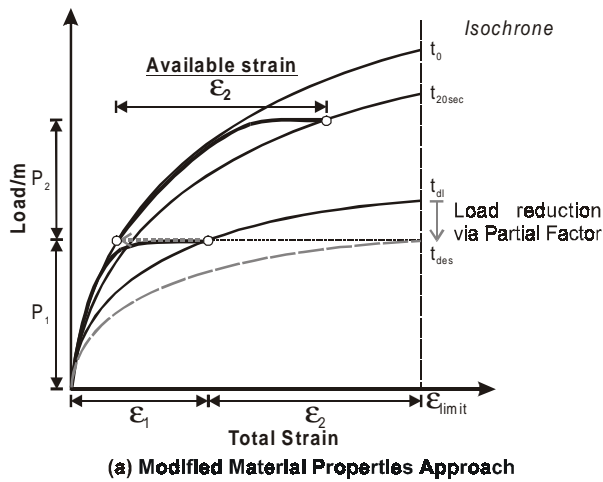


Figure 28 Modified Material Properties Approach for design against sustained loading plus short-term loading

The Geosynthetic Reinforced Soil Structure may then be further analysed for the Short-term load using the Isochronous Load-Strain plots in Fig. 29. Combining these provides the Load-Total Strain relationship, Fig.30. This plot shows that the geosynthetic exhibits quite different stiffnesses during the two stages of loading.



P_{MS} = Short-Term Load
 P_{SUS} = Long-Term Load
 ϵ_1 = Strain developed during long-term loading
 ϵ_2 = Strain developed during short-term loading

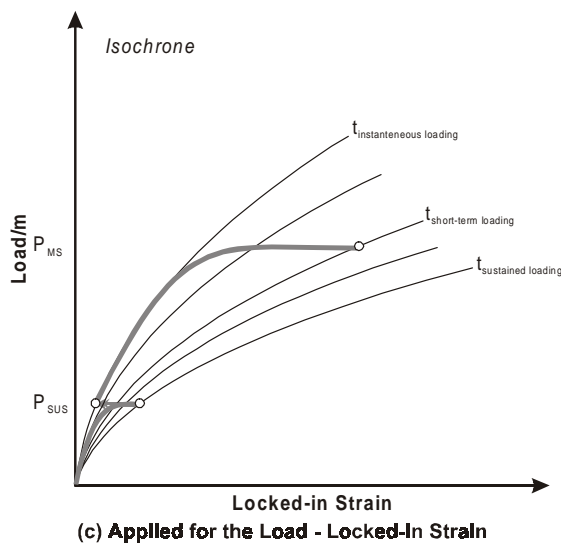
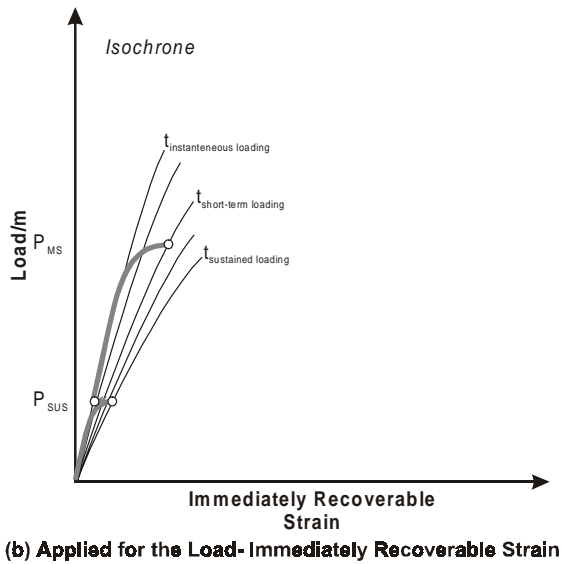


Figure 29 Modified Materials Approach for geosynthetics

Figure 30 Secant Stiffnesses for short and long-term loading

6. SOME IMPLICATIONS OF THE ISE APPROACH

6.1 General

As previously stated, current practice does not effectively take account of the elasto-visco-plastic nature of geosynthetics. Thus it has been suggested that the changes in isothermal behaviour of geosynthetics with time, strain level, damage and environmental exposure are not being effectively taken into account in current design codes / methods. In addition, most design codes / methods treat all Actions as pseudo-static loads although Actions vary greatly in nature and duration.

