COMBINED SUSTAINED AND SHORT TERM LOAD TESTING OF GEOSYNTHETICS

by

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A thesis submitted in partial fulfilment of the degree of
Master of Science by Research

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July 2000

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THIS THESIS IS DEDICATED TO MY DAD

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DIE FOLGENDEN SEITEN SIND MEINEM VATER GEWIDMET

Acknowledgements

I wish to express my gratitude to Professor Alan McGown. He made it possible for me to study at the University of Strathclyde under his supervision and he has been very supportive during the final stage of the writing. It was his encouragement which kept me in the laboratory instead of climbing some Scottish mountains or paddling down rivers. His guidance led finally to the successful finish of this work.

I would also like to take the opportunity to express his appreciation for the support of Dr. Abdul Jabbar Khan. Our discussions enabled me to look behind the complicated schemes of my work. He also helped to develop major parts of this thesis.

My parents, especially my late father who passed away during the late stage of my work, gave me invaluable psychological and financial support and I would like to take this opportunity to express my deepest thanks to them. Further thanks are to be acknowledged to my brother Ludwig who not only gave me advice towards more efficient work with my computer, but helped me to organise all important affairs at home.

Furthermore I have to acknowledge that no test could be conducted without the excellent support of our technician team that consists of Gerry Carr, Bill Gemmill, Alex Brown and Jim Milne.

Additional thanks go to all my friends in Glasgow and all over the world, especially to Sabine who became a very good friend to me. Thanks to Sylke, Guillaume, Céline, Peter, Sebastian, Elke, Kai, Piia, Boksun and Sacha for their friendship and all the activities which made my life in Glasgow far more enjoyable. In this connection, my heartfelt appreciation to Yaso.

Abstract

Polymer based geosynthetics currently used in reinforced soil structures exhibit a salient feature, that is they show a complex time and temperature dependent elasto-visco-plastic strain response under load. To date, most of the designs are based on the assumption that the loads applied to the structure are of a sustained nature. Field conditions, however, often consist of a sustained load (due to self-weight) and transient loads (due to traffic) with the possibility of seismic loads (earthquakes), explosions and other transient or dynamic loadings occurring. Therefore, a sustained load does not fully represent the field situation.

This thesis investigates the behaviour of geosynthetic reinforcements under a combination of a sustained load and short-term loading conditions. To simulate the behaviour of reinforcements under such loading conditions, a test apparatus was developed. Test results were analysed using the Isochronous Strain Energy approach and the Recoverable and the Locked-in ISE components for a geosynthetic were identified. Following this the significance of the use of the ISE approach to the design of geosynthetic reinforced soil structures subject to combined loading is assessed.

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Glossary of Terms

γ	Calculated Factor of Safety
μ	Coefficient of Friction between Soil and the Reinforcements
σ'_{vi}	Total Vertical Stress in the i^{th} Layer of Reinforcement
γ_1	Density of the Reinforced Fill
β_f	Slope Angle of the Face of the Structure
γ_m	Partial Factor
σ_v	Vertical Stress on Reinforcing Element
$[\phi_\mu]$	True Angle of Friction
$[\phi'_0]$	Angle of Friction at Rest
$[\phi'_{cv}]$	Constant Volume Angle of Friction
$[\phi'_m]$	Mobilised Angle of Friction
$[\phi'_{mEDL}]$	Mobilised Angle of Friction at the End of Design Life
$[\phi'_{mRL}]$	Mobilised Angle of Friction in the Soil, Less than the Peak Angle of Friction $[\phi'_p]$
$[\phi'_p]$	Peak Angle of Friction
$[\sigma]$	Average Normal Stress
$[\tau]$	Shear Stress
$[\varepsilon_D]$	Design Strain
$[\varepsilon_{des,eq}]$	Factored Limiting Strain
$[\varepsilon_I]$	Instability Strain Limit
$[\varepsilon_L]$	Limiting Strain
$[\Delta L]_t$	Additional 'Locked-in' ISE
$[\varepsilon_P]$	Performance Limit Strain
$[\varepsilon_{pc}]$	Actual Post-Construction Strain
$[\Delta P_s]$	Additional Short Term Load
$[\varepsilon_R]$	Rupture Strain
$[\Delta R]_t$	Additional Recoverable ISE
$[\varepsilon_t]$	Lateral Tensile Strain
$[A]_t$	Absorbed ISE
$[A]_{t0}$	Absorbed ISE at Time $[t_0]$

$[A]_{t1}$	Absorbed ISE at Time $[t_1]$
$[C]_t$	ISE Capacity at a Limiting Strain
$[DC]_t$	Design ISE Capacity
[F]	Shear Force
[F]	Shearing Resistance in the Soil
$[F_{perm}]$	Permissible Working Load of the Reinforcing Element
[i]	Intercept
$[L]_t$	'Locked-in' ISE at any Time [t]
$[L]_{t1}$	'Locked-in' ISE at the End of Stage1
$[L]_{t2}$	'Locked-in' ISE at the End of Stage2
$[L]_{t3}$	'Locked-in' ISE in Stage3
$[P_a]$	Earth Pressure on the Wall
$[P_{Ref}]$	Reference Strength
$[P_s]$	Disturbing Shear Force
$[P_s]$	Sustained Load
$[P_t]$	Additional Transient Loading
[Q]	Normal Force
$[q_r]$	Imposed Bearing Pressure
$[R]_t$	Recoverable ISE at Time [t]
$[R]_{t0}$	Recoverable ISE at Time $[t_0]$
$[R]_{t1}$	Recoverable ISE at the End of Stage1
$[S]_t$	Slope Value
[T]	Temperature
[t]	Time
$[t_0]$	Equivalent Instantaneous Loading Time
$[T_{1pcs}]$	Force Required in Order to Mobilise 1% Post-Construction Strain
$[T_{al}]$	Long-term Strength of the Reinforcing Element
$[t_{CP}]$	Construction Period
$[T_d]$	Design Strength of the Reinforcing Elements for Single-Stage Actions
$[t_{dl}]$	Design Lifetime
$[t_{EDL}]$	Time at the End of Design Life
$[T_g]$	Glass Transition Temperature

$[T_g]$	Mobilised Force in the Reinforcement
$[t_{RL}]$	Reinforcement Loading Period
$[t_{SOG}]$	Time of Switch-on of Gravity
A_1	Reduction Factor for Long - term Strength
A_2	Reduction Factor for Damage from Placing and Compaction
BS	British Standard
c'	Effective Cohesion of Soil
c'_p	Effective Cohesion of Soil
c'_r	Adhesion between the Fill and Reinforcement
CAD	Computer Aided Design
CRS	Constant Rate of Strain
CSL	Critical State Line
c_u	Undrained Cohesion
DIBt	Deutsches Institut für Bautechnik
DIN	Deutsche Industrie Norm
D_m	Depth of Embedment below Ground Level
EDL	End of Design Life
e_L	Void Ratio of Loose Soil
EOC	End of Construction
f_b	Factor of Safety for Bearing Capacity
F_B	Short Term CRS Rupture Strength of the Reinforcing Element per metre Width
f_f	Partial Factor for Surcharge Load
FHWA	American Federal Highway Administration
f_k	Characteristic Long Term Strength (at 10% Limiting Strain) of the Reinforcement
f_m	Partial Factor
f_{m1}	Partial Factor to Allow for Material Manufacturing Variations and Confidence in the Extrapolated Strength
f_{m11}	Partial Factor Related to the Consistency of the Manufacturer
f_{m111}	Partial Factor Related to whether or not a Standard for Specification, Manufacture and Control Testing of the Reinforcement Exist

f_{m112}	Partial Factor for whether or not Standards for the Dimensions and Tolerances Exist
f_{m12}	Partial Factor Related to the Extrapolation of Test Data Dealing with Base Strength
f_{m121}	Partial Factor for the Assessment of Available Data
f_{m122}	Partial Factor for Extrapolation of the Statistical Envelope over the Expected Service Life of the Reinforcement
f_{m2}	Partial Factor Concerned with the Effects of Construction and Environmental Effects
f_{m21}	Partial Factor for the Effects of Construction Activities
f_{m21}	Partial Factor which Deals with the installation Damage of the Reinforcements
f_{m211}	Partial Factor Related to Short Term Effects of Damage Prior to and During Installation
f_{m212}	Partial Factor for the Long Term Effects of the Short Term Damage
f_{m22}	Partial Factor for the Effects of Environmental Degradation
f_{m22}	Partial Factor which Deals with the Environmental Effects of the Reinforcements
f_{m3}	Overall Factor of Safety
f_{ms}	Partial Factor for c'_r
f_{ms}	Partial Factor for Material
f_n	Partial Factor for Economic Ramifications of Failure
FOS	Factor of Safety
f_p	Partial Factor against Pullout
f_s	Factor of Safety against Base Sliding
GCL	Geosynthetic Clay Liner
GRSSs	Geosynthetic Reinforced Soil Structures
H	Height of the Structure above Ground Level
HDPE	High Density Polyethylene
h_i	Depth of Reinforcement of i^{th} Layer
h_s	Height of Sloped Retained Fill
h_{wt1}	Depth Within the Structure
h_{wt2}	Depth Outside the Structure
ISE	Isochronous Strain Energy

K_0	Coefficient of Earth Pressure at Rest
K_a	Coefficient of Active Pressure
L	Effective Base Width for Sliding
L_{ei}	Anchorage Length beyond Failure Plane
LTADS	Long-Term Allowable Design Strength
LVDT	Linear Variable Displacement Transducers
MDPE	Medium Density Polyethylene
M_w	Moment Magnitude
OBF	Out of Balance Forces
$P_{des,eq}$	Modified Design Strength
PE	Polyethylene
PP	Polypropylene
PVC	Polyvinyl chloride
PVR	Product Value Rating
q_{ult}	Ultimate Bearing Capacity of the Foundation
R	Resistance
RF	Reduction Factor
RF_{CR}	Reduction Factor for Long Term Rupture Strength
RF_D	Reduction Factor to Prevent Rupture of the Reinforcement Due to Chemical and Biological Degradation
RF_{ID}	Reduction Factor for Damage from Placing and Compaction
R_h	Horizontal Factored Disturbing Force
R_v	Vertical Factored Resultant Force
SLS	Serviceability Limit States
S_v	Vertical Spacing of Reinforcing Element
T	Total Tensile Force to be Resisted By Reinforcing Elements
T_b	Unfactored Reinforcement Base Strength
TBW	TENSAR® Tie-back Wedge
T_c	Extrapolated Creep Strength (at 10% Limiting Strain) of the Reinforcement at Specified Design Life Time and Operational Temperature
T_i	Maximum Tensile Force in the i^{th} Layer
T_{max}	Maximum Disturbing Force
TPVR	Total Product Value Rating

T_{ult}	Short Term CRS Rupture Strength of the Reinforcing Element per metre Width
ULS	Ultimate Limit State
w_s	Surcharge Load

Reference Strength	Ex-works strength of geosynthetics
Partial Factors	Factors those take account of any changes of geosynthetics due to construction damage, environmental degradation and/or any uncertainties involved in the manufacturing and extrapolation of test data
Design Strength	Reference Strength divided by the Partial Factors

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Current soil retaining methods may be broadly divided into two categories; namely *Externally* and *Internally* stabilised systems. An externally stabilised system uses a structural wall, against which stabilising forces are mobilised. An internally stabilised system involves reinforcements installed within and extending beyond the potential failure plane. Within this latter system, out-of-balance forces carried by the tensile capacities of closely spaced reinforcements remove the need for a structural facing, however, a non-structural facing is required to prevent degradation of the lateral soil surface. Thus the role of this facing is to prevent local loss of soil and deterioration of the reinforcements.

Conventional retaining walls may be regarded as externally stabilised systems. Internally stabilised systems are typified by reinforced soils with predominantly horizontally layered reinforcing elements, such as metallic strips or polymeric straps, sheets or grids. A key aspect of internally stabilised systems is their incremental form of construction. In effect, the soil mass is partitioned into horizontal sections so that each section receives support from local reinforcing elements.

Since its inception, the application of reinforced soil systems has become rapidly and increasingly acceptable. The basic attributes of soil reinforcement which have led to this are reductions in costs and ease of construction. The cost benefits obtained from the reinforced soil structures may range from 20-80% for different heights of soil retaining applications, compared to conventional solutions, Jones (1996).

1.2 Development of Geosynthetic Reinforced Soil Structures

The concept of earth reinforcement was proposed by Casagrande who idealised the method as weak soil reinforced by high-strength membranes laid horizontally in layers, Westergaard (1938). The modern form of earth reinforcement was introduced by Vidal (1969). Vidal's concept was for a composite material formed from flat (metal) reinforcing strips laid horizontally in a frictional soil, the interaction between the soil and the reinforcing members being solely generated by soil-reinforcement friction. The introduction of Vidal-type structures led to the development of a better understanding of the fundamental concepts involved and later the introduction of different forms of reinforcement, such as polymeric strips, sheets and grids.

Soil reinforcement materials development is related to the development of reinforced soil structures. Historically reinforced soil structures were formed using organic materials as reinforcements, such as timber, straw or reeds, but as early as the 19th century, Pasley (1822) recognised the potential of more advanced forms of reinforcement. His use of canvas as a reinforcing membrane is arguably the first application of geotextile reinforcement, Jones et al (1996). Canvas could only have been expected to have a limited life before deterioration, thus Pasley's structures were not expected to be very durable in the long term.

The use of non-metallic materials for permanent reinforcement could not be seriously contemplated until the development of synthetic polymer based materials. Synthetic fabrics were known prior to 1940 but it was not until late 1960s and early 1970s that the advances in geosynthetics manufacturing meant that their longevity could be assured, Jones et al (1996). A salient feature of these polymer-based geosynthetics is that they exhibit a rather complex time and temperature dependent elasto-visco-plastic strain response to loading. For this reason, the design of Geosynthetic Reinforced

Soil Structures [GRSSs] requires careful attention with respect to controlling within tolerable limits the deformations of the structures over time.

1.3 Design Approaches and Computer Programs

With the development of polymeric geosynthetics GRSSs were progressively introduced into modern civil engineering practice during the 1970's. The earliest applications were in road pavements and embankment slopes, although there were some notable steep slopes and walls constructed, McGown and Ozelton (1973), Holtz and Massarsch (1976) and Jarrett et al (1977). In most cases, a Limit Equilibrium Approach was used to design these structures. In these designs, the soils were usually represented by their peak strengths and the geosynthetics by either their short term or long-term rupture strengths.

Little or no account was taken of the deformation characteristics of the soil compared to the geosynthetics or their interaction behaviour. Large global *Factors of Safety* were applied which were intended to ensure *Explicitly* that collapse did not occur and to ensure *Implicitly* that deformations under working conditions were not excessive. As implemented, many of these design methods were semi-empirical and they proved to be generally acceptable and economic compared to other technical solutions available at that time.

Coincidentally as geosynthetics were being introduced, a fundamental change in the approach to the design of civil engineering structures was underway. This involved the introduction of deformations and strains as design criteria to be assessed and controlled in an explicit manner. These were incorporated into what is now known as the *Limit State Approach*, in which both collapse conditions, *Ultimate Limit States* [ULS], and operational conditions, *Serviceability Limit States* [SLS], are analysed. Another feature of this approach is the introduction of risk factors, *Partial Factors*, to replace the use of global Factors of Safety.

The Limit State Approach has been adopted in structural engineering since 1980's and recently it has been gaining acceptance in geotechnical engineering. Some design codes/methods take account of Limit State principles (e.g. BS 8006, 1995 and TBW, 1998), however in most cases, these include factors, mis-termed Partial Factors, that "*Manipulate*" outcome designs to ensure that they are close to outcome designs based on pre-existing semi-empirical Limit Equilibrium Approaches. These methods have been termed *Hybrid Approaches* to the design of GRSSs, Pradhan (1996).

With respect to Limit State designs, Pradhan (1996) and McGown et al (1998) suggested valid mechanisms appropriate to ULS and SLS analyses. To date, their suggested approach may be considered to represent the only *"True"* Limit State Approach available for the design of GRSSs, being based on internal strain compatibility and force equilibrium between the soils and geosynthetic reinforcements. Their approach to the design of GRSSs involves rigorous analyses and iterations to be performed.

Almost all of the above methods of analyses are too complex to perform manually. Therefore, Computer Aided Design [CAD] programs are required. Amongst those available are; Winwall, Winslope and Tensoil programs. The Winwall and Winslope programs have been developed by Netlon (1998) and are Hybrid designs methods based on both Limit Equilibrium Approach and Limit State principles. The Tensoil program, developed by Pradhan (1996), is based on the "True" Limit State Approach.

1.4 IDENTIFICATION OF THE PROBLEMS

Despite much research and considerable evidence from case histories, outcomes of most design methods/codes have remained essentially the same or have become more conservative over the last 25 years, Berg et al (1998) and Greenway et al (1999). Part of this is due to the *"Manipulations"* built into the design approaches themselves, but some of this may be attributed to the choice of input values for the material properties used in the analyses. It has been suggested by McGown et al (1998) that the "True" Limit

State Approach is the most applicable method for the design of GRSSs and to date contains no "*Manipulations*". However, the appropriate input parameters for this approach and their values still remain to be optimised.

Important input parameters for soils are the mobilized angle of friction and its relationship to the tensile strain in the soil. For geosynthetics, important input parameters are their Reference Strengths, Partial Factors and Design Strengths. Reference Strengths are the strengths of *Ex-works* materials. Partial Factors are the factors that allow for any change of geosynthetics due to construction damage, environmental degradation and uncertainties in production. Design Strengths are the strengths obtained by dividing the Reference Strengths by the Partial Factors.

1.4.1 ACTIONS

An important aspect related to the design of GRSSs is the need to take account of different types of Actions. GRSSs can be subjected to *Single-Stage Actions*, e.g. self-weight, however, in most operational conditions, Actions imposed on GRSSs are likely to be a combination of long-term sustained (self-weight) and transient loads or deformations (from traffic or earthquakes etc.). These may be termed *Multi-Stage Actions*.

Although the current design codes/methods, e.g. BS8006 (1995), AASHTO (1997) and DIBt (1998), recognise the Multi-Stage Actions, the input parameters for soils and geosynthetics are determined from Single-Stage Loading tests only. These testings do not simulate the actual Actions, therefore, the data obtained from these tests may not be appropriate for use in the design of GRSSs.

1.4.2 SOIL PROPERTIES FOR SINGLE-STAGE ACTIONS

For granular soils, the important input parameter for "True" Limit State Approach is the relationship between the *Mobilised Angle of Friction* [ϕ'_m] and the *Lateral Tensile Strain* [ε_t]. In fact, the large strain *Constant Volume Angle of Friction* [ϕ'_{cv}] of granular soils should be used for ULS analyses where the rupture strain and limiting strain of the GRRS exceed 10%, Bolton (1996), Jewell (1996), Pradhan (1996) and McGown et al (1998). For SLS analyses relationships between Mobilised Angle of Friction [ϕ'_m] and Lateral Tensile Strain [ε_t] under plane strain condition should be used with soil strengths in the range of $\phi'_{at\text{-}rest}$ to ϕ'_{peak}. Although some design methods suggest *Peak Angle of Friction* [ϕ'_p] for ULS analyses, the design values chosen for this parameter by practitioners are often very low, sometimes lower than the actual ϕ'_{cv} of the soil. To date, relationships between Mobilised Angle of Friction [ϕ'_m] and Lateral Tensile Strain [ε_t] under plane strain condition have not been widely investigated for the reinforced fills recommended by various codes for use in GRSSs. However, Khan (1999) has investigated the properties of compacted granular fills in this manner.

1.4.3 Soil Properties for Multi-Stage Actions

It should be noted that the properties of granular fills do not vary with the types of loading, Shimming and Saxe (1964). Therefore, the same input parameters for granular fills are applicable to both Single-Stage Actions and Multi-Stage Actions. Nevertheless, it should be appreciated that other types of fills often will exhibit different properties under Single-Stage Actions and Multi-Stage Actions.

1.4.4 Geosynthetic Properties for Single-Stage Actions

For Single-Stage Actions, the *Reference Strength* of a geosynthetic is often defined in various codes/methods, but in different ways. Some codes/methods define it as a factored short-term strength, some as long-term creep rupture strength and others as long-term creep strength for a particular limiting strain. It should be noted that the properties of a geosynthetic are likely to be strain level and time dependent. Therefore, the Reference Strength is not an unique value and should be determined for a particular limiting strain and time.

Most commonly the *Partial Factors* are determined by comparing the strength of a geosynthetic *before* and *after* construction damage and/or environmental degradation. Different design codes/methods and researchers suggest different material test methods for determining these Partial Factors, e.g. on the basis of the strength at rupture or a pre-defined limiting strain obtained from Constant Rate of Strain [CRS] or long-term sustained load [creep] tests. Because of the elasto-visco-plastic nature of the polymeric geosynthetics, the values of Partial Factors obtained from different testing methodologies are, therefore, likely to be different from each other even for a single type of geosynthetic, Esteves (1996).

Although many different materials testing techniques have evolved and have been employed for determining Partial Factors for geosynthetics, to date, it has not been possible to compare in a general manner or correlate data obtained from these different testing techniques using a single approach. Further, in most of the design codes and methods, Partial Factors obtained from short term CRS tests are applied to the ex-works long-term strengths to obtain the long-term design strengths for geosynthetics. The inherent assumption is that the effects of construction damage and environmental degradation on the properties of geosynthetics are the same for both the short-term and the long-term. It will be shown later that this may not be true.

Furthermore, Partial Factors are likely to be strain level and time dependent. Therefore, it may not be appropriate to specify a single value for a Partial Factor either for ULS or SLS analyses.

Due to differences in the definitions of Reference Strengths and the means of determining Partial Factors, it is likely that Design Strengths will be different even for a particular geosynthetic. Further, strains corresponding to the Design Strengths will be different from one method to the other. For use in the "True" Limit State Approach, it is important to know the exact values of strains corresponding to Design Strengths for ULS and SLS analyses.

1.4.5 Geosynthetic Properties for Multi-Stage Actions

Traffic loads are transient in nature. In most of the design codes, traffic loads are specified or are calculated as the wheel load divided by the contact area, for example BS 5400 (1978), and AASHTO (1997). No consideration is given for the transient nature of this Action. The external and internal stability analyses are then carried out for GRSSs for the critical loading combination as outlined in BS8006 (1995) or AASHTO (1997). For internal stability analyses, Reference Strengths are considered as the creep rupture strength or factored short-term strength in these codes.

In contrast, it has been shown by Müller-Rochholz (1994) and Khan (1999) that the strain response of a geosynthetic due to cyclic loading can be obtained by applying a sustained load equal to half or less of the amplitude of cyclic load. This indicates that the present approaches are conservative.

For designs against short term transient loadings such as earthquake forces, it has been suggested by Fukuda et al (1994), AASHTO (1994) and Jones (1996) that the Reference Strength of a geosynthetic for Single-Stage Actions should be increased by 1.5 times. Some recent design codes/methods suggest that a CRS strength of the geosynthetics should be used as the Reference Strength for design against earthquake loading, for example AASHTO (1997), NCMA (1997) and DIBt (1998). The basis for the consideration of the use of these strengths is, however, not justified.

It is essential that when designing a GRSS subject to sustained loads against earthquake forces the time of occurrence of an earthquake is predicted in order to determine, whether or not a geosynthetic is capable of withstanding any additional short-term loads. Current codes/methods suggest a single value of Design Strength for a geosynthetic for the design of a GRSS against earthquake loading, i.e. no matter when a GRSS is hit by an earthquake, a geosynthetic is attributed with a constant strength which is used to determine if a particular amount of additional short term transient load can be supported.

In addition, no design codes/methods have to date, dealt with the determination of Partial Factors for geosynthetics under Multi-Stage Actions. Partial Factors are usually obtained from short term Single-Stage CRS testing. This will be shown later to be inappropriate. Therefore appropriate Design Strengths for Multi-Stage Actions need be identified.

1.5 Objectives of the Present Research

On the basis of the problems identified in the previous section, the objectives of the present research are:

- To characterize Actions, according to their effects on GRSSs, using the classifications and combinations identified in Eurocode 1 (1996) and 7 (1995).
- To develop protocols for Multi-Stage Load testing in a similar manner to the protocols in BS 6906 Part 1 (1987) and Part 5 (1991).
- To develop a means of correlating test data from different Single-Stage Loading tests.
- To present a means of analysing test data using the Isochronous Strain Energy [ISE] Approach.
- To analyse the load-strain-time-temperature behaviour of geosynthetics under Multi-Stage Loadings, e.g. sustained loading plus short-term loadings, using the ISE approach and identify an appropriate approach for the selection of Design Strength to resist these loadings.

1.6 Layout of the Thesis

Chapter Two presents a brief historical background of GRSSs and describes various reinforcing applications. Two different reinforcing systems, (steel and polymeric), are identified and compared. The various types of polymeric reinforcements are then briefly described.

In Chapter Three, a general review of governing concepts of GRSSs are presented, including the internal force equilibrium and the internal strain compatibility. Salient features of design codes/methods based on Limit Equilibrium Approach and the Hybrid Approach are described briefly and valid mechanisms for the "True" Limit State Approach are presented in this Chapter. Also, problems associated with input parameters related to soils and geosynthetics for Single-Stage Actions and Multi-Stage Actions, are considered.

In Chapter Four, a new classification of Actions is suggested and discussed in comparison to the Eurocode classification. Implications of the review are summarized at the end of this chapter and this lead towards the development of new Combined Sustained-Short Term Loading test protocols.

The presentation and employment of the Isochronous Strain Energy [ISE] Approach for a range of geosynthetics is given in Chapter Five. This shows that the ISE Approach, developed by Khan (1999), can be used to compare and correlate data obtained from different test methodologies and that the geosynthetics can be characterised in terms of their ISE Capacity and 'Immediately Recoverable' and 'Locked-in' ISE components.

No standardised testing protocols for Multi-Stage Actions have been evolved to date. This study recognises the need for such testing hence in Chapter Six, such protocols for Multi-Stage Load testing are presented.

Chapter Seven presents results obtained from Combined Sustained-Short Term Load testing, using the Multi-Stage Load testing protocol described in Chapter Six. These results are then analysed on the basis of the ISE Approach. This leads to the development of design approaches for combined loading, viz. the Equivalent Sustained Action Approach and the Modified Material Properties Approach.

Chapter Eight contains discussion and conclusions drawn from the present study. The recommendations for future work are presented thereafter.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

Henry Vidal, a French architect and engineer investigated in the middle of the last century, the frictional effects of reinforcing elements in soil with the aim of improving the mechanical properties of the soil in the direction in which the soil is subjected to tensile strain. As a result of these studies, he launched a new civil engineering material known as Reinforced Earth or Terre Armée. In fact, the idea of earth reinforcement is not new. There are many historical instances of construction techniques which use reinforcements to improve weak soils.

One of the most impressive reinforced earth structures is the large pyramid of Ziguratt near Baghdad, which was built from clay bricks reinforced by separate layers of reed in the 14^{th} century B.C. Some parts of this structure are still visible today.

Based on the basic idea of soil reinforcement a number of applications were developed and different synthetic materials evolved in the late 1960s. These materials are now termed Geosynthetics. A description of the basic mechanism, soil reinforcement, geosynthetic materials and applications are presented below.

2.2 BASIC MECHANISM OF BEHAVIOUR

Reinforced Earth is a composite construction material in which the strength of engineering fill is enhanced by the addition of tensile elements. This technique provides a versatile and economical construction method for a variety of engineering applications, including retaining walls, bridge abutments, embankments, dams, foundations and roads.

The basic behaviour mechanism of reinforced soil involves the generation of frictional forces between the soil and the reinforcement. These forces are manifested in the soil in a form analogous to an increased confining pressure which enhances the strength of the composite. Additionally, the reinforcement has the ability to unify a mass of soil that would otherwise shear along a failure surface.

2.3 Types of Reinforcements

The type of materials used today as reinforcing elements include steel, fibreglass and geosynthetics in the form of sheets, strips or grids. The choice of material and then the form in which it is used generally dictates the load transfer mechanism from the soil to the reinforcement. In the case of strip or sheet reinforcement, the load transfer mechanism at the soil-reinforcement interface is principally frictional, while for grid reinforcement the principal mode of stress transfer is bearing stress developed at the junctions or protrusions.

When stressed, reinforcements have to be capable of sustaining the load without rupture or unacceptable strains during their lifetime. The wide variety of reinforcements and the variation in their behaviour has led to the classification of reinforcements into various categories.

2.3.1 Inextensible Reinforcement

According to BS 8006 (1995) reinforcing elements, subject to design loads of a sustained nature, may be classified as *Inextensible* when the total axial tensile strains are less than 1%, viz. steel, fibre glass.

Embedding inextensible reinforcements in soil, in the direction of the principal tensile strain results in a net increase in load carrying capacity of the soil or a reduction of boundary movements when compared to the soil alone. This continues to be effective until rupture of the reinforcements occurs, after which the composite reverts back to the properties of the soil alone.

Inextensible metallic reinforcement, were the first form of reinforcing elements, Vidal (1969). These reinforcements have two main inadequacies: they are susceptible to corrosion, King (1978) and hinder the development of active soil pressures due to their inextensibility, McGown et al (1988) and Yogarajah (1993). To overcome the corrosion aspect, certain building codes recommended a sacrificial thickness.

The lack of predictability of the extent of corrosion and the need to allow adequate soil movement for active pressure development, has led to these materials being in part replaced with materials made from synthetic polymers.

2.3.2 Extensible Reinforcements

According to BS 8006 (1995) reinforcing elements, subject to design loads of a sustained nature, may be classified as *Extensible* when the total axial tensile strains exceeds 1%, viz. synthetic polymeric reinforcements, natural fibres.

McGown et al (1978) classified extensible reinforcement into the *Relatively Inextensible* and *Relatively Extensible* reinforcement, Fig. 2.1.

Relatively inextensible reinforcements that have rupture strains that are less than the maximum tensile strain in the soil without reinforcement under the same operational conditions.

Relatively extensible reinforcements have rupture strains that are larger than the maximum tensile strain in the soil without reinforcement under the same operational conditions.

Generally, extensible reinforcements allow larger strains to occur without reinforcement rupturing. In these circumstances, the benefit of the mobilised tensile strength of the reinforcement is available even after the peak strength of soil is reached, Fig. 2.2.

The variety and availability of extensible materials have greatly increased in recent years. Most are manufactured from polymeric synthetic materials. These reinforcements have their own disadvantages such as being susceptible to long term creep and stress relaxation. Additionally, they exhibit degradation due to ultraviolet light when exposed to the sunrays together with thermal instability and decay caused by biological agencies.

This thesis covers extensible synthetic polymeric (geosynthetic) reinforcements only. The different materials in this categories are presented below.

2.4 GEOSYNTHETICS

Geosynthetics comprise a large family of materials and as stated previously, they are usually made from synthetic polymers. They include geotextiles, geogrids, geonets, geomats, geocomposites, geomembranes and pipes, Fig. 2.3. They are used with soil, rock or other geotechnical engineering related materials to enhance the performance or to reduce the cost of man-made products, structures or systems.

Geotextiles include textiles in the traditional sense, produced by standard weaving machinery or by being melted or needled together. A major point is that they may be porous to water flow across their plane and also to some extent along their plane.

Geotextiles are often classified as water permeable and impermeable. Further classification separates them into woven or non-woven products. There is another major group of materials which are used, namely geotextile related materials. These include grids, nets, meshes and composites. They are not made by traditional methods but by means of new innovative processes developed over the last thirty years or so. Further there are impermeable flat sheets of polymer known as membranes and structured sheets and pipes which may be either permeable or impermeable.

2.4.1 HISTORY

The Egyptians used natural fibres to strengthen clay bricks and walls, some 4,000 years ago. Much later, Ancient Romans laid woven mats over marshy grounds and covered them with stone in a manner similar to present day techniques. Moreover, in the Middle and Far East it has been a common practice for many millennia to reinforce large earth structures with bundles or woven mats of reeds, rushes or bamboo. Bamboo and other natural fibres fascines are still frequently used in embankment construction in South East Asia and Africa. Natural fibres, like jute, have been and are still in use as silt fences and agricultural drains in many countries, Kabir (1984). In addition, it is reported that jute fabric was used in military supply routes in India and Burma during the WW II, Lawson (1982).

Natural fabrics were used to reinforce roads by South Carolina Highway Department in 1936. They used cotton fabric on a prepared base and then applied hot asphalt mix. Results showed a reduction in cracking, ravelling and localized road failures. Although good results were obtained, cotton fabric never became properly established as a geotextile because the cotton fabric lacked the long-term durability, Koerner et al (1980).

It is believed that the first applications of polymer-based geotextiles were woven industrial fabrics used beneath concrete block revetments in water front structures buil7 in Florida in 1958, however, Nicolon Ltd (USA) have claimed that they used the same material in 1957. Moreover, Dutch engineers used geotextiles in 1956 in an attempt to find innovative construction solutions for use on their massive Delta Works Scheme which was commenced at that time, Koerner et al (1980).

The first report of the adoption of synthetic materials, in the form of a woven geotextile was by Agerschou (1961), who used a geotextile as a filter in a coastal protection works.

Terzaghi and Lacroix (1964) are known to have used a continuous polyvinyl chloride [PVC] plastic sheet as a water tight stretchable membrane to control cracking in the construction of the Mission Dam in British Columbia, Canada.

Barrett (1966) reported that wovens were first used in connection with erosion control applications and were intended to be an alternative for granular soil filters. He described how effective these woven geotextiles were in a variety of coastal structures, Kabir (1984).

From 1968 onwards the American Federal Highway Administration [FHWA] monitored many pavements overlay repair schemes where textiles were installed with the intention of controlling reflective cracking in the asphalt surfacing. As recently as 1982 the FHWA stated that the variable performance observed, suggested that it would not be cost effective and that they were not encouraged. Only with the development of geogrids and new installation methods does the use of pavement reinforcement look justifiable, Ingold and Miller (1982).

Nets and Meshes started to become established around 1968 when the Japanese used polyethylene nets exported from Britain by Netlon Ltd. as a means of alleviating the damage caused to embankments by seismic activity and heavy rainfall. Initially these were mainly used to strengthen the face of embankments. At the same time they were used to increase stability and to reduce the risk of base failures when embankments were constructed over weak soil.

The development work on the first non-woven textiles was undertaken during the mid to late 1960`s, by Rhone-Poulenc in France (Bidim, thick needle-punched polyester fabric) and ICI in Britain (Terram, thin heat-bonded non-woven fabric).

In about 1971, other geotextiles or related products first appeared, namely fin drain, woven geotextile base reinforcements and reinforced soil wall reinforcements which opened up new application areas.

McGown and Ozelton (1973) identified the three basic functions of *Separation, Filtration* and *Reinforcement* for textiles used in Civil Engineering. Leflaive and Puig (1974) also recognized these three functions and added Drainage as a fourth function.

In the mid 1970`s, Giroud coined the phrase *Geotextiles* to describe the textiles used in Civil Engineering, which was later extended to *Geotextiles* and *Related Materials* and then to *Geosynthetics*.

The classification of geosynthetics is shown in Fig. 2.3.

Boyes (1981) reviewed the progressive use of Geosynthetic products and reported a number of new applications, including erosion control and scour control on river flows with water-impermeable liners. Today three more functions are recognized, namely *Screening, Sealing* and *Wrapping*.

Murray and McGown (1982) developed and reported upon a number of specifications and testing methods for quality control of geosynthetics. Various research institutions have run research programmes from the 1970`s to the present to develop testing techniques for geosynthetics. The objectives of these programmes not only included the development of rapid easy to perform, so called *Index Tests*, they also included tests which could be used to obtain data for use in analytical design techniques, so called *Performance Tests*.

Giroud (1984) reviewed the potential of using Geosynthetics in geotechnical engineering and concluded that the greatest research needs are in the areas of durability, soil interaction and field performance.

Despite the development of different types of design approaches, codes and methods, Berg et al (1998) and Greenway et al (1999) have shown that the outcomes of the design methods have become more conservative or remained the same over the last 20 years or so, Fig. 2.4. The figure shows the Resistance, R, as the Long-Term Allowable Design Strength [LTADS] in terms of percentage of ultimate wide width strength and Load, L, as the total load on the wall, without surcharges. The lower the value of R the more conservative the procedure. Also, with other factors remaining the same, the higher the assumed load the more conservative the method. Therefore, the lower the R/L ratio, the more conservative the design method.

Khan (1999) showed that the increasing conservatism was due to "*Manipulation*" of the present design methods.

2.4.2 POLYMERIC MATERIALS USED IN GEOSYNTHETICS

Although some materials such as jute are still in use, most geosynthetic are made from synthetic polymers. The basic polymers, viz. Polyethylene, Polypropylene, Polyamide and Polyester, used in geotechnical engineering are briefly described below. Further details are given in Chapter 4. However, they almost always are mixed with a variety of additives.

2.4.2.1 POLYETHYLENE

Polyethylene [PE] is a member of the polyolefin family. The structure is a chain of carbon atoms with hydrogen atoms attached. It exists in highly crystalline regular-arrayed linear chains sometimes with various degrees of branching leading to less-crystalline, more random, amorphous structures. The regularly arranged crystalline form is slightly more compact and dense, and it is called high density polyethylene [HDPE]. The medium density form, MDPE, is less sensitive to stress cracking and stress ageing than the HDPE counterpart. HDPE is widely used in a variety of geotechnical applications.

2.4.2.2 POLYPROPYLENE

Polypropylene [PP] is a sub-family of the polyolefins. Polypropylenes are similar in many respects to the polyethylenes. However, the molecular structure of polypropylene is arranged to encourage crystal formation which leads to a better balance of chemical resistance and heat resistance. These are influenced, as in other thermoplastics, by molecular weight and molecular weight distribution. Tensile strength and impact resistance increase with molecular weight. However, the long-term creep resistance is not outstanding, but its fatigue endurance limit is excellent.

2.4.2.3 POLYAMIDE

There are several different types of nylons, e.g. Nylon 6, Nylon 66, Nylon 11, but as a family their characteristics of strength, stiffness and toughness have earned them a good reputation for use as engineering plastics. The two most important types of Polyamides used for geosynthetics are Polyamide 6 and Polyamide 66. Polyamide 6 is an aliphatic polyamide obtained by the polymerisation of the petroleum derivate caprolactam while Polyamide 66, also an aliphatic polyamide, is produced from the polymerisation of a salt of adipic acid and hexamethylene diamine which are both petroleum derivatives. At low pH levels, however, the material is susceptible to hydrolysis and can lose its flexibility, becoming brittle and ultimately losing completely its tensile strength.

2.4.2.4 POLYESTER

Polyesters are usually highly crystalline and exhibit toughness, strength, abrasion resistance, low friction, chemical resistance and low moisture absorption. They are made by polymerising ethylene glycol with dimethyl terephthalate or with terephthalic acid. The amorphous region, although relatively large, is stiff at normal working temperatures. In the crystalline regions the molecules cannot easily slide under load because of the complex molecular structure and polar cross-bondings. As a result the material is less sensitive to creep than polypropylene and high-density polyethylene. However, it is to some extent susceptible to hydrolysis and the associated loss of strength.

2.4.3 METHODS OF MANUFACTURING OF PERMEABLE GEOSYNTHETICS

2.4.3.1 GEOTEXTILES

Geotextiles are similar to soil in that they have physical characteristics. They are permeable, have mechanical properties and suffer the effects of the environment.

The classification of geotextiles is shown in Fig. 2.5.

2.4.3.1.1 Wovens

Woven geotextiles were the first to be developed from synthetic fibres. Woven geotextiles are manufactured using traditional or modified weaving techniques. The weaving process gives these geotextiles their characteristic appearance of two sets of parallel threads known as yarns, interlaced at right angles to each other. The term warp and weft are used to distinguish between the two different directions of yarn. The yarn running along the length of the loom and hence along the length of the geotextile roll is known as the warp. The yarn running in the traverse direction, across the width of both the loom and the roll, is known as the weft. Although this form of construction may appear quite simple there are many different variations of the basic concept, giving a range of different forms of woven geotextiles.

The yarn used in either the warp or weft direction of a woven geotextile can either be a monofilament, multifilament or a mixture of both. Monofilament yarns consist of a single element or filament whereas multifilament yarns are composed of many separate fine filaments bundled together to form the yarn. The typical diameter of a round monofilament is about 0.5mm, whereas the typical diameter of an individual filament from a multifilament yarn is about 25μm. Extruding molten polymer through dies produces individual filaments. The filaments are usually stretched either during or shortly after extrusion to align the polymer molecule chains and so strengthen them. Simultaneously drawing and twisting a number of filaments form multifilament yarn. The greater surface area of multifilament yarns enables them to be more uniformly heated during drawing, facilitating a greater draw ratio and hence a greater strength and elastic modulus.

Some early woven geotextiles were made from monofilaments that were roughly rectangular and about three times as wide as they were thick. More recently, slit film tapes, which are generally much thinner are manufactured by slitting a thin sheet of polymer with sharp blades. Slit film tapes are generally about ten to twenty times as wide as they are thick with a width that is typically in the region of 3 mm.

Two basic weave patterns are employed in woven geotextiles. These are known as: plain and twill weaves. In the plain weave both the warp and the weft yarns follow an "over one, under one, over one" sequence while twill weave has a characteristic diagonal patterns to the exposed lengths of warp and weft in the form of "over two, under one, over two" sequence, Fig. 2.6.

It is generally found that woven geotextiles have a relatively high strength and relatively low extensibility when compared to other types of geotextiles of the same weight.

2.4.3.1.2 Non-wovens

Many products are known as non-woven geotextiles. These are composed of elements bonded together by mechanical, melting or chemical means to form a relatively smooth uniform mat. Each non-woven geotextile manufacturing system includes four basic steps:

1) Fibre preparation
2) Web formation
3) Web bonding
4) Post-bonding

Non-wovens are formed from filaments or fibres arranged at random and then bonded together into planar structure. The absence of any preferred orientation of the filaments results in properties which are approximately equal in all directions. Common types of non-woven geotextile are:

(a) Needle punched due to mechanical bonding

A fibrous web is introduced into a machine equipped with a group of specially designed needles. The needle density can reach up to 500 needles/cm². In some cases, the needles may vibrate or rotate to enhance the degree of entanglement. These materials have high density and considerable bulk and their thicknesses are generally between 0.25mm and 6mm. This basic technique is also used to manufacture composites, like geomembranes.

(b) Melt bonded due to thermal bonding

These materials are manufactured by spraying continuous polymer filaments onto a moving belt which is then passed through heated rollers which cause partial melting leading to thermal bonding of the filament cross over points. These materials are tough, compact and thin with thicknesses ranging between 0.5mm and 3.0mm.

(c) **Resin bonded due to chemical bonding**

Acrylic or similar resins are sprayed onto a fibrous web. After cutting or rolling, strong bonds are formed between the filaments. The thickness of such types ranges between 0.5mm and 3.0mm.

2.4.3.1.3 Knitted

Knitted geotextiles are made from one yarn or several yarns employed in a rectangular or diagonal system. Opposite to the woven geotextiles, the yarn is interweaved by means of loops. The interaction between knitted geotextiles and soil is equal to the parameters obtained from woven geotextiles. Knitted geotextiles can take at low extension high tension forces and compared to the woven geotextiles they show very low pre-extension. Some products with specially aligned yarns can take diagonal tension forces.

2.4.3.1.4 Composites

In geotextile composite products, at least one geotextile is employed. The materials are formed as different structures and arranged in layers. The different materials are interconnected over the whole width. Composites incorporate the best features of different materials in such a way that the interaction of the one material to another provides an optimal solution for a specific problem, viz. filtration and reinforcement within one product.