The ISE Approach to the characterisation of the isothermal load-strain-time behaviour has identified a number of differences with current practice which have significant implications for the design of Geosynthetic Reinforced Soil Structures. Some of these implications are discussed in the following sections.

6.2 Sustained Actions

For Sustained Actions the use of the ISE Approach has led to the suggestion that the isothermal Reference Strength of geosynthetics should be determined from the Isochronous Load-Strain curves. This is not a new suggestion, rather it confirms the previously stated views of many researchers. However, the ISE Approach has given further support to this view.

Further, the ISE Approach has led to the suggestion that Partial Factors are likely to be time and strain level dependent. Also, that they should be based on the change in Absorbed ISE 'before' and 'after' damage or environmental exposure. This is a novel idea and may lead to some factors being more conservative and others less conservative than current values. Importantly, it very much calls into question the validity of using short-term CRS tests to determine Partial Factors related to long-term applications, as is the present practice.

Overall, the above suggestions relating to the Reference Strength and Partial Factors, mean that the current approach in many design codes / methods of identifying a single value of Design Strength for a geosynthetic without regard to the design lifetime or a limiting strain level must be seriously questioned.

6.3 Equivalent Sustained Actions

The design implications of the ISE Approach are the same for both Sustained Actions and Equivalent Sustained Actions. Additionally, the ISE Approach can provide justification for the use of Equivalent Sustained Action to represent combinations of some types of Actions and can result in the use of less conservative values of the Equivalent Load. For example, for road pavement design sustained loading plus traffic loading is commonly modelled by an Equivalent Sustained Loading with the value of the traffic loading taken as a static wheel load acting over the design lifetime of the road. This is a very conservative approach which does not take account of the transient nature of the wheel loading nor of the possibility of recovery of some of the Absorbed ISE between loading cycles.

To illustrate the use of the ISE Approach to determine the Equivalent Sustained Loads that should be used to represent combined loads, Khan (1999) undertook laboratory tests with combined sustained loading plus cyclic loading intended to model sustained loading plus traffic loading. The tests were conducted at 20°C on the uniaxial geogrid described in Section 4. He used sustained loads of 10, 15, 20 and 25 kN/m and a cyclic load of 5 kN/m applied at a frequency of 0.1 Hz for 36,000 cycles without any rest periods. The sustained loads were applied for 100 hours prior to the application of the cyclic loading. In addition, he carried out sustained load (creep) testing on the same geosynthetic at load levels equal to the upper, lower and average of the combined loads in the cyclic testing. The test data obtained from these tests are presented in Fig. 31.

Using the ISE Approach the data from the testing may be interpreted in the following manner:

- At the end of the 100 hours of sustained loading there would be a certain amount of Immediately Recoverable ISE $[R]_t$ and Locked-in ISE $[L]_t$ within the geosynthetic.
- On application of the first transient load, both the Immediately Recoverable ISE $[R]_t$ and Locked-in ISE $[L]_t$ within the geosynthetic would increase.
- On removal of the first transient load, the Immediately Recoverable ISE $[R]_t$ would revert back to its original value but most of the Locked-in ISE $[L]_t$ would remain within the geosynthetic.
- With successive cycles the Immediately Recoverable ISE $[R]_t$ would continue to develop and recover but the Locked-in ISE $[L]_t$ within the geosynthetic would gradually increase.
- With the loading regime applied in this test series, the net effect was to produce a behaviour close to that for a sustained load equal in value to the sustained load applied plus half the cyclic loading, Fig. 32.

Thus the analysis of the test data using the ISE Approach confirms that some forms of combined loading may be reasonably modelled by an Equivalent Sustained Action and that the current practice of including all of the transient loading within this may be very conservative.

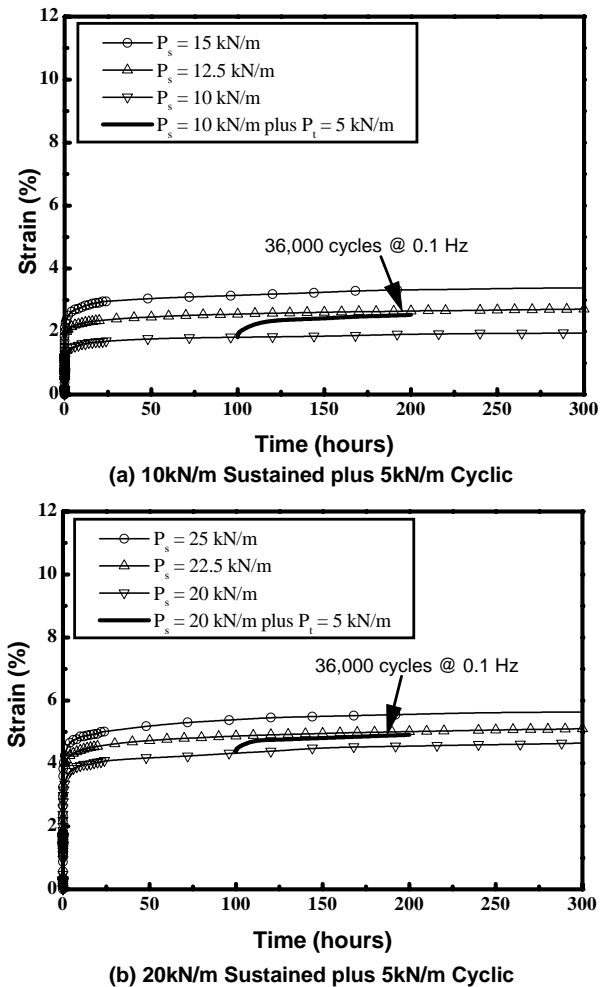


Figure 31 Results of combined sustained-cyclic loading for the uniaxial geogrid at 20°C

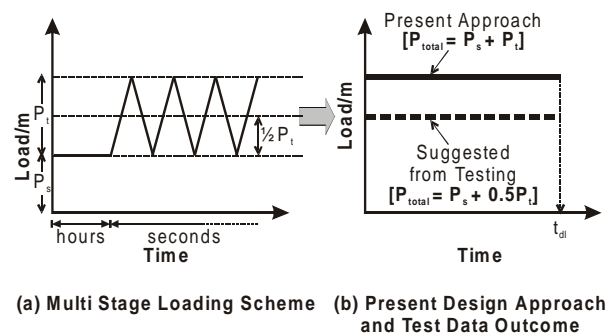


Figure 32 Present approach and test outcome for design against sustained plus cyclic loading

6.4 Multi-Stage Actions

For Multi-Stage Actions, the use of the Modified Materials Approach was suggested for use in the design of Geosynthetic Reinforced Soil Structures. An example of Multi-Stage Action is sustained loading plus short-term (earthquake) loading on walls and slopes.

To date the development of designs for walls and slopes involving earthquake forces has been empirically based. Fukuda et al (1994) reported that until 1993, designs

for earthquake loading were based on the procedure for structures subject to sustained loading suggested by Jewell et al (1984). In this procedure the long-term creep rupture strength of geosynthetics was used as the Reference Strength. The structures so designed, were reported by Collin et al (1992) to have maintained their stability during the Loma Prieta earthquake in 1989, which had a magnitude of 7.1. Fukuda et al (1994) also reported a similar situation following the Kushiro Offshore earthquake in 1993, which had a magnitude of 7.8. These data were taken to indicate that geosynthetics were capable of taking higher loads applied rapidly, than the long-term creep strength used in their design. On this basis, Fukuda et al (1994), AASHTO (1994) and Jones (1996) suggested that the Reference Strength of geosynthetics for sustained loading should be increased by 1.5 times when designing for sustained loading plus short-term earthquake loading. In more recent design codes / methods, as even more confidence was gained from the performances of Geosynthetic Reinforced Soil Structures during earthquakes, factored short-term CRS strengths of geosynthetics were suggested for use in designs against sustained loading plus short-term earthquake loading, e.g. AASHTO (1997), NCMA (1997) and DIBt (1998). However, it should be noted that these suggestions are all empirically based and have not been technically justified in detail.