2.4.3.2 PRODUCTS RELATED TO GEOTEXTILES

Figure 2.7 shows the classification of products related to geotextiles.

2.4.3.2.1 Geogrids

Geogrids with integral junctions are usually manufactured from polymer sheets using the production sequence shown Fig. 2.8. The first stage involves precise punching of a pattern of holes into the sheet. This is followed by carefully stretching of the sheet in one direction while it is gently heated. The action of stretching the sheet aligns the long chain molecules of the polymer in the direction of stretch, giving the grid a high tensile stiffness in this direction. If no further processing is carried then the greatest strength properties lie in this direction. This type of grid is named Uniaxial Grid.

Incorporating a second stretching stage when the uniaxial grid is pulled in the traverse direction may produce another form of geogrid; this is then called Biaxial Grid. In this case the term biaxial refers to the fact that stretched ribs are aligned in two directions.

Subsequent to the development of geogrids with integral junction, welded/ heat bonded strip grids have been developed. These consist of two sets of polymer strips which intersect at 90° and are heat welded at the intersection. In this form of grid, the stretching of the polymer chains is carried out during extrusion of the high tenacity filaments that are used for the core material. In this manner it is possible to achieve a greater degree of polymer alignment within the prime strength directions.

2.4.3.2.2 Products related to Geogrids

Geogrid related products include nets, strips and mats or any other geosynthetic with aperture width greater than the strand width.

Geonets consist of two roughly rounded polymer strands that cross at a constant angle to give a very open structure with large diamond or rectangle shaped apertures. This shape is called a mesh or net. A typical size of the strands is 2mm with 7mm apertures, although nets with apertures up to 75mm have been manufactured. The strands are usually brought together whilst still molten to produce thermal bonds where they cross. The intersection angle between the two sets of strands is often constant for any particular product and generally lies between 60° and 90°.

The form of geogrids and geonets is that of a two dimensional structure.

Geomats have comparatively large opening size with a distinct three-dimensional structure. Most geomats are made from semi-rigid monofilaments, less than 1mm in diameter, that are tangled together to form a very open crinkly three-dimensional matting. Small heated plungers are inserted in a rectangular pattern to produce their shapes. Moreover geomats can be manufactured from two or more sheets of fine geotextile mats.

2.4.3.2.3 Geocomposites

Geocomposites can be manufactured from two or more geosynthetic products. A composite can combine the properties of the constituent members in order to meet better the needs of a specific application. The basic philosophy of geocomposites is to incorporate the best features of different materials in such a way that the interaction of the one material to another provides an optimal solution for a specific problem. Geocomposites will generally be synthetic materials, but certainly not always. Sometimes, a natural material may be used with a synthetic one for optimal performance and cost. The number and application range of geocomposites possibilities is almost limitless.

2.4.4 Methods of Manufacturing of Impermeable Geosynthetics

Impermeability is desired to prevent contained materials escaping into the biosphere. The degree of impermeability depends on the contained material, viz. liquid or gaseous. Membranes are used in various geotechnical applications where it is necessary to prevent the loss of water or toxic materials.

2.4.4.1 Membranes

Membranes are produced as sheets and then connected on site to each other by different means, viz. welding, gluing, vulcanisation, to create an impermeable barrier. Two different types of materials are used to impart impermeability either synthetic or bituminous materials. The classification of membranes is shown in Fig. 2.9.

2.4.4.1.1 Homogeneous

Homogeneous membranes are made from one material. However, the material may show different stages of polymerisation.

2.4.4.1.1.1 Foils (d < 1.0mm)

Foils show a maximum thickness of 1.0mm. They are not used in geotechnical long-term application. Their main use is more in wrapping or protection during application or transport. However, various materials are employed in the production of thin foils.

2.4.4.1.1.2 Foils (d > 1.0 mm)

The required minimum thickness for landfill sites is 2.5mm. The used materials are PE and bitumen or a mixture of both which is based on a mix-polymerisation. The membrane surface is dependant on the application it may be smooth to reduce friction between the membrane and the material or structured to increase friction. The connection between two sheets is obtained by:

- Welding: gas, extrusion, flame and heat wedge welding
- Gluing: diffusion, adhesion and hot gluing
- Vulcanisation

2.4.4.1.2 Heterogeneous

In general, two different heterogeneous membranes exist which may be classified according to their production technique. Membranes are produced from a mixture of at least two different materials and products where homogeneous membranes are connected in layers.

2.4.4.1.2.1 Composite Foils

Basically two categories exist which are the composite foils with connected protection layers and without. The function of the protection layer is to increase the resistance of the membrane against the environment and/or the contained material.

Present practice is to reinforce a membrane with a textile layer. The textile reinforcement can be used in geomembranes for different reasons:

- Impart stability to the polymer compound during manufacturing
- Provide dimensional stability to compounds that would excessively shrink or suffer expansion as a result of temperature changes
- Increase the operational strength of the membranes
- Reduce the elongation of membranes when they are subject to stress.

To make an impermeable connection between individual elements the above mentioned techniques are often employed. The overall strength has to remain same as a reduction due to a connection is not allowed. The connections have to remain impermeable during all working conditions.

2.4.4.1.2.2 Bituminous Membranes

Bituminous membranes with synthetic support are available. The synthetic support stabilizes the membrane, increases the impermeability and reduces the penetration of in-situ material due to reinforcement of the outer layers.

2.4.4.2 PRODUCTS RELATED TO MEMBRANES

Products related to membranes are often produced off-site, however, some manufacturing techniques allow an on-site production. In general, for geotechnical use, the water impermeability factor has to guaranteed (k □ 10^{-9} m/s). The membrane layout can consist of one or several layers. The classification of products related to membranes is shown in Fig. 2.10.

2.4.4.2.1 Geomembranes

The geomembrane is a composite material of at least one geotextile and a layer of clay. The main use is the base foundation of landfill sites to form an impermeable layer which prevent contaminated water to escape into the biosphere.

2.4.4.2.2 Geosynthetic Clay Liners

Bentonite is specially prepared clay that is then bonded by friction, cohesion and/or adhesion between two layers of a geotextile. This composite is then called a Geosynthetic Clay Liner [GCL]. The difference between a geomembrane and a GCL is the material used. However, only bentonite is presently used in the production of geomembranes, therefore geomembranes and GCL may be considered to be the same.

GCLs are self-sealing barriers with a composite structure that utilises the strength of needle-punched fibres to secure a uniform layer of high swelling natural sodium bentonite. This combination of polymer fibres and a naturally occurring clay mineral creates the sealing layer with high long-term shear strength. GCLs are the industry standard for mineral sealing layers worldwide. Once hydrated, bentonite is an effective barrier against liquids, vapours, and gases.

2.4.4.3 OTHERS

Other impermeable membranes or sheets exist but it is beyond the scope of this thesis to cover them all.

Furthermore, there exist plastic pipes, slotted or perforated, which are used for groundwater, effluent or leachate collection and removal.

2.5 FUNCTIONS OF GEOSYNTHETICS

Geosynthetics always perform at least one of the basic functions, McGown et al (1973) and Leflaive et al (1974).

- Separation
- Filtration
- Drainage
- Reinforcement

However, Geosynthetics have also been described as performing one of these four additional functions, Giroud (1981) and Saathoff et al (1996).

- Screening
- Sealing
- Wrapping
- Erosion Control

2.5.1 SEPARATION

Separation is defined as preventing different materials, placed next to each other, from mixing.

A geosynthetic placed between two materials functions as a separator when it prevents the two materials from mixing under the action of applied loads. When placing coarse materials on a fine material there are two mechanisms that tend to occur; one is that the fines attempt to enter into the voids of the coarse material so reducing its strength and its drainage capability, while the other is that the coarse intrudes into the fine material thereby reducing its strength.

The distribution of local pressures is positively influenced by separation. Therefore, it is often possible to install soil layers of high bearing capacity on weak soil.

2.5.2 FILTRATION

Filtration is defined as the transport of liquid or gas normal to the plane (across) of the geosynthetic.

In the past filters were mostly granular soils arranged in layers. Today a geosynthetic functions as a filter when placed in contact with a soil or fine material and allows water to pass through whilst retaining most particles. The selection of product types is important since permeability requires an open structure whilst particle retention requires a tight structure. It is required that the permeability in the long term use is assured and the voids are not blocked due to the clogging of fine soils.

2.5.3 DRAINAGE

Drainage is defined as the transport of liquid or gas in the plane (along) of the geosynthetic.

A product functions as a drain when it collects a liquid or a gas and conveys it towards an outlet. In engineering applications are mineral drains commonly used where the geotextile acts as a wrapping to prevent the clogging of the drain. However, recently developed geosynthetic materials act as drains by themselves. Different products provide widely varying degrees. There is an overlap of drainage and filtration and a product may perform mainly in drainage with a secondary filter function or vice versa.

2.5.4 Reinforcement

Reinforcing soil is defined as the improvement of the operational soil tensile strength within geotechnical structures.

Geosynthetic reinforcements improve the strength of a soil in three different ways:

- By a membrane effect, when a vertical load is applied to a geosynthetic on a deformable soil.
- By a shear reinforcement effect, when a geosynthetic is placed on a soil loaded in a normal direction and then the two materials are sheared at their interface.
- By an anchorage effect, when a tensile force is developed and then the geosynthetic is prevented from pulling out of the soil.

Usually these tension forces have to be taken over a long-term and involve material strain compatibility. The commonly used reinforcing materials are woven geotextiles or geogrids.

The main application area is in GRSSs, such as steep slopes, walls and embankments.

2.5.5 Screening

Screening is defined as the stopping of fine particles carried in suspension, whilst allowing the fluid to pass.

A geosynthetic placed across the path of a flowing fluid (water stream or wind) carrying particles in suspension, functions as a screen when it stops fine particles whilst allowing the fluid to pass through.

After some time, particles accumulate against the screen such that it requires to be able to withstand pressures generated by the accumulated particles and the increasing fluid pressure. It may be used as silt fence, vertical silt curtain or similar. The transition between screening, drainage and filtration is fluent.

2.5.6 Sealing

Sealing is defined as preventing the escape of contained materials.

An impermeable geosynthetic may be used to prevent contained materials escaping into the biosphere. They have been used to seal wastes, landfills and contaminated dredged harbour muds.

2.5.7 Wrapping

Wrapping is defined as maintaining a mass of material in its shape.

A geosynthetic functions as a wrapper when it maintains a mass of materials such as sand, rocks or fresh concrete in position.

A geosynthetic may be formed in to a tube or bag shaped form and then filled with material. The material chosen depends on the application, e.g. it may be waterproof or permeable wrapping. It is used to form a concrete mattress for slope protection or protection against scouring, concrete filled bags or sand filled tubes and wrapping of dredged materials.

2.5.8 Erosion control

Erosion control is defined as preventing soil particles from dispersing due to environmental forces.

A geosynthetic placed on a slope, functions as an erosion control mat when it restricts movements and prevents dispersion of soil particles subjected to erosion actions of rain and wind, often while vegetation, which will eventually perform this function, is growing. Various product types are used in engineering applications.

2.6 APPLICATIONS

It is beyond the scope of this thesis to cover all applications of geosynthetics. However, some common applications in different fields of civil engineering may be described as follows.

2.6.1 GEOTEXTILES

The ease of application, the relatively low unit cost and the multi-functions performed (one or more main functions with one or more secondary functions) have made geotextiles very attractive materials for many applications. Geotextiles are utilized in major projects such as access and unpaved roads. One of the best surveys of geotextile applications is reported by Koerner (1998).

(a) Geotextile applications involving separation of dissimilar materials

- Between a subgrade and the stone base in unpaved roads and airfields
- Between a subgrade and the stone base in paved roads and airfields
- Between a subgrade and the ballast in railroads
- Between the foundation and embankment soils for earth and rock dams
- Between the foundation soils and rigid retaining walls
- Between the foundation soils and flexible retaining walls
- Between the foundation soils and storage piles
- Beneath sidewalk slabs
- Beneath curb areas
- Beneath parking lots
- Beneath sports and athletic fields
- Between old and new asphalt layers

(b) Geotextile application involving reinforcement of weak soils

- Over soft soil for unpaved roads
- Over soil for airfields
- Over soft soil for railroads

- Over soft soil in sports and athletic fields
- To construct geosynthetic-reinforced walls
- To reinforce embankments
- To aid in the construction of steep slopes
- To reinforce earth and rock dams
- To reinforce jointed flexible pavements
- To stabilize unpaved storage yards and staging areas
- To anchor facing panels in reinforced earth walls
- To contain soft soils in earth dam construction
- For use with in-situ compaction and consolidation of marginal soils
- To aid in the bearing capacity of shallow foundations

(c) Geotextile applications involving filtration (cross plane flow)

- In place of granular soil filters
- Beneath stone base for unpaved roads and airfields
- Beneath stone base for paved roads and airfields
- Beneath ballast under railroads
- Around crushed stone surrounding underdrains
- Around crushed stone with underdrains
- Around perforated underdrain pipes
- Around stone and perforated pipes in the fields
- Beneath landfills that generate leachate
- To filter hydraulic fills
- To protect chimney drain material
- To protect drainage gallery material
- Between backfill soil and voids in retaining walls
- Between backfill soil and gabions
- Against geogrids to prevent soil intrusion
- Around sand columns in sand drains
- As a filter beneath stone riprap
- As a filter beneath precast block

(d) **Geotextile applications involving drainage (in-plane flow)**

- As a drainage gallery in an earth dam
- As a drainage interceptor for horizontal flow
- As a drainage blanket beneath a surcharge fill
- As a drainage behind a retaining wall
- As a drainage beneath railroad ballast
- As a water drain beneath geomembranes
- As a drain beneath sport and athletic fields
- As a drain for roof gardens
- As a pore water dissipator in earth fills
- As a replacement for sand drains

2.6.2 GEOGRIDS

Geogrids generally exhibit high moduli and strengths which make them suitable for reinforcement. Also they have a second use, as a drainage core in geocomposites. The following uses of Geogrids have been reported by Al-Mudhaf (1993), Koerner (1998) and OCED Road Transport Research (1991).

- Beneath aggregates in unpaved roads
- Beneath aggregates in paved roads
- Beneath ballast in railroad construction
- Beneath surcharge fills or temporary construction sites
- Reinforcement of embankment fills and earth dams
- As sheet anchors for retaining-wall facing panels
- As sheet anchors and facing panels to form a retaining wall
- As asphalt reinforcement in pavements
- As cement or concrete reinforcement in a wide variety of applications
- To reinforce disjointed rock sections
- To reinforce disjointed concrete sections
- As inserts between geotextiles
- As inserts between geomembranes
- As insert between a geotextile and geomembrane

2.6.3 MEMBRANES

Membranes are used in geotechnical applications where it is necessary to prevent the loss of the contained materials and entry in the containment.

- Liquid containment: to prevent the loss of liquids from the containment structures
- Gas containment: to prevent the loss of gases
- Waste containment: to prevent the loss of toxic chemicals waste from impound structures
- Pavements: to control moisture movements in pavements, enabling them to maintain their structural integrity

2.7 SUMMARY

The following important aspects can be identified for geosynthetics from the literature review.

Geosynthetics show a broad variety of products used in many different applications. This thesis deals with polymeric reinforcements used in retaining walls and steep slopes, as this is an important commercial aspect of the use of geosynthetics.

The design of GRSSs has to be carefully chosen to prevent loss of human life due to failure of the reinforcing element and therefore a loss of stability of the structure. Furthermore, the University of Strathclyde provided major contributions to the research of GRSSs in the past. This research indicated that there is a need for the understanding of the load-strain-time-temperature material behaviour if subject to a combination of loadings. Therefore, this thesis deals with polymeric reinforcing elements under such combinations of loadings.

CHAPTER 3

DESIGN METHODOLOGIES FOR GRSSs

3.1 GENERAL

Traffic, earthquakes, explosions and a variety of intermittent loads apply cyclic or short-term loading to Geosynthetic Reinforced Soil Structures [GRSSs] in addition to sustained load. Very little is known about the behaviour of GRSSs under such Multi-Stage Actions. Nevertheless it is appreciated that the duration of the cycle or the short-term loading during these events has an important influence on the response of the foundation materials and the structures itself.

Therefore, in this Chapter, the design methodologies for GRSSs are identified and the present practice for designing against cyclic or short-term loading events is discussed.

3.2 REVIEW OF PRESENT DESIGN METHODOLOGIES

The present design approaches to external stability have been based on classical methods for gravity retaining walls, with the addition of internal stability calculations to take the effect of reinforcements into account. The majority of design methods for GRSSs are based on the Limit Equilibrium Approach. Although some design codes and methods have considered the Limit State principles, they may be termed Hybrid Approaches as they are based partly on the Limit Equilibrium Approach and partly on the Limit State Approach. The existing design methods involving these three design approaches are given in the following sections.

3.2.1 LIMIT EQUILIBRIUM APPROACH

This method of analysis is based on the assumption that all the components simultaneously reach their limiting stress conditions. This assumption is not valid for GRSSs as the limiting stress conditions for soils and geosynthetics may occur at quite different strain levels. As with all the GRSS's design methods, the following stability analyses have to be carried out to ensure the safety of the structures:

(i) External Stability

(ii) Internal Stability

The analysis of the external stability for the GRSSs is similar to the conventional retaining wall analyses. It is assumed that the reinforced soil zone acts as monolithic structure. The external stability analysis consists of the analysis of:

- Sliding of the structure at the base
- Overturning of the structure
- Bearing failure of the subsoil
- Overall stability of the structure

The internal stability analysis consists of the following:

- Tension failure of the reinforcing elements
- Wedge/Pull-out failure of the reinforcing elements

No direct or explicit analysis for deformation of the soil boundary is considered in this approach. It is assumed that once the external and internal stability criteria are satisfied, the deformation criteria will also be satisfied. Although this assumption may be overlooked for inextensible reinforcements, for extensible reinforcements this may be seen to be a critical issue.

Many design methods have evolved on the basis of the Limit Equilibrium Approach. Examples of these are the AASHTO (1997), the DIBt (1998) and the HA 68/94 (1997) methods. The salient features of these design methods are given below:

3.2.1.1 AASHTO Standard Specifications for Highway Bridges (1997)

This design method suggests the procedure for designing walls and steep slopes.

The soil property used in the analysis is the Peak Angle of Friction [ϕ'_p] and the Long-term Strength [T_{al}] of the reinforcing element for Single-Stage Actions and Multi-Stage Actions such as sustained loading plus traffic loading is calculated by:

$$T_{al} = \frac{T_{ult}}{RF} \tag{3.1}$$

where,

RF = Reduction Factor

= $RF_{CR} \times RF_{ID} \times RF_D$

T_{ult} = short-term CRS rupture strength of the reinforcing element per metre width.

RF_{CR}= Reduction Factor for long-term rupture strength.

RF_{ID} = Reduction Factor for damage from placing and compaction.

RF_D = Reduction Factor to prevent rupture of the reinforcement due to chemical and biological degradation

The Reduction Factor RF_{CR} is determined by comparing the CRS rupture strength (at 10% per minute strain rate) to the long-term rupture strength of a geosynthetic at a specific time and temperature. The Reduction Factor RF_{CR} thus depends on the type, design life time and operational temperature of the reinforcement.

Reduction Factors RF_{ID} and RF_D are determined by comparing the CRS rupture strengths of a geosynthetic 'before' and 'after' construction damage and environmental degradation respectively.

For design against sustained loading plus traffic loading, the live load surcharges are treated as uniform distributed surcharge loads that are effective over the whole design life.

For design against sustained loading plus short term earthquake loading, [T_{al}] is increased by reducing RF to eliminate creep as a consideration, i.e. by considering $RF_{CR} = 1$.

Global Factors of Safety are applied at the end of both external and internal stability analyses, Table 3.1. This design method requires checking the rupture and pullout failures of the reinforcements. The single wedge method is employed in the analysis calculating the disturbing force. A critical wedge is found by trial and error and that is which gives the maximum disturbing force. This disturbing force should not exceed the minimum of (i) the long-term strengths, or (ii) the pullout resisting forces provided by reinforcements within a failure wedge.

3.2.1.2 DEUTSCHES INSTITUT FÜR BAUTECHNIK (DIBt) METHOD (1998)

This design method is based on the Deutsches Institut für Bautechnik Certificate No. Z20: 1-102 and is capable of designing walls and steep slopes. Stability analyses have to conform to different DINs, e.g. the sliding check has to conform to DIN 1054 and the bearing check to DIN 4017.

The soil property used in the analysis is the Constant Volume Angle of Friction [ϕ'_{cv}] and the Permissible Working Load [F_{perm}] of the reinforcing element for Single-Stage Actions and Multi-Stage Actions such as sustained loading plus traffic loading is calculated by:

$$F_{perm} = \frac{F_B}{A_1 A_2 \gamma} \tag{3.2}$$

where,

F_B = short-term CRS rupture strength of the reinforcing element per metre width.

A_1 = Reduction Factor for long-term strength, (= 2.40 to 2.75).

A_2 = Reduction Factor for damage from placing and compaction, (= 1.05 to 1.5).

γ = calculated Factor of Safety, (= 1.75).

The Reduction Factor A_1 is determined by comparing the CRS rupture strength (at 33% per minute strain rate) to the long-term rupture strength of a geosynthetic at a specific time and temperature. The Reduction Factor A_1 thus depends on the type, design lifetime and operational temperature of the reinforcement.

The Reduction Factor A_2 is determined by comparing the CRS rupture strengths of a geosynthetic 'before' and 'after' construction damage. The values of A_2 thus depends on the type of the soil and the type of geosynthetic.

For design against sustained loading plus traffic loading, the live load surcharges are treated as uniform distributed surcharge loads that are effective over the whole design life. Two loading cases are considered: Permanent traffic loads (load case 1) where the Overall Factor of Safety [FOS] is given as $\gamma_1 = 2.0$ and for temporary traffic loads (load case 2) where the Overall FOS is given as $\gamma_2 = 1.5$.

For design against sustained loading plus short term earthquake loading, the Permissible Working Load [F_{perm}] is then increased by considering $A_1 = 1.0$ and $\gamma = 1.31$.

Global Factors of Safety are applied at the end of both external and internal stability analyses, Table 3.1. This design method does not require checking the individual reinforcing element against its permissible working load. The structure is checked against the pullout failure. A single wedge method is employed in the analysis of calculating the disturbing force. A critical wedge is found by trial and error that gives maximum disturbing force. This disturbing force should not exceed the minimum of (i) the permissible working load, or (ii) the pullout resisting force. The pullout check is carried out at places only where the reinforcement spacing or type changes.

3.2.1.3 HA 68/94 METHOD (1997)

The British HA design method is based on the Department of Transport Advice Note HA 68/94. It provides a single unified Limit Equilibrium design approach for all types of reinforced highway earthworks with slope angle ranging from 10° to 70° to the horizontal. The design assumes that the foundation is competent.

The philosophy of the design method is to use the soil strength parameters which represent the minimum conceivable values so that no further overall Factor of Safety needs to be applied to the design. The design suggests the use of the Constant Volume Angle of Friction [ϕ'_{cv}] and the Design Strength [T_d] of the reinforcing elements for Single-Stage Actions calculated as:

$$T_d = \frac{f_k}{\gamma_m} \tag{3.3}$$

where,

f_k = characteristic long - term strength (at 10% limiting strain) of the reinforcing element per metre width

γ_m = Partial Factor obtained from CRS tests

Within this method no specific Design Strength [T_d] is suggested for sustained loading plus traffic loading or sustained loading plus earthquake loading.

The analysis is based on a two-part wedge analysis. The reinforcing element length at the base of the structure is determined using a trial and error method which finds the wedge which gives the disturbing force equal to zero. This is termed the T_{ob} mechanism. The length at the top of the structure is based on the so-called T_{max} mechanism. In this process a trial and error method is used to find the wedge which gives the maximum disturbing farce which is known as the T_{max} mechanism. Different wedges are analysed at different depths, to ensure that the disturbing force does not exceed the resisting force from the reinforcing elements. In this method, individual reinforcing elements are not checked against rupture failure. No Factor of Safety is applied in the analysis.

3.2.1.4 SUMMARY

i) AASHTO (1997) design method uses the Peak Angle of Friction [ϕ'_p], but HA 68/94 (1997) and DIBt (1998) use the Constant Volume Angle of Friction [ϕ'_{cv}] as the design parameters for soils.

ii) Both AASHTO (1997) and DIBt (1998) define Reference Strengths for Single-Stage Actions and Multi-Stage Actions (sustained loading plus traffic loading) as the 'Ex-works' long-term rupture strengths, obtained by applying a Reduction Factor to the short-term strengths. However, it should be appreciated that the short-term strength in the AASHTO (1997) method is obtained at a strain rate of 10% per minute whereas in the DIBt (1998) method it is obtained at a strain rate of 33% per minute. The Reduction Factors used to derive the 'Ex-works' long-term rupture strength according to these two methods will, therefore, be different.

iii) For sustained loading plus earthquake loading, Reference Strength is considered to be the short-term CRS strength by AASHTO (1997) and DIBt (1998). However, because the strain rates used for CRS tests are different, the Reference Strengths will also be different.

iv) The Reduction Factors or Partial Factors for construction damage and environmental degradation are determined by comparing the short-term CRS rupture strengths 'before' and 'after' the event in the AASHTO (1997), HA 68/94 (1997) and DIBt (1998) methods. The Design Strengths for Single-Stage Actions and Multi-Stage Actions are then derived by dividing the Reference Strengths by the Reduction Factors or Partial Factors.

v) It may be noted that for both external and internal stability analyses, the global Factor of Safety approach is adopted in the AASHTO (1997) and DIBt (1998) design methods. The HA 68/94 (1997) does not apply any Factors of Safety to the external and internal stability analyses.

3.2.2 Hybrid Approach

Some design codes and methods have used Hybrid Approach for the design of GRSSs, for example BS 8006 (1995) and the TBW Method (1998).

3.2.2.1 BS 8006 (1995)

BS 8006 adopts a limit state design approach whereby individual Partial Factors are applied to the various forces acting on the structure and the soil/reinforcement properties. The object of this approach is to apply appropriate Partial Factors where they are required, i.e. the greatest Partial Factors should be applied where there is the greatest uncertainty.

In this method, the peak effective shear strength parameters are recommended for the soils for both Ultimate and Serviceability Limit State analyses, i.e. ϕ'_p, c'_p. For walls, steep slopes and embankments the Design Strength [T_d] of the reinforcement for Single-Stage Actions and Multi-Stage Actions (sustained loading plus traffic loading) should be taken as

$$T_d = \frac{T_b}{f_m} \tag{3.4}$$

where,

T_b = unfactored Reinforcement Base Strength

f_m = Partial Factor

Reinforcement Base Strength [T_b] should be the following:

a) For the Ultimate Limit State [ULS] the base strength is the tensile creep rupture strength [T_{cr}] at the appropriate times and design temperature.

b) For the Serviceability Limit State [SLS] the base strength is the tensile load in the reinforcement [T_{cs}] which induces the prescribed post construction limit state strain (0.5% for abutments and 1.0% for retaining walls).

For design against sustained loading plus traffic loading, the traffic loads are calculated as the wheel load divided by the contact area and considered as a uniform surcharge load over the whole design lifetime.

Within this code, no Design Strength [T_d] is suggested for sustained loading plus earthquake loading.

The Partial Factor [f_m] has two components:

$$f_m = f_{m1} \times f_{m2} \qquad (3.5)$$

where,

f_{m1} = Partial Factor related to the intrinsic properties of the material

f_{m2} = Partial Factor concerned with the effects of construction and environmental effects

Partial Factor f_{m1} is further divided into several components as:

$$f_{m1} = f_{m11} \times f_{m12} \qquad (3.6)$$

$$f_{m1} = (f_{m111} \times f_{m112}) \times (f_{m121} \times f_{m122}) \qquad (3.7)$$

where,

f_{m11} = Partial Factor related to the consistency of the manufacturer

f_{m12} = Partial Factor related to the extrapolation of test data dealing with Base Strength

f_{m111} = Partial Factor related to whether or not a standard for specification, manufacture and control testing of the reinforcement exist

f_{m112} = Partial Factor for whether or not standards for the dimensions and tolerances exist

f_{m121} = Partial Factor for the assessment of available data

f_{m122} = Partial Factor for extrapolation of the statistical envelope over the expected service life of the reinforcement

Partial Factor f_{m2} is also divided into several components as:

$$f_{m2} = f_{m21} \times f_{m22} \tag{3.8}$$

$$f_{m2} = (f_{m211} \times f_{m212}) \times f_{m22} \tag{3.9}$$

where,

f_{m21} = Partial Factor which deals with the installation damage of the reinforcements

f_{m22} = Partial Factor which deals with the environmental effects of the reinforcements

f_{m211} = Partial Factor related to short term effects of damage prior to and during installation

f_{m212} = Partial Factor for the long term effects of the short term damage

For applications within UK all the Partial Factors are certified and specified by the British Board of Agreement (BBA).

3.2.2.1.1 Ultimate Limit State Analyses

a) Sliding Analysis

The Factor of Safety against sliding failure [f_s] is given by the equation:

$$f_s R_h \le R_v \frac{\tan\phi'_p}{f_{ms}} + \frac{c'_p L}{f_{ms}} \tag{3.10}$$

where,

f_s = Factor of Safety against base sliding (=1.20)

R_v = vertical factored resultant force

R_h = horizontal factored disturbing force

ϕ'_p = peak angle of friction

c'_p = effective cohesion of soil

L = effective base width for sliding

f_{ms} = Partial Factor for material (1 for $\tan\phi'_p$ and 1.6 for c'_p)

b) Bearing Failure

The imposed bearing pressure [q_r] must be compared to the factored ultimate bearing capacity as follows:

$$q_r < \frac{q_{ult}}{f_b} + \gamma D_m \tag{3.11}$$

where,

f_b = Factor of Safety for bearing capacity (=1.35)

q_{ult} = ultimate bearing capacity of the foundation

D_m = embedment depth

c) Reinforcement Rupture

Each layer of reinforcement requires checking against rupture. The maximum tensile force in the i^{th} layer T_i is then calculated by:

$$T_i = K_a \sigma_v S_v \tag{3.12}$$

where,

K_a = coefficient of active earth pressure within the reinforced zone

σ_v = vertical stress on reinforcing element

S_v = vertical spacing of reinforcing element

In order to ensure stability with regard to rupture failure, the following relationship must be satisfied:

$$\frac{T_d}{f_n} \geq T_i \tag{3.13}$$

where,

T_d = Design Strength of the reinforcement

f_n = Partial Factor for economic ramifications of failure

d) **Pullout Failure**

The resistance provided by each individual reinforcing element must be taken to be the lesser of (i) pullout resistance, or (ii) the design strength. The total resistance from all reinforcing elements is checked by:

$$\sum_{i=1}^{n}\frac{T_d}{f_n} \; or \; \frac{2L_{ei}}{f_p f_n}\left[\mu\left(f_f \gamma_1 h_i + f_f w_s\right) + \frac{c'_r}{f_{ms}}\right] \geq T \qquad (3.14)$$

where,

L_{ei} = anchorage length beyond failure plane

f_p = Partial Factor against pullout [=1.30]

μ = coefficient of friction between soil and the reinforcements

f_f = Partial Factor for surcharge load

γ_1 = density of the reinforced fill

h_i = depth of reinforcement of i^{th} layer

w_s = surcharge load

c'_r = adhesion between the fill and reinforcement

f_{ms} = Partial Factor for c'_r

T = total tensile force to be resisted by reinforcing elements

3.2.2.1.2 Serviceability Limit State Analyses

For a polymeric reinforcement where short-term axial tensile stiffness decreases with time through the agency of creep, the strain occurring between the end of construction and the end of selected design life can be estimated from isochronous load strain curves for these two times. Figure 3.1 demonstrates this procedure, where T_{cs} is the capacity of the reinforcement at a prescribed limiting value of post construction strain.

3.2.2.2 THE TENSAR TIE-BACK WEDGE (TBW) DESIGN METHOD (1998)

In this method, due to the relatively large strains occurring within GRSSs, critical state or constant volume shear strength properties are used for the soils i.e. ϕ'_{cv}, c'_{cv}. The Design Strength [T_d] of the reinforcement for Single-Stage Actions is given by:

$$T_d = \frac{T_c}{f_{m1} x f_{m21} x f_{m22} x f_{m3}} \qquad (3.15)$$

where,

T_c = extrapolated creep strength (at 10% limiting strain) of the reinforcement at specified design life time and operational temperature

f_{m1} = Partial Factor to allow for material manufacturing variations and confidence in the extrapolated strength

f_{m21} = Partial Factor for the effects of construction activities

f_{m22} = Partial Factor for the effects of environmental degradation

f_{m3} = overall Factor of Safety (=1.35)

Within this method, no Design Strength [T_d] is suggested for any Multi-Stage Actions.

The Partial Factor f_{m1} and the overall Factor of Safety f_{m3} are specified by the manufacturers. Partial Factors f_{m21} and f_{m22} are obtained by comparing the short-term CRS rupture strengths 'before' and 'after' an event, i.e. construction damage or environmental degradation.

3.2.2.2.1 Ultimate Limit State Analyses

The procedure for ULS analyses adopted in this method is in accordance with BS 8006 (1995) with the exception that no Partial Factor is applied to the soil properties. Another difference is that in this method, a Factor of Safety of 2.0 is applied to sliding, bearing and pullout failure. No Factor of Safety is applied against rupture failure.

3.2.2.2.2 Serviceability Limit State Analyses

In order to ensure that the SLS is satisfied, two checks are undertaken during the analysis.

(a) The $K_a \times \sigma'_v$ check

This check examines the post-construction strain occurring in the reinforcements as a result of overburden pressure. It is assumed that wedges emanating from the toe at angles greater than 45° - $\phi_p'/2$ mobilise the full active pressure given by K_a, while wedges at angles less than ϕ_p' are subjected to zero active pressure i.e. K is zero. The active pressure acting on wedges at angles between ϕ_p' and 45° - $\phi_p'/2$ are obtained by linear interpolation.

At any point, the force mobilised in the i^{th} reinforcement T_i is given by $K_a \times \sigma'_{vi}$ For normal walls, the force required in order to mobilise 1% post-construction strain [T_{1pcs}] (usually around two-third of the creep limited strength) is determined for each reinforcement type from isochronous curves. The actual post-construction strain [ε_{pc}] in an individual reinforcement is therefore approximated by the equation:

$$\varepsilon_{pc} = \frac{T_i}{T_{1pcs}} \tag{3.16}$$

A similar approach is adopted for bridge abutments, but in this case the serviceability limit is reduced to 0.5% post-construction strain.

(b) The Wedge Check

In this check, the Out of Balance Force [OBF] for a series of wedges is determined. The OBF for each wedge is then distributed amongst the reinforcements cut by the wedge in proportion to their strength, Fig. 3.2.

Groups of wedges at 2° intervals are analysed at the base of the wall and levels of one tenth of the height of the face. The strain from each wedge in the individual reinforcement is then determined in accordance with Equation 3.16 and a strain distribution curve is obtained, Fig. 3.3. The area under the curve divided by the reinforcement length defines the average post-construction strain for each grid.

3.2.2.3 SUMMARY

i) BS 8006 (1995) uses the Peak Angle of Friction [$\phi'p$] for the soils for both ULS and SLS analyses, but the TBW (1998) method uses the Constant Volume Angle of Friction [ϕ'_{cv}].

ii) Partial Factors are applied to soil properties in BS 8006 (1995). The TBW (1998) method does not apply any such Partial Factors.

iii) In BS 8006 (1995), the Reference Strength for Single-Stage Actions and Multi-Stage Actions (sustained loading plus traffic loading) is defined as the 'Ex-works' long-term creep rupture strength. In the TBW (1998), the Reference Strength for Single-Stage Actions is defined as the 'Ex-works' long-term creep strength at a limiting strain of 10%. None of these methods specify Reference Strength for sustained loading plus earthquake loading.

iv) In both BS 8006 (1995) and TBW (1998), the Design Strengths of the reinforcements are derived by dividing the Reference Strengths with several Partial Factors. These Partial Factors are determined from the short-term CRS tests.

v) Factors of Safety are applied to some of the analyses in both BS 8006 (1995) and TBW (1998) method.

vi) The SLS analysis is based on the assumption that the design soil strength will be mobilised at the end of post-construction creep. This may not always be valid. Further, the SLS analysis does not take account of the strain compatibility between soil and reinforcement.

vii) Overall, these design methods may be considered as a combination of Limit Equilibrium Approach and Limit State Approach.

3.2.3 "TRUE" LIMIT STATE APPROACH

The soil and the geosynthetics both influence the behaviour of GRSSs, and the equilibrium is too complex to be reflected by a single 'lumped' factor or 'global' Factor of Safety. Rather the procedure should be to select representative design values for the parameters influencing stability and then to perform calculations to show that possible undesirable design situations will not occur. This is considered in the Limit State Approach, Jewell (1996).

The Limit State Approach does not use global Factors of Safety, instead Partial Factors are applied to the calculations where the uncertainties lie. The use of Partial Factors aims to distribute margins of safety to the places in the calculation where there are uncertainties. Also Partial Factors have the advantage that "margins of safety" can be shared appropriately among the main parameters employed in the design. For example, a structure may be subjected to two Actions, one adverse and the other beneficial, so that the nominal value of each cancels the other. Hence the resistance required for the structure will be very small or zero, on the basis of the nominal Action. However, if in reality one of the Actions is different from the expected value, the required resistance may be very high. This possible situation can be anticipated using Partial Factors, but could be missed in the global Factor of Safety approach, McGown et al (1998).

There are two main areas of concern for design in Limit State Approach. First, the risk of collapse must be shown to be acceptably small by means of Ultimate Limit State [ULS] calculations. Second, the risk that deformations may interfere with the functioning of the structure must be shown to be acceptably small by means of Serviceability Limit State [SLS] calculations.

The Limit State Approach of design is set out in International and European Standards (ISO 2394 and CEB Bulletin III) and is being widely introduced in geotechnical engineering. The draft Eurocode 7 (1995) for geotechnical engineering is based on the Limit State Approach.

The Limit State Approach to the design of GRSSs should be based on internal force equilibrium and strain compatibility as set out by Jewell (1985) and (1988). The valid mechanisms for ULS and SLS analyses need to be identified first and then the appropriate parameters of the materials can be identified on the basis of these mechanisms.

The mechanisms related to the external stability and operational performance of GRSSs are taken to be the same as for conventional soil structures. The main differences between conventional soil structures and GRSSs are the mechanisms related to internal stability and operational performance.

3.2.3.1 A Model for the Internal Ultimate Limit State Mechanism

Long-term sustained load [creep] test data for geosynthetics are most appropriate to the design of GRSSs under Single-Stage Actions. To simulate the creep test conditions and to facilitate the direct application of the test data to the design of a prototype structure, a constant *Out of Balance Force* [OBF] must be applied to the reinforcements from the *End of Construction* [EOC] to the *End of Design Life* [EDL]. Such a situation can be achieved only if the tensile strain in the soil is large enough to mobilise the large strain Constant Volume Angle of Friction, [ϕ'_{cv}], at the End of Construction. Thereafter, until the End of Design Life, the soil may strain further but the mobilised shear strength of the soil will remain constant, i.e. the Constant Volume Angle of Friction, [ϕ'_{cv}], will continue to be mobilised.

For the above condition, a *Model Limit State Mechanism* for GRSSs can be identified. In this model it is assumed that a constant load is applied to the reinforcements from the time of *Switch-on of Gravity* [t_{SOG}] to the *End of Design Life*, [t_{EDL}].

In practice all the loads are not applied instantaneously, although it is generally assumed that they are applied simultaneously at some point during the *Construction Period*, [t_{CP}]. An important point to realise is that the overall Construction Period may be much longer than the *Reinforcement Loading Period*, [t_{RL}], depending on the construction procedures adopted. For example, a wall may be fully propped until the fill is placed to full height, during which time the reinforcements will not be loaded. Only on removal of the props will the reinforcements be loaded. Thus great care must be taken when selecting the Reinforcement Loading Period, [t_{RL}].

When the loads are "switched-on", at say two-thirds the Reinforcement Loading Period, (2/3 t_{RL}), they will develop a strain in the soil, [ε_{SOG}], sufficient to mobilise the large strain Constant Volume Angle of Friction [ϕ'_{cv}] of the soil. If such a situation is achieved, there will be no change in the Out of Balance Force from the time of Switch-on of Gravity to the End of the Design Life and the geosynthetic reinforcements will always carry the same load. Long-term sustained load [creep] test data thus directly apply and the Model Limit State Mechanism applies.

3.2.3.2 A Model for the Internal Serviceability Limit State Mechanism

As stated above, in practice all loads are not applied instantaneously. As for the Ultimate Limit State condition, it can be assumed that all the Serviceability Limit loads are applied at say two-thirds of Reinforcement Loading Period (2/3 t_{RL}), although this depends on the construction procedures.

Normally, these loads mobilise an angle of friction in the soil, [ϕ'_{mRL}] less than the peak angle of friction [ϕ'_p]. With time, the soil and the reinforcement strain and at the End of Design Life the mobilised angle of friction [ϕ'_{mEDL}] is likely to remain less than the peak angle of friction, but higher than the mobilised angle of friction at the End of Construction Loading. As the mobilised shear strength of the soil increases with time, the force required to be carried by the reinforcements will gradually become less, which means that the applied force on the reinforcements will change with time. If the change is significant, then an iterative approach to the determination of the deformations in the soil and the reinforcements must be undertaken.

3.2.3.3 SUMMARY

i) These mechanisms suggest that Constant Volume Angle of Friction [ϕ'_{cv}] of soils should be used for ULS analyses and a relationship between Mobilised Angle of Friction [ϕ'_{m}] and Lateral Tensile Strain [ε_t] should be used for SLS analyses. No Partial Factor requires to be applied to these soil parameters, McGown et al (1998).

ii) For geosynthetics, different strengths should be used for ULS and SLS analyses. It may be noted for ULS analysis, since a large strain equal to [ε_{SOG}] is required for Constant Volume Angle of Friction [ϕ'_{cv}] to develop, strength corresponding to a strain greater or equal to [ε_{SOG}] of geosynthetics should be used. Most of the geosynthetics exhibit rupture at a strain higher than that is required for the development of Constant Volume Angle of Friction [ϕ'_{cv}] in the soil. However, the Rupture Strains [ε_R] are always found to be widely scattered, Fig. 3.4, and therefore should be avoided in the design of GRSSs. The most appropriate choice of strength made for geosynthetics lie between the range of Performance Limit Strain [ε_P] and Instability Limit Strain [ε_I]. The Instability Limit Strain [ε_I] is defined as the strain where the onset of rupture of the geosynthetics initiates, Fig. 3.4. Most often the design strengths are obtained on the basis of a Performance Limit Strain [ε_P] which is less than the Instability Limit Strain [ε_I], Fig. 3.4. Even this Performance Limit Strain [ε_P] may be found to be high compared to what is required for the development of the Constant Volume Angle of Friction [ϕ'_{cv}] in the soil. Thus for most of the geosynthetics, the Performance Limit Strain is limited to 10% strain.

iii)For SLS analyses, as the soil and reinforcement strain together, the load in the reinforcement changes with time. Therefore, the strength of the reinforcement should be obtained from the Isochronous Stiffness using the mobilised tensile strain data.

At present, all the Actions are considered sustained in nature. Therefore, input parameters are not identified separately for Single-Stage Actions and Multi-Stage Actions.

CHAPTER 4

ACTIONS RELATED TO THE DESIGN STRENGTH OF GEOSYNTHETICS

4.1 INTRODUCTION

An *Action* is a force (load) applied to the structure or an imposed or constrained deformation caused by temperature changes, moisture variation or uneven settlement, Eurocode 1 (1996) and DIN 1055 (1976).