Having regard to the elasto-visco-plastic nature of geosynthetics it can be stated that the strength available to resist short-term earthquake loading of geosynthetics following different periods of sustained loading will vary. Thus the current practice of using a single value of Design Strength based on either factored long-term sustained loads or factored short-term CRS strengths must be questioned. In addition, no design codes / methods have dealt with the determination of Partial Factors for geosynthetics under Multi-Stage Actions. Usually, Partial Factors are obtained from short-term CRS test data which has been questioned in previous sections. Thus it is suggested that the Modified Materials Approach is likely to be more appropriate.

To illustrate the use of this Modified Materials Approach, Kupec (2000) undertook a series of laboratory tests involving sustained loading plus short-term loading. The short-term load was chosen to represent short-term earthquake loading. Usually, earthquakes are cyclic in nature with irregular frequency, however to avoid the complexities of simulating these cycles, it was represented by a uniform load applied over 20 seconds. The 20 second duration was chosen on the basis of the durations of the main strokes of the Kushiro Offshore and Northridge earthquakes, as reported by Fujii et al (1996) and Frankenberger et al (1996), respectively, which is more critical than actual earthquake loading.

The tests were all carried out at 20°C using the same uniaxial geogrid described in Section 4. The sustained loading [P_s] of 25 kN/m was applied for 200 hours. Short-term loading [ΔP_s] was applied after 100 hours for 20 sec at five load levels from 10 to 50 kN/m, in increments of 10 kN/m, Fig. 33. The maximum total load of 75 kN/m was the same value as the strength obtained from CRS testing at 20°C and a strain rate of 25% per minute.

Figure 34 shows the test data for the period of the short-term loading and shortly afterwards. It can be seen that only with the additional Short-term load of 50 kN/m in Stage2 did the material strain to rupture. For the lower levels of Short-term load, it can be seen that the Total Strain in Stage3 reduced to an almost constant value within the next 100 hours. Indeed for Short-term loads of 10 and 20 kN/m, the strain behaviour in Stage3 rapidly approached that obtained from the sustained load (creep) test under the load of 25 kN/m.

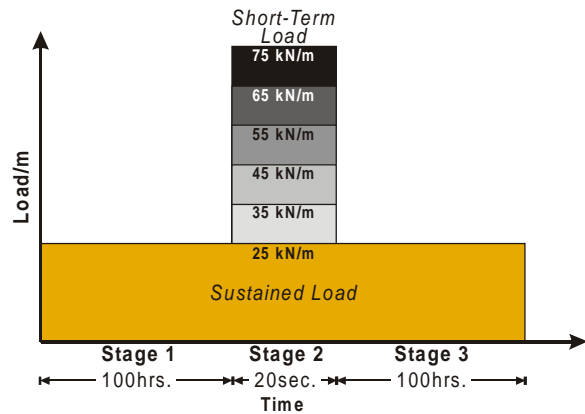


Figure 33 Loading scheme for the sustained plus short-term loading tests

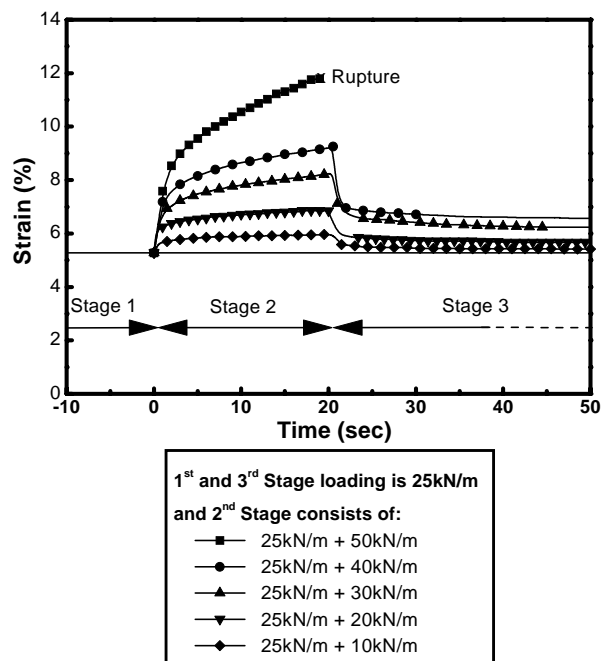


Figure 34 Sustained plus Short-term Loading test results for the uniaxial geogrid at 20°C

Using the ISE Approach the test data may be interpreted in the following manner:

- At the end of the 10 hours of sustained loading there would be a certain amount of Immediately Recoverable ISE [R_t] and Locked-in ISE [L_t] within the geosynthetic.
- On application of the Short-term load both the Immediately Recoverable ISE [R_t] and Locked-in ISE [L_t] within the geosynthetic would increase.
- On removal of the Short-term Load, the Immediately Recoverable ISE [R_t] would revert back to its original value but most of the Locked-in ISE [L_t] would remain within the geosynthetic.
- With time, the Locked-in ISE [L_t] from the Short-term Load would gradually decrease but over the same time the Locked-in ISE [L_t] from the sustained load would gradually increase. Overall the Absorbed ISE would decrease until the Absorbed ISE developed from sustained loading caused it to gradually increase again.

- (e) With the loading regime applied in this test series, the net effect of the Short-term load was therefore to produce rupture at a very high load or a temporary increase in the strain developed in the geosynthetic over those developed by the sustained loading.

With regard to Short-term loads such as earthquakes and temporary surcharges, the implication of the ISE Approach is that the timing of the loading within the design lifetime of a structure is important. For example, if an earthquake occurs after say 15 years then it may not cause failure / rupture of the geosynthetic reinforcement. If the same earthquake occurs after 120 years then it may do so, Fig. 35. Thus the critical design condition for Short-term loads, such as an earthquake or temporary surcharge, must be the maximum possible Short-term load occurring at the end of the design lifetime of the structure. By allowing for the amount of strain induced by the Short-term load at the end of the design lifetime a safe design may be achieved using the proposed Modified Material Approach.

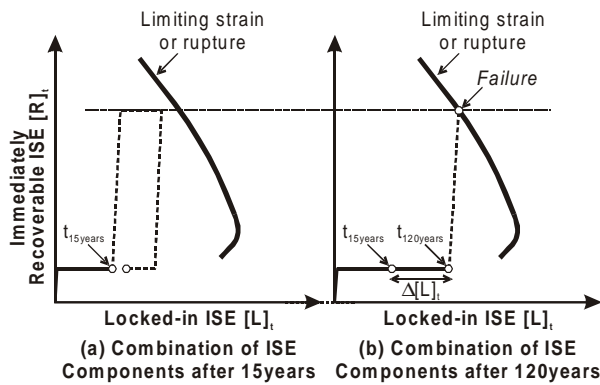


Figure 35 Combinations of ISE Components at different times during the lifecycle

7 DISCUSSION

A number of aspects of the current design practice for Geosynthetic Reinforced Soil Structures have been critically assessed and suggestions made for modifications and changes to these. Some of these modifications and changes involve significant technical differences and will not be easy to implement without considerable further research and development work. Although these may prove to be technically justifiable, it must be shown that they have sufficient cost benefits to make them worthwhile.

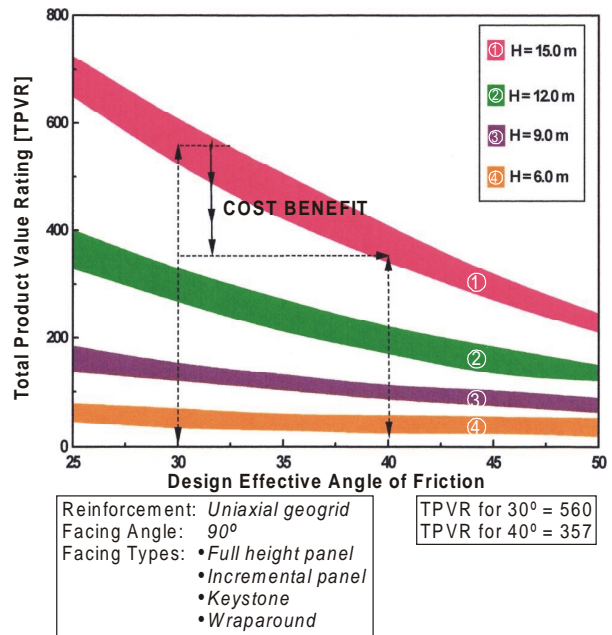
In order to investigate the economic implications of the choice of design values for the angle of friction of the soil on outcome designs, analyses of some typical Geosynthetic Reinforced Soil Structures were undertaken using the Limit State “Tensoil” program. This was developed by Pradhan (1996) and reported on in detail by McGown et al (1998). With the exception of the soil parameters all the design parameters were those specified in current design codes / methods. Walls of different heights, viz. 6m, 9m, 12 m and 15m, with a flat surface resting on a competent foundation and subject to Sustained Actions (self weight), were analysed with several different facing types. Compacted granular reinforced fill was assumed and the Constant Volume Angle of Friction adopted to represent its strength. The values adopted were varied from 25° to 50° in increments of 5°. The most economical reinforcement layouts were determined for all the cases. The reinforcement costs were then calculated