Actions for GRSSs should include all loads, forces and displacements related to any specific Limit State condition. Further the *Duration of Actions* needs to be considered, including changes in Actions resulting from changes in the properties of materials with time.

4.1.1 EUROCODE CLASSIFICATION

Different types of Actions have been identified in Eurocode 1 (1996) for structural engineering. These Actions have also been suggested in Eurocode 7 (1995) for geotechnical engineering.

4.1.1.1 DIRECT ACTIONS

Direct Actions can be classified in the following manner:

a) Actions varying with time:

Permanent Actions e.g. self-weight of structures, fittings, fixed equipment, road surfacing etc.

Variable Actions e.g. imposed loads, wind loads or snow loads.

Accidental Actions e.g. explosions or impacts from vehicles.

b) Actions which vary spatially:

Fixed Actions e.g. self-weight

Free Actions e.g. movable imposed loads, wind loads, snow loads

c) Actions related to a structural response:

Static Actions, which do not cause significant acceleration of the structure or structural member.

Dynamic Actions, which cause significant acceleration of the structure or structural member. Dynamic effects may be taken into account either by increasing the magnitude of the static Actions or by the introduction of an equivalent static Action.

4.1.1.2 INDIRECT ACTIONS

Indirect Actions can be classified as:

Permanent Actions e.g. settlement of support

Variable Actions e.g. temperature effect.

4.1.1.3 SUMMARY

It may be noted that the Actions imposed on the GRSSs are not wholly static, intermittent or dynamic. The Actions when they are applied alone are termed Single-Stage Actions. In most of the applications there will be a combination of any of the Single-Stage Actions, which are termed Multi-Stage Actions. Examples of Multi-Stage Actions are sustained loading plus traffic loading and sustained loading plus seismic loading.

However, to date GRSSs are designed under mainly static loads as very little is known about the behaviour under Dynamic Loading regimes, Müller-Rochholz (1994).

4.1.2 SUGGESTED CLASSIFICATION

Soils and geosynthetics are likely to exhibit different behaviours under Single-Stage Actions and Multi-Stage Actions. Therefore, the input parameters for soils and geosynthetics under these Actions require to be considered separately. Different design approaches consider these input parameters in different ways.

It is suggested that various combinations of the wide variety of Actions identified in Eurocode 7 (1995), may act upon GRSSs, Fig. 4.1. However, for design purposes it is recommended that these Actions or combinations of Actions may be resolved into three critical groups, Fig. 4.2.

These are Sustained Actions, Equivalent Sustained Actions and Sustained plus Short Term Actions. Appropriate material input parameters then require to be used in design to take account of these three critical groups of Actions.

Sustained Actions are Single-Stage Actions and can be represented for testing purposes with a sustained load over the critical duration of this particular type of Actions. The appropriate testing technique is creep testing as the long term material behaviour is obtained, viz. the visco-plastic components. Short term testing, viz. CRS testing does not represent the long term material behaviour, different material properties are tested, viz. the elastic material component. For further details see Chapter 7.

Equivalent Sustained Actions are Multi-Stage Actions that can be taken as an equivalent sustained load, viz. sustained loading plus traffic loading or snow loads. Khan (1999) and Müller-Rochholz (1994) dealt with sustained loading plus traffic loading and showed that an intermittent Action can be represented with a sustained load. Thus the appropriate testing technique is creep testing.

Sustained plus Short-Term Actions represent Actions, which are applied to the structures on one or more occasions and are of a short-term nature, in addition to sustained loads over the design life. To date there is no testing technique which can deal with these type of actions. Khan (1999) showed that the present approaches and design implications are mostly empirical.

Thus there is a need to investigate the influence of such Multi-Stage Loading regimes on geosynthetics and to observe their behaviour under such loadings. To find such a testing method was the objective of this study

4.2 The Strength of Geosynthetics in GRSSs Designs

In the design of GRSSs, two categories of strength are employed, viz. Reference and Design Strengths. Various testing techniques are used to obtain the Reference Strength [P_{Ref}]. The Design Strength [P_D] is obtained by dividing the Reference Strength with a Partial Factor [f_m].

4.2.1 Reference Strength

The Reference Strength [P_{Ref}] is obtained from different testing techniques, most frequently by CRS and creep testing. Various design codes and methods define Reference Strength [P_{Ref}] in different ways for Single-Stage and Multi-Stage Actions.

4.2.1.1 Single-Stage Actions

DIBt (1998) and AASHTO (1997) use the maximum load at rupture of the Ex-works materials under CRS tensile testing as the basis of defining the Reference Strength of geosynthetics. The Reference Strength [P_{Ref}], to avoid long-term creep rupture, is then obtained by dividing the CRS rupture strength by a Reduction Factor, Fig. 4.3 (a). The DIBt and the AASHTO design methods specify 33% per minute and 10% per minute strain rates respectively, for the CRS tests employed. Kabir (1984) and Yeo (1985) have shown that the CRS test can give different strengths at different test strain rates and temperatures. Therefore, even for a particular type of geosynthetic the same Reduction Factor may not be applicable to all CRS data in order to obtain the long-term creep rupture strength, i.e. the Reduction Factor is dependent on the strain rate used in the CRS test.

Troost and Ploeg (1990), BS8006 (1995) and Jewell (1996) defined the Reference Strength [P_{Ref}] as the load to cause creep rupture of 'Ex-works' specimens at the end of design life [t_{dl}], Fig. 4.3 (b). However, as shown in Chapter 3 many geosynthetics exhibit a wide range of scatter of creep rupture strains for different sustained [creep] load levels. Hence, the Reference Strength [P_{Ref}], defined on the basis of load at creep rupture for a specific design life time [t_{dl}], can be very difficult to select with any certainty related to the strain level developed at creep rupture, McGown et al (1998).

Some design methods define the Reference Strength [P_{Ref}] as the load obtained from the Isochronous Load-Strain curves, corresponding to a Performance Limit Strain [ε_P], for example the TBW Method (1998) and the HA 68/94 Design Method (1997). This value is always less than the strength at creep rupture, Fig. 4.4. Adoption of the Performance Limit Strain [ε_P] to define the Reference Strength [P_{Ref}] for Ultimate Limit State design can be seen to be a most appropriate and conservative choice of strength value.

Thus, the Reference Strengths for Single-Stage Actions have been defined in two different ways, i.e. on the basis of long-term creep rupture strength or on the basis of long-term creep at a limiting strain.

4.2.1.2 MULTI-STAGE ACTIONS

One example of Multi-Stage Actions is sustained loading plus traffic loading that may be applied to GRSSs in reinforced roads. The traffic loads are transient in nature. In most of the design codes, the traffic loads are, however, calculated as the wheel load divided by the contact area, for example BS 5400 Part 2 (1978), and considered as a uniform surcharge load over the whole design life time, DIBt (1998), Geoguide1 (1993), BS 8006 (1995) and AASHTO (1997). This means that these codes allow the Reference Strengths for Single-Stage Actions to be used in designs for sustained loading plus traffic loading. The external and internal stability analyses for the sustained loading plus traffic loading are carried out for the critical loading combination as outlined in the respective codes.

The consideration of wheel contact pressure as a sustained load over the whole design lifetime of the structure does not consider the transient nature of the load which may be beneficial to the elasto-visco-plastic behaviour of geosynthetics.

Another example of Multi-Stage Action is sustained loading plus short-term seismic loading. The development of designs for GRSSs involving earthquake forces is presently empirical. Fukuda et al (1994) reported that until 1993, the GRSSs were designed for earthquake on the basis of the procedure for design under ordinary static conditions, as given by Jewell et al (1984). In this procedure the long-term creep rupture strength of geosynthetics was used as Reference Strength. The structures, so designed, were found to maintain stability during the Loma Prieta Earthquake in 1989 having a magnitude of 7.1, Collin et al (1992), and Kushiro Offshore Earthquake in 1993 having a magnitude of 7.8, Fukuda et al (1994). These data indicate that the geosynthetics were actually capable of taking higher loads applied rapidly, than the long-term creep strength. Later, it was suggested by Fukuda et al (1994), AASHTO (1994) and Jones (1996) that the design strength of geosynthetic for static condition should be increased by 1.5 times when designing for sustained loading plus short term earthquake loading. In the recent codes/methods, as more confidence is gained from the performances of GRSSs during recent earthquakes, factored short-term CRS strengths of geosynthetics are suggested for use in designs against sustained loading plus short term earthquake loading, AASHTO (1997), NCMA (1997) and DIBt (1998). However, it should be noted that the considerations of higher strengths of geosynthetics, from 1994 to 1998, were all empirical.

Higher strengths of geosynthetics were suggested, presumably, on the basis that the earthquake loadings will be very transitory, but they have not been technically justified in detail. It is unlikely that the same amount of high strength of geosynthetics can be used over the whole design lifetime of the structure, irrespective of the time of occurrence of an earthquake. If this is true, current practice of designing against earthquakes may be considered unsafe.

4.2.1.3 Partial Factors

The strengths of the geosynthetics for ULS and SLS analyses require modification by applying Partial Factors [f_m]. Four Partial Factors of major concern have been identified for geosynthetics, Voskamp and Risseeuw

(1987), Jewell and Greenwood (1988), Greenwood and Jewell (1989) and Troost and Ploeg (1990). These are:

(a) The Damage Factor; to allow for mechanical damage during construction.

(b) The Environmental Factor; to allow for the chemical environment and microbiological exposure in the ground.

(c) The Material Factor; to allow for the uncertainty inherent in the extrapolation of test data, and

(d) The Overall Factor; to allow for the properties of materials not meeting the manufacturer's specification.

Conventionally, the Design Strengths [P_D] for the geosynthetics is determined by applying Partial Factors [f_m] to the Reference Strengths [P_{Ref}].

$$[P_D] = \frac{[P_{REF}]}{[f_m]} \qquad (2.17)$$

BS 8006 (1995), AASHTO (1997), DIBt (1998) and TBW (1998) suggest that the Partial Factors should be determined by comparing the material strengths of the geosynthetics "before" and "after" an event, as obtained from CRS tests.

Specified by the manufacturer of Geogrid B is the Damage Factor for sandfill is equal to 1.1, Environmental Degradation is factored with 1.0 and the Material Factor and the Overall Factors are both given with 1.0.

It should be noted that no Partial Factors for the combined effects of Multi-Stage Actions are given.

4.2.1.4 Soils - Geosynthetic Interaction

Interaction between soil and geosynthetic reinforcement is important for internal stability analyses of GRSSs. The mechanisms involved in the interaction and the theoretical expressions for calculating the Coefficient of Direct Sliding and the Coefficient of Bond were set out by Jewell (1996).

Clearly, these two coefficients are governed by the geometry of the reinforcements and the soil properties. However, the uncertainties involved in the determination of these coefficients have been recently identified by Dyer (1985), Palmeira and Milligan (1989) and Milligan (1990). Later, McGown et al (1998) suggested that different values of the Partial Factors should be applied to the Coefficient of Direct Sliding and Coefficient of Bond, to cater for the uncertainties involved. To date, the values for these Partial Factors have not been decided and this is beyond the scope of the present study.

4.2.1.5 SUMMARY

The Reference Strength for Geogrid B is according to BS 8006 (1995) and DIBt (1998) 33.33 kN/m and according to TBW (1998) 30.50 kN/m, both at an operational temperature of 20°C and a design life of 120 years. It may be appreciated that these strengths were all obtained under Single-Stage Loading regimes.

4.2.2 DESIGN STRENGTH

The Design Strength [P_D], suggested by DIBt (1998), BS 8006 (1995) and TBW (1998), for Geogrid B at 20°C for a design life of 120 years are shown in Table 4.1. Properties of Geogrid B are given in Table 5.1.

Although both DIBt (1998) and BS 8006 (1995) use creep rupture strength as Reference Strength, the differences in the Design Strengths are due to an Overall Factor of 1.75 used in DIBt (1998) and a Material Factor of 1.2 used in BS 8006 (1995).

4.2.3 SUMMARY

The design code review indicated that considerations of higher strengths for geosynthetics, in the time from 1994 to 1998, were all empirical. Although the higher strengths were suggested on the basis that seismic loadings will be very transitory, but they have not been technically justified in details.

Besides the review showed that the option for seismic analysis is not included in some national design codes, e.g. BS 8006 (1995).

The sustained load of 25 kN/m used in the Combined Sustained-Impulse Loading test was selected according to the highest Design Strength used in the reviewed design codes at 20°C of operational temperature.

4.3 Review Implications

The review of the present design methodology shows that to date there are no appropriate testing protocols or standards for any Multi-Stage loadings. The objective of the present research was to study the effects of short-term loadings on GRSSs, viz. seismic loadings, explosions, and accidents.

The impact on the human society due to major disastrous earthquakes and the availability of data were reasons to investigate seismic loading regimes.

Bommer et al (1999) showed that a seismic event is of a cyclic nature; hence CRS testing is not necessarily an appropriate test methodology. Therefore, the short-term loading in the Combined Sustained-Impulse Loading test sequence was selected in a way to consider the cyclic nature of an earthquake. The problem was to find such a Loading Regime which could simulate a seismic event appropriately.

4.3.1 Seismic Loading Regimes

The duration of seismic events is important to obtain realistic test results. If during an earthquake a structure is deformed beyond its elastic limit, then the amount of permanent deformation will depend on how long the shaking is then sustained.

This shaking is defined by Bommer et al (1999) as the *Earthquake Strong Motion*. Therefore, it is important to ensure that the duration of the Strong Motion is consistent within the design scenario. The Effective Duration includes 91% of the total recorded energy released by the Strong Motion and also considers the rest period between sub-events in multiple events of this type.

Khan (1999) showed the creep deformation depends on these rest periods. A specimen is likely to show higher deformation when subjected to Cyclic Loading without rest periods.

The dependence of Strong Motion duration on distance is complex, e.g. dependent on the geology. On one hand, as the energy wave separates due to different propagation velocities and scattering, the duration will increase with increasing distance. On the other hand, energy will decrease with distance due to attenuation of the motion.

The Effective Durations for recent major earthquakes at a distance of less than 10 km from Epicentre for soil and rock sites are shown in Figure 4.5 (a) and (b). Effective Durations for recent major earthquakes versus distance from Epicentre, for rock and soil sites, are shown in Fig. 4.6

In this connection Fujii et al (1996) and Frankenberger et al (1996) reported that the Kushiro Offshore Earthquake in 1993 and the Northridge Earthquake in 1994 also showed very similar durations.

Nevertheless, the review also indicates that there exist earthquakes, which exceed the duration of 60 seconds. For most cases, however, the duration of the Earthquake Strong Motion is below 20 seconds, therefore the duration of the short-term loading was chosen to be 20 seconds.

However, to simulate such very dynamic loading regimes presents major difficulties. No standard was found which defines an appropriate dynamic short-term loading regime.

4.3.2 Implications from Cyclic Loading

A seismic event can be described as a sequence of repeated cycles, which are additionally applied over a sustained load. This situation consists of a dynamic and static loading part.

Müller-Rochholz (1994) studied the effects of dynamic loading. The tests showed that a dynamic loading has no significant influence on the creep performance. In fact if peak load is constant, the dynamically loaded specimen shows less creep deformation. This kind of load history is close to the deformation of mean level of load. Results from Combined Sustained-Cyclic Load testing confirmed these results, Khan (1999).

Therefore, it is suggested that dynamic loading can be conservatively simulated by an appropriate sustained loading regime.

4.4 Simulation of Sustained plus Seismic Loading

Specimen testing under dynamic conditions is extremely problematic therefore a solution involving a sustained loading needs to be achieved. This load is then called the Multi-Stage Sustained Load Equivalent, Fig. 4.7.

Müller-Rochholz (1994) and Khan (1999) showed that the creep deformation corresponds to the mean load. Thus a Multi-Stage Sustained Load Equivalent at peak load is therefore more critical than any cyclic loading condition.

Chapter 5

The Isochronous Strain Energy Approach

5.1 Introduction

Geosynthetics are known to show time and temperature dependent load-strain behaviour. Single-Stage Loading tests, viz. CRS, creep and cyclic load tests are presently employed to determine these load-strain behaviours. The strength data obtained from these testing methodologies differ from each other. For example, rupture strengths or strengths at a limiting strain from CRS tests, at different rates of strain, and creep tests are not same. Geosynthetics show different properties, viz. elastic, visco-plastic and plastic, and these components respond differently in these tests. Therefore, an approach requires to be developed that will allow the data obtained from these various Single-Stage tests to be correlated and compared. It is then necessary to identify the most appropriate input parameters for geosynthetics, i.e. Reference Strength, Partial Factors and Design Strength, for GRSSs applications.

To compare and correlate data the Isochronous Strain Energy [ISE] approach has been developed by Khan (1999). Using this all strength data are obtained at a constant temperature, i.e. isothermal conditions. Khan (1999) defined ISE as the Strain Energy absorbed by a geosynthetic at a specific time. It includes the "Immediately Recoverable" and "Locked-in" Strain Energy components and their significance needs to be identified.

In this Chapter, the ISE approach developed by Khan (1999) is presented and the isothermal test data obtained from different testing methodologies are compared and correlated within ISE–Time plots. The test data at different temperatures are presented in the ISE-Time-Temperature plots, to obtain the failure surfaces at different limiting strains. The ISE components are then identified and test data are plotted and interpreted in terms of these components.

5.2 THE NATURE OF GEOSYNTHETIC MATERIALS

Geosynthetics are generally manufactured from polymeric materials mixed with additives and produced in a variety of plastics.

The two most important plastics are:

1) *Thermoplastic materials*: Intermolecular Van-der-Waals forces dominates their behaviour: When these materials are heated they become soft and flexible and eventually, at high temperatures, exhibit a viscous melt. When they are allowed to cool, they solidify again. This process of softening by heating and solidifying on cooling may be repeated more or less indefinitely.

2) *Thermosetting materials*: Cross-linking of the long chain-like molecules dominates their behaviour. This takes place during moulding under heat and pressure. His material is rigid when cooled and does not soften on heating.

The vast majority of the geosynthetics are manufactured using thermoplastic materials. The thermoplastic sheets, strands, fibres, etc. comprising the geosynthetics can exhibit a range of mechanical behaviours, from brittle-elastic at low temperatures, through plastic, to visco-elastic or leathery, to rubbery and finally to viscous at high temperature. Their behaviours are determined by the nature of the constituent materials, by the processing to which they are subjected during manufacturing and the temperature at which they operate.

Much more important under normal operational temperatures is the fact that the load-strain-time-temperature behaviour is dependent upon their micro and macro-structures. The principal micro-structural features that characterise all polymers are the molecular size, the basic polymer network structure, the chain stiffness and rigidity.

Very often there are no network structures or cross-linking within engineering thermoplastics, however, small amounts can occur as a result of particular polymer processing operations. During this processing the molecular state of the material can be manipulated from the amorphous to semi-crystalline and finally to crystalline state. The degree of crystallinity within the thermoplastic components of geosynthetics is very significant. Most of the thermoplastic materials and processes used to produce the various geosynthetics result in micro-structures which are partly amorphous and partly crystalline. As a consequence of this, the mechanical behaviour of the components is often elastic initially then becomes visco-plastic. This is a result of the long molecular chains unfolding, (if chain-folded), or drawing out of the amorphous tangle, (if glassy), together with straightening and aligning. When drawing is complete, the stress-strain curve rises steeply to complete rupture.

If subjected to a constant load, i.e. sustained load (creep) testing, the geosynthetics exhibit an instantaneous elastic strain followed by primary creep, secondary creep and tertiary creep, Fig. 5.1 Primary creep is associated with fast rate of creep and occurs in a very short period of time. Secondary creep occurs at a very slow rate and requires load to be applied over a longer period of time. Tertiary creep occurs at higher load levels where non-liner visco-elasticity becomes predominant. The material rupture under tertiary creep conditions is influenced by various factors and therefore difficult to predict.

It should be noted that the strain in a thermoplastic component can be represented in terms of its degree of crystallinity. Due to straightening and aligning of the polymer molecules the degree of crystallinity increases, i.e. under isothermal conditions is a 2 percent strained specimen less crystalline than a 10 percent strained sample.

Another important aspect of the effect of micro-structure is the basic structure of the polymer itself. The structure defines ability of molecular chains to slip past one another. Polyester have "knots" in their structure and therefore they show a relative inability of the molecular chains to slip past one another.

Thus they are little time dependent and show minor creep at low load levels. On the contrary the smoother and straighter molecules of polypropylenes can slip past one another more easily, which is why polypropylenes can exhibit a significant creep even at low load levels.

The effect of an applied load on a thermoplastic component can then be interpreted in terms of the absorbed Strain Energy. For example, if it is required to reach 5 percent strain in 100 hours then the absorbed Strain Energy will be much smaller than if the same strain has to be reached in 10 seconds. Thus the Strain Energy required to develop a particular limiting strain in a thermoplastic component is time (strain rate) dependent.

In the final stage of manufacture of geosynthetics the various thermoplastic components are linked and arranged geometrically to form the macro-structure. Many different generic types of geosynthetics are then produced, including wovens, needle punched and melt bonded non-wovens, nets, grids and linear composites as described in Chapter 2. These various macro-structures have a further highly significant influence on the time dependent load-strain behaviour of geosynthetics. However, the temperature dependency of their behaviour may not be greatly influenced by their macro-structures, as this is a micro-structure issue.

As a consequence of the range of polymer types, additives, processing and final manufacturing techniques used to produce geosynthetics, an extensive range of micro and macro structures are to be found in the range of geosynthetic product types now in use. Hence when comparing two product types which have the same basic polymeric composition, they are unlikely to have the same mechanical behaviour. Equally so, they are unlikely to respond in the same manner to alternative forms of testing regimes. Consequently, good correlations between data from different testing regimes, involving monotonic, cyclic and sustained loading, are only likely to occur within a specific product type.

5.3 LOAD-STRAIN-TIME BEHAVIOUR UNDER ISOTHERMAL CONDITIONS

Geosynthetics exhibit elasto-visco-plastic behaviour. This means that their mechanical behaviour is in part similar to that of elastic solid, in part similar to that of a viscous liquid and in part similar to that of a plastic, with all these parts being temperature dependent and acting at the same time. Therefore, when subjected to an externally applied load as shown in Fig. 5.2 (a), they respond by exhibiting a combination of elastic displacement, viscous flow and irrecoverable plastic deformation, Fig. 5.2 (b). Various rheological models have been employed to characterise the elasto-visco-plastic behaviour of the geosynthetics at strain levels less than the strains that induce rupture.

The simplest rheological model that represents the response of geosynthetics to loading and unloading is the four-element model, Fig. 5.3. The spring [S1] in this model represents the elastic displacements, the linked spring and dashpot system [S2+D2] represents the viscous flow and the dashpot [D3] represents the irrecoverable plastic deformations.

Rupture of geosynthetics may occur in either a brittle or ductile mode, depending on the micro and macro structures, operational temperature, etc. For most of the geosynthetics this rupture domain is difficult to identify in a consistent manner, Kabir (1984) and Yeo (1985). Only if a geosynthetic exhibits a clearly defined rupture domain, should a rupture criterion be used. For the majority of geosynthetics a limiting strain criterion is the more appropriate criterion. It should be noted that the limiting strain criteria for different applications are being set within some of the latest design codes and methods, for example BS8006 (1995) and the TBW (1998).

5.4 STRENGTH TESTING AND TEST DATA PLOTS

A wide range of strength Single-Stage Loading tests for geosynthetics have been adopted or specially developed over the last 25 years, for example CRS, creep, stress relaxation and cyclic tests. These tests utilise different specimen sizes and shapes, loading methodologies, rates of load application and test temperatures for measuring the mechanical behaviour of geosynthetics, McGown et al (1981). To represent the macro-structure of the product the test specimens must be of sufficient size and shape, Kabir (1984) and Yeo (1985). However, even when this condition is satisfied, each test may investigate the response of different features of the micro and macro-structures of geosynthetics. Indeed changing a single test condition may result in a change in the load-strain response. For example, for wide width specimens tested at a given test temperature a change in the rate of strain in monotonic tests has a significant effect on the measured properties of geosynthetics as has changing the test temperature whilst using the same rate of strain has a significant effect, Kabir (1984) and Yeo (1985).

The behaviour of geosynthetics under sustained load [creep] testing has been widely studied and the influence of time and temperature clearly identified, Kabir (1984), McGown et al (1984a), Yeo (1985) and Wrigley et al (1999). To a lesser extent, the behaviour of geosynthetics under sustained strain, stress relaxation testing has been studied and the influence of time and temperature similarly identified, Soong et al (1994). Most recently cyclic loading has been systematically investigated by Müller-Rochholz et al (1994). Significantly, all these test data produce different load-strain-time-temperature responses even for a particular geosynthetic product type.

Each of these test methodologies involves different forms of data presentation and for some particular tests several forms of data presentation can be used. The most widely applicable approach to geosynthetics is to present the data as Isochronous Load-Strain curves at a constant temperature. Commonly these are produced from sustained load [creep] test data. However, they can also be produced from most test procedures, for example CRS test, Fig. 5.4. Recently it has been shown by Müller-Rochholz et al (1994) that the data obtained from cyclic load test can be represented by an equivalent sustained load test data, Fig. 5.5. This allows Isochronous Load-Strain curves to be obtained from cyclic load test data as well.

It should be noted that the area under the Isochronous Load-Strain curves represents the absorbed Strain Energy at the specific time and limiting strain and may be termed the *Isochronous Strain Energy.*

5.5 THE ISOCHRONOUS STRAIN ENERGY [ISE] APPROACH

Khan (1999) stated that for Single-Stage Loadings, i.e. CRS or creep, under isothermal conditions, 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].

Thus from Isochronous Load-Strain plots, the areas under the curves represent the Isochronous Strain Energies at different times, Fig. 5.6. The units of Isochronous Strain Energy [ISE] for geosynthetics are:

Force per unit width times unit strain = [kN/m]×[m/m] = [kN/m]

To avoid confusion with load per unit width and classical definitions of Strain Energy, consideration needs to be given to the introduction of a new term for the unit of ISE.

It should be noted that for any Single-Stage Loading regime the ISE, so defined, is time and strain level dependent at isothermal conditions. For example, if a sustained load [P_1] is applied to a geosynthetic, the ISE absorbed will increase with time due to creep, Fig. 5.7. Similarly, if a constant strain is maintained, ISE will decrease with time due to stress relaxation, Fig. 5.8. If, however, both load and strain increase gradually, as would happen in a CRS test, ISE will increase with time, Fig. 5.9.

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. The amount of ISE to develop a limiting strain at that temperature [T] for a particular Single-Stage Loading regime is termed the ISE Capacity [C]$_t$ of the geosynthetic at the specified time, Fig. 5.10 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. 5.11

In addition, the data from different temperatures may be plotted on an ISE-Time–Temperature plot, allowing the influence of temperature dependence to be identified, Fig. 5.12.

5.6 Components of ISE Identified by Using Rheological Models

The load-strain-time behaviour of a geosynthetic in terms of the absorbed ISE $[A]_t$ is shown in Fig. 5.13. Upon application of load $[P_1]$ at time $[t_0]$, there will be an absorbed ISE $[A]_{t0}$ (Point 1) within the geosynthetic. Thereafter, over a period of time between $[t_0]$ and $[t_1]$ more Strain Energy will be absorbed by the geosynthetic, i.e. the absorbed ISE will increase to $[A]_{t1}$, (Point 2). At time $[t_1]$ when the load is removed, a part of absorbed ISE $[A]_{t1}$ will be recovered and this is termed the Recoverable ISE $[R]_{t1}$, (Point 2 to 3). At this point of time, i.e. time $[t_1]$, the absorbed Strain Energy remaining in the geosynthetic is termed the Locked-in ISE $[L]_{t1}$. If no further load is applied to the geosynthetic, then with time (between t_1 and t_2) part of this Locked-in ISE $[L]_{t1}$ will be recovered due to viscous rebound. However, part of this Locked-in ISE will never be recovered and this irrecoverable Locked-in ISE at time $[t_2]$ is denoted by $[L]_{t2}$.

Khan (1999) suggested that this indicates that ISE comprises two components, which are:

- "Immediately Recoverable" ISE $[R]_t$
- "Locked-in" ISE $[L]_t$

These components are likely to vary with time for any limiting strain condition as shown in Fig. 5.14.

5.7 Determination the ISE Capacities

Five geosynthetics were tested using different test methodologies by Kabir (1984) and Yeo (1985), see Table 5.1. Khan (1999) used these data to construct the ISE Capacities $[C]_t$ of these five products.

For example, CRS tests were carried out on uniaxial geogrid A at different rates of strain and at a constant temperature 20°C, Fig. 5.15 (a), and the Isochronous Load Strain curves drawn from the test data are shown in Fig. 5.15 (b). Sustained load [creep] tests were carried out on the same geosynthetic at different load levels and the test data were plotted as Isochronous Load-Strain curves, Fig. 5.16 (a) and (b). The areas under these isochronous curves were calculated for 2, 5 and 10 per cent strains, and plotted in an ISE – log.Time plot, as shown in Fig. 5.17 (a). These data were re-plotted to a log-log scale and the best-fit curves were drawn through the data, Fig. 5.17 (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, which for the log-log plot are straight lines at different strain levels. The best-fit curves at each strain level represent the ISE Capacity $[C]_t$ for this material.

Figure 5.17 (c) shows that the values ISE Capacity $[C]_t$ for a particular limiting strain and time are almost equal for CRS and creep tests, within the range of test data. This means that the loads required for reaching a particular limiting strain at a specified time from CRS and creep tests are almost equal.

Strictly speaking, this may not always be true. Consider a CRS test where a Limiting Strain $[\varepsilon]_L$ is reached in 1 hour. Let the Load corresponding to this Limiting Strain $[\varepsilon]_L$ be $[P_1]$. It should be appreciated that in CRS test load taken by the geosynthetic increases with time and the duration of lower level of load is higher than the higher level of load, Fig. 5.18 (a). This means that the Load $[P_1]$ corresponding to Limiting Strain $[\varepsilon]_L$ will be acting on the geosynthetic for an instant of time only, but loads smaller than $[P_1]$ will act for a longer period of time, with some of them acting almost over the whole duration of test. On the contrary, load in the creep test is applied over the whole duration of testing, Fig. 5.18 (b). Therefore, strictly speaking, the strain due to a sustained load equal to $[P_1]$ applied over a period of 1 hour in a creep tests will be higher than the Limiting Strain $[\varepsilon]_L$ in the CRS test. In other words, this means that for reaching a particular Limiting Strain $[\varepsilon]_L$ at a specific time, the load required in CRS tests will be higher than in creep tests, Fig. 5.19 (a). Therefore, theoretically, the ISE Capacity $[C]_t$ for reaching a particular limiting strain at a specific time in CRS tests should always be higher than in creep tests, Fig. 5.19 (b).

Further testing confirmed that CRS and creep test data for different geosynthetic product types may be compared and correlated within the ISE-Time plots. The tests data show that the ISE Capacity $[C]_t$ obtained from CRS tests and creep tests are scattered and that they do not show any general trend. This may be attributed to the accuracy of measurements, test conditions and material variability. Therefore, for all practical purposes, the ISE Capacity $[C]_t$ represented by the best-fit curves can be used to compare and correlate the data obtained from CRS and creep tests, within the range of test data. For longer period of time, it is impractical to carry out CRS tests and, therefore, data should be obtained from creep tests. For a biaxial geogrid, a woven and two non-woven geotextiles were similarly CRS and creep test data obtained and analysed using the ISE approach, see Fig. 5.20 to 23.

As a part of this study data were obtained from CRS and creep testings for uniaxial geogrid B. Cyclic testing was conducted on this geosynthetic and analysed by Müller-Rochholz et al (1994). The Isochronous Load-Strain data from the different testing regimes were used to calculate the Isochronous Strain Energies and these values were plotted in the ISE-Time plot for the limiting strain conditions of 2, 5 and 10 per cent, Fig. 5.24. This figure confirms that the data obtained from cyclic loading may be plotted together with CRS and creep test data and that this also lies close to the best best-fit curve, if interpreted in the manner proposed by Müller-Rochholz et al (1994).

Figure 5.17 (c) and Figs. 5.20 to 5.24 show that the ISE approach allows comparison and correlation of the data obtained from CRS and creep tests within the ISE – Time plot, using best-fit curves at different strain levels. These plots show that different amounts of ISE are required in order to reach the same limiting strain in the short term compared to the long term. This indicates that it will be erroneous to apply the properties obtained from short-term CRS tests and apply it to long-term applications.

Further, the plots show that amount of ISE Capacity $[C]_t$, at different limiting strains and times, is dependent on the polymeric composition, micro and macro structures of geosynthetics. Therefore, as suggested by Khan (1999) the ISE Capacity $[C]_t$ can be seen to represent an *intrinsic* property of geosynthetics under Single-Stage Loading.

5.8 DETERMINATION OF ISE COMPONENTS

5.8.1 TESTS AND CALCULATION PROCEDURE

Calculation of ISE components requires derivation of Isochronous Load-Recoverable Strain and Isochronous Load-Locked-in Strain curves. In doing so, Isochronous Load-Total Strain and Isochronous Load-Recoverable Strain curves are first developed from loading/unloading tests. The test procedure for loading test is similar to conventional creep test and requires therefore no further consideration. However, the test procedure for the recovery test was developed by Yeo (1985) at the University of Strathclyde and is described in Appendix A.

Since at any time [t] the Total Strain $[\varepsilon_T]$ is equal to the summation of Recoverable Strain $[\varepsilon_R]$ and Locked-in Strain $[\varepsilon_L]$, the Isochronous Load-Locked-in Strain curves can be obtained by using this equation:

$$[\varepsilon_L] = [\varepsilon_T] - [\varepsilon_R] \qquad (5.1)$$

The 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.

5.8.2 ISE COMPONENTS FROM TEST DATA

The above mentioned procedure has been applied by Khan (1999) to the analyses of the test results for uniaxial geogrid B to obtain the isochronous load-strain curves obtained from sustained load [creep] test data, Fig. 5.25.

This figure shows that the Isochronous Load-Recoverable Strain relationship for this geosynthetic is essentially time independent.

Figure 5.26 shows the variation of each of the ISE components with time [t] at 10% and 5% limiting strains. It may also be noted that the summation of the Recoverable ISE $[R]_t$ and Locked-in ISE $[L]_t$ is equal to the ISE Capacity $[C]_t$ for any particular limiting strain, Fig. 5.27. Further, this figure shows that if a limiting strain (say 10%) is reached in 0.1 hours, the amount of 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 is likely to be higher than the Recoverable ISE. This means that totally different components of ISE are measured in a short-term test than in a long-term test.

The combinations of ISE components, i.e. Recoverable and Locked-in ISE is unique at any time [t]. These combinations of Recoverable and Locked-in ISE at a particular limiting strain establish therefore unique boundary for a particular geosynthetic, i.e. the material cannot reach under a Single-Stage Loading a particular limiting strain in a specific time without a certain combination of Recoverable and Locked-in ISE.

5.8.3 EXTRAPOLATION

GRSSs are often designed for an operational period of time which is greater than the duration of long-term creep tests. It is therefore necessary to extrapolate creep test data to the required design lifetime $[t_{dl}]$ of the structure.

Khan (1999) suggested that the extrapolation of ISE capacity $[C]_t$ and Recoverable ISE $[R]_t$ can be obtained by determining the best fit curves for these two parameters, Fig. 5.28 (a). The difference between $[C]_t$ and $[R]_t$ then gives the extrapolated value of Locked-in ISE $[L]_t$ for the required design life time. This allows $[R]_t$ and $[L]_t$ values to be plotted beyond the test duration, Fig. 5.28 (b).

The procedure is illustrated in Fig. 5.29 for uniaxial geogrid B up to 2 log cycles, i.e. up to 1,000,000 hours. It may be noted that this extrapolation is confirmed by the data obtained from the rheological models given by Kabir (1984) and Yeo (1985).

5.9 Identification of Geosynthetic Product Types for Different Applications

It was stated previously that the ISE Capacity $[C]_t$ for a geosynthetic is unique for any Single-Stage Loading regime and limiting strain, therefore, the ISE Capacity $[C]_t$ for a limiting strain can be considered to be an intrinsic property of a geosynthetic.

Two important indicators of this behaviour are the short-term ISE Capacity and the rate of decay of the ISE Capacity with time under sustained loading. Khan (1999) defined these as the *Intercept* value [I] and the *Slope* value [S], taken from the ISE plots, as indicated in Fig. 5.30.

Any convenient short period of time can be taken for the Intercept value. Khan (1999) suggested that the Equivalent Instantaneous Loading Time $[t_0]$ as suggested by Kabir (1984) and Yeo (1985), Fig. 5.31, may be used to define the time for Intercept value. The Intercept value at time $[t_0]$, i.e. the ISE capacity at time $[t_0]$, is taken to be equal to the Immediately Recoverable ISE at that time and is denoted by $[R]_{t0}$. Such an Intercept value will be controlled by all the compositional, processing and manufacturing factors identified in earlier sections, including the polymer and additive comprising the plastic, the processing applied and manufacturing technology employed to develop the micro and macro-structural features of the product. The Slope value will be principally controlled by the micro and macro-structural changes that develop on sustained loading, although any possible chemical changes that might develop with time will also influence this parameter.

It is suggested that different product types will exhibit different combinations of Intercept and Slope values as indicated in Fig. 5.32, Khan (1999). This plot may be used to identify the geosynthetics that are most appropriate for use in particular applications. For example, Type I may be most appropriate to walls and steep slopes on competent soil whereas the Type II may be appropriate to embankments on soft soil. Similarly, Types III and IV may be more applicable to road sub-base separation and filtration applications.

To illustrate this, the Intercept value [i] was obtained from the test data and Slope value $[S]_t$ data were abstracted from ISE-time plots presented previously. The results were plotted for different limiting strain conditions, Fig. 5.33.

5.10 Effects of Environmental Degradation and Construction Damage on the ISE components

The ISE components were shown to be the "Immediately Recoverable" ISE $[R]_t$ and the "Locked-in" ISE $[L]_t$. The ISE components are likely to be affected differentially by construction damage and environmental degradation. For a particular geosynthetic the change of the Immediately Recoverable ISE $[R]_t$ may be more than on the Locked-in ISE $[L]_t$ and vice-versa. It may be appreciated that the changes on each of the components are both time and strain level dependent.

Present design methods/codes apply Partial Factors obtained from short-term CRS test to the long-term Reference Strength for obtaining the Design Strength. This is based on the assumption that an event affects the Immediately Recoverable and Locked-in ISE components in the same manner during the whole lifetime. This is not likely to be the case.

The procedure for the calculation of the ISE components was set out earlier. Similar procedures can be used to calculate the Immediately Recoverable ISE $[R]_t$ and the Locked-in ISE $[L]_t$ 'before' and 'after' an event. This allows the Immediately Recoverable ISE $[R]_t$-Time and Locked-in ISE $[L]_t$-Time plots to be developed 'before' and 'after' an event. The effects on the Immediately Recoverable ISE $[R]_t$ and the Locked-in ISE $[L]_t$ can be determined by comparing the amount of change of these components to their original value. Khan (1999) suggested that these changes can be defined as the ISE Component Change.

Thus,

$$\text{ISE Component Change (\%)} = \frac{\text{ISE component}_{(after)} - \text{ISE component}_{(before)}}{\text{ISE component}_{(before)}} \times 100$$

Partial Factors, determined by different testing methodologies, vary in terms of their values and nature and it is not possible to integrate the Partial Factors obtained from these different testing methodologies.

The ISE approach allows a uniform means of comparing and correlating the test data obtained from different testing methodologies. The ISE approach identifies an intrinsic material property, the ISE Capacity $[C]_t$, of the geosynthetics which takes account of any alterations in their polymeric composition, micro or macro structures as a result of construction damage or subsequent environmental degradation. Therefore, this approach can be used to eliminate the difficulties of interpreting the Partial Factors obtained from different testing methodologies.

In the method suggested by Khan (1999), the Partial Factors for the geosynthetics can be appropriately determined by comparing the ISE Capacity $[C]_t$ 'before' and 'after' an event, e.g. before and after constructional damage and environmental degradation.

The Partial Factors obtained in this way are both time and strain-level dependent. Therefore, for Ultimate Limit State and Serviceability Limit State analyses different Partial Factors need to be used over the design lifetime of the GRSSs.

The changes of the ISE components due to an event may vary significantly with time. It is emphasised that the short-term CRS tests may measure totally different changes of ISE components than the long-term sustained load creep tests and for some geosynthetics this can be very significant. Therefore, it is suggested that the Partial Factors obtained from CRS tests may not applicable to determine the long-term Design Strength of the geosynthetics.

The current test protocols of determining Partial Factors do not truly represent the actual field conditions and therefore the data obtained from these tests are not appropriate for use in designs. Hence, test protocols simulating actual field conditions require to be developed.

5.11 SUMMARY

Different testing methodologies are employed to determine the mechanical behaviour of the geosynthetics. The strength obtained from each test method is different for the same geosynthetic and to date, it has not been possible to compare and correlate the data obtained from these testing methodologies using a single approach.

The ISE approach introduced by Khan (1999) allows a means of representing the isothermal data obtained from different testing methodologies within one plot. Thus, comparisons and correlations of these data can be made at different limiting strains. In addition, Khan (1999) has shown that the ISE approach allows identification of limiting strain surfaces to represent the load-strain-time-temperature behaviour of geosynthetics. These surfaces are useful for analysing whether or not a geosynthetic has reached a limiting criterion. Further, the ISE approach also allows identification of the product types most appropriate to various applications.

Khan (1999) has shown that the ISE approach for any Single-Stage Loading regime identifies the ISE Capacity $[C]_t$ at any limiting strain and that this represents an Intrinsic Property of a geosynthetic. However it changes with time in response to alterations of the micro or macro structures of the geosynthetic. On this basis, Khan (1999) suggests that the ISE approach provides a means of measuring changes in mechanical behaviour of geosynthetics due to changes in the micro or macro structures of geosynthetics induced by construction damage or environmental degradation.

Further, Khan (1999) identified the components of the ISE Capacity $[C]_t$ as the Immediately Recoverable ISE $[R]_t$ and the Locked-in ISE $[L]_t$. These ISE components indicate that the geosynthetics behave differently under different Single-Stage Loading regimes, i.e. CRS and creep tests. In Chapter Six this understanding will be used to identify the appropriateness of the currently employed test protocols.

In Chapter Seven, the concepts of the intrinsic property represented by the ISE Capacity $[C]_k$ and the combinations of ISE components at different limiting strains and time will be used to identify appropriate input parameters for geosynthetics under Single-Stage Loadings. Further, these concepts will be used to develop an understanding of the behaviours of geosynthetics under Multi-Stage Loadings, i.e. combined sustained-short term loading. It will be shown that the ISE Capacity $[C]_k$ and the combinations of ISE components at different limiting strains and times for Single-Stage Loadings may not be applicable to Multi-Stage Loadings, suggesting a totally different approach should be adopted when dealing with the Multi-Stage Loading situations.

CHAPTER 6

MULTI-STAGE LOADING TEST PROCEDURES

6.1 INTRODUCTION

Multi-Stage loading comprises a combination of two or more Single-Stage loadings, viz. sustained, cyclic or CRS. These Single-Stage loadings can be represented by various standardised testing techniques, viz. creep, cyclic or CRS. In existing testing protocols no consideration is given to the combination of these various Single-Stage loadings. A literature review reveals that to date there are few published test results on the behaviour of GRSSs under Multi-Stage loading using clearly defined test protocols. Test protocols are important as the loads have to be applied and removed consistently in a rapidly and smooth manner to the specimen in the correct sequence and not cause rebound, swinging or any other movements leading to specimen distortion. Thus clearly defined test protocols and test apparatus are essential for Multi-Stage Loading tests and require to be developed.