and expressed in terms of “Total Product Value Rating” (TPVR):

$$TPVR = \frac{\text{Number of Reinforcing Elements} \times \text{Length [m]} \times PVR}{\text{Horizontal Spacing [m]}}$$

Where the “Product Value Rating” (PVR) is a relative costing for the reinforcing elements.

Figure 36 shows that the TPVR decreases with increase of the Angle of Friction for the soil at a particular height of wall. Further it shows that the influence of this parameter becomes more significant as the height of the wall increases. It is evident that if a value of 30° for the Constant Volume Angle of Friction is used, similar to the value used in the case histories referred to previously, then the TPVR for the 15m high wall would be 527. However, if the average value for the Constant Volume Angle of Friction of 40° is employed, the value suggested by the test data referred to in Section 3.4.1, then the TPVR for the same wall would be 345. This represents a cost benefit related to the geosynthetic reinforcement costs of around 35% and highlights the need to use realistic, less conservative soil strength values in designs.



Benefit of more than 30% in terms of geosynthetics costs

Figure 36 Cost benefits in terms of the Total Product Value Rating

Alternatively, the current use of low design values for the shear strength of the reinforced fill could be taken to infer that poorer quality fill could safely be used with the same amount of geosynthetic reinforcement. This could once again represent a substantial cost saving.

Further, design examples were analysed involving Sustained Actions (self weight), Equivalent Sustained Actions, (self weight plus traffic loading) and Multi-Stage Actions (self weight plus earthquake loading). In these examples the values for the geosynthetic design parameters most commonly used in current practice were employed. The outcome designs from these were then compared to outcome designs obtained by using the new values of geosynthetics properties, as identified in this paper. The results obtained showed that cost benefits could be obtained in terms of the geosynthetic reinforcement costs

of the order of 30 to 50% or again that poorer quality fill could be used with substantial cost savings.

Nevertheless, it must be appreciated that great care needs to be taken in seeking to achieve these benefits. Most of the current designs are based on Limit Equilibrium Methods with some using a Hybrid Approach involving aspects of both Limit Equilibrium and Limit State Analysis. These design codes / methods have nearly all been “manipulated” to produce consistent design outcomes. Thus they contain both factors and procedures which modify outcome designs. This is perhaps illustrated by the outcomes from the three design codes / methods shown in Table 1. The data in this Table show that for the same wall design using the same soil and geosynthetic reinforcements, the quantity of geosynthetic required is only modestly different. This is in spite of the fact that the Design Strengths for the geosynthetic varied much more. Indeed it can be seen that the code adopting the highest design strength actually indicated the highest required amount of reinforcement.

Design Codes / Methods	Design Strength [kN/m]	Length of reinforced section [m]	Number of Reinforcements [i]	Quantity [m ² /metre length of wall]
DIBt (1998)	17.32	6.0	19	131
BS8006 (1995)	26.17	7.0	20	157
TBW (1998)	20.61	6.1	21	146

Frictional Fill: $\phi'_{peak} = 43^\circ$ and $\phi'_{cv} = 36^\circ$

Facing: Wraparound

The quantity includes:

- Length of geogrid turned up face
- 0.3m overlap at wraparound
- 1.5m return on highest geogrid

Table 1 Outcome design for a 10m high wall

In conclusion, it appears possible that if appropriate input parameters and values of these parameters are used in Limit State Approach designs, considerable technical improvements and cost benefits can be achieved for Geosynthetic Reinforced Soil Structures. These would allow reductions in the amount of geosynthetic reinforcements or a reduction in the quality of the reinforced fill. Both could significantly reduce costs and lead to greater use of this type of geotechnical structure in the future. However, care must be exercised when seeking such benefits by modifying existing design codes / methods within which there are factors or procedures aimed at constraining design outcomes. The combination of these with new input parameters and values may cause unexpected and perhaps unsafe outcome designs.

REFERENCES

- AASHTO 1994. *Standard Specifications for Highway Bridges*, Washington DC, 50th edition 1992 with 1993 and 1994 interims.
- AASHTO 1997. *Standard Specifications for Highway Bridges*, Division II, Section 7: Earth Retaining Systems.
- Al-Hassani, M.M. 1978. *Investigation of Stress-Strain Behaviour of Sand Tested in Plane Strain With and Without Sheet Inclusions*. Ph.D. Thesis, University of Strathclyde, Glasgow, UK.
- Al-Mudhaf, H.F. 1993. *The Behaviour of Geotextiles and Geogrids Following Environmental Conditioning in Kuwait*, Ph.D. thesis, University of Strathclyde, Glasgow, UK.
- Barden, L., Ismail, H. and Tong, P. 1969. *Plane Strain Deformation of Granular Material at Low and High Pressures*. Geotechnique 19, p: 441-452.
- Berg, R.R., Allen, T.M. and Bell, J.R. 1998. *Design Procedures for Reinforced Soil Walls - A historical Perspective/ 6th International Conference on Geosynthetics*, Atlanta, Georgia, USA, v.2, 25-29 Mar., pp.491-496.
- Bolton, M.D. 1986. *The Strength and Dilatancy of Sands*. Geotechnique 36, p: 65-78.
- Bonaparte, R., Holtz, R.D. and Giroud, J. P. 1985. *Soil Reinforcement Design Using Geotextiles and Geogrids*. Geotextile Testing and the Design Engineer, Symposium Sponsored by ASTM Committee D-35 on Geotextiles, Geomembranes and Related Products, Los Angeles, p: 69-116.
- Broms, B.B. 1971. *Lateral Earth Pressure Due to Compaction of Cohesionless Soils*. Proc. 6th International Conference of Soil Mechanics and Foundation Engineering, Budapest, p: 373-384.
- BS8006 1995. *Code of Practice for Strengthened/Reinforced Soils and Other Fill*, British Standard Institution.
- Collin, J.G., Chouery-Curtis, V.E. and BERG, R.R. 1992. *Field observation of reinforced soil structures under seismic loading*. Proc. Int. symposium on earth reinforcement practice, v.1: 223-228.
- Cornforth, D.H. 1964. *Some Experiments on the Influence of Strain Conditions on the Strength of Sand*. Geotechnique 14, p: 143-167.
- DIBt 1998. *Theory Used in the Deutches Institut für Bautechnik Design Method*, Netlon Ltd., Blackburn, UK.
- Dyer, M.R. 1985. *Observation of the stress distribution in crushed glass with applications of soil reinforcement*. D.Phil. Thesis, University of Oxford, UK.
- Esteves, S.A.C. 1996. *The Strength of Geosynthetics and Construction Effects*, M.Phil. Thesis, University of Strathclyde, Glasgow, UK.
- Eurocode 7: Geotechnical design. DD ENV 1997-1 1995. *General rules (together with UK application document)*, p. 5-7, 12-15, 72-86.
- Fukuda, N., Yamanouchi, T., Sakai, N. and Shinatani, H. 1994. *Applicability of Seismic Design Methods to Geogrid Reinforced Embankment*. Proc. 5th International Conference on Geotextiles, Geomembranes and Related Products, Singapore, pp. 533-536.
- Greenway, D., Bell, R. and Vandre, B. 1999. *Snailback Wall-First Fabric Wall Revisited at 25-Year Milestone*. Proc. *Geosynthetics*, Boston, USA, v.2, pp. 905-919.
- Ingold, T.S. 1979. *The Effects of Compaction on Retaining Walls*. Geotechnique 29, No.3, p: 265-283.
- Jewell, R.A. 1980. *Some Effects of Reinforcement on the Mechanical Behaviour of Soils*. Ph.D Thesis, Cambridge University, UK.
- Jewell, R.A. 1996. *Soil reinforcement with geotextiles*. Special publication 123, CIRIA, London, UK.
- Jewell, R.A., Paine, N. and Woods, R.I. 1984. *Design method for steep reinforced slopes and embankments*. Symposium of polymer grid reinforcement in civil engineering, London, UK.
- Jones, C.J.F.P. 1993. *Discussions: Slopes and Excavations*. Proc. of International Symposium on Earth Reinforcement Practice, Fukuoka/ Kyushu/ Japan/ 11-13 Nov.1992, Ochiai, Hayashi & Otani eds., Balkema, p: 972-975.