6.2 MULTI-STAGE SITUATIONS

Single-Stage Loading tests, e.g. CRS and creep tests, are conducted by a Single-Stage Loading regime over the full test period. Change of the loading regime during the testing is not allowed. In contrast, in the field a GRSS is subjected to various forms of Actions applied over different periods. These Actions may change in amount, frequency, time of application and in other ways. Therefore a Single Stage Loading test does not reflect the Multi-Stage Actions to which GRSSs are subject. Thus it is suggested that to model the behaviour of a GRSS under Multi-Stage Actions a combination of loading regimes must be employed. In most situations, a sustained load acts over the entire design lifetime but this is accompanied by live loads or an impact load occurring over short terms.

For example, for Load Supporting Structures such as roads, a sustained load (due to self-weight) and repeated loading (due to traffic) occur. It is to be noted that in this case the amount of the repeated load is often higher than the amount of sustained load. To model the behaviour of a Load Supporting Structure subject to such Multi-Stage Actions, a Multi-Stage Loading test has to be employed which involves a sustained load and repeated (cyclic) loading. The cyclic load can vary in frequency and amount, with rest periods between cycles as shown in Fig. 6.1 (a). Such a Multi-Stage Loading test may be called a *Combined Sustained-Cyclic Loading Test.*

Earth Structures such as retaining walls, steep slopes, embankments and bridge abutments, experience during their design life, sustained loading (due to self-weight) and additional permanent or transient sustained loads (surcharges or traffic), with the possibility of seismic loads (earthquakes) occurring. Therefore a Multi-Stage Action may occur. This may be represented as shown in Fig. 6.1 (b) in a Multi-Stage Loading test, which may be called a *Combined Sustained-Short Term Loading Test.*

To date a small number of Multi-Stage Loading tests has been conducted at the University of Strathclyde. Pradhan (1996) made Combin*ed Sustained-CRS Tests*. These consisted of sustained loads over 100 hours and followed by CRS loading up to rupture. Khan (1999) undertook *Combined Sustained-Cyclic Loading Tests*. These consisted of 100 hours sustained load and then a further 100 hours of combined sustained and cyclic loading. *Combined Sustained-Short Term Loading tests* are the latest to be conducted and are presented in this thesis. These tests consist of 200 hours of sustained load, with an additional sustained load applied for 20 seconds at 100 hours and then removed.

6.3 Test Protocols for Multi-Stage Loading

The test protocols for Multi-Stage Loading is primarily based on the procedures and methods given by BS 6906 Part 5 (1989): Methods of Test for Geotextiles; Determination of Creep, (with adjustments as necessary).

6.3.1 Scope

Multi-Stage Loading test protocols describe the procedures for determining the load-strain-time relationship of geotextiles at a given temperature under two or more loading regimes.

Methods of presenting and evaluating test results are in a form which enables Isochronous Load-Strain curves to be determined. Such curves can be used to identify the strain of a material at a specified load and time. Load-Strain curves for a range of times are generally plotted on the same diagram. Calculations using the ISE approach then allows the ISE versus time plots and $[R]_t - [L]_t$ plots to be derived from the data.

6.3.2 Principle of Multi-Stage Load Testing

Clamps are used at both ends to grip the test specimen across its width. A sustained load is applied as a longitudinal force to the test specimen. It is maintained over the whole test period. An additional load is applied after a certain time and after a defined time removed. This load may be applied on one occasion or on a repeated basis. Extension is measured at specified time intervals and recorded by dial gauges or Linear Variable Displacement Transducers [LVDT's] and recorded with an interfaced computer or data logger.

Various test procedures are used which involve taking the test specimen to rupture, to a limiting strain, limiting time or for a limiting number of cycles.

6.3.3 Test Apparatus for Multi-Stage Load Testing

Several researchers Müller-Rochholz (1994), Pradhan (1996) and Khan (1999) combined Creep, CRS and/or Cyclic testing to simulate Multi-Stage Actions in a GRSS.

The Multi-Stage Loading test apparatus consists of a steel frame sufficiently strong not to deform and vibrate when in use. It provides access for the specimen to be mounted, allows the load to be correctly applied and the strain monitored. The connections between the clamps, load cell, LVDT's and loading apparatus have sufficient freedom to ensure that the load is applied uniformly across the specimen width. In tests where the specimen ruptures, all loads require to be free to fall or move without endangering the apparatus, instrumentation or operators.

6.3.4 Test Specimens: Sampling, Conditioning and Loading

6.3.4.1 Test Specimens

Test specimens are prepared according to ISO 9862 (1990) and BS 6906 (1989), Part 1 and 5.

6.3.4.2 Sampling

All involved parties must agree the number of batches. Each selected batch has to be undamaged and the wrapping intact, if any existed. Material for ex-works testing must not contain dirt, irregular areas, creases, holes or other visible defects of accidental origin.

6.3.4.2.1 Cutting

The first two turns of the roll are not used for sampling. Samples are cut from a roll, over its width, perpendicular to the machine direction. The required number of specimens are cut from positions evenly distributed over the full width and length of the sample, but not closer than 100 mm to the seam or roll edges. Identical longitudinal or traverse position of two or more specimen must be avoided, i.e. cutting should be in a staggered way. Identification markings on the sample, e.g. batch number, machine direction, sample number, etc. must be carefully transferred to all test specimens. The cut specimens should be kept in a dry, dark place free from dust, at ambient temperature of 20°C and protected against chemical and physical damage until testing.

6.3.4.2.2 Dimensions

The prepared geotextile test specimens are required to have a minimum dimension of 100mm in any direction of testing. This is generally considered satisfactory to account for the local variability of most geotextiles. Test specimens with a width to length ratio of 2 and a gauge length of 100mm are used for the majority of geotextile types to ensure reproducible test data. However, it should be pointed out that for a few geotextiles, such as staple fibre needle-punched products, larger width to length ratios (of 5 or more) may be required.

Geogrid test specimens must contain at least three complete tensile elements (ribs) within the width and at least one row of nodes or cross-members along the length, excluding the nodes or cross-members by which the sample is held in the clamps. All ribs must be cut at least 10 mm away from any node. The gauge length for geogrids is taken to be the distance between the centre-lines of the elements.

All test specimens are examined for any signs of imperfection which can cause realignment and measurement failures. Figure 6.2 shows typical specimens used in Multi-Stage Loading tests.

6.3.4.3 Conditioning and Testing Atmosphere

Test specimens are conditioned and the test conducted, in the standard temperature atmosphere for geosynthetics testing as defined in BS EN

20139 (1992). The cut samples are stored for at least 24 hours and the tests are conducted at a relative humidity of 65 ± 2% and a temperature of 20 ± 2°C. The prepared sample must not be subjected to any stress or strain that could affect their subsequent performance.

6.3.4.4 LOADING

Determine the total load and starting time for loading. The starting time is the moment when the full load is firstly applied to the specimen. The load must be applied smoothly to the sample at a rate which will not cause rebound or any other distortion. This is achieved by monitoring the extension using LVDT's and a load cell.

6.3.4.5 UNLOADING

Determine the starting time for unloading. The starting time is the moment when the load is removed from the specimen. The load must be removed smoothly from the sample at a rate which will not cause rebound or any other distortion. The Unloading method is test dependent and approaches must be developed as necessary. It may be achieved by using an unloading guide mechanism which prevents rebound and by monitoring the extension using LVDT's and a load cell.

6.3.4.6 APPLICATION OF PRE-TENSION

Due to the mass of the clamps or test apparatus, a pre-tensioning load is sometimes applied. If any significant extension develops due to this load, it must be recorded.

6.3.5 CLAMPS

The clamps must be selected to grip the specimen with sufficient firmness to avoid slippage. According to BS 6906: Part 1, they should not cause damage or produce areas of weakness outside the gauge length. Each clamp has faces extending over the width of the specimen or longer. Types of clamps, which have been found satisfactory in practice, are shown in Fig. 6.3.

6.3.6 INSTRUMENTATION

The following instruments are used in Multi-Stage testing for load application and data acquisition.

6.3.6.1 STRAIN MEASUREMENT

Extension of the sample under load is measured between two lines across the width of the sample, parallel to each other and to the clamps and at a minimum spacing of 100 mm. In general two extensometers are used on opposite sides and ends of the specimen, from which the average readings are taken in order to compensate for alignment errors. The extensometers must have a resolution of 0.02% strain, and their range must be selected such that it accommodates material extension during at least the first hour of testing without exceeding the limits of calibration or any adjustment. Further a fast reaction time is necessary to measure the initial strain. The extension is measured at specified time intervals that are test dependent.

6.3.6.2 LOAD MEASUREMENT

For some Multi-Stage Loading tests it is not necessary to measure the applied load because dead load is being used, but it is important to obtain the loading and unloading time for later analysis, thus a load cell must be chosen. The reaction time is vital therefore a fast reacting load cell must be used. The load cell has to accommodate the overall load of all stages without exceeding the calibration limits and have sufficient resilience for loading, unloading, impact, impulse and vibrations.

6.3.6.3 DATA RECORDING AND STORAGE

The obtained data has to be recorded and stored. A computer interface has proven to be useful. The data is acquired by a data logger or an interface and then stored on the hard disk of a connected computer. The recording device must be capable of recording and storing data at a rate of 0.2 seconds or faster. This is important for the strain measurement while loading and unloading to obtain data to determine the instantaneous strain and loading time for later analysis.

One of the main problems is that data-loggers and computers may require to operate constantly for the whole test duration without failing or re-calibration. Some tests can take up to 10,000 hours. However, it may be possible to change to dial gauges and then the read-outs must be taken manually at pre-set time intervals. Where data is collected at small time intervals, this approach is not possible because of the considerable amount of data which has to be acquired. Khan (1999) has reported such a problem.

6.3.7 NUMBER OF TESTS

Tests are required to be conducted on a number of occasions to check for reproducability. They also need to be conducted over the full range of operational loadings, strains, temperatures, etc.

Loads, strain and temperatures are selected in such a way as to model the behaviour of a geosynthetic under working and special loading conditions, e.g. average design sustained load together with increasing short-term or cyclic loading at specified temperatures.

6.3.8 DATA PROCESSING

The data should be stored on a computer hard disk and later analysed using commercially available spreadsheets and graph plotting programs.

6.4 Analysis of Test Results

Several approaches to represent and analyse the obtained test results have evolved to date, some are shown below.

6.4.1 Available presentation techniques

Over the time several presentation techniques were being developed to display test data. The simplest one may be the Strain-Time plot, where Strain is plotted over the Time. This presentation technique is particularly used to present Creep test data; Multi-Stage Loading test results may be presented in this way as well. However, for a complex analysis, viz. to understand the strain components and the material behaviour under special loading conditions, this presentation technique is not useful.

The use of *Rheological Models* has become popular, where simple physical models like ideal elastic springs, Newtonian dashpots and friction-elements are combined in series or parallel to simulate the load-strain behaviour of a specimen. Such models are usually acceptable only in the linear extension domain and the predictions for the non-linear extension range are not accurate or simply wrong, Seeger (1996). However, rheological models provide an understanding of the material behaviour and the affected strain components. Test results are plotted as Strain-Time, Load-Strain or *Isochronous Load-Strain curves* with the disadvantages presented below.

Isochronous Load-Strain curves present test data as a load-strain plot where each time develops a specific curve. Like strain-time plots this technique does not fully allow the analysis of strain components, but it allows the understanding of the loading and unloading behaviour of a specimen. A major problem is that it is very difficult to illustrate the loading history of a specimen with Isochronous Load-Strain curves, e.g. after unloading a specimen the Isochronous Load-Strain curves shows a single horizontal line.

In contrast the *Isochronous Strain Energy [ISE]* approach allows the data to be presented in one single plot and shows the loading and unloading sequences by presenting the specimen's energy state at any given time.

6.4.2 Calculation of ISE Components under a Multi-Stage Loading Regime

The procedure used to calculate Isochronous Load-Strain curves has been described by Kabir (1984). The calculation and presentation of ISE components was also suggested by Khan (1999) and is shown by way of example for the Combined Sustained-Short Term Loading test results in Chapter 7. Similar analysis techniques can be used for other Multi-Stage Loading test results.

CHAPTER 7

TEST RESULTS

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INTERPRETATION USING THE ISE APPROACH

7.1 INTRODUCTION

The research objective was to apply a loading sequence to model the behaviour of geosynthetic reinforcements before, during and after a seismic event. The test results were obtained from Combined Sustained-Short Term Loading tests, using the suggested test protocol outlined in Chapter 6. They were then interpreted using the ISE approach. It is hoped that this will lead towards a better understanding of geosynthetic behaviour under Multi-Stage Loading regimes in the future. It also allows an interpretation and a back analysis of why GRSSs in seismic active regions often did not fail during catastrophic earthquakes.

7.2 STRAIN COMPONENTS UNDER LOADING

To study the effects on the Immediately Recoverable and the Locked-in Strain components due to a Multi-Stage Loading regime was one of the research objectives. In a Single-Stage Loading regime the Immediately Recoverable Strain increases instantaneously the moment load is applied. The Locked-in Strain increases with time. In the moment the load is removed the Immediately Recoverable Strain component totally decreases. The Locked-in Strain component, in contrast, recovers a certain amount partially with time and then remains permanent.

Under a Multi-Stage Loading regime it is likely that the behaviour of the Immediately Recoverable and Locked-in Strain components changes, thus Multi-Stage Loading tests were conducted to observe the changes in the strain components under Short-time Loading.

7.2.1 TEST SEQUENCE

This Multi-Stage Loading test consisted of three stages. In the 1st Stage, a sustained load was applied and maintained over the whole test period. The response of the strain components was equal to a Single-Stage loaded specimen.

In the 2nd Stage, an additional load was applied for 20 secs at 100 hours. The short-term load, or additional load, consisted of one of five load levels from 10 kN/m to 50 kN/m at 10 kN/m increments. At the end of the 2nd Stage the additional load was instantaneously removed. Thus the 3rd Stage consisted of a sustained load equal to the first Stage. The sustained load was maintained for a further 100 hours.

Rheological models indicated that the Immediately Recoverable Strain should decrease an amount equal to the 1st Stage. However, the behaviour of the Locked-in Strain component could not been predicted using conventional techniques.

Figure 7.1 shows the schematic Load-Time plot for the Combined Sustained-Short Term Loading tests.

7.2.2 LOAD ARRANGEMENT

The sustained load of 25 kN/m is suggested by BS 8006 (1995) as the long-term design strength for uniaxial Geogrid B at 20°C. This also represents the maximum long-term design strength used in a variety of other national design codes obtained by a Single-Stage loading test, see Chapter 3.

The maximum additional load of 50 kN/m was chosen as this plus the sustained load of 25 kN/m provides the short-time strength obtained in CRS tests made at the University of Strathclyde using the regulations of BS 6906 Part 1 (1987). The extension rate suggested by BS 6906 (1987) for CRS testing is 7 %/min to 13 %/min. The manufacturer of Geogrid B gives the short-time strength as 80 kN/m using the extension rate of 33 %/min. This later extension rate was suggested by the Deutsches Institut für Bautechnik

[DIBt] and used in the Manufacturers Approval Certificate No: Z-20.1-102 to obtain the design parameters for Geogrid B. The DIBt test results, however, could not be confirmed, as the maximum extension rate of the CRS testing machine at the University of Strathclyde is 25 %/min for the standard specimen length.

Thus the maximum load level to simulate a major seismic event was selected to be 75 kN/m. To simulate other seismic events or transient loads the load levels were reduced by 10 kN/m increments.

7.3 Test Protocol for the Combined Sustained-Short Term Loading Test

This test protocol was developed to apply a certain combination of sustained loading and short-term loading to polymeric reinforcements as mentioned above. This test protocol is based on the assumptions stated previously in Chapter 6.

7.3.1 Scope

The Combined Sustained-Short Term Loading test protocol describes the procedures for determining the load-strain-time relationship of geosynthetics at a given temperature under two loading regimes. These loading regimes simulate a sustained load and seismic load applied to the structure. The extension is measured to obtain the load-strain-time behaviour before, during and after the event.

Isochronous Load-Strain curves are used to obtain the ISE components. These results allow ISE versus time plots and $[R]_t - [L]_t$ plots to be derived.

7.3.2 Test Procedure

The test specimen was centrally mounted in the clamps with the clamps parallel. These were used to grip the test specimen across its full width. The sample was positions as close as possible to the centre line of the top and bottom clamp. This line was drawn previously at the centre of the width of the specimen and the clamp. A sustained load was applied as a longitudinal force to the test specimen. It was maintained over the whole test period of 200 hours. The Short-term Load was applied after 100 hours and removed after 20 seconds. This load was applied on one occasion only. Extension was measured at specified time intervals using LVDT's and was recorded with a data logger.

7.3.3 Combined Sustained-Short Term Loading Test Apparatus

The Combined Sustained-Short Term Loading test apparatus fulfilled all the conditions mentioned in the Multi-Stage Loading tests procedure. The space between the bottom clamp, ground and apparatus was sufficient to ensure freedom for all load tables to move without touching the apparatus or ground while the specimen extended. The schematic of the test apparatus to be used in Combined Sustained-Short Term Loading tests is shown in Fig. 7.2. The test apparatus during various Loading Regimes is shown in Fig. 7.3.

An important objective was that the additional load had to fall free without disturbing the specimen and the sustained load. A release mechanism was developed to satisfy these conditions, Fig. 7.4.

7.3.4 Test Specimen: Sampling, Conditioning and Loading

Test specimens were chosen according to ISO 9862 (1990) and BS 6906 (1989), Part 1 and 5.

7.3.4.1 Sampling

One ex-works batch of Geogrid B was used for the Combined Sustained-Short Term Loading tests. This batch showed no visible damage or any other alteration.

7.3.4.1.1 Cutting

The first two turns of the roll were not used for sampling. The required numbers of specimen were cut from position evenly distributed over the full width and length of the sample, but not closer than 100 mm to the seam or roll edge. Identification markings on the sample e.g. longitudinal and traverse position, sample number, etc. were carefully transferred to all test specimens by using stickers. The cut specimens were kept in a dry, dark place free from dust, at ambient temperature of 20°C and were protected against chemical and physical damage until testing.

7.3.4.1.2 Dimensions

The geogrid test specimen contained three complete tensile elements (ribs) within the width one row of nodes or cross-members along the length, excluding the nodes or cross-members by which the sample was held in the clamps. All ribs were cut at least 10 mm away from any node. The dimensions of the geogrid test specimen were 105mm wide and 300mm long.

Figure 7.5 shows a prepared sample of a geogrid test specimen with its dimensions.

7.3.4.2 Conditioning and Testing Atmosphere

Test specimen was conditioned and the test conducted, in the standard temperature atmosphere defined in BS EN 20139 (1992).

The geogrid test specimen was known to show no influence from relative humidity but was known to be affected by temperature. Storage, conditioning and test conditions were chosen to be 20 ± 2°C and 65 ± 2 % relative humidity in a climate-controlled environment.

7.3.4.3 Loading

A hydraulic forklift was used to apply a sustained load in form of dead load. Lead bricks, each 25 kg, piled upon a specially constructed load table were used. The load was applied by opening a valve, which was connected to the hydraulic cylinder of the forklift. This load was applied smoothly and rapidly within 5 seconds.

7.3.4.4 UNLOADING

As stated previously a static load, called a Multi-Stage Sustained Load Equivalent may be used to substitute a dynamic load, see Chapter 4. However, Instantaneous unloading is necessary to obtain the Immediately Recoverable ISE component $[R]_t$.

For high load levels, the forklift device was not satisfactory to unload the sample instantaneously. As mentioned previously a release mechanism was developed. The principle of the device is based upon cutting a connection thread while the load is still applied. To prevent the clamped specimen from shooting back, thus disturbing the test and damaging the LVDT's, a guide mechanism was developed. Figure 7.6 shows a schematic of this mechanism. The test results obtained using this method were found to be entirely safe and satisfactory.

7.3.4.5 APPLICATION OF PRE-TENSION

The geogrid test specimen showed no extension upon application of pre-tension by the mass of the bottom clamp. The weight of the profiled bottom clamp (8.50 kg) was regarded as negligible compared to the stiffness of the geogrid test specimen. Therefore no detailed measurements were made.

7.3.5 CLAMPS

For the Combined Sustained-Short Term Loading test profiled clamps were used as recommended by the manufacturer of the geogrid test specimen, see Fig. 7.7. Specimen rupture occurred at the middle node, thus the specimen was not affected by clamping.

7.3.6 INSTRUMENTATION

7.3.6.1 STRAIN MEASUREMENT

Two Linear Variable Displacement Transducers [LVDT's] were chosen for the Combined Sustained-Short Term Loading test. A pair of universal couplings was used to connect the LVDT's to the top clamp. Therefore it was possible to determine the relative displacements between the top and bottom clamp and thus the strain was calculated. These universal couplings were developed at the University of Strathclyde and ensured that the LVDT's were parallel, showed no inclinations or eccentricities during the test. A schematic of these couplings is shown in Fig. 7.8.

The LVDT's pistons had a total travel length of 50mm. For high load levels the material extension exceeded the calibration limit of the 50-mm LVDT's and 150-mm LVDT's were used. The reaction time was below 0.2 seconds and therefore satisfactory.

The extension was measured at frequent intervals during loading and unloading as specified below. The fastest record mode of the Data Logger was 0.2 seconds. This time interval was used to cover the loading and unloading time. Following readings were taken:

Within the first minute	@ 0.2 second intervals
Between the 1st and the 6th minute	@ 10 seconds intervals
Between the 6th and the 13th minute	@ 1 minute intervals
Between the 13th minute and 1st hour	@ 3 minutes intervals
Between the 1st hour and the beginning of additional loading	@ 30 minutes intervals

7.3.6.2 LOAD MEASUREMENT

The maximum load used in the Combined Sustained-Short Term Loading test were 6 kN. The chosen load cell had a capacity of 20 kN for tension and compression, Fig. 7.9. There was no indication for any compression while unloading, hence there was no impact from the instantaneous unloading. Calibrations in lower load levels were necessary to assure a sensitivity sufficient to identify minor changes in loads.

7.3.6.3 DATA RECORDING AND STORAGE

A Schlumberger data logger model 3530 was chosen. The data logger was connected to a 486DX computer, which stored the transferred data on hard disk. A program was written in Quick Basic to program the data logger and to establish an interface connection. The data logger was capable of simultaneous multi channel reading using a recording rate of 0.2 seconds. Three channels were observed one for load cell and two for the LVDT's.

7.3.7 NUMBER OF TESTS

The product type used in the Combined-Sustained Earthquake Loading test showed small load-strain-time-temperature dependencies in the creep and CRS tests (nearly no specimen variation from the mean). Repeated identical Combined Sustained-Short Term Loading tests confirmed this. Therefore all other load levels were tested once.

7.3.8 DATA PROCESSING

The data were stored as a text file that was then converted to a spreadsheet where calculations were done. The results were then displayed as a graph. The used software was the spreadsheet program Microsoft Excel and the graph plotting software Microcal Origin.

Macros were written and used for data processing in Excel, because of the great amount of accumulated data in one 200-hour test.

7.4 COMBINED SUSTAINED-SHORT TERM LOADING TEST RESULTS

7.4.1 CALCULATION OF ISE COMPONENTS

Kabir (1984) described the procedure to calculate Isochronous Load-strain curves. The calculation and presentation of ISE components was suggested by Khan (1999) and is shown by way of example on the Combined Sustained-Short Term Loading test results.

To obtain the $[R]_t$ and $[L]_t$ components at the end of each stage was important as this was the starting point for the next stage.

The 1st Stage for all Combined Sustained-Short Term Loading tests consisted of an identical load level and was represented by a Single Stage

Loading regime. $[R]_t$ and $[L]_t$ were calculated according to procedures given in Chapter 5.

From the Strain-Time plots in the 2nd Stage Isochronous Load-strain curves were derived, Fig. 7.10. For the 20 second duration of the 2nd Stage 5 seconds interval were chosen, thus at 100 hours plus 5 seconds, 100 hours plus 10 seconds, 100 hours plus 15 seconds and 100 hours plus 20 seconds Isochronous load-strain curves were constructed, Fig. 7.11. This provided results which helped to understand the loading of the specimen and its deformation. The obtained Isochronous Load Strain curves were then separated into Isochronous Load total, recoverable and locked-in strain curves. $[R]_t$ and $[L]_t$ for a certain load level were obtained by calculating the area under the corresponding Isochronous Load-strain curve. These results were then presented in a $[R]_t$ - $[L]_t$ plot, Fig. 7.12 and Fig. 7.13.

The time intervals to create the Isochronous Load Strain curves in the 3rd Stage were chosen to be 100 hours plus 21 seconds, 100 hours plus 25 seconds, 100 hours plus 35 seconds, 100 hours plus 80 seconds and at 200 hours. This assured that the Unloading at the beginning of the 3rd Stage, the Recovery of time dependent ISE component and the ISE components at the end of the test could be observed.

All 3rd Stages consisted of a sustained load of 25 kN/m. Therefore all Isochrone are at the same load level; thus they appear to be a single horizontal line in a plot, Fig. 7.14. This turned out to be a major disadvantage of the Isochronous Load Strain curves for Multi-Stage Loading tests analysis. It is not possible to present more than one load level and time in the same plot, Fig. 7.15. Nevertheless, the calculation of ISE components follows the same principles as described above, Fig. 7.16. The data is given in a $[R]_t$ - $[L]_t$ plot, Fig. 7.17.

7.4.2 INTERPRETATION USING LOAD-STRAIN PLOTS

Figure 7.18 shows the test results for the whole test period of 200 hours. After the load was removed the material recovered and then the strain remained constant. It is remarkable that the strain in the 3rd Stage compared to the 2nd Stage is very low. Also very important to notice is that although 25 kN/m of sustained load was applied during the 3rd Stage, there was no indication of further creep for higher load levels.

Figure 7.19 shows the strain-time relationship for the 20 seconds of the 2nd Stage loading and after the unloading sequence for 20 seconds at the beginning of the 3rd Stage. It should be noted that up to a load level of 65 kN/m the material behaves within the linear extension domain. For 25 kN/m + 50 kN/m (a total load of 75 kN/m), which caused a specimen rupture after 18 seconds of 2nd Stage loading, the extension behaviour is clearly non-linear. This particular linear and non-linear extension behaviour for Geogrid B has been confirmed in the later *Recovery tests*, see Appendix A.

Figure 7.20 (a) and (b) show that the strain-time relationship for 10 kN/m 2nd Stage loading in both linear and semi-logarithmic plots. It has to be remarked that for an additional load of 10 kN/m, which is 40% more additional load over the sustained load, there is no indication of any change in the 3rd Stage creep behaviour compared to 25 kN/m Single Stage Loading. All additional strain was recovered; thus the amount of Locked-in time independent (permanent) Strain developed during the 2nd Stage was negligible. After the initial Recovery the sample behaved in a similar manner to the Single Stage loading specimen. The same effect was observed for 20 kN/m + 25 kN/m loading (80% more additional load) in the 2nd Stage, Fig. 7.21 (a) and (b). The Locked-in time dependent Strain component was recovered within the first minute of the 3rd Stage. Therefore the amount of Locked-in permanent Strain, developed during the 2nd Stage loading, was very small.

Moreover Figs. 7.22 to 7.24 show the strain-time relationship for 30 kN/m, 40 kN/m and 50 kN/m 2nd Stage loading in linear and semi-logarithmic plots.

After the 2nd Stage load was removed the material initially recovered a high amount and then did not recover or creep any further but remained constant compared to Single Stage Loading of 25 kN/m. Figure 7.22 (a) and (b) confirmed this behaviour in a 1000-hours test. This indicates that this stretched high-density polyethylene [HDPE] material shows minor visco-elastic extension behaviour for high load levels. Seeger (1996) mentioned this might occur for some materials.

The short-term strength was reached after 18 seconds with 75 kN/m loading in the 2nd Stage and caused specimen rupture. This confirmed that even after a previous loading sequence of 100-hours-sustained loading the short-term strength was reached. The specimen showed apparently no degradation due to pre-straining of the 1st Stage loading. However, further testing is required to confirm this particular behaviour.

7.4.3 Interpretation using Isochronous Load-Strain Curves

As stated above the Immediately Recoverable and Locked-in Strain components cannot be presented in a strain-time plot. However, it is possible to represent the Immediately Recoverable and Locked-in Strain components after their calculation as Isochronous Load-Strain curves.

7.4.3.1 Loading in the 1st Stage

The 1st Stage 100-hour Isochronous Load-Total strain curve is shown in Fig. 7.25 (a). The Isochronous Load-Immediately Recoverable Strain and the Isochronous Load-Locked-in Strain curve is plotted in Fig. 7.25 (b) and 7.25 (c) respectively.

These curves were obtained from the Combined Sustained-Short Term Loading test results and confirmed by other tests, viz. creep tests.

7.4.3.2 Loading in the 2nd Stage

The specimen was pre-strained due to 1st Stage sustained loading, all additional loads were applied at the end of Stage 1, thus the Isochronous

Load-strain curves for the 2nd Stage started on the 100 hours Isochronous Load-strain curve.

The Isochronous Load-Immediately Recoverable Strain curve was obtained by using the method of obtaining the *Equivalent Instantaneous Loading Strain* as suggested by Kabir (1994), Fig. 7.26. Recovery tests confirmed that the elastic behaviour of Geogrid B show no dependency of time, thus no degradation over time was recognised and only a single Isochronous Load-Immediately Recoverable Strain curve could be plotted. The Locked-in Strain, however, increased with time, thus several Isochronous Load-Locked-in Strain curves were plotted for the 2nd Stage loading.

The Isochronous Load-Total strain curves for 2nd Stage loading are shown in Fig. 7.27 (a); the Isochronous Load-Immediately Recoverable Strain curve and Isochronous Load-Locked-in Strain curves for 2nd Stage loading are presented in Fig. 7.27 (b) and 7.27 (c) respectively.

7.4.3.3 LOADING IN THE 3RD STAGE

The main problem with this particular Multi-Stage Loading test was that the load in the 3rd Stage for all load levels remained constant, thus all Isochronous load-strain curves were horizontal lines at 25 kN/m load level.

The Isochronous Load-strain curves for 100 hours plus 21 seconds and 100 hours plus 25 seconds are presented in Fig. 7.28 (a) to (f), 100 hours plus 35 seconds and 100 hours plus 80 seconds, Fig. 7.29 (a) to (f), and 200 hours, Fig. 7.30 (a) to (c). Each plot represents the Isochronous Load-strain curves of 25 kN/m, 35 kN/m, 45 kN/m, 55 kN/m and 65 kN/m load applied during the 2nd Stage.

The Isochronous Load-strain curves for previously applied loads are presented at 100 hours plus 35 seconds, Fig. 7.31 (a) to (f) and 7.32 (a) to (f).

7.4.4 INTERPRETATION USING THE ISE APPROACH

The results presented below are obtained according to procedures given in Chapter 5. The area under the Isochronous Load-Total strain curve gives the amount of Absorbed Strain Energy $[A]_t$ and under the Isochronous Load-Immediately Recoverable Strain curves the Immediately Recoverable Strain Energy $[R]_t$ at a certain time for a certain load. The calculation of the Locked-in Strain component $[L]_t$ follows the equation:

$$[L]_t = [A]_t - [R]_t \tag{7.1}$$

These results were compared with the $[L]_t$ results obtained from the area under Isochronous Load-Locked-in Strain curves. Geogrid B showed a good accordance.

7.4.4.1 LOADING IN THE 2ND STAGE

These results for the 2nd Stage loading are plotted in the $[R]_t$ - $[L]_t$ plot, Fig. 7.33. The $[R]_t$ - $[L]_t$ plot shows the Immediately Recoverable over the Locked-in Strain component. The suggested time dependent 10%-Limiting Strain curve is shown.

The 2nd Stage loading sequence is dominated by a fast increasing $[R]_t$ for different load levels. The Immediately Recoverable Strain component that represents the elastic material component increases while loading and remains constant. For load levels of less than 50 kN/m additional load the deformation is within the linear extension domain.

Immediately after the loading sequence is accomplished (full load applied) the Locked-in component starts to develop. The rate of development is dependent on the amount of applied load. The results also suggest that for lower load levels, where the material is still in the linear extension domain the Locked-in Strain also increase linearly.

Only the 75 kN/m loading regime reached the 10%-Limiting strain after 7 seconds and was therefore regarded as failed, although the specimen ruptured after 18 seconds of loading. This shows that a structure subjected to a 17-seconds earthquake with 75-kN/m peak load and very high frequency would exceed the Ultimate Limit State [ULS], but would not collapse. Although heavily damaged the structure would maintain its stability. If however the earthquake would last only 7 seconds then the conditions for the ULS criterion would be maintained. It may be noticed that after this seismic event (3rd Stage) the total strain recovers and is then again below the limiting strain. It may be noted that the Serviceability Limit State [SLS] for GRSSs is application dependent and in the range of 1% to 2% of post construction strain.

The 75 kN/m-loaded specimen shows non-linear extension behaviour in the 2nd Stage. This is shown in Fig. 7.34 where the 40 kN/m and 50 kN/m 2nd Stage load are compared in a $[R]_t$ - $[L]_t$ plot. The plot shows that the development of $[L]_t$ at 40 kN/m is within the linear domain (compared to the lower load levels) and for 50 kN/m it follows a non-linear trend. The developing rate of $[L]_t$ before rupture decreases slightly. This is due to material alteration. The material changes rapidly from amorphous; semi-crystalline to a fully crystalline state, due to these alterations the material becomes stiffer but also more brittle. The result is on one hand a decreasing of the $[L]_t$ development and one the other hand, specimen rupture from further loading.

7.4.4.2 LOADING IN THE 3RD STAGE

The $[R]_t$ - $[L]_t$ plot for the 3rd Stage loading is shown in Fig. 7.35. The $[R]_t$ - $[L]_t$ behaviour is similar to a loop. $[R]_t$ recovers completely whereas $[L]_t$ experience a initial recovery and then remains constant.

The $[R]_t$ - $[L]_t$ loop closes completely for 10 kN/m at 101 hours test time and follows then the path of a Single-Stage loaded specimen. This shows once again that 40% additional load applied as a 20-second Impulse has no effect on the long-term material behaviour. If between two 10 kN/m-loading regimes a minimum of 1-hour rest periods is considered then this induces no additional deformation of the specimen.

A similar effect can be observed for 20-kN/m additional loading, where the rest period is with more than 100 hours longer. This indicates that 80% of additional load causes no additional deformation if applied with a 100 hours rest period. Figure 7.36 (a) and (b) shows the $[R]_t$ - $[L]_t$ plots for the 10 kN/m and 20 kN/m additional loading.

Figure 7.37 (a) and (b) shows the $[R]_t$ - $[L]_t$ loops for 30 kN/m and 40 kN/m additional load. It may be noticed that at the end of 200 hours in both cases $[L]_t$ recovers a certain amount and then remains constant compared to the Single-Stage loaded specimen. This shows that an additional load of more than 20 kN/m causes permanent strain.

$[L]_t$ consists of different contributions; time dependent and permanent $[L]_t$ due to 2nd Stage loading and time dependent and permanent $[L]_t$ due to sustained load. Therefore, $[L]_t$ decrease on one hand and increase on the other hand. This particular behaviour causes that the Total strain remain constant.

After a certain time, which is dependent on the previously applied load, $[L]_t$ starts to increase again. At this time the Single-Stage and the Combined Sustained-Short Term loaded specimens reach a similar state. After this time both show identical creep behaviour. This behaviour was observed for minor loads after short time. However, more testing is required to obtain the long-term behaviour.

7.5 SUMMARY

Previous design methodology considered a factor of 1.5 times the sustained load as satisfactory for design against earthquakes. The Combined Sustained-Short Term Loading test results from this series show that even a load of 80% (1.8 times) the sustained load alters the material only temporally and that with time this alteration decreases. Therefore, previously used design codes are likely to be conservative.

Present design codes consider for seismic loading the transient nature of earthquakes. They use for design against earthquakes at the end of design life factored CRS test results. The Reference Strength may be overestimated with this consideration, as the CRS testing technique does not consider any pre-straining or previous loading history. Figure 7.38 (a) shows the $[R]_t$ - $[L]_t$ for the present design methods. It may be noticed that the loading history is omitted and the CRS test is assumed to start for an ex-works material. This consideration is not correct as shown in Fig. 7.38 (b) where the implication from the Combined Sustained-Short Term Loading test results are shown.

It is important to notice that during a CRS test minor development of $[L]_t$ was observed but that the Immediately Recoverable Strain component $[R]_t$ dominates. In contrast however, the Combined Sustained-Short Term Loading test show major $[R]_t$ and $[L]_t$ development. The interpretations using the ISE approach consider the previous loading history and show the available ISE to a limiting strain.

Therefore, it may be appreciated that the design methods using the CRS strength consider the wrong strain components and assume an available strength which is too high. Hence, they must be considered unsafe.

The Limiting Strain that is used for the ULS analysis for the group of geogrids, where Geogrid B belongs to, shows an Instability Strain at 12% to 14%. As shown only the 75-kN/m Loading regime exceeded after 7 seconds the 10%-Limiting strain. If these results are employed in a design methodology then the Reference Strength of 75 kN/m for seismic events of less than 7 seconds can be used. If, however, the Reference Strength of 65 kN/m is used than this assures that a 20-seconds earthquake does not exceed the 10%-Limiting strain. This covers most earthquakes with a Magnitude $[M_W]$ of 7 or less, see Chapter 4.

It may be appreciated that these considerations are only valid for a loading period of 100 hours. No stiffness degradation, thus alteration of $[R]_t$ and $[L]_t$ is considered in these assumptions. To observe the stiffness alteration during the design lifetime should be part of future research.

CHAPTER 8

DISCUSSION AND CONCLUSIONS

8.1 DISCUSSION

In Chapter One, it was pointed out that despite considerable research and evidence from case histories, outcomes of the design methods/codes for the GRSSs have remained essentially the same or have become more conservative over the last two decades. Apart from the calculation models used in the analyses themselves, the reason for this lack of progress may be attributed to the choice of very conservative and sometimes inappropriate input parameters related to the soils and the geosynthetics. Further it has been indicated that the input parameters for soils and geosynthetics should be separately identified for Single-Stage Actions and for Multi-Stage Actions.

In Chapter Two, the basic mechanism of behaviour of GRSSs was introduced and the different reinforcing elements compared. It was shown that the presently used extensible geosynthetic reinforcing elements exhibit a rather complex time and temperature dependent elasto-visco-plastic behaviour.

In Chapter Three, various design codes/methods were presented which were based on the Limit Equilibrium Approach and the Hybrid Approach. These various design methods were the AASHTO Standard Specifications for Highway Bridges (1997), the Deutsches Institut für Bautechnik (DIBt) Method (1998), the HA 68/94 Method (1997) and the BS 8006 (1995). Furthermore, the model mechanisms for "True" Limit State Approach, developed by McGown et al (1998), were also presented in this Chapter.

In Chapter Four it was shown that different input parameters for the same soils and geosynthetics may require to be used for both Single-Stage Actions and Multi-Stage Actions. The main input parameters for geosynthetics were identified as their Reference Strengths, Partial Factors and Design Strengths. It was shown that these input parameters are dealt with in a variety of ways by different existing codes/methods and researchers.

For Single-Stage Actions, the Reference Strengths have been determined either by factoring the short-term CRS strengths so that the end results are equal to the long-term creep rupture strength, or by taking the load corresponding to rupture or a particular limiting strain from sustained load (creep) tests.

For Multi-Stage Actions, such as sustained loading plus short-term seismic loading, most design codes consider the Reference Strengths to be equal to the short-term strength obtained from CRS tests or factor up the Reference Strength used for Single-Stage Actions, (e.g. multiply the Reference Strength by 1.5).

Additionally, it was shown in Chapter Four that the Partial Factors are generally determined on the basis of short-term CRS tests for use in designs involving either Single-Stage Actions or Multi-Stage Actions. Although some researchers suggested that sustained load (creep) test should be used for determining the Partial Factors for geosynthetics, to date, no design codes/methods are known to have adopted this.

The Design Strengths for both Single-Stage Actions and Multi-Stage Actions are always obtained by dividing the appropriate Reference Strengths by the Partial Factors.

Later in Chapter Four, the implications of seismic, cyclic and combined sustained cyclic loading were taken into account and the Multi-Stage Sustained Load Equivalent was developed. It is based on the assumption that a combined sustained load at peak load of an intermitting load for the same application time show a more critical creep behaviour than the intermitting load alone. The test parameters for the Combined Sustained-Impulse Loading tests were selected accordingly to these results.

In Chapter Five, the ISE approach developed by Khan (1999) was introduced to characterise the load-strain-time-temperature behaviours of geosynthetics under Single-Stage loading regimes. Within this approach, it was shown that the ISE Capacity [C_t] varies with time and strain levels. This indicates that the Reference Strengths for Single-Stage Actions should be determined for different times and strain levels. Further, it was identified within this approach that the ISE Capacity [C_t] comprises the Immediately Recoverable ISE [R_t] and the Locked-in ISE [L_t] components which respond totally differently in the short and long-terms. The identification of the ISE components thus indicates that the data obtained from short-term tests might not be appropriate for use in designs for long-term applications.

The values of the ISE Capacity [C_t] were found to vary with the type of geosynthetics even for a particular limiting strain and time. This indicates that the ISE Capacity [C_t] represents an intrinsic property which changes with response to alterations of micro or macro structures of the geosynthetics. In other words, there is no unique Reference Strength, no unique Partial Factor and no unique Design Strength for a geosynthetic.

Further, it was shown in Chapter Five that the changes of the ISE components due to an event may vary significantly with time. It was emphasised that the short-term CRS tests may measure totally different changes of the ISE components than the long-term sustained load creep tests and for some geosynthetics this can be very significant.

It was also suggested that the current test protocols may not truly represent the field conditions and the data obtained from these may not be appropriate for use in designs. Therefore, test protocols simulating the actual field conditions require to be developed.

In Chapter Six, a test protocol for Multi-Stage Load Testing was developed. It consisted of a test scheme based upon established protocols but aimed to deal with Multi-Stage Loading regimes applied on one occasion or repeated.

This test protocol was later employed in the Combined Sustained-Short Term Loading test.

Tests data obtained in the Combined Sustained-Short Term Loading test were presented in Chapter Seven and interpreted using the ISE approach. On these bases, it was suggested that when designing for sustained loading plus short-term (seismic) loading, geosynthetics are likely to be able to carry a much larger load than the long-term sustained load. Tests indicated that one particular geosynthetic could carry an additional short term load [ΔP_s] of some 50 kN/m (for a period of 18 seconds) in addition to a Sustained Load [P_s] of 25 kN/m, whereas its long term Reference Strength was 30.5 kN/m at 10% limiting strain. This ability of geosynthetics to carry large short-term loads greater than the long term Reference Strength, explains why GRSSs designed according to the current codes/methods survived the Northridge Earthquake in 1994 and the Kobe Earthquake in 1995. Whether or not these structures will survive any earthquake in the future, when they have carried a sustained load for much longer periods, still remains open to question. Thus, it is suggested that the amount of additional short term load [ΔP_s] taken by a geosynthetic is not a constant but will depend *significantly* on when this short-term load is applied, i.e. when during its operational lifetime the GRSS is hit by an earthquake. It should be appreciated that the period of sustained loading is likely to have a profound influence on the changes of micro and macro structures and therefore the stiffness properties of the geosynthetic.

The additional short term load [ΔP_s] that a geosynthetic will be able to take is likely to depend on the available strain, i.e. the difference between the strain before the earthquake and the limiting strain. For example, if for a sustained load [P_s] the strain in a geosynthetic is equal to the limiting strain of 10% at the end of design life, i.e. there is no available strain, then no further load can be applied. However, if the strain at the end of design life is 8%, i.e. available strain is 2%, the geosynthetic will be able to take a certain amount of additional load. However, further research requires to be carried out to confirm the relationships between time of occurrence and the amount of

Additional Short-Term Load [ΔP_s], the Available Strain and Stiffness Properties for the geosynthetics.

The above indicates that the current practice of using a single value of Design Strength over the entire design lifetime is *not appropriate* and could be *unsafe*. More understanding of the response of the ISE components under combined sustained plus short-term loadings requires to be developed. Further, it is suggested that in order to design for sustained loading plus short-term loading, a Modified Material Properties Approach should be used. Within this approach, a new Partial Factor, called the Short Term Loading Factor, should be applied to the Limiting Strain [ε_L] in order to obtain a Factored Limiting Strain [$\varepsilon_{des,eq}$]. The load corresponding to this Factored Limiting Strain may be termed the Modified Design Strength [$P_{des,eq}$]. It should be appreciated that the purpose of applying the Short Term Loading Factor to the Limiting Strain [ε_L] is to allow a certain amount of Available Strain. In this approach, the GRSSs should be first designed for Single-Stage Actions with a Modified Design Strength [$P_{des,eq}$] and then the reinforcement layout should be checked for Additional Short Term Load [ΔP_s].

8.2 CONCLUSIONS

The following conclusions are drawn from the present study:

1. Current design codes/methods are based on the Limit Equilibrium Approach or the Hybrid Approach, the latter being a Limit Equilibrium Approach incorporating Limit State Design principles.
2. The existing design codes/methods use different calculation models, global Factors of Safety and several Partial Factors for the analyses of the GRSSs.
3. No design code/method is known to have used the "True" Limit State Approach, i.e. deformation analysis on the basis of internal strain compatibility and force equilibrium.

4. The existing design codes/methods recognise that actual Actions are generally Multi-Stage Actions but treat them as Single-Stage Actions.

5. When designing for sustained loading plus short-term loading, the combined load is considered as a short-term load. This ignores the history of sustained loading which is likely to have great influence on the micro and macro structure and so the behaviour of the geosynthetics acting as reinforcements.

6. In the current codes/methods, which all use a Single-Stage Actions approach, the Reference Strengths for geosynthetics are defined either as a load for creep rupture or for a performance limit strain.

7. In the current codes/methods, Multi-Stage Actions, such as for sustained loading plus traffic loading, the Reference Strengths are defined either as a load for creep rupture or for a performance limit strain. For sustained loading plus seismic loading situations, the Reference Strengths are defined either as the short-term CRS strengths or a multiple of the Reference Strengths used for Single-Stage Actions.

8. The Partial Factors for both Single-Stage Actions and Multi-Stage Actions are determined by comparing the strengths of the geosynthetics 'before' and 'after' an event using short-term CRS tests and the Design Strengths for Single-Stage Actions and Multi-Stage Actions are calculated by dividing the Reference Strengths by the Partial Factors. The Partial Factors for soil-geosynthetic interactions are applied in some codes, for example BS8006 (1995). The values of these Partial Factors are not widely agreed.

9. It has been shown that the ISE approach may be used to characterise the load-strain-time temperature behaviours of the geosynthetics. For any Single-Stage Loading regime the ISE Capacity $[C]_t$ at any limiting strain has been shown to represent an Intrinsic Property of a geosynthetic. It changes with time in response to alterations of the micro or macro structures of the geosynthetic. Further, the components of the

ISE Capacity $[C]_t$ have been identified as the Recoverable ISE $[R]_t$ and the Locked-in ISE $[L]_t$. These ISE components indicate that the geosynthetics will behave differently under different Single-Stage Loading regimes, i.e. CRS and creep tests.

10. Partial Factors can be obtained using the ISE approach by comparing the ISE Capacity $[C_t]$ 'before' and 'after' an event. The Partial Factors so obtained have been shown to be time and strain level dependent. This means there cannot be any unique value for a Partial Factor, rather it should be chosen for particular time and strain level. Further, as short-term tests measure totally different changes of the ISE components than long-term tests, Partial Factors for long-term applications should not be determined by the CRS testing.

11. The current test protocols for the determination of the Partial Factors for geosynthetics do not simulate the actual field conditions. To date, no test protocol has been developed for determining the Partial Factors for geosynthetics under Multi-Stage Actions.

12. No Multi-Stage Load testing protocols for determining material strength under special loading conditions are given by any national or international standards.

13. The results from Combined Sustained-Short Term Loading test, simulating the sustained loading plus seismic loading conditions, interpreted using the ISE approach, reveals that the geosynthetics are capable of carrying a larger amount of additional short-term loadings than their long-term Reference Strengths suggests.

14. The amount of additional short-term load a geosynthetic can carry is unlikely to be the same over the whole design lifetime and is likely to decrease with time.

15. The current practice of considering a single value of the Design Strength for geosynthetics in order to design for earthquake loading might be unsafe.

16. In the present study, for designing against sustained loading plus earthquake loading, a Modified Material Properties Approach has been envisaged. Within this approach it is suggested that a Short-Term Loading Factor should be applied to the Limiting Strain to allow for a certain amount of Available Strain which would be developed by the application of additional short term loading.

Recommendations for Future Research

1. The test protocols for determining the Partial Factors for geosynthetics simulating the actual field conditions should be developed. These test protocols should make provision for determining the Partial Factors under Multi-Stage Loading regimes. Further research is necessary to obtain more understanding of the parameters involved.

2. In order to obtain more confidence about the Sustained Load Equivalent for sustained loading plus short-term loading, tests require to be carried out for different load levels and for different loading times. A wider range of geosynthetics also requires to be tested.

3. The Modified Material Properties Approach for design against sustained loading plus seismic loading should be developed. The values of the Short Term Loading Factor should be identified on the basis of more understanding of the changes of the ISE components and the Stiffness Properties of the geosynthetics under the sustained loadings applied over different periods of time.

4. To understand the response of the ISE components of geosynthetics and to identify the changes in the Stiffness Properties of the geosynthetics under combined sustained-short term loadings, tests should be carried out for different combinations of loadings. One example of these combinations may be where the duration of a particular sustained loading will be varied and then the short-term loadings will be applied until the specimen ruptures or reaches a particular limiting strain.

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APPENDIX

APPENDIX

RECOVERY TEST

A1 LOADING SCHEME

The loading scheme for the recovery test is shown in Fig. A1. In this test, the load was applied instantaneously, maintained over a period of time and then unloaded fully to measure the amount of recoverable strain and to observe the behaviour of the time dependant locked-in strain. The applied loads were varied from 5 kN/m to 65 kN/m in increments of 10 kN/m. For lower load levels the duration of loading was between 30 minutes and 1 hour and for the highest load level the duration of loading was 30 seconds only, being governed by the rupture of the material.

A2 TEST MATERIAL, SAMPLING AND CONDITIONING

Uniaxial Geogrid B described in Table A1 was used for testing. The samples were taken from the roll as described in BS6906: Part 1 (1987). First two turns were discarded and 100mm from either edge were avoided. None of the samples was taken from same row and column.

The geogrid test specimen contained five ribs within the width and one cross-member along the length, excluding cross-members by which the sample was held in the clamps. Of the five ribs, two outer ribs were cut at least 10mm away from any node. Therefore, the effective numbers of ribs subjected to tensile force were three. The dimensions of the geogrid test specimen are shown in Fig. A2.

Samples were conditioned for 24 hours at 20 ± 2°C temperature and 65 ± 2% relative humidity before commencing the tests.

A3 TEST APPARATUS AND METHODOLOGY

A special testing rig was developed at the University of Strathclyde for this test, Fig. A3 and Plate A1. A calibrated load cell was placed in the central position of this bar. The top clamp for holding the specimen was attached to the load cell via a mild steel saddle. A specimen of uniaxial geogrid B was then slide from one side through the jaw-shaped gap in the top clamp. The clamps used were as recommended by the manufacturer, Fig. A4. The bottom clamp was also slide through the bottom part of the hanging specimen of the geogrid. Two calibrated LVDTs were attached to either side of the top clamp for measuring deformations. The load cells and the LVDTs were then connected to a programmable data logger. This data logger was further connected to a 486DX computer to facilitate transfer of data from the data logger to the hard disk. The data logger was programmed to record data at an interval of 0.2 seconds for the first minute and then at an interval of 1.0 second.

The loading was then placed on a forklift in the form of 25 kg lead bars. The arrangement of lead bars consisted of a vertical screw rod having a stainless steel saddle connected to the bottom clamp. The data logger was then started and the dead loads were applied to the specimen instantaneously.

For unloading, it was essential to ensure instantaneous recovery of the material and free fall of the load without disturbing the specimen. To these ends, a release mechanism was developed, Fig. A5. This mechanism consisted of a connection thread which was cut to remove the loading instantaneously. To prevent the clamped specimen from shooting back, thus disturbing the test and damaging the LVDT's, a guide mechanism was developed, Fig. A6.

After unloading, data were recorded for up to 1,000 hours. These data were then copied to a floppy disk, which were analysed and plotted using Microsoft Excel and Microcal Origin respectively, see Fig. A7.

The immediately recoverable strain component [R] (elastic) was determined using the procedures given in Fig. A8.

Uniaxial Geogrid B			
Mass per Unit Area [g/m^2]	Polymer Composition	Structure	
		Micro	Macro
700	High Density Polyethylene	Semi-crystalline	Uniaxially stretched punched sheet

Table A1 Properties of geogrid B

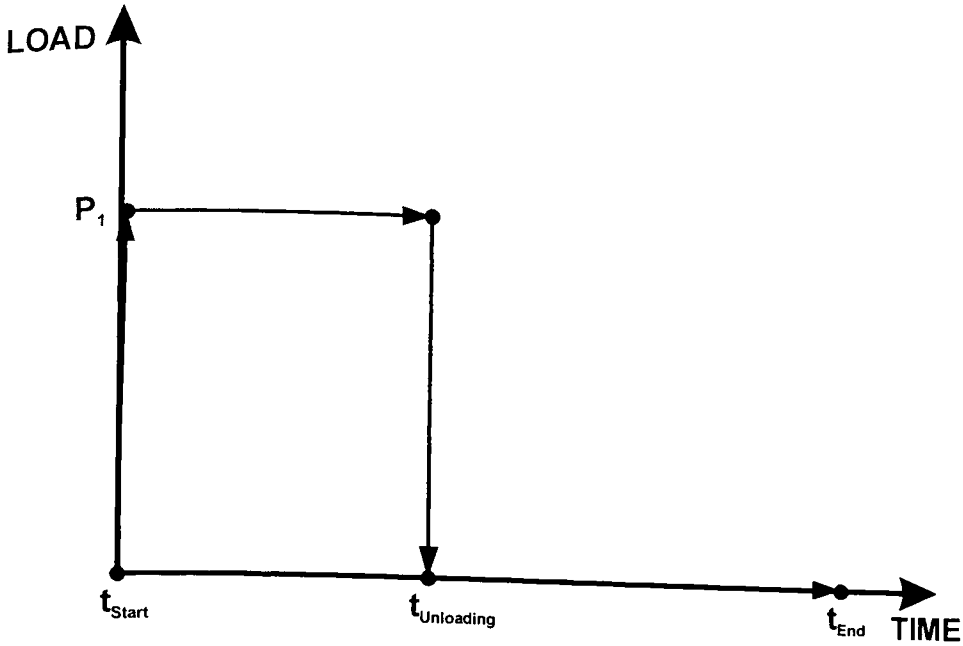


Figure: Load sequence

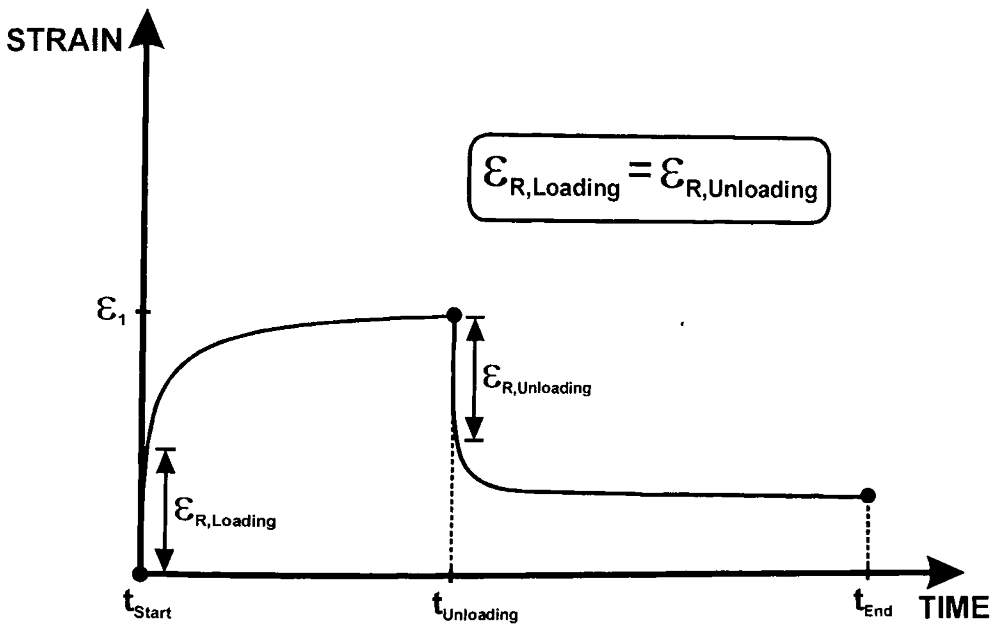


Figure: Strain response

Figure A1 Loading scheme and expected strain response

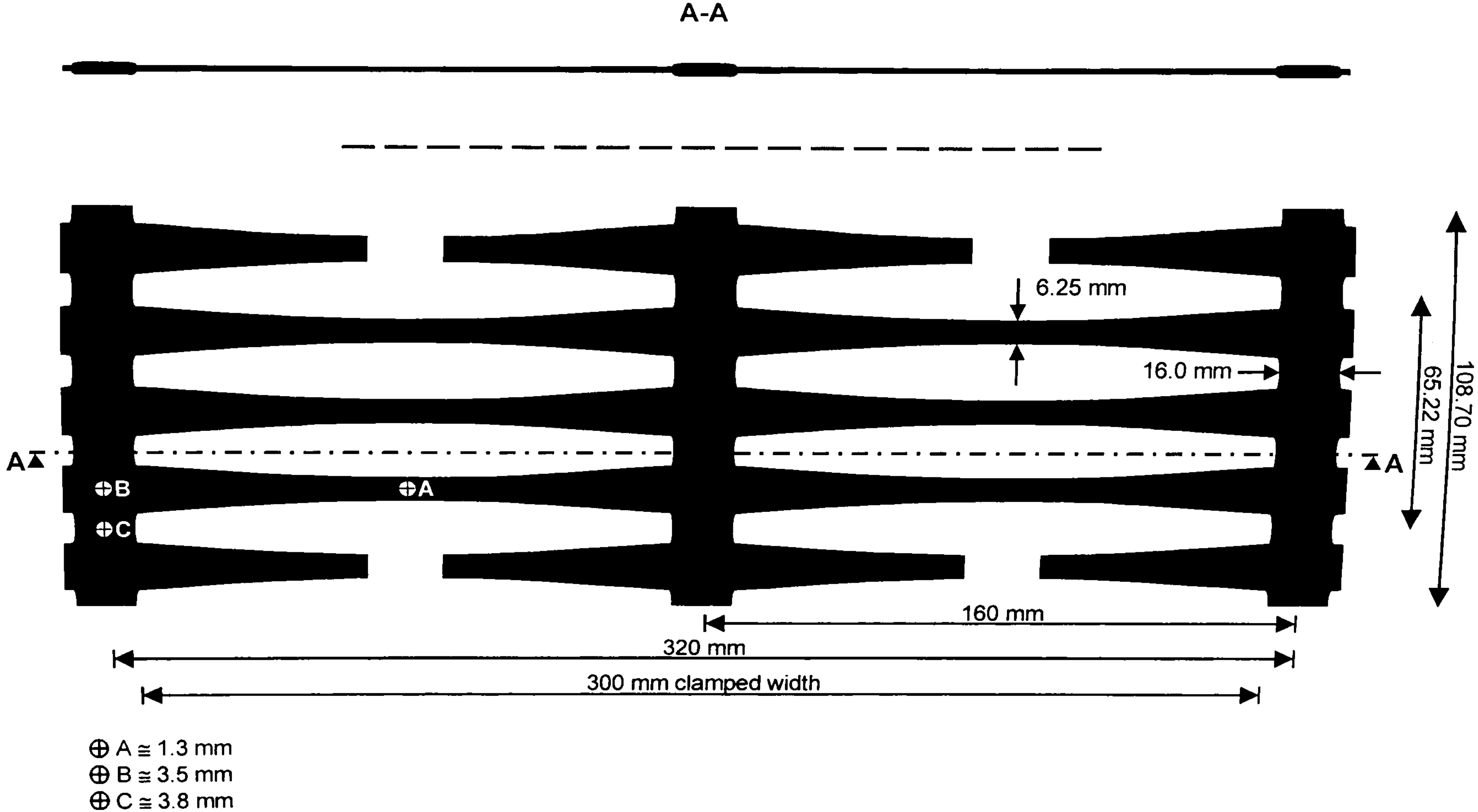
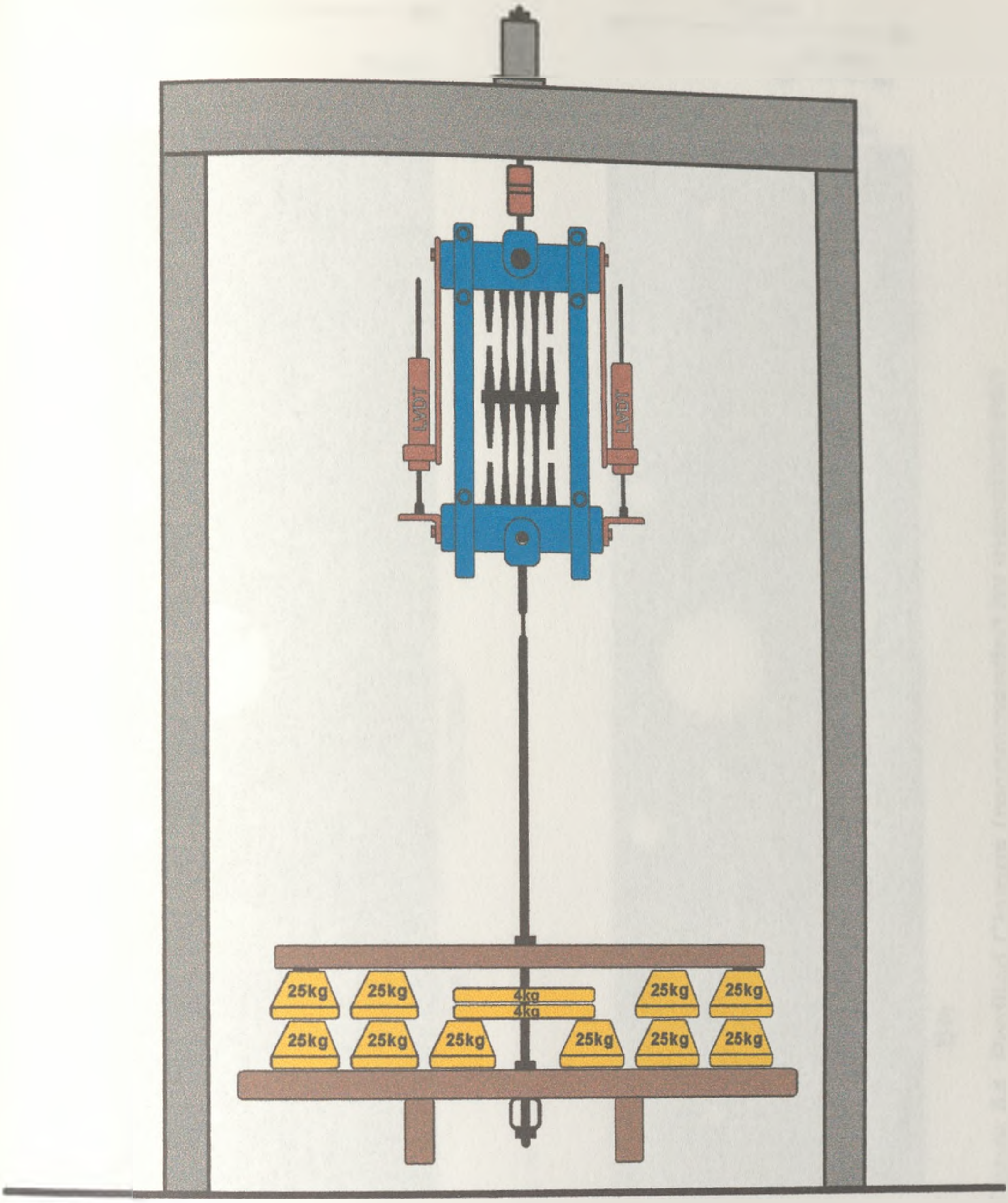


Figure A2 Prepared geogrid B specimen



Legend:

- Test Rig
- Loadcell & Extensometers
- Clamps & Guide mechanism
- Specimen
- Sustained Load
- Load Table

Figure A3 Scheme of test apparatus used in recovery tests

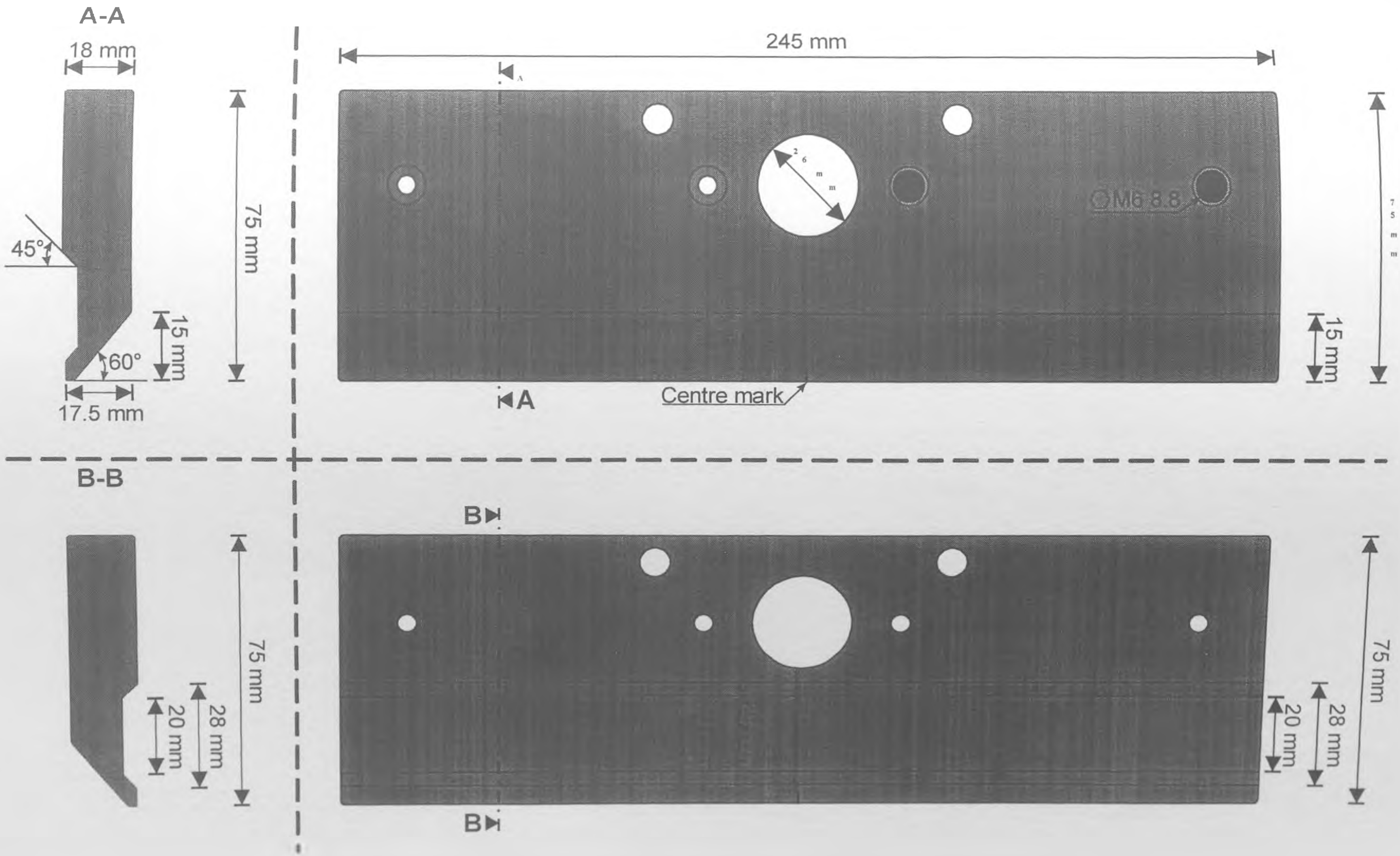


Figure A4 Profiled Clamps (recommended by manufacturer)

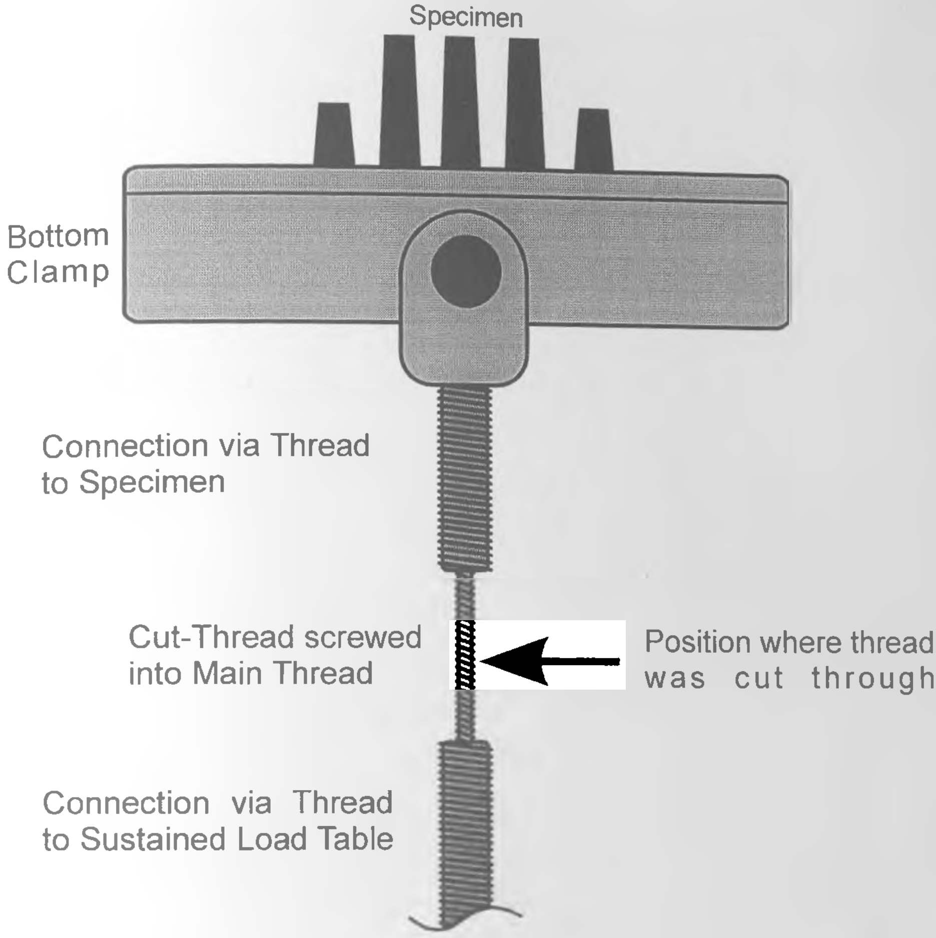
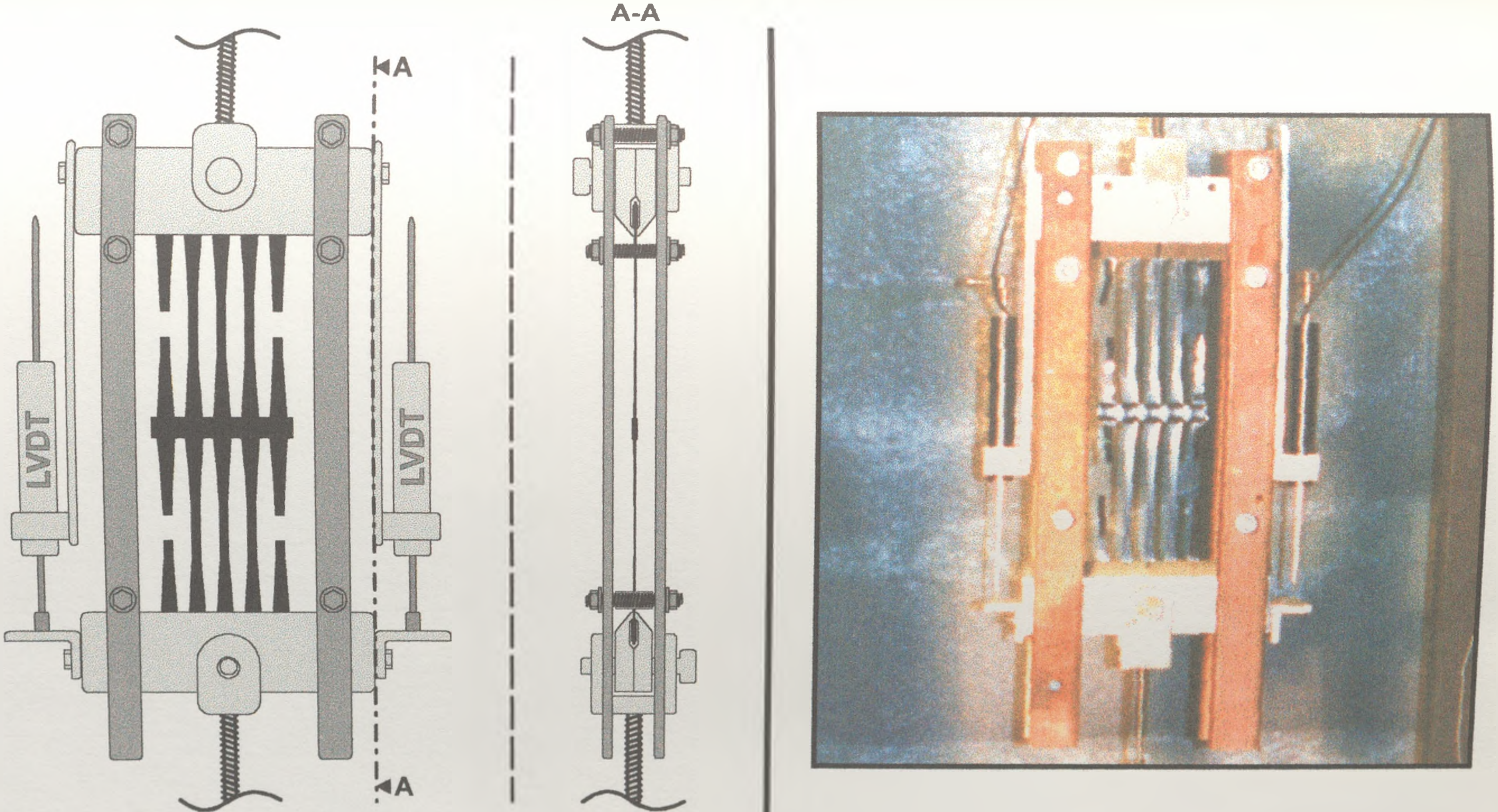


Figure A5 Scheme of release mechanism



(a) Scheme

(b) Plate

Figure A6 Unloading guide mechanism

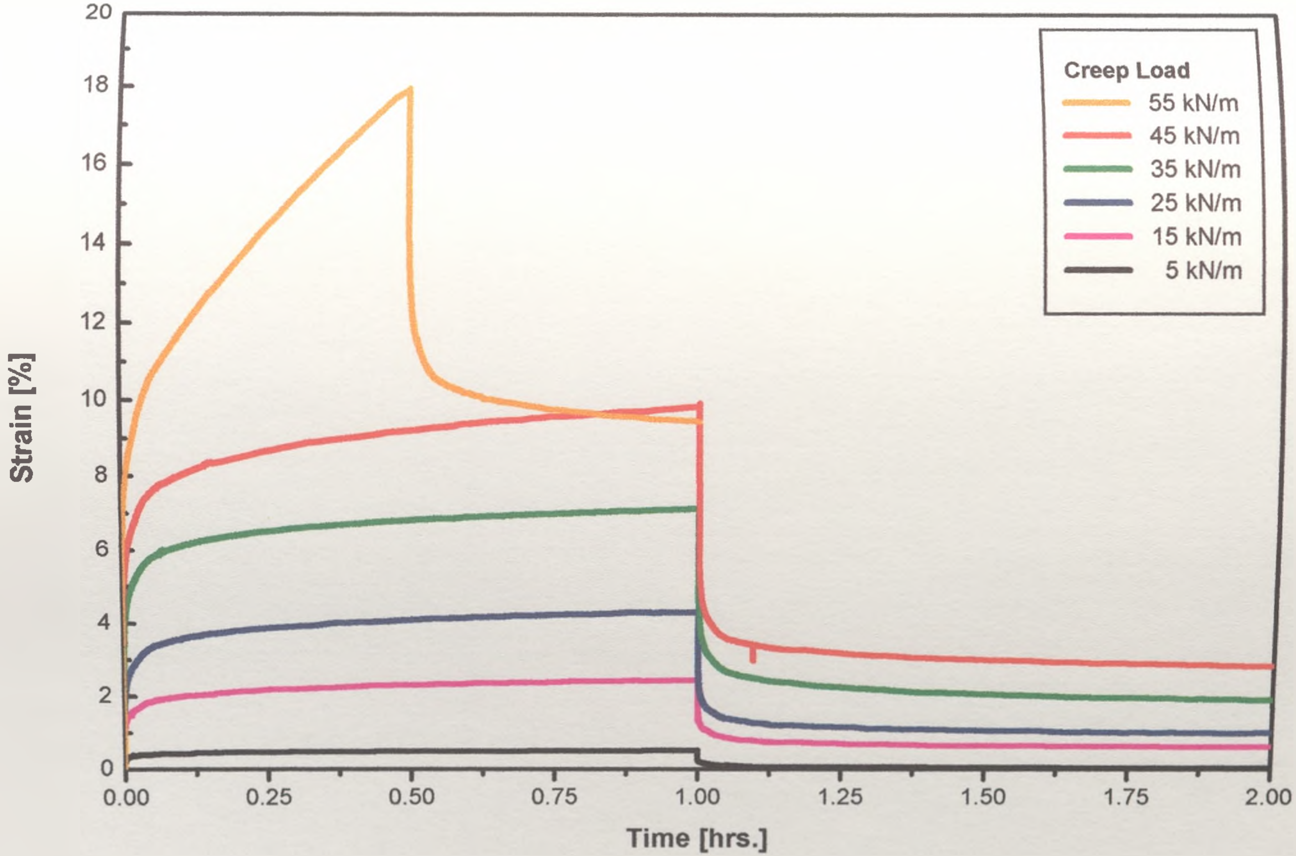


Figure A7 Creep-recovery test results for geogrid B at 20°C

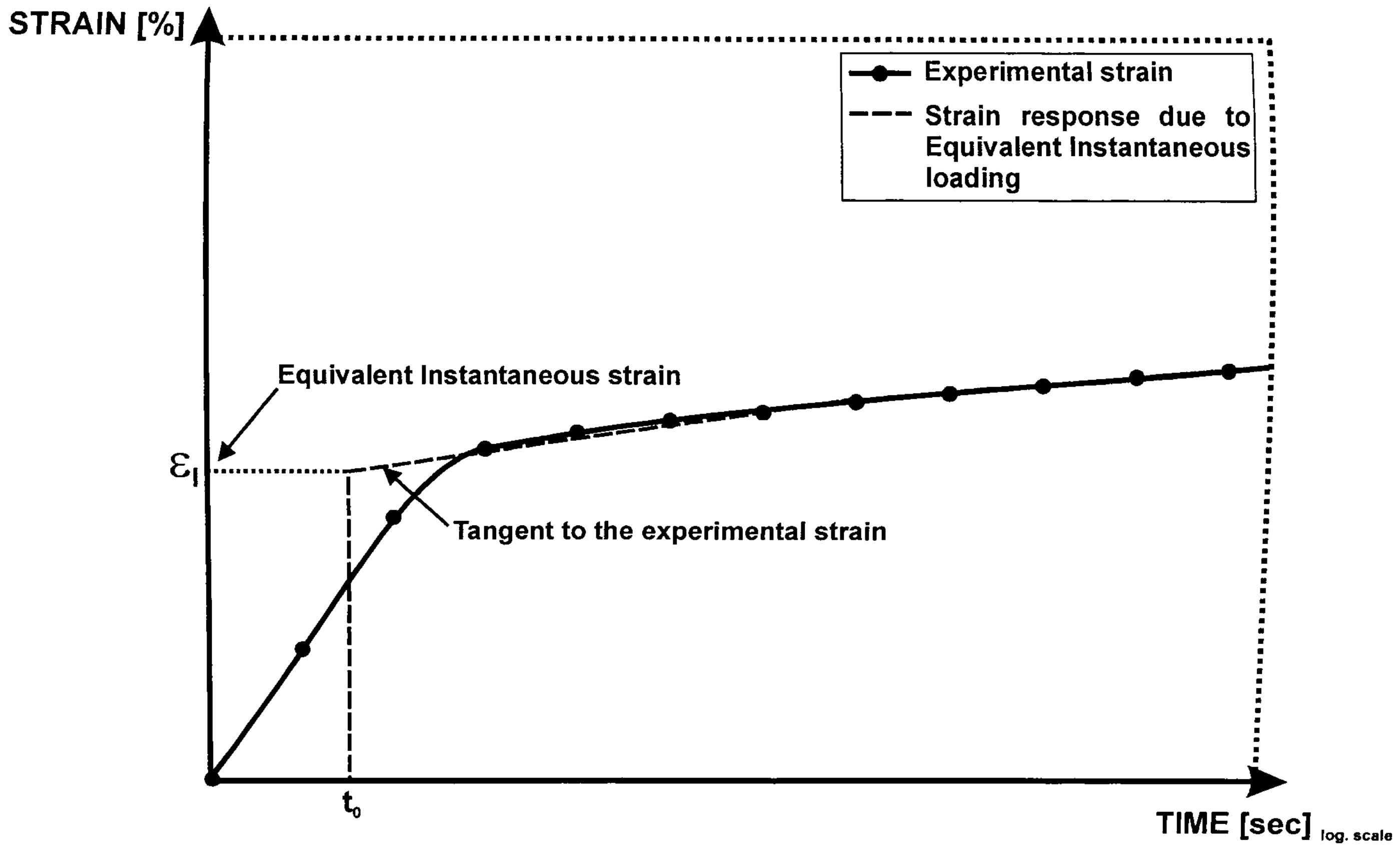


Figure A8 Method of obtaining equivalent instantaneous loading strain

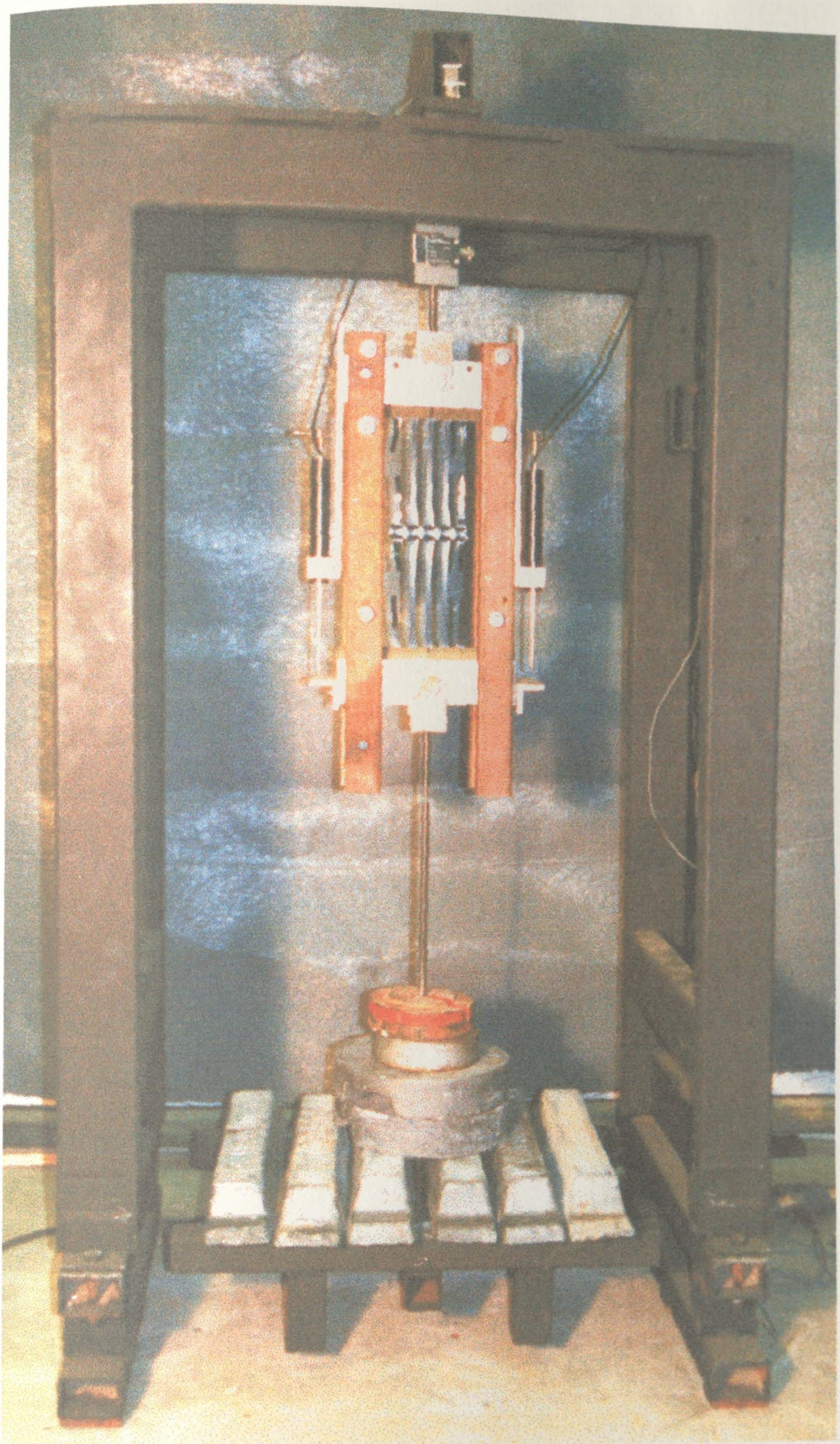


Plate A1 Recovery test apparatus

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	AASHTO (1997)	DIBt (1998)
Sliding	1.5	1.5
Overturning	2.0	-
Bearing	2.5	2.0
Rupture	1.2 - 1.5	-
Pullout	1.5	2.0

Table 3.1 Factors of Safety in different design methods

Product type:
Uniaxial geogrid B

Design Codes / Methods	**Design Strength** [kN/m]
DIBt (1998)	17.32
BS 8006 (1995)	26.17
TBW (1998)	20.61