- Jones, C.J.F.P. 1996. *Earth Reinforcement and Soil Structures*, Thomas Telford Publishing, Thomas Telford Services Ltd., London, UK.
- Kabir, M.H. 1984. *In-isolation and in-soil behaviour of geotextiles*. Ph.D. Thesis, University of Strathclyde, Glasgow, UK.
- Kasahara, K., Kataoka, H., Yokota, Y., Arai, K., Machihara, H. and Nishikawa, S. 1992. *Development of New FTRP-Geogrid and its Application to Test Embankment*. Proc. International Symposium on Earth Reinforcement Practice, Japan, p: 357-362.
- Khan, A.J. 1999. *A reassessment of the design of geosynthetic reinforced soil structures*. Ph.D. Thesis, University of Strathclyde, Glasgow, UK.
- Kupec, J. 2000 *Combined Sustained and Short Term Load Testing of Geosynthetics*. M.Sc. Thesis, University of Strathclyde, Glasgow, UK.
- McGown, A., Andrawes, K.Z., Paul, J., Austin, R.A. 1993. *Limit State Design of Reinforced Soil Walls Slopes and Embankments*. Danish Geotechnical Society, ISLSD93, Vol.1/3, p: 275-284.
- McGown, A., Andrawes, K.Z., Pradhan, S. and Khan, A.J. 1998. *Limit State Design of Geosynthetic Reinforced Soil Structures*. Keynote Lecture, 6th International Conference on Geosynthetics, Atlanta, Georgia, USA, v.1, 25-29 Mar., pp.143-179.
- McGown, A., Andrews, K.Z. and Al-Hasani, M.M. 1978. *Effect of inclusion properties on the behaviour of sand*. Geotechnique, v. 28, no. 3: 327-346.
- McGown, A., Yeo, K.C. and Yogarajah, I. 1990. *Identification of a Dynamic Interlock Mechanism*. Proc. of International Reinforced Soil Conference, University of Strathclyde, Glasgow, UK, pp. 377-379.
- McGown, A., Yogarajah, I., Andrawes, K.Z. and Saad, M.A. 1994. *Strain Behaviour of Polymeric Geogrids Subjected to Sustained and repeated Loading in Air and in Soil*. Geosynthetics International, vo.1, no.3, p: 1-15.
- Milligan, G.W.E., Jewell, R.A., Houlsby, G.T. and Burd, H.J. 1990. *A new approach to the design of unpaved roads – part II*. Ground Engineering, Nov.: p. 37-42.
- NCMA 1997. *Segmental Retaining Wall Seismic Design Procedure*, Final draft, Netlon Ltd., Blackburn, UK.
- Pradhan, S. 1996. *The Design of Synthetic Polymeric Reinforced Soil Structures: A Limit State Design Approach*, Ph.D. Thesis, University of Strathclyde, Glasgow, UK.
- Yeo, K.C. 1985. *The behaviour of polymeric grids used for soil reinforcement*. Ph.D. Thesis, University of Strathclyde, Glasgow, UK.
- Yogarajah, I. 1993. *Effects of construction procedures on the behaviour of geogrid reinforced soil walls*. Ph.D. Thesis, University of Strathclyde, Glasgow, UK.