Table 4.1 Design strengths according to the design codes and methods

Product Type	Mass Per Unit Area [g/m²]	Polymer Composition	Micro Structure	Macro Structure
Uniaxial geogrid A *	971.6	High density polyethylene	Semi-Crystalline	Uniaxially stretched punched sheet
Biaxial geogrid *	320.6	Polypropylene	Semi-Crystalline	Biaxially stretched
Woven geotextile **	120	Polypropylene	Semi-Crystalline	Woven tapes
Melt bonded non-woven geotextile **	140	67% Polypropylene, 33% Polyethylene	Semi-Crystalline	Non-woven melt-bonded continuous drawn filaments
Needle punched non-woven geotextile **	210	Polyester	Semi-Crystalline	Non-woven needle punched continuous drawn filaments
Uniaxial geogrid B ***	700	High density polyethylene	Semi-Crystalline	Uniaxially stretched punched sheet

Note:

* Yeo (1985); ** Kabir (1984), *** Tensar (1998)

Table 5.1 Description of various geosynthetics

Product Type	Limiting Strain Conditions	Intercept (i) = $[R]_{t0}$ [kN/m]×[10^{-3}]	Slope [S_t] ×[10^{-3}]
Uniaxial geogrid A	2	263.3	-61.6
	5	-	-59.6
	10	-	-55.5
Biaxial geogrid	2	157.8	-134.1
	5	-	-134.8
	10	-	-122.6
Woven geotextile	2	13.8	-112.6
	5	85.2	-119.4
	10	297.8	-111.7
Melt bonded non-woven geotextile	2	7.5	-157.6
	5	46.9	-146.9
	10	-	-131.6
Needle punched non-woven geotextile	2	4.1	7.3
	5	26.6	-0.5
	10	107.6	-12.1
Uniaxial geogrid B	2	243.3	-50.8
	5	1556	-54.2
	10	-	-54.3

Table 5.2 Intercept and slope values for various geosynthetics

FIGURES

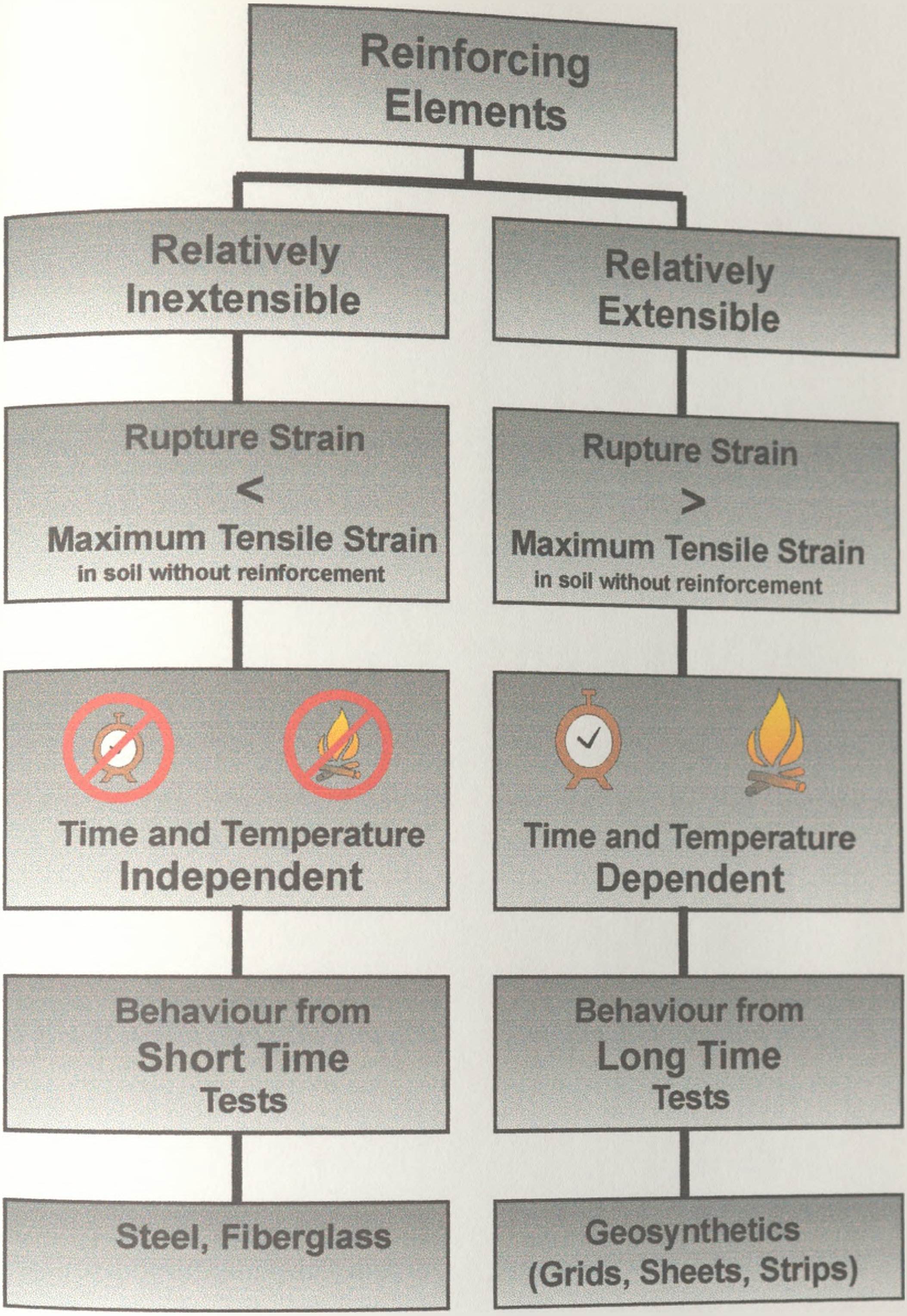


Figure 2.1 Classification of reinforcing elements after McGown et al (1978)

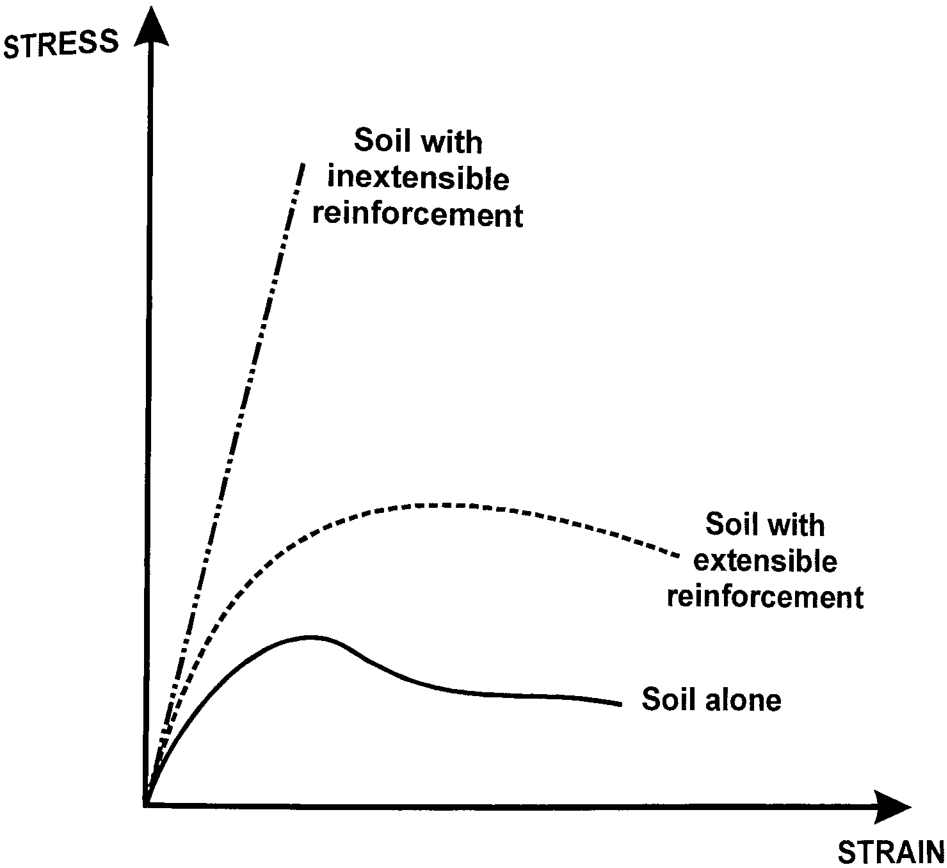


Figure 2.2 Stress-strain behaviour the soil-reinforcement composite

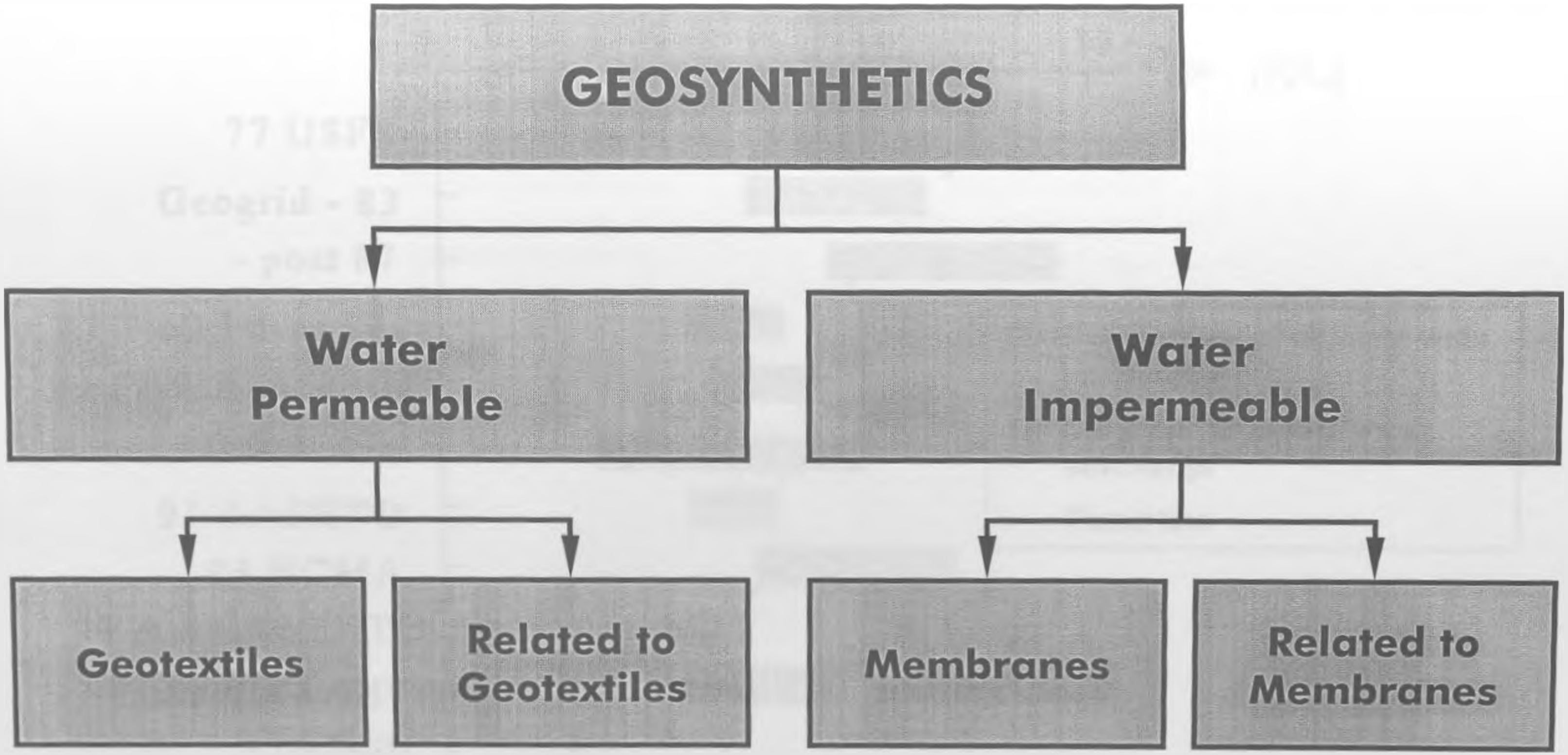


Figure 2.3 Classification of Geosynthetics

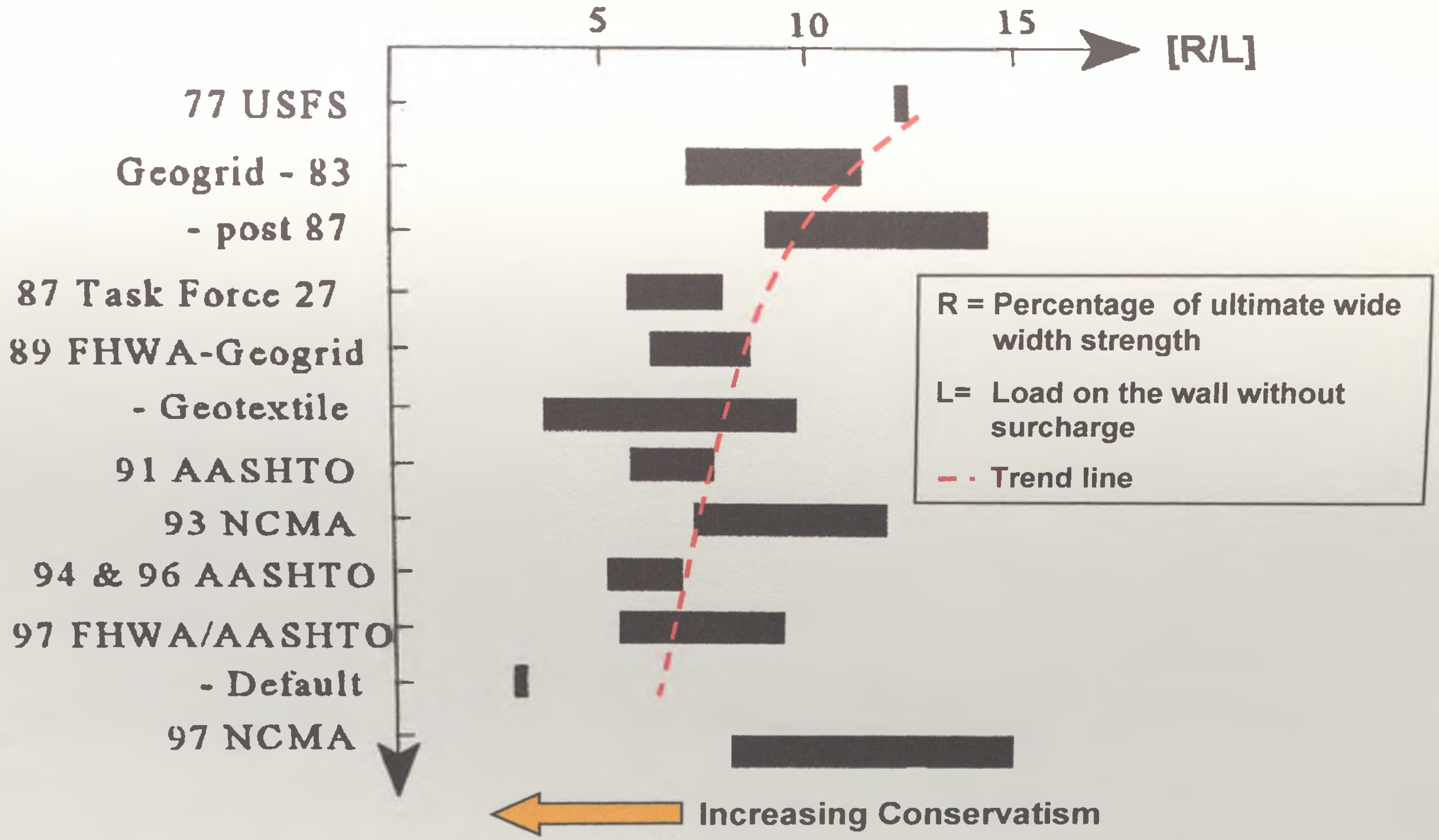


Figure 2.4 The conservatism in the design methods after Berg et al (1998)

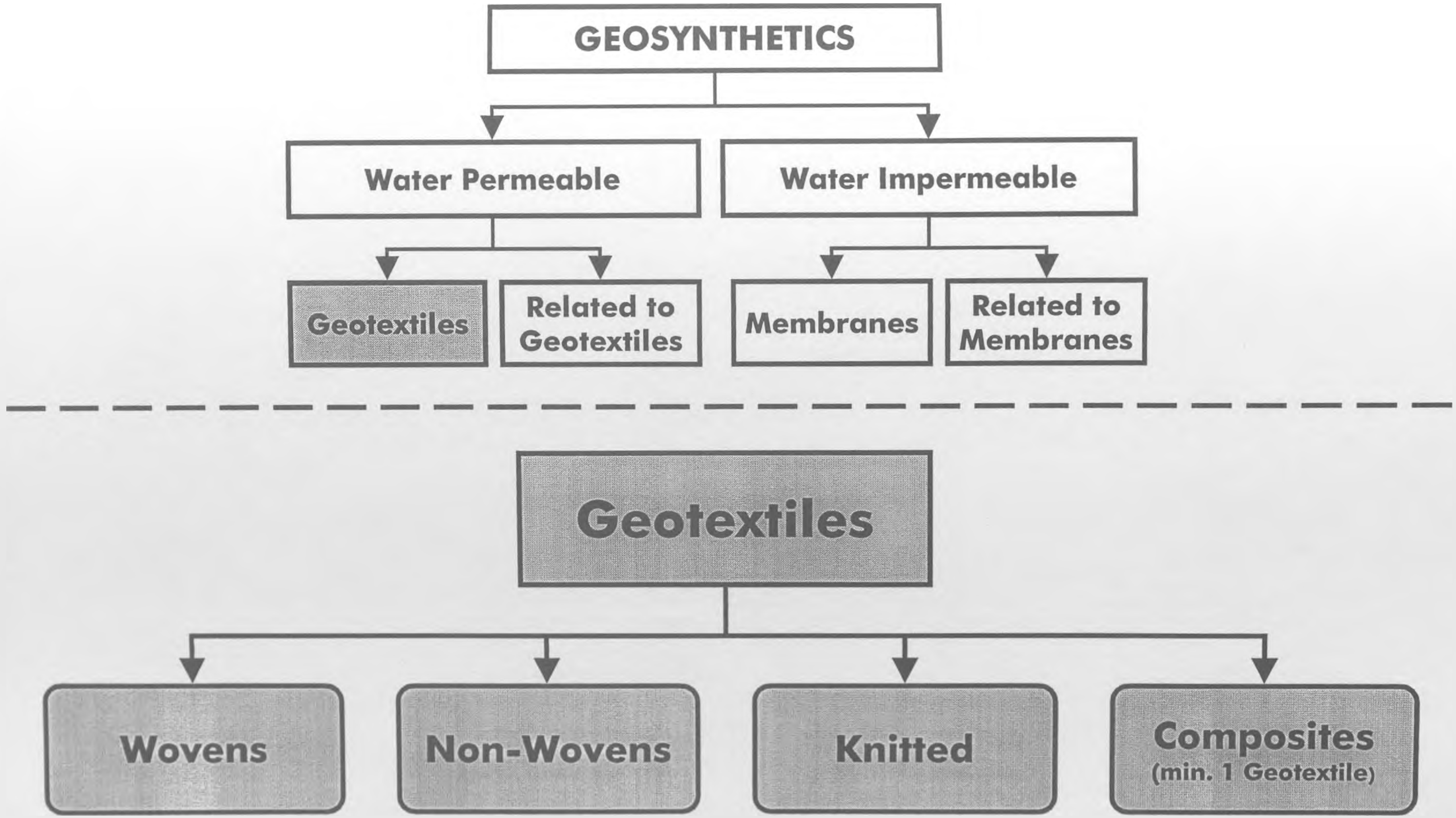


Figure 2.5 Classification of Geotextiles

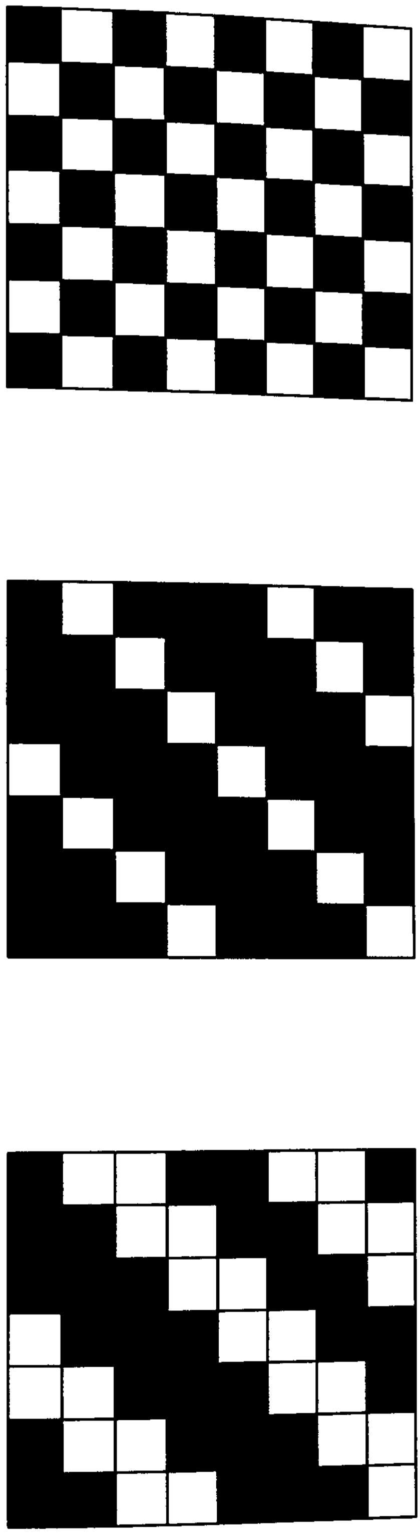


Figure 2.6 Weave patterns of geotextiles

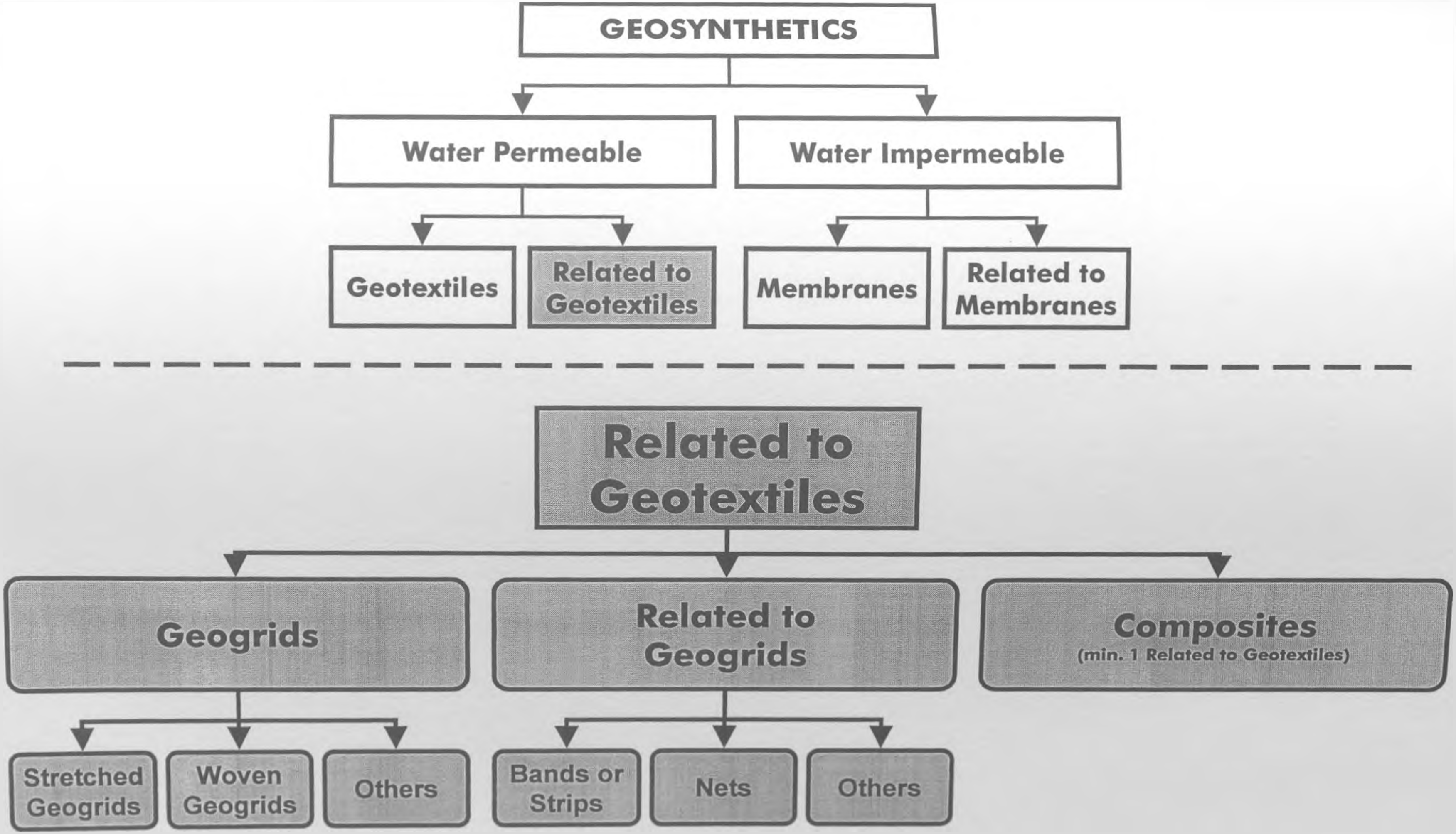


Figure 2.7 Classification of Products Related to Geotextiles

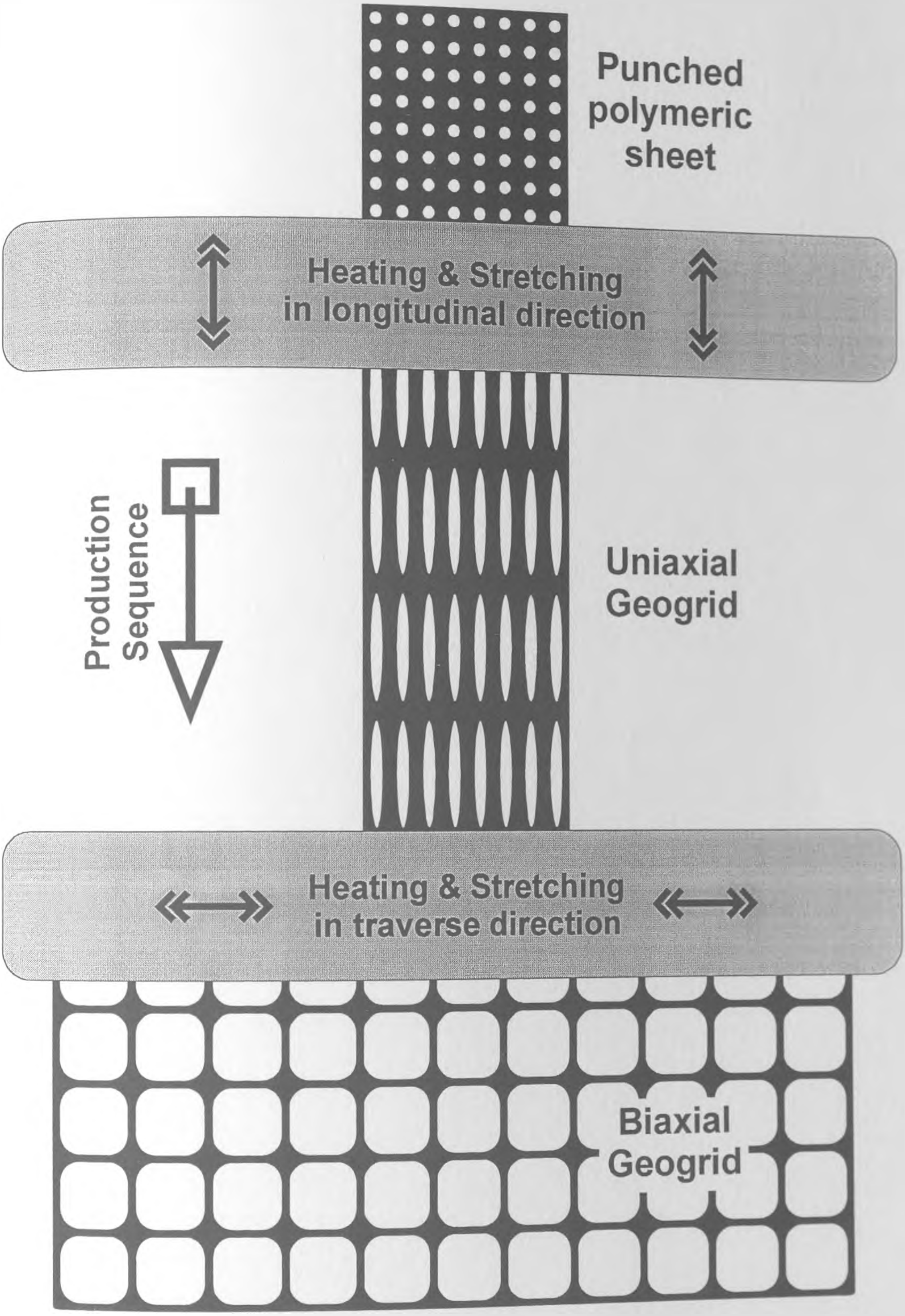


Figure 2.8 Geogrids with integral junctions - the production sequence

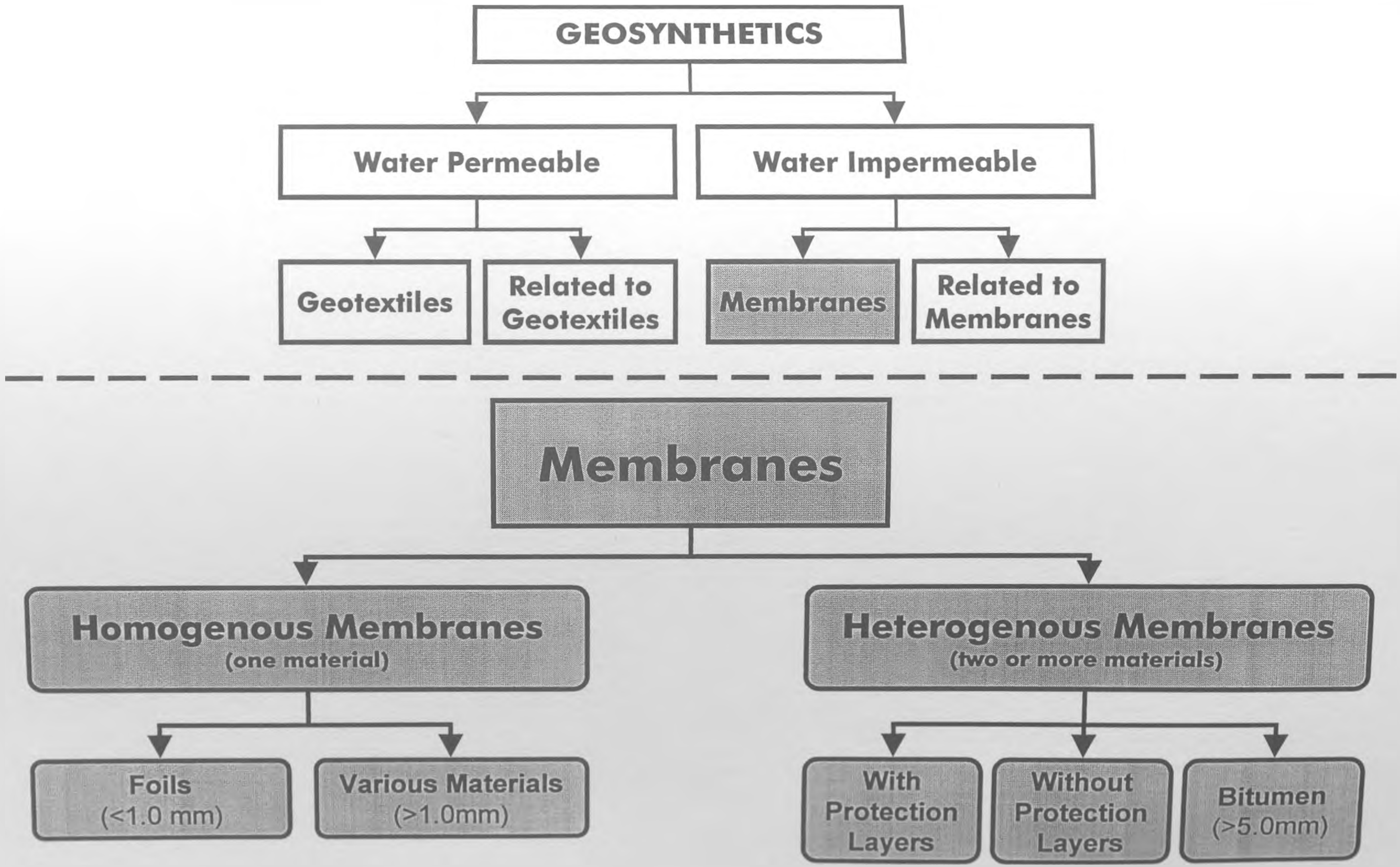


Figure 2.9 Classification of Membranes

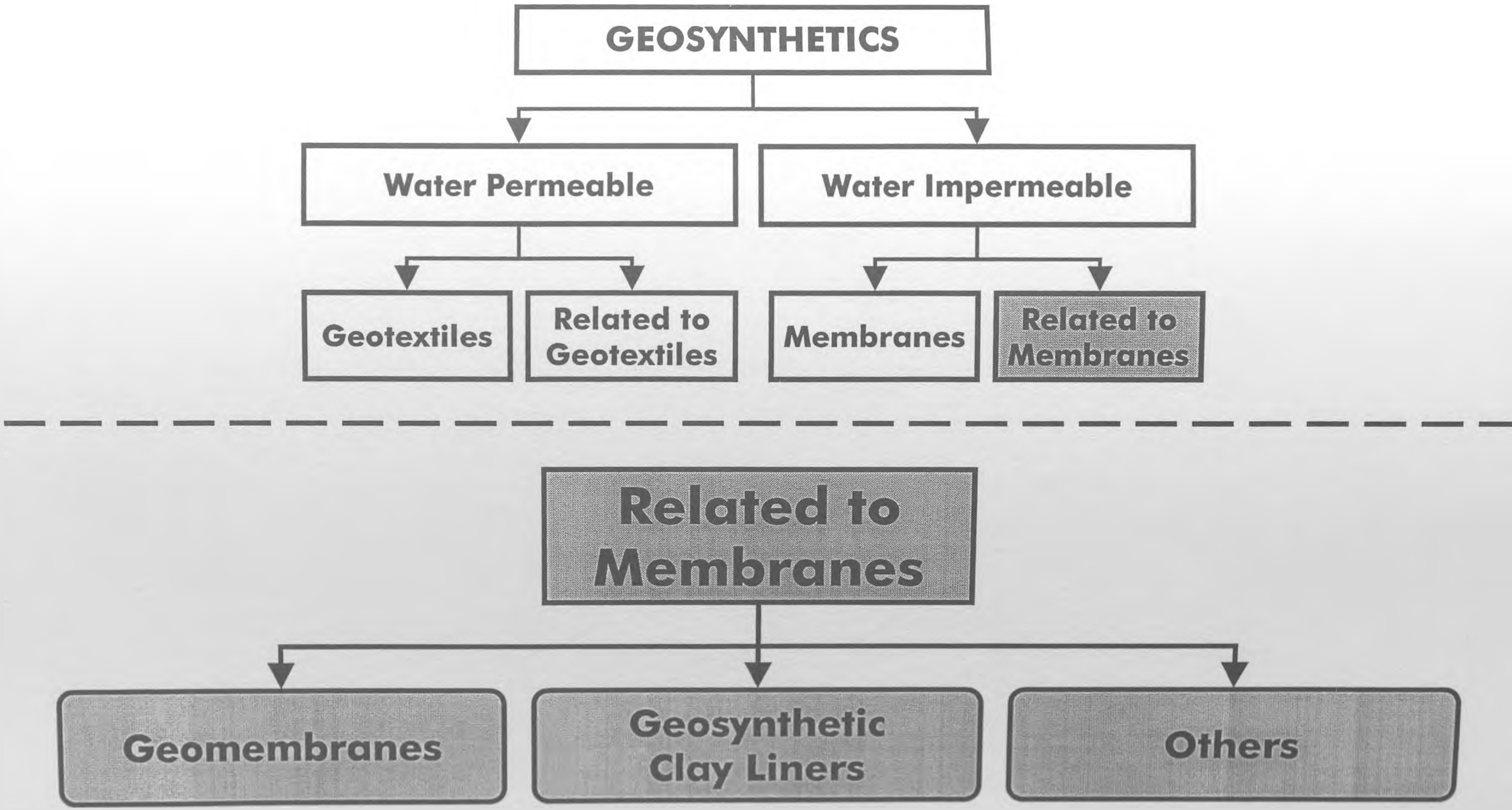


Figure 2.10 Classification of Products Related to Membranes

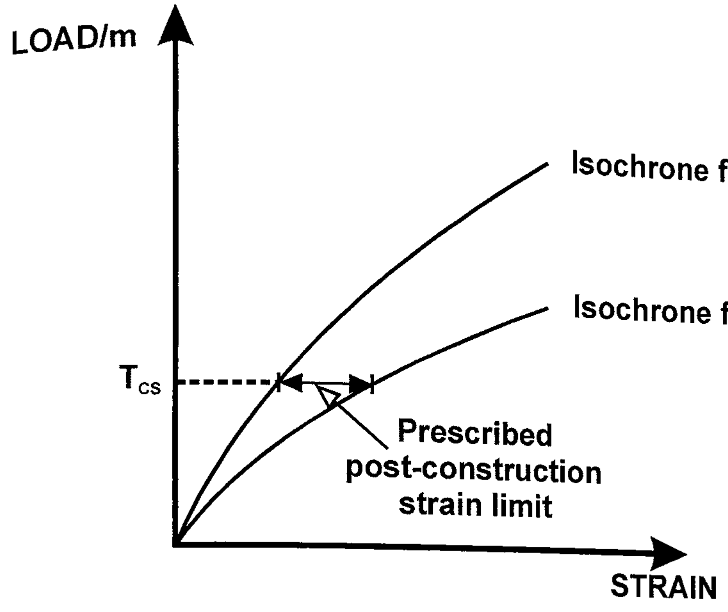


Figure 3.1 Assessment of SLS base strength T_{cs} after BS 8006 (1995)

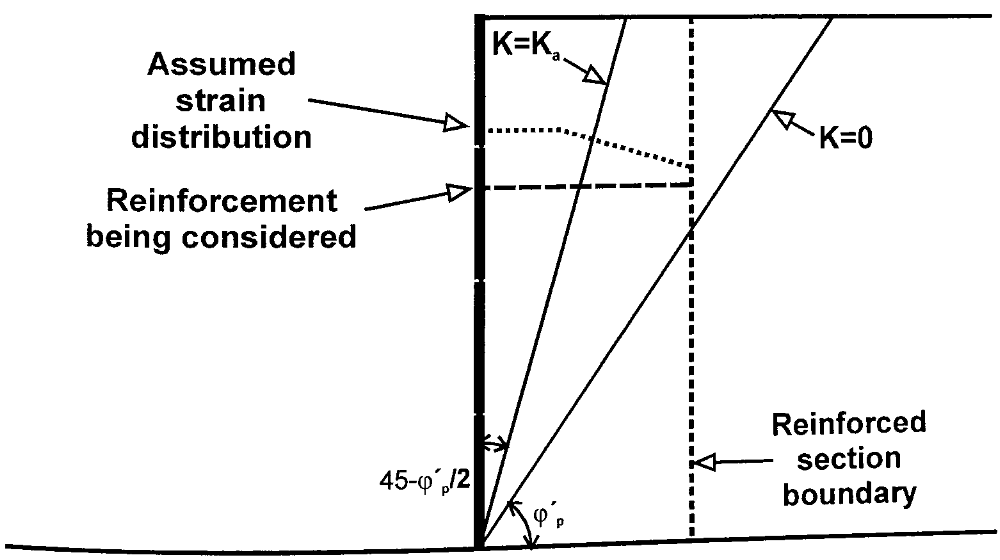


Figure 3.2 The $K_a \times \sigma_v$ check

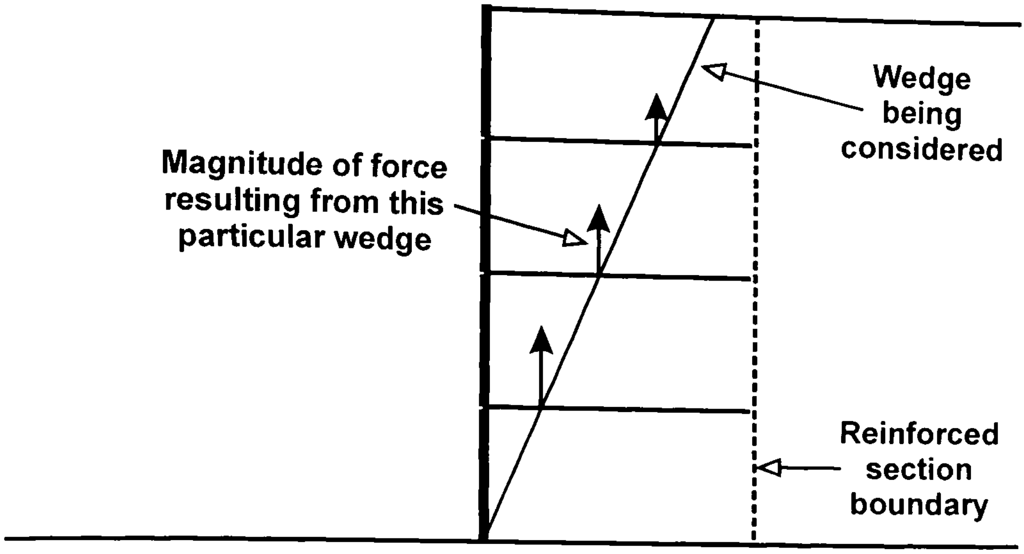


Figure 3.3 The force distribution in the wedge check

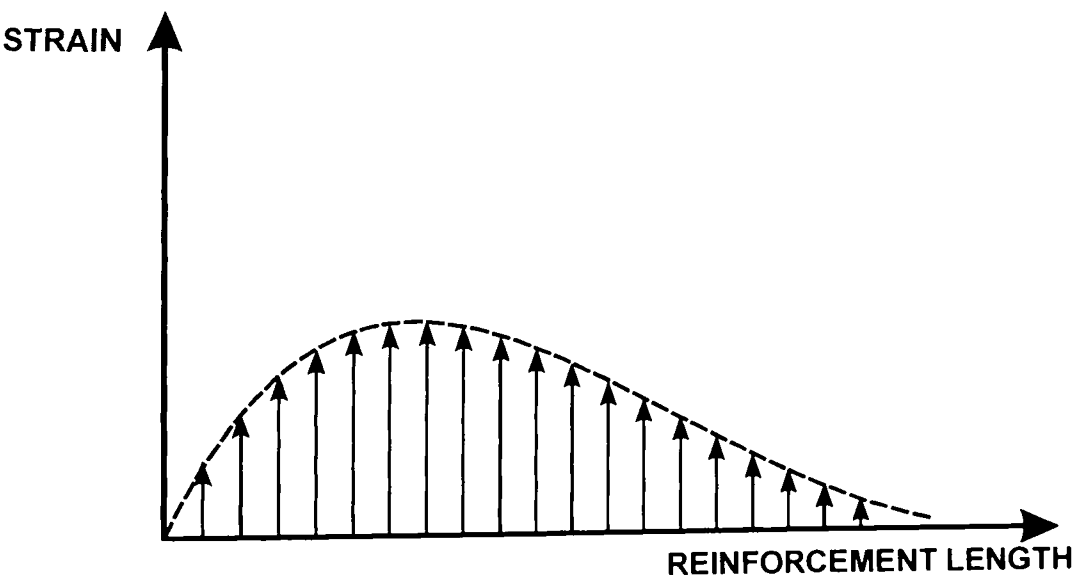


Figure 3.4 The strain distribution in the wedge check

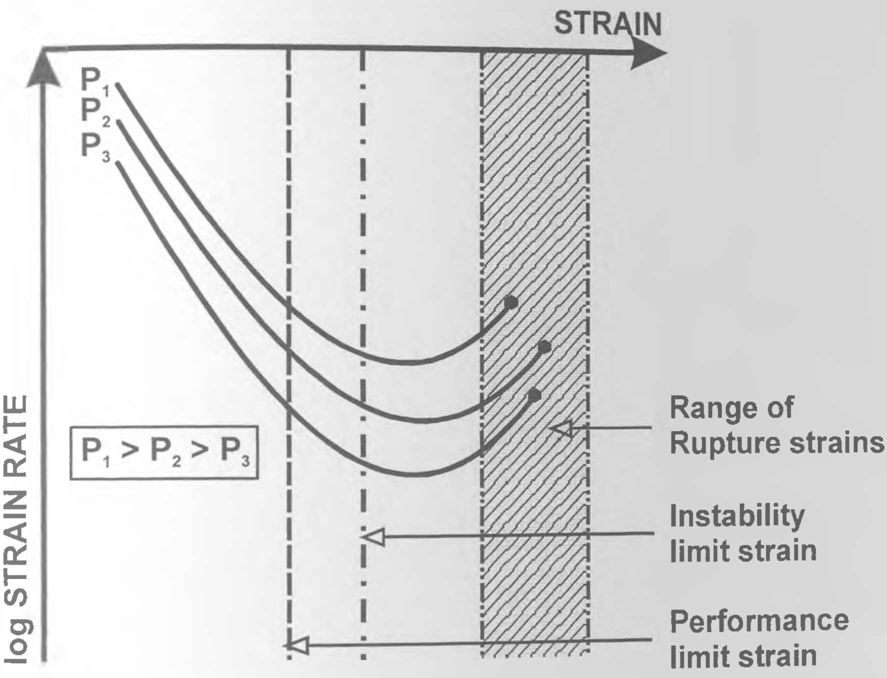


Figure 3.5 Typical Sherby-Dorn plot

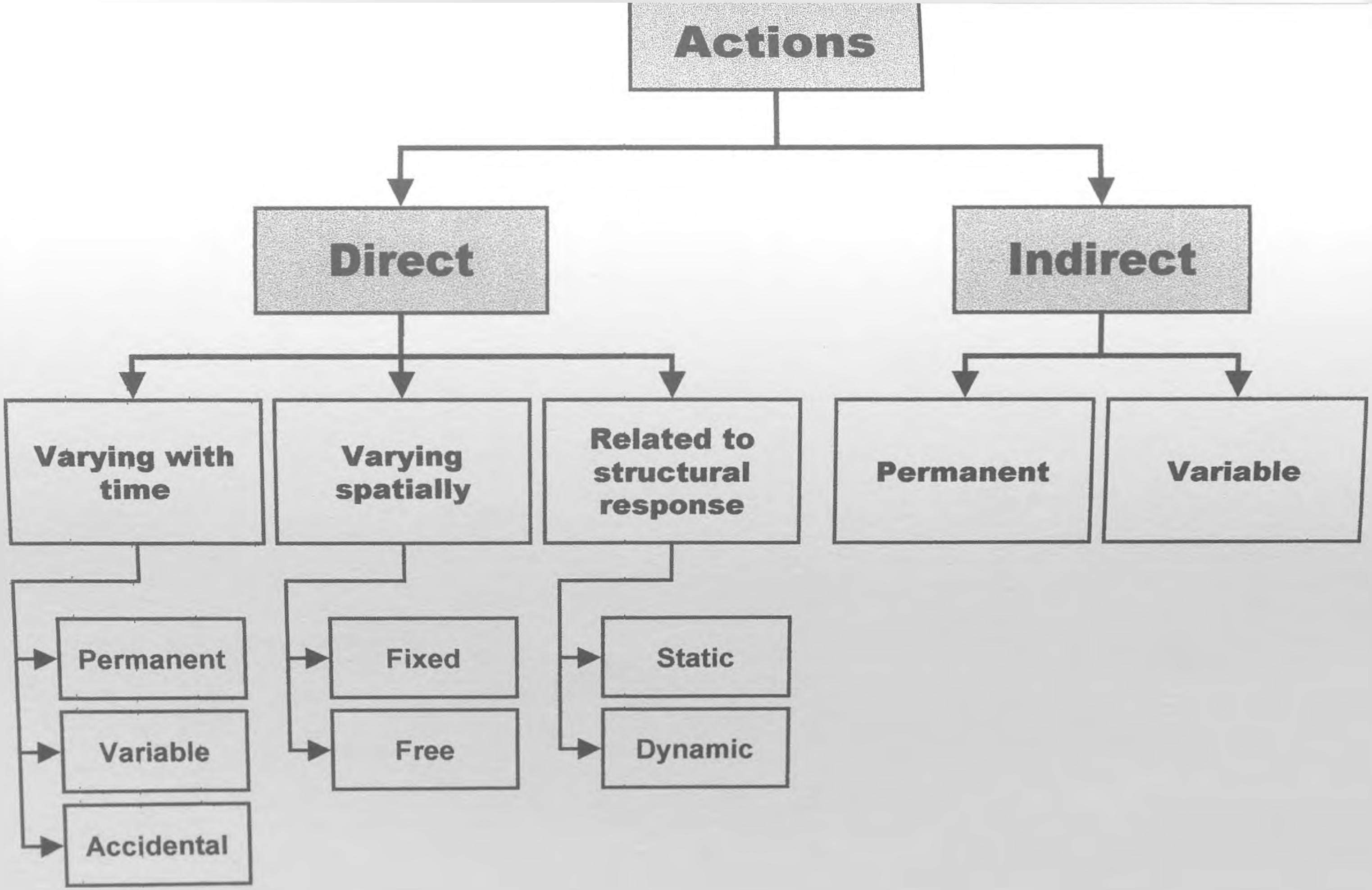


Figure 4.1 Classification of Actions according to Eurocode 7

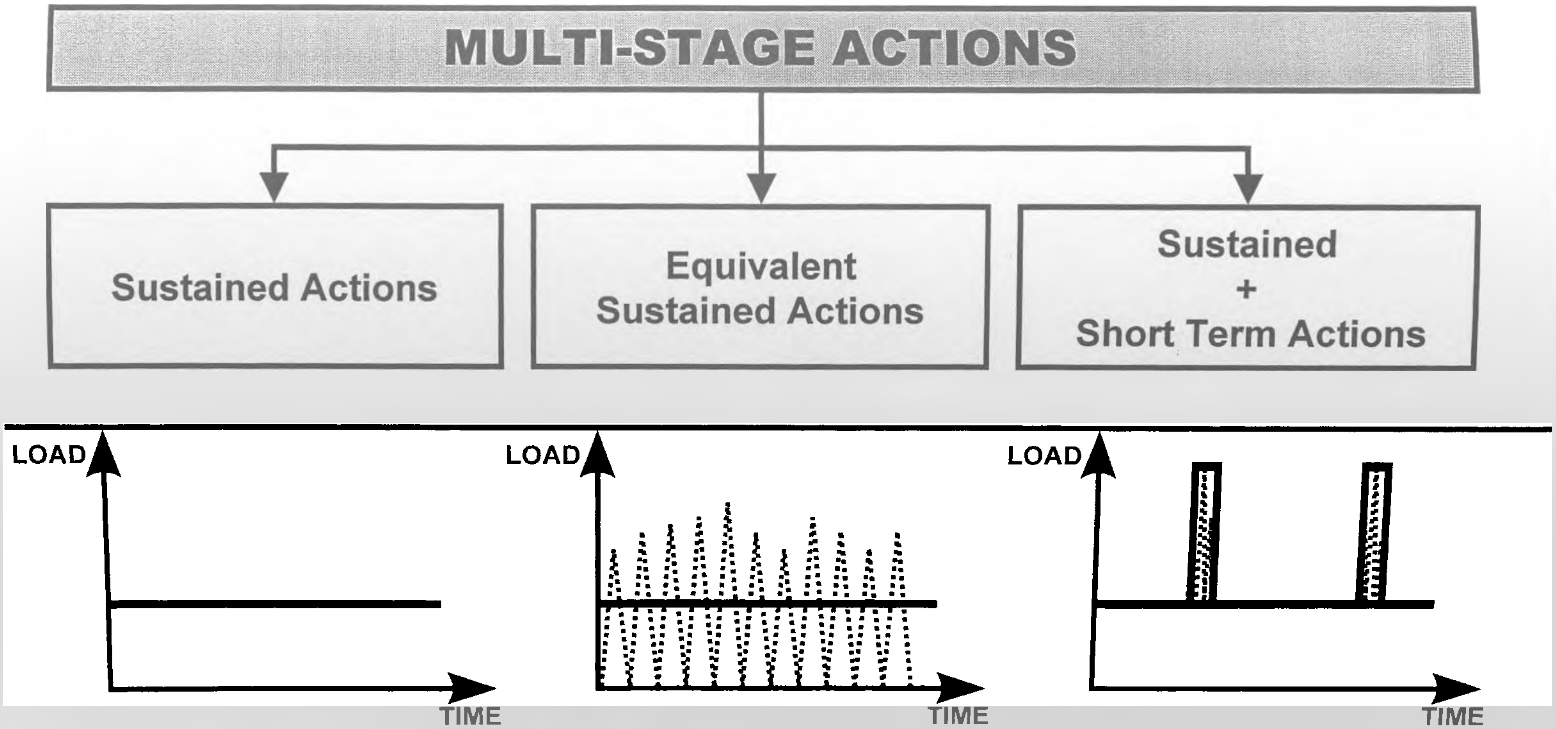
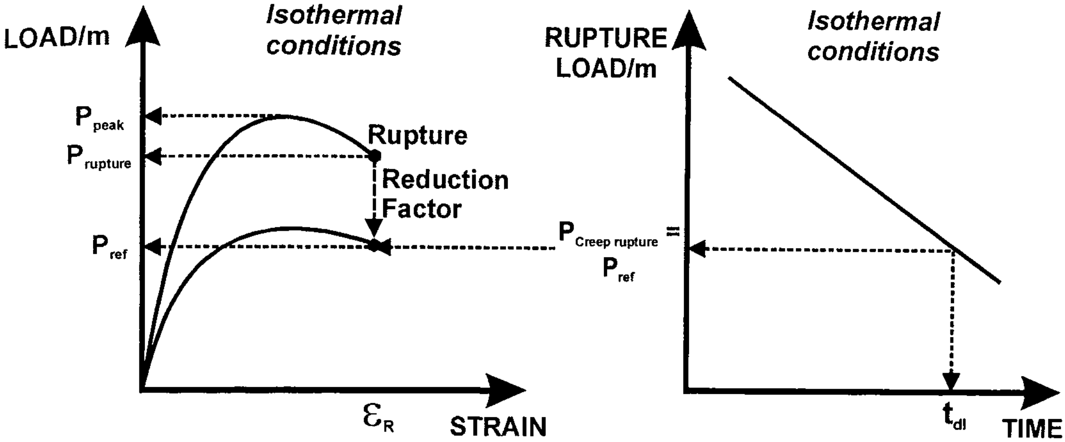


Figure 4.2 Suggested classification of Actions in critical groups



(a) Based on CRS test data (b) Based on creep rupture data

Figure 4.3 Definitions of reference strength based on creep rupture

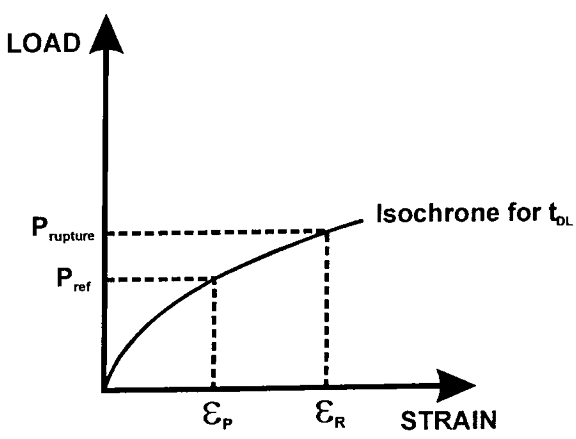
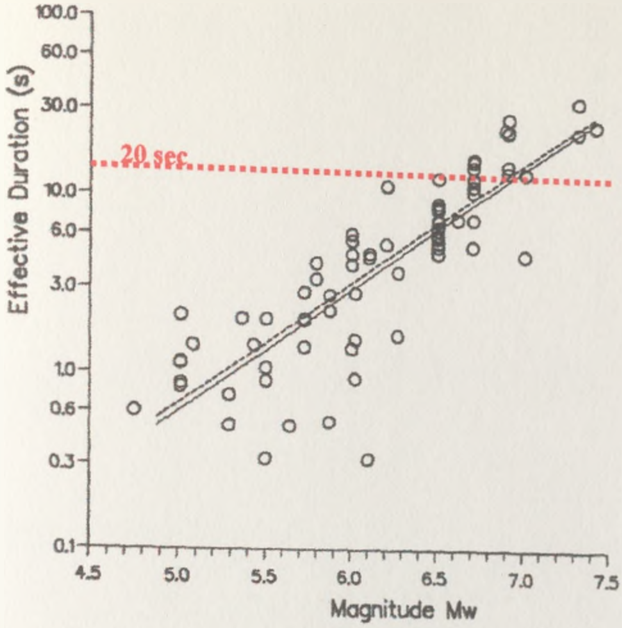
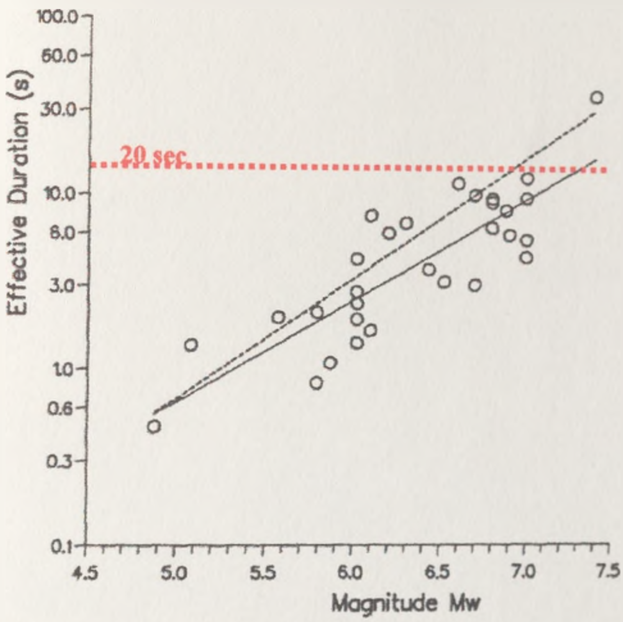


Figure 4.4 Definition of a reference strength based on a limiting strain



(a) Soil sites



(b) Rock sites

Figure 4.5 Effective Durations versus Moment Magnitude at distances of less than 10km from Epicentre

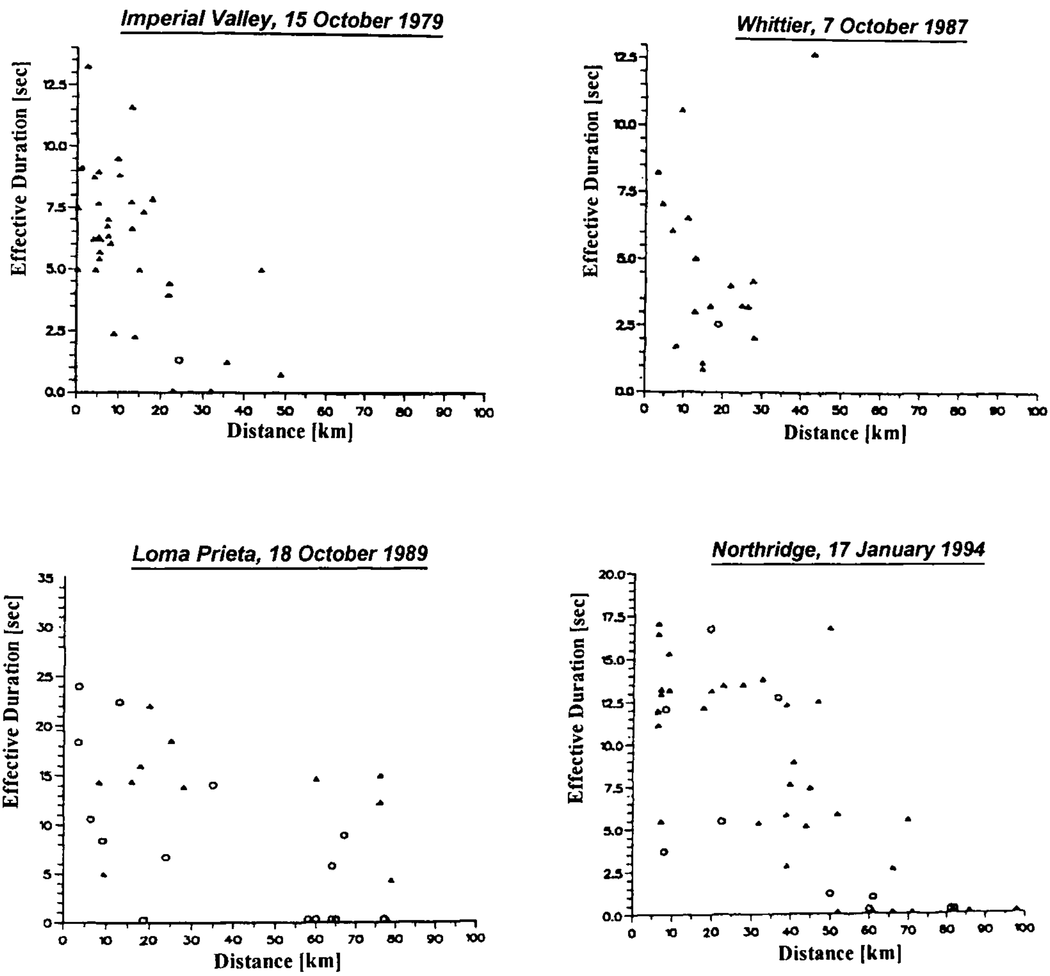
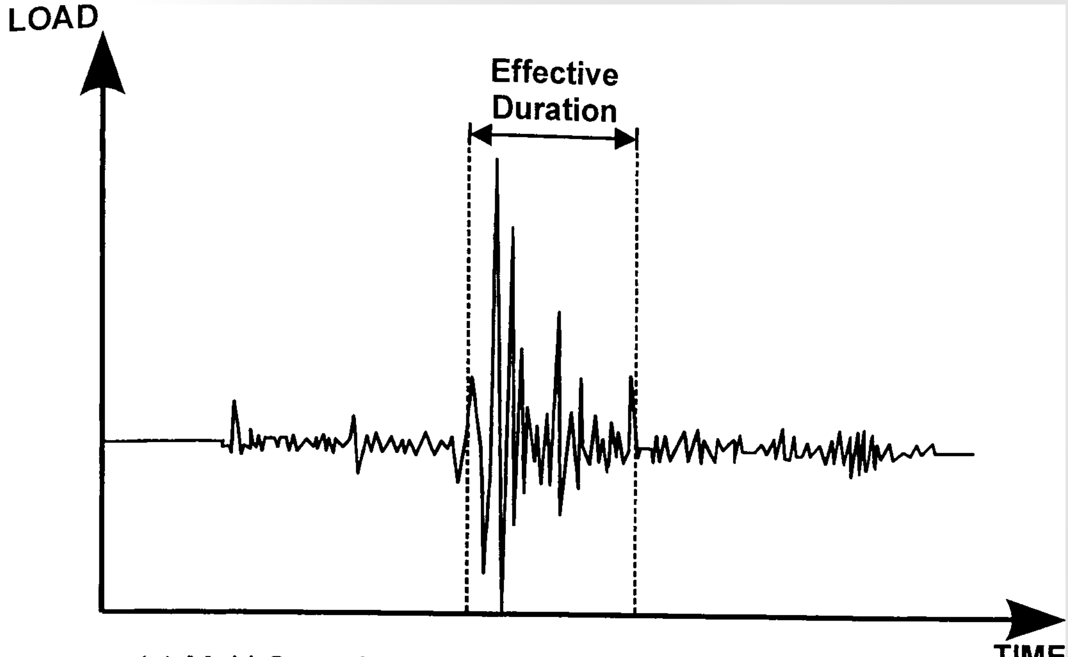
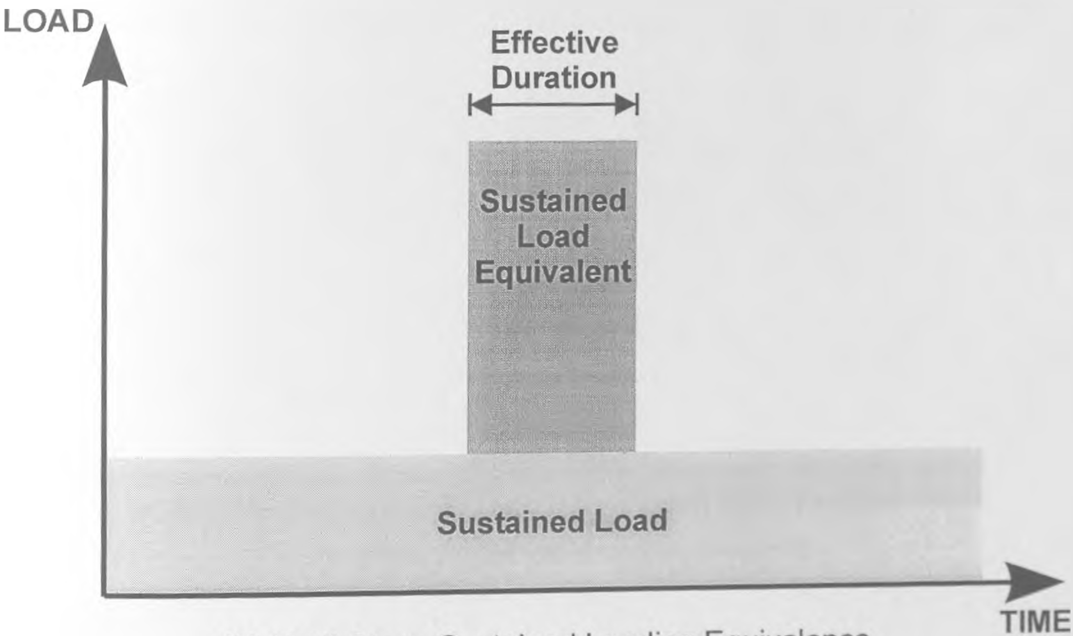


Figure 4.6 Effective Durations for soil (▲) and rock (O) sites as a function of distance



(a) Multi-Stage Loading due to Static and Dynamic Actions



(b) Multi-Stage Sustained Loading Equivalence

Figure 4.7 Multi-Stage Sustained Loading Equivalent

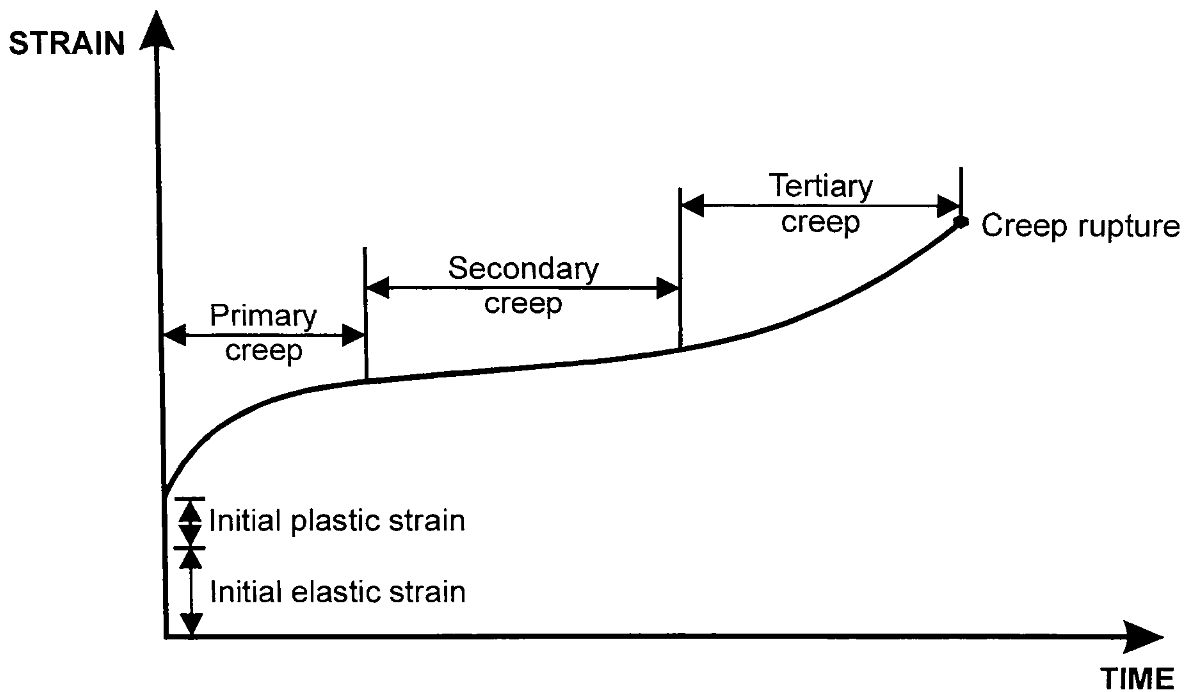
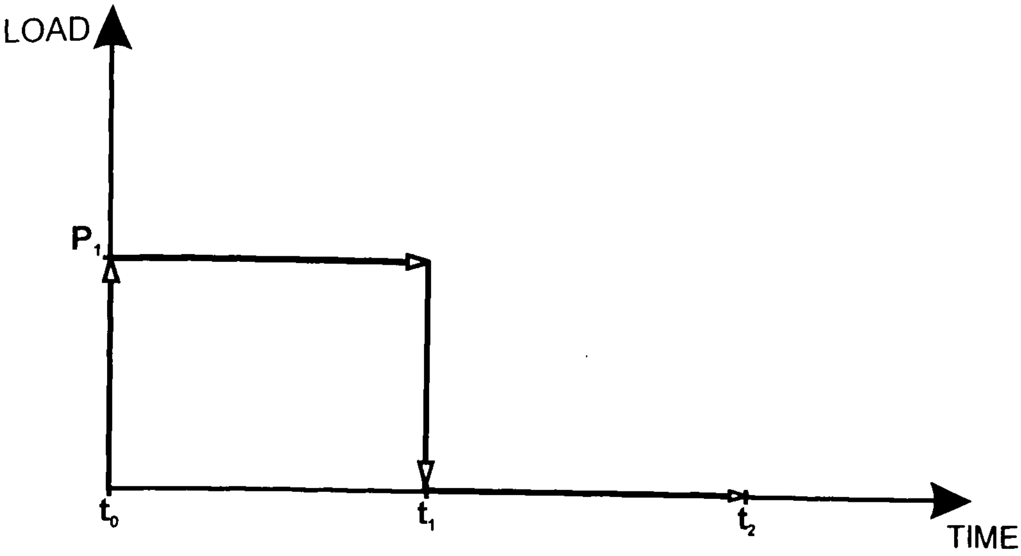
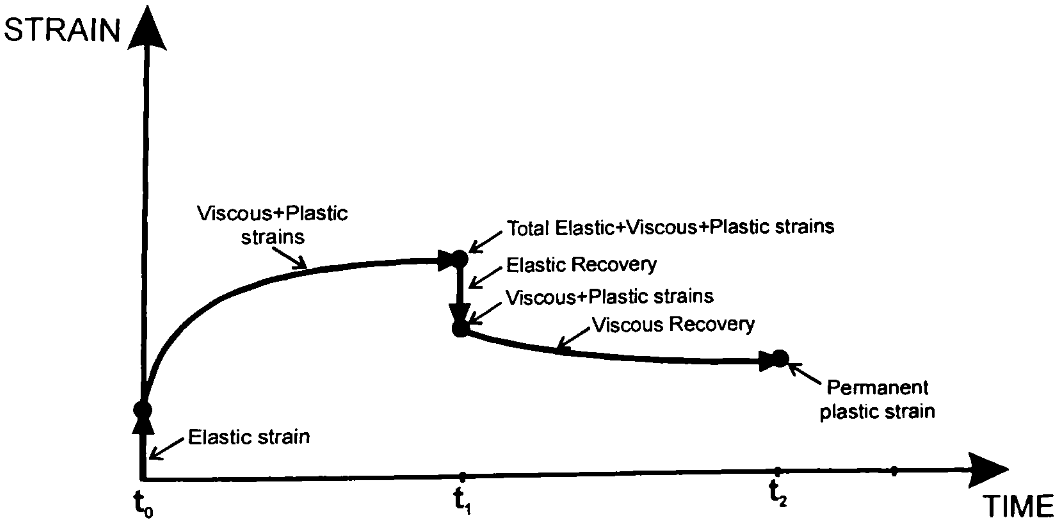


Figure 5.1 Idealised constant load creep curve under isothermal conditions



(a) Loading sequence to a geosynthetic specimen



(b) Load response of the geosynthetic

Figure 5.2 Characteristic behaviour of geosynthetics under sustained loading

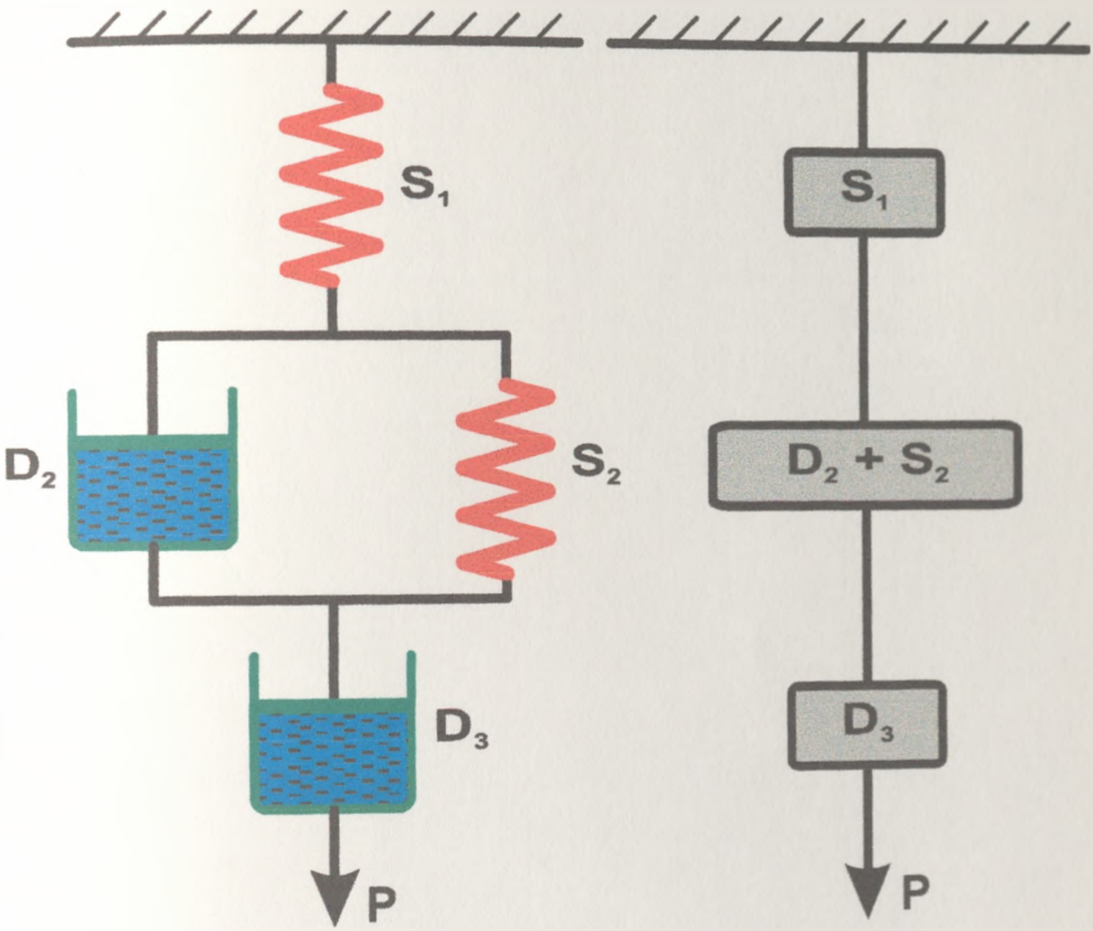


Figure 5.3 Four element rheological model for geosynthetics

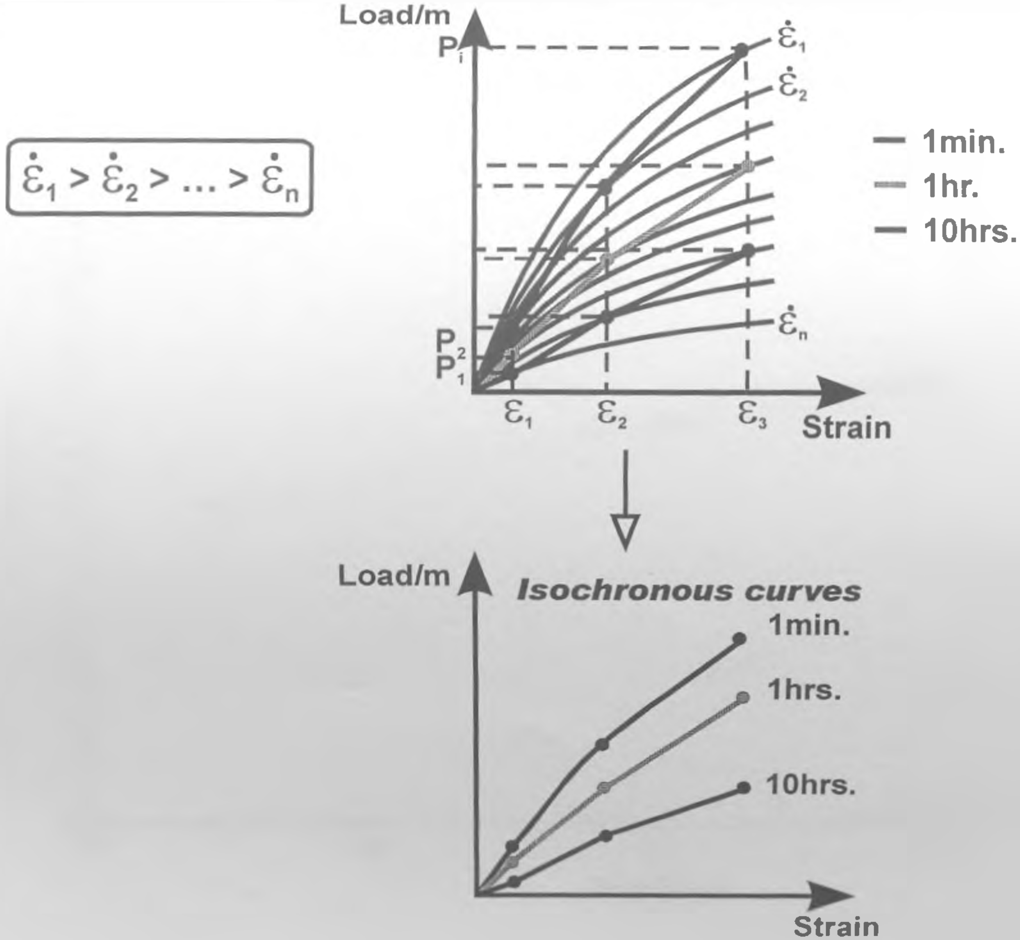


Figure 5.4 Derivation of isochronous curves from CRS testing after Khan (1999)

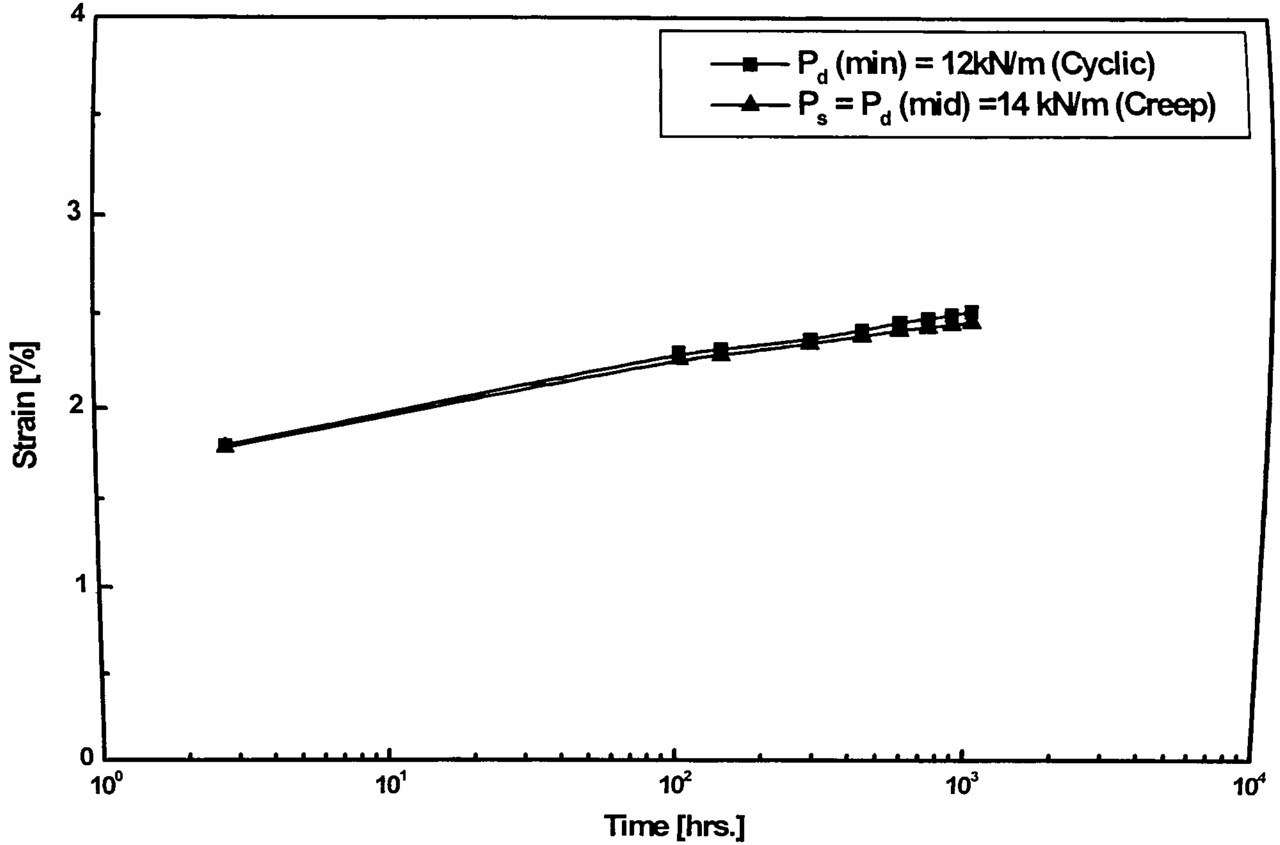
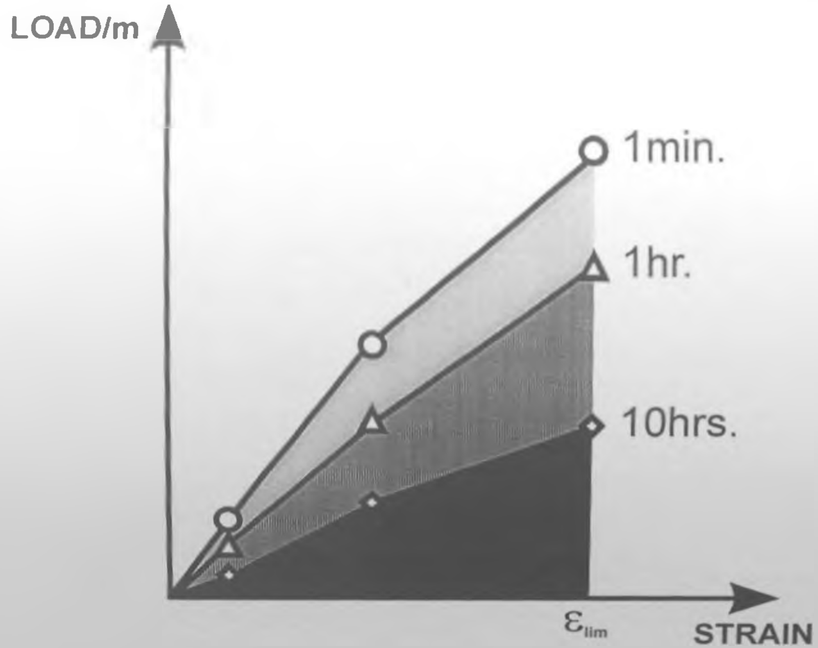


Figure 5.5 Cyclic load test data for uniaxial geogrid B at 20°C after Müller-Rochholz et al (1994)



$$\boxed{ISE_{1min.} > ISE_{1hr.} > ISE_{10hrs.}}$$

Figure 5.6 Isochronous strain energies at a limiting strain after Khan (1999)

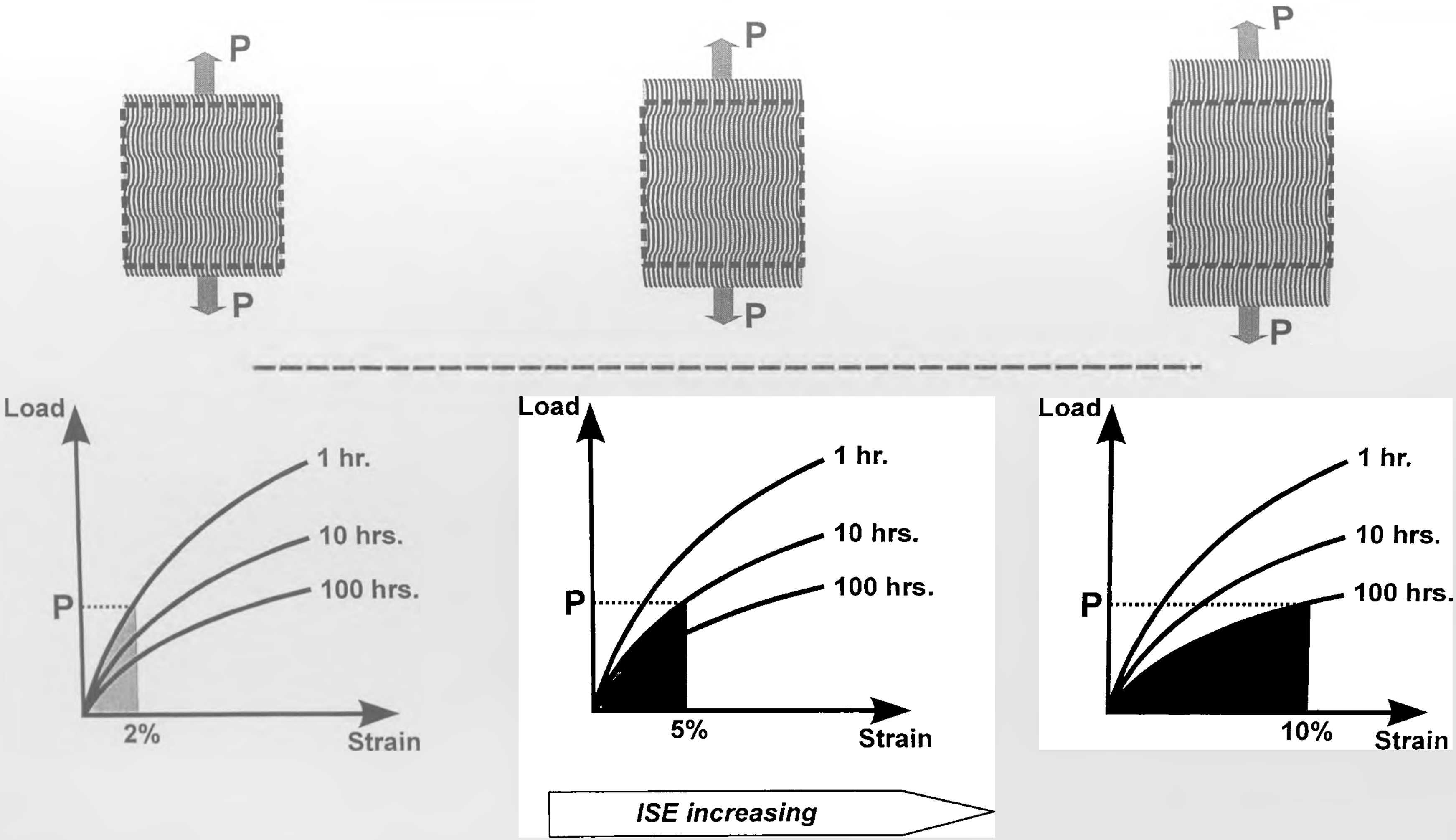


Figure 5.7 Absorbed ISE under sustained loading after Khan (1999)

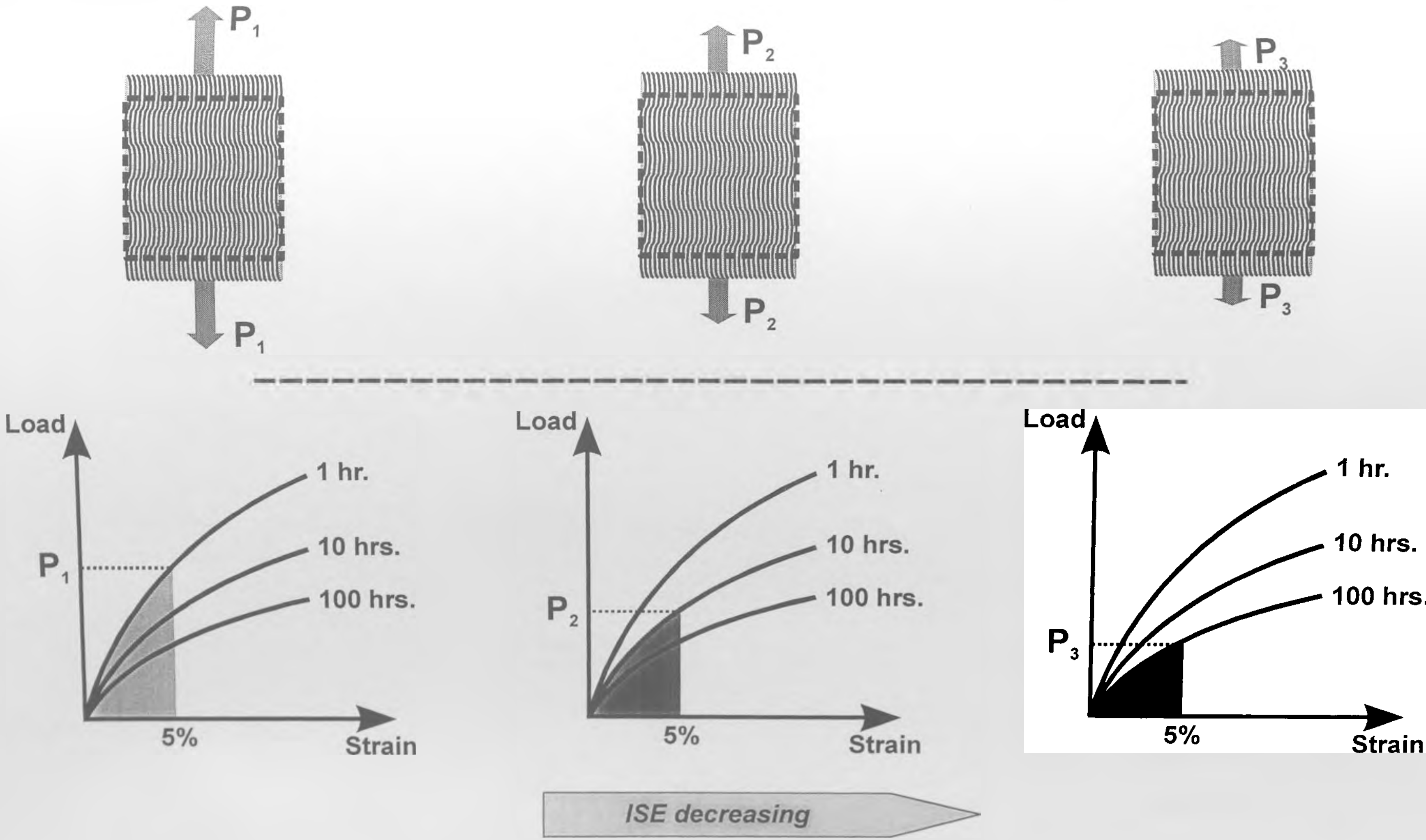


Figure 5.8 Absorbed ISE under constant strain after Khan (1999)

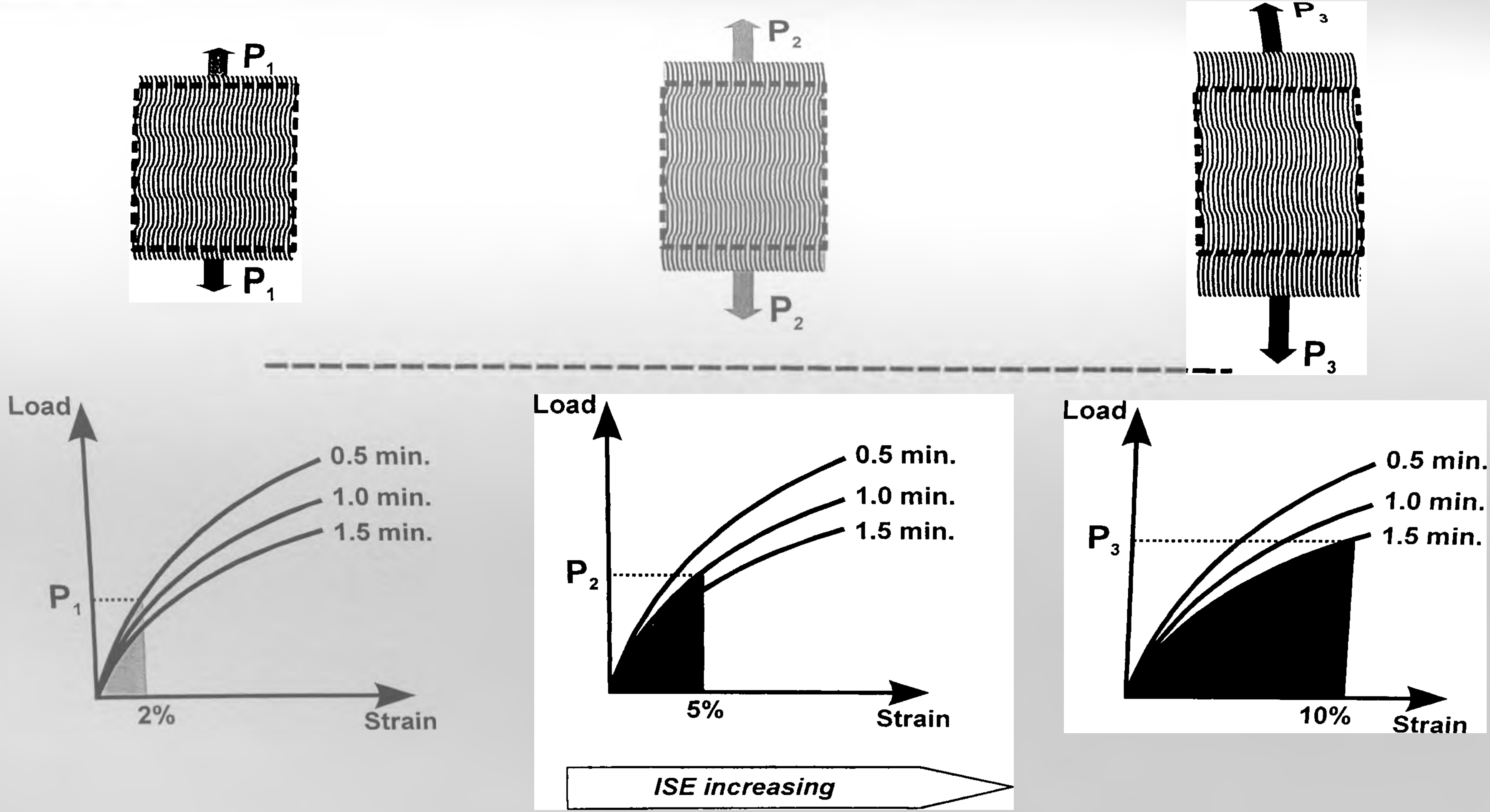


Figure 5.9 Absorbed ISE under increasing load and strain after Khan (1999)

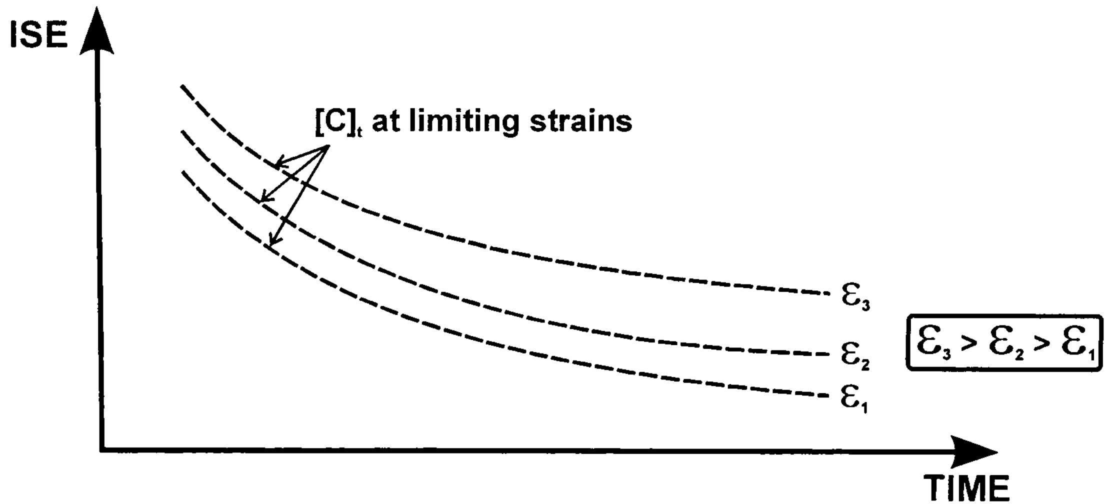


Figure 5.10 ISE capacity at different strains after Khan (1999)

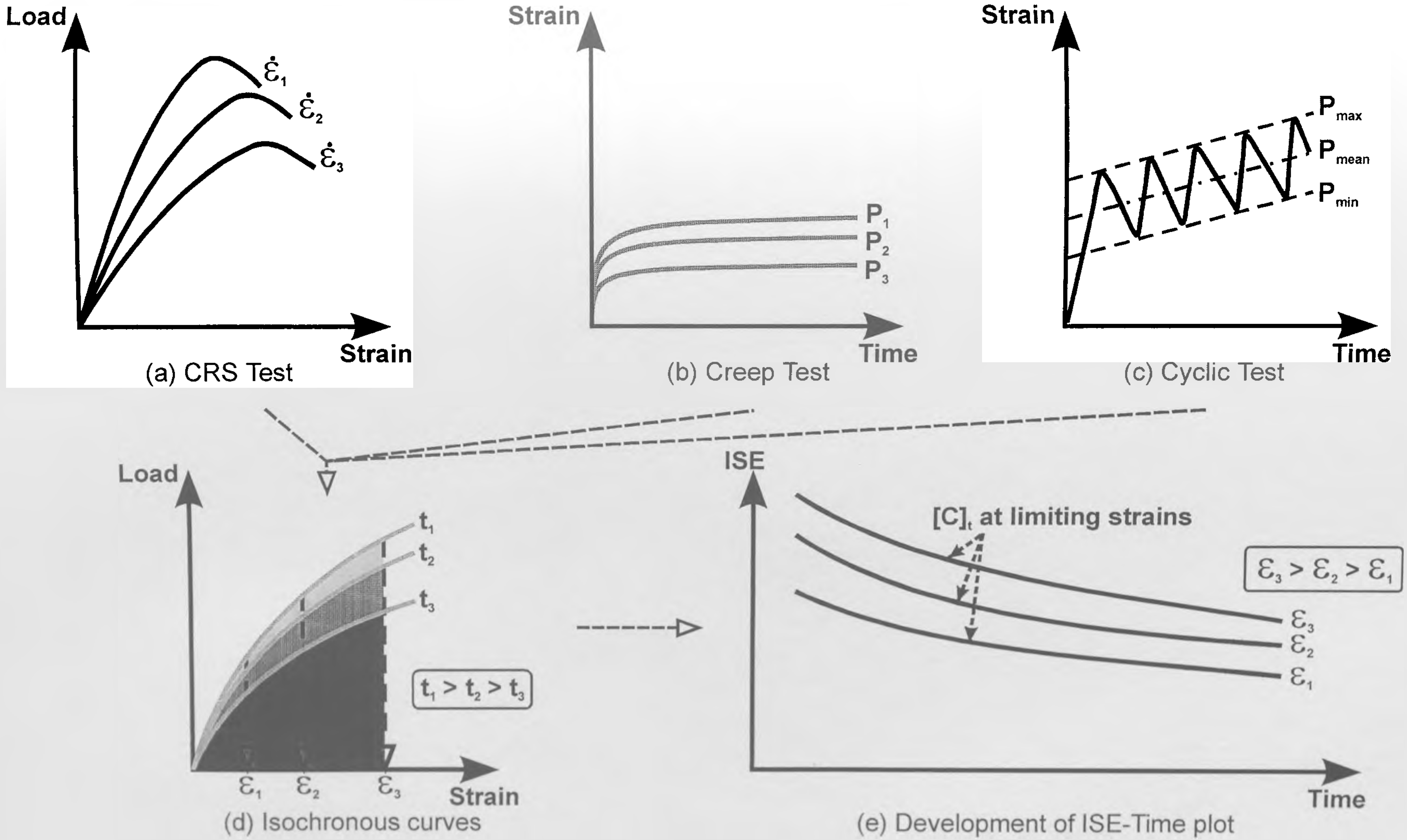


Figure 5.11 Development of ISE-Time plots from various tests after Khan (1999)

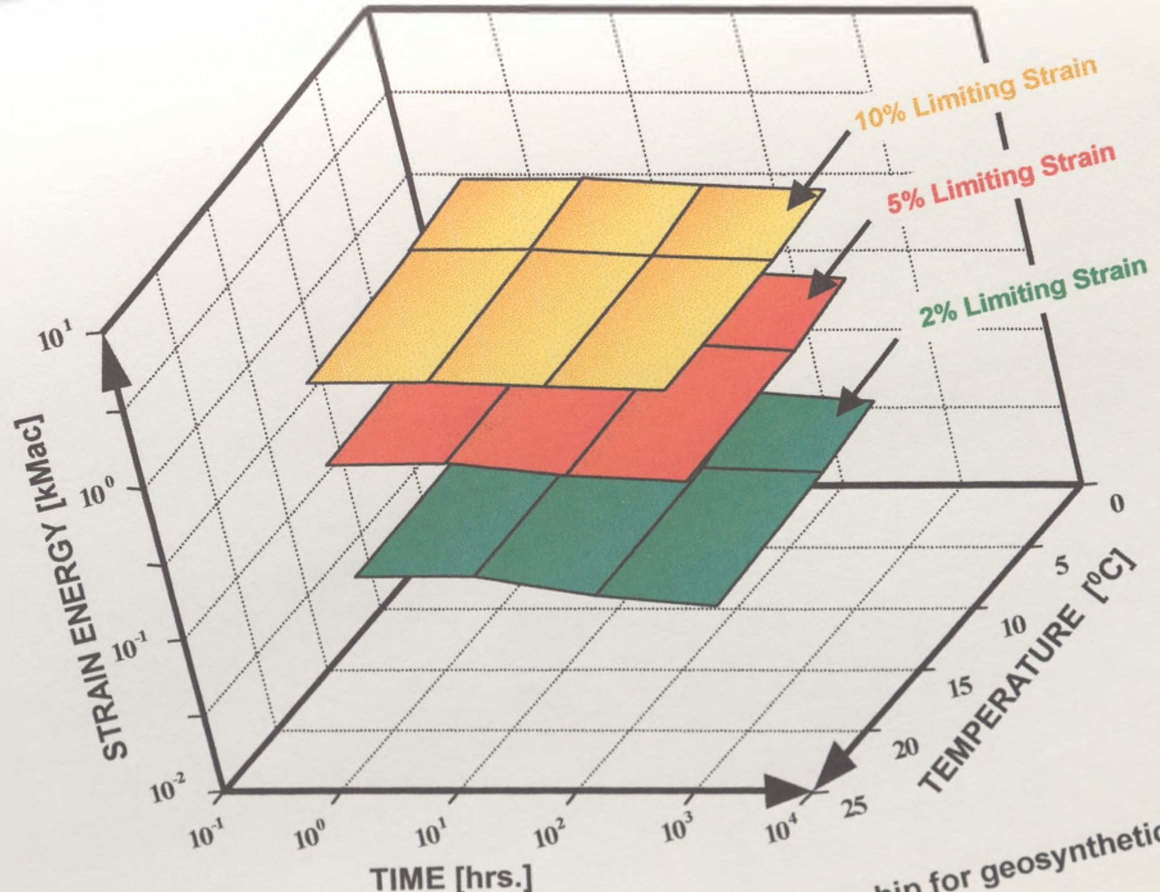
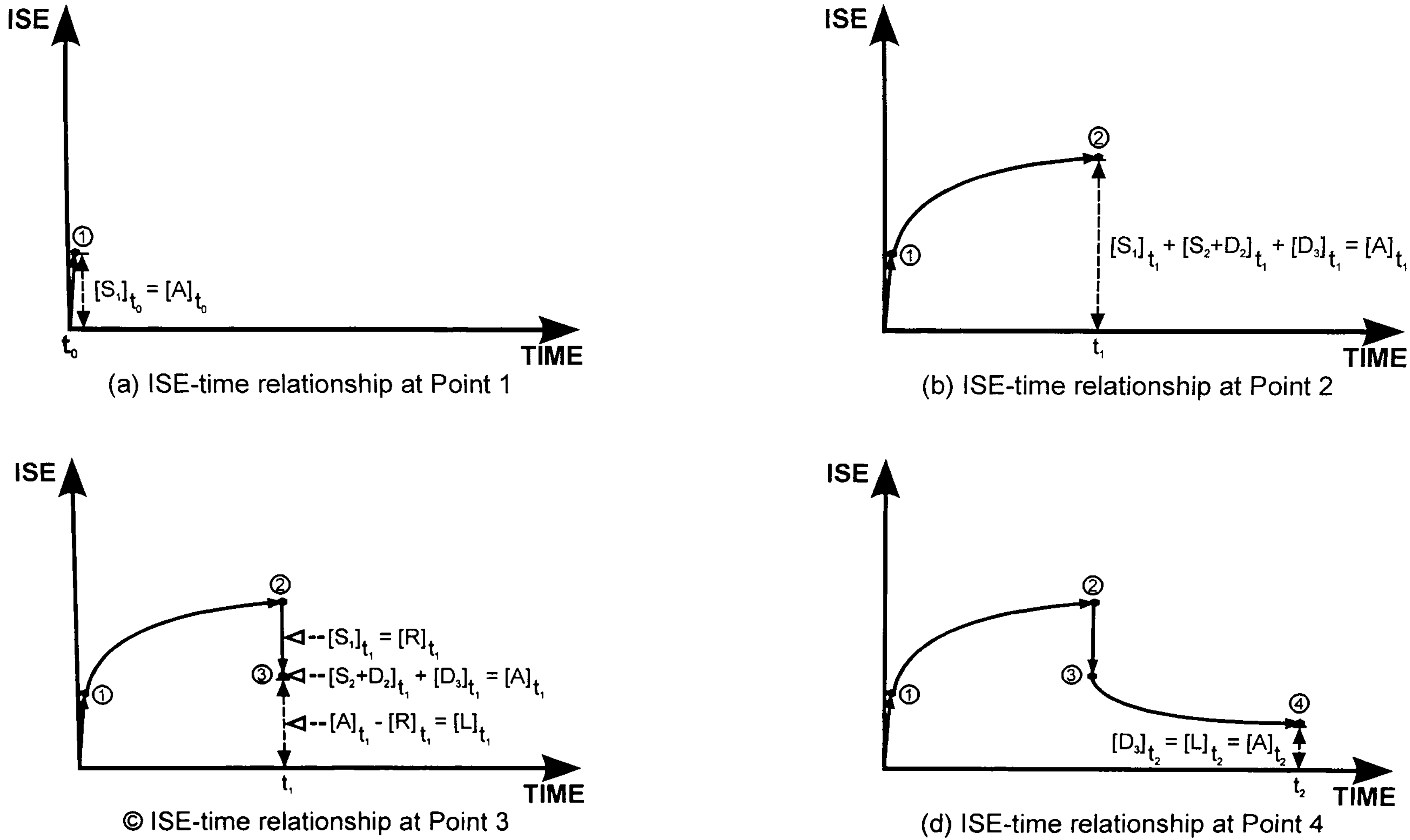


Figure 5.12 ISE – time – temperature relatinship for geosynthetics



(a) ISE-time relationship at Point 1

(b) ISE-time relationship at Point 2

© ISE-time relationship at Point 3

(d) ISE-time relationship at Point 4

Figure 5.13 Absorbed ISE representation of geosynthetics behaviour after Khan (1999)

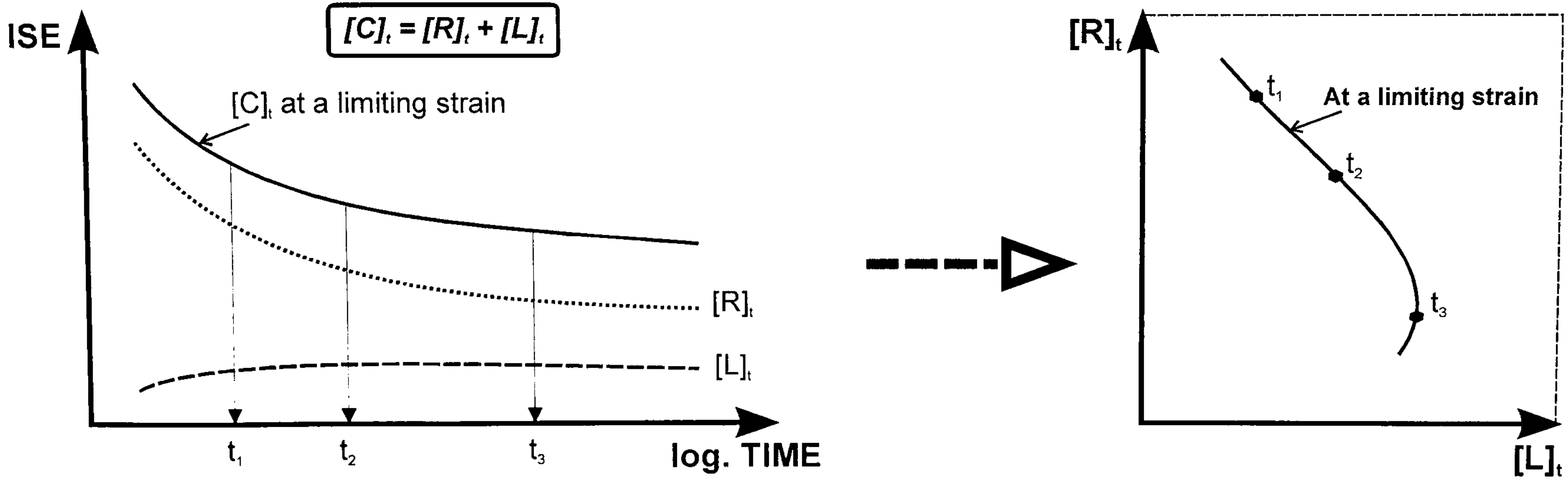


Figure 5.14 Variation of ISE components with time after Khan (1999)

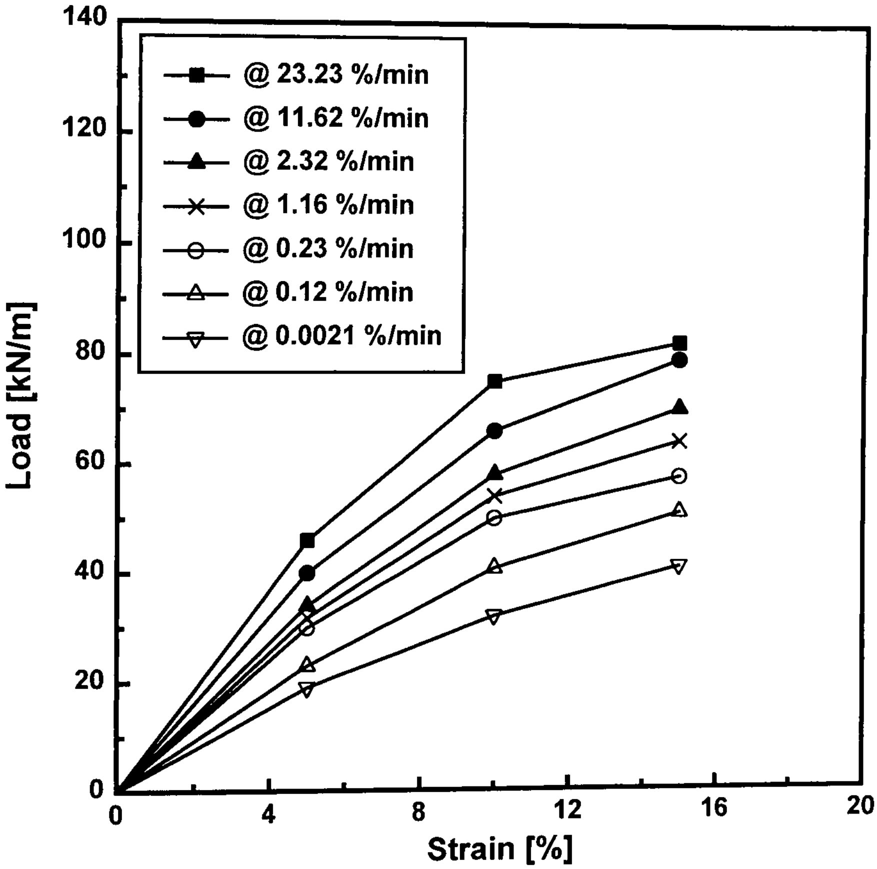


Figure 5.15(a) Load-strain curves for uniaxial geogrid A at 20°C in CRS test after Yeo (1985)

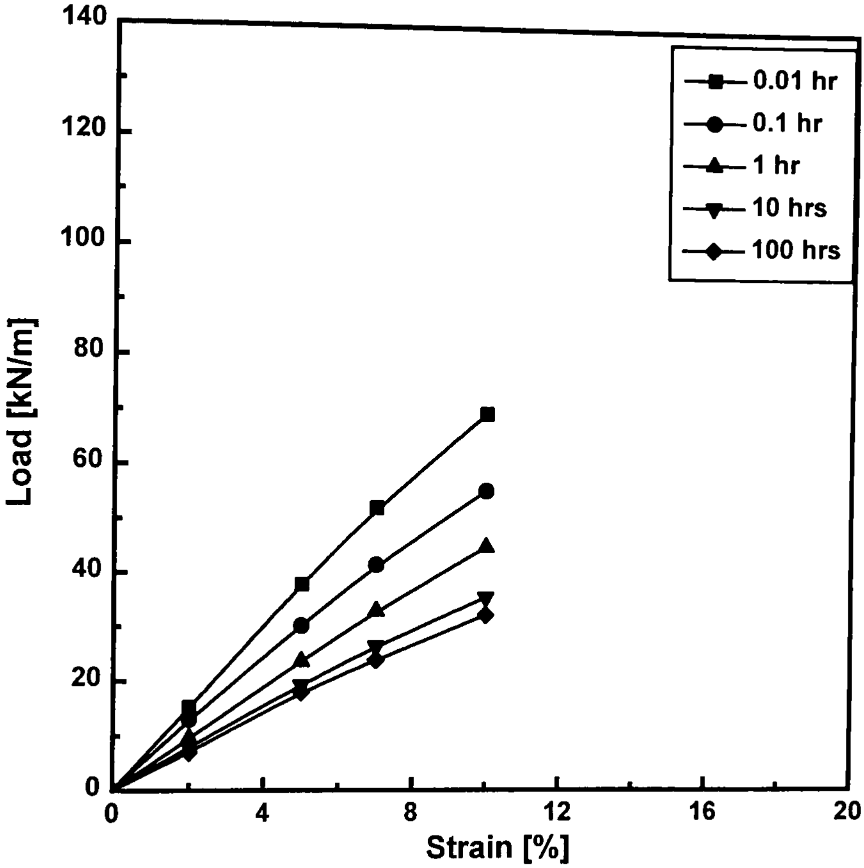


Figure 5.15(b) Isochronous load-strain curves for uniaxial geogrid A at 20°C from CRS test after Khan (1999)

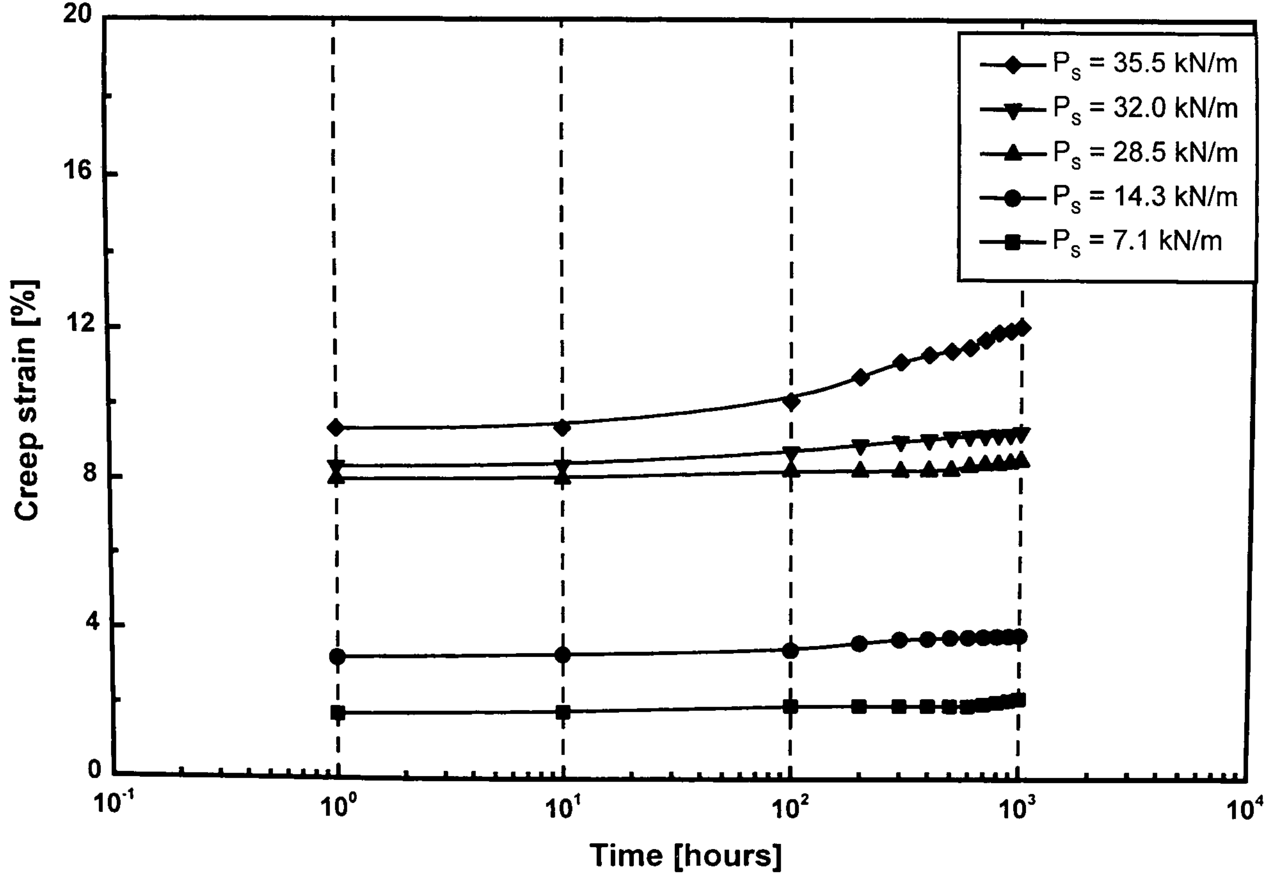


Figure 5.16(a) Creep curves for uniaxial geogrid A at 20°C after Yeo (1985)

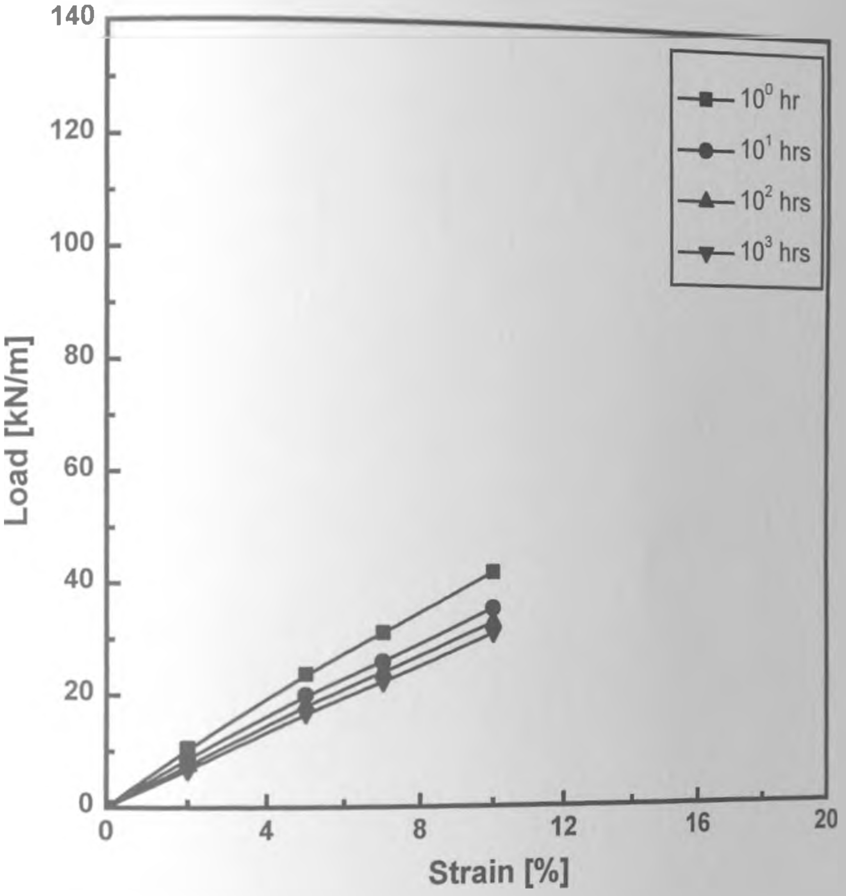


Figure 5.16(b) Isochronous load-strain curves for uniaxial geogrid A at 20°C from creep test after Khan (1999)

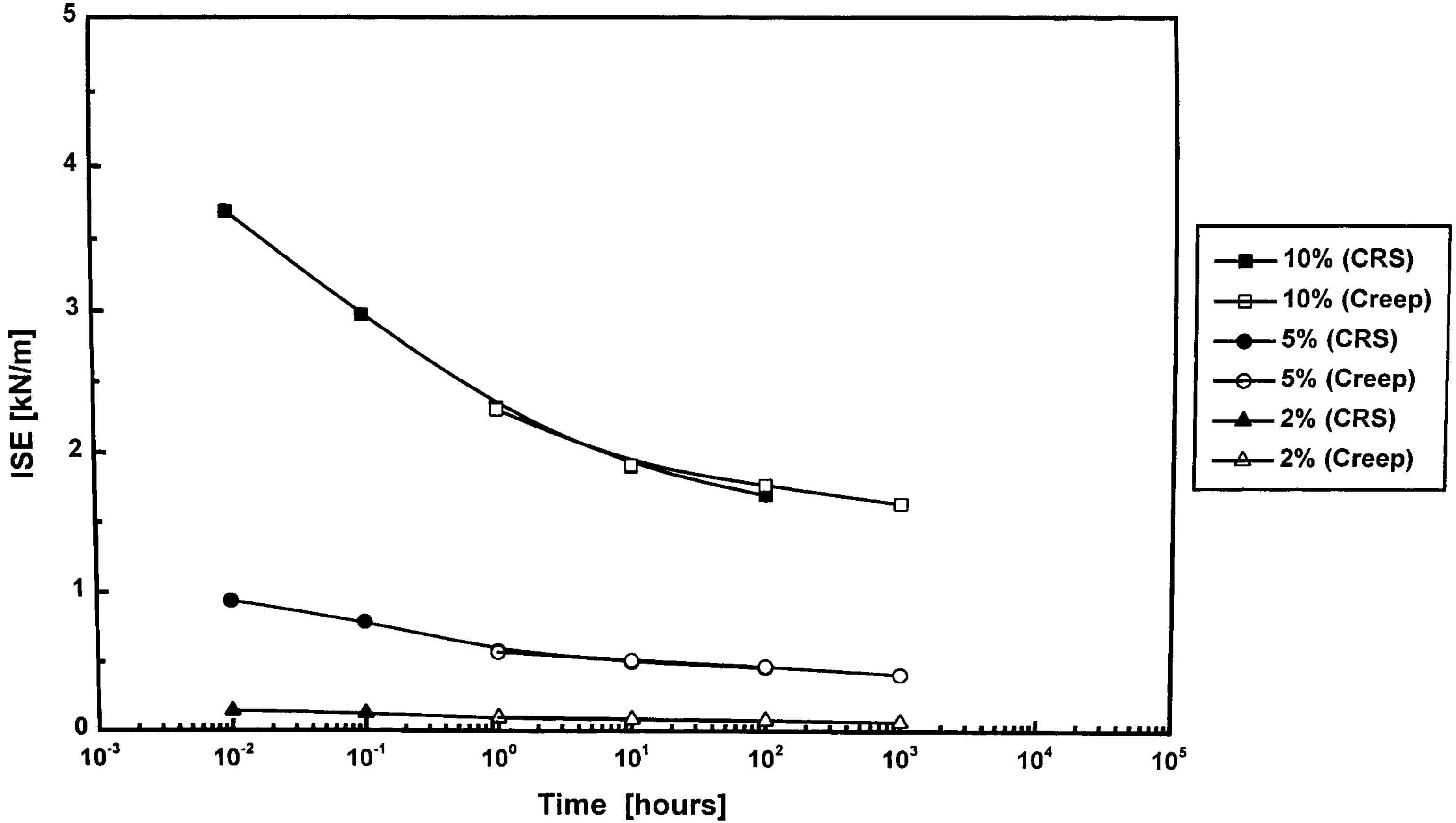


Figure 5.17(a) ISE - time relationships for uniaxial geogrid A at 20°C in linear-log scale after Khan (1999)

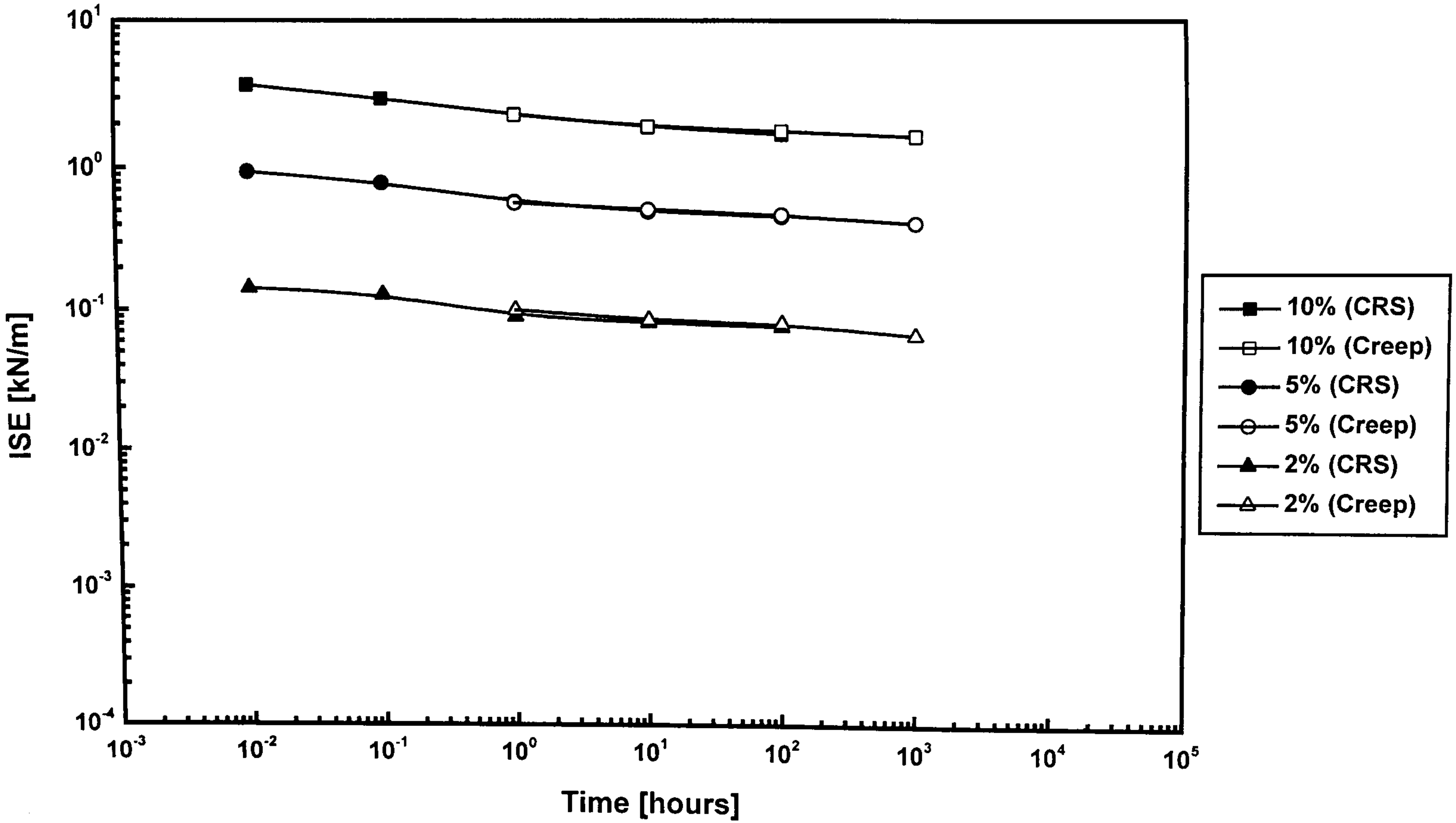


Figure 5.17(b). ISE - time relationships for uniaxial geogrid A at 20°C C in log-log scale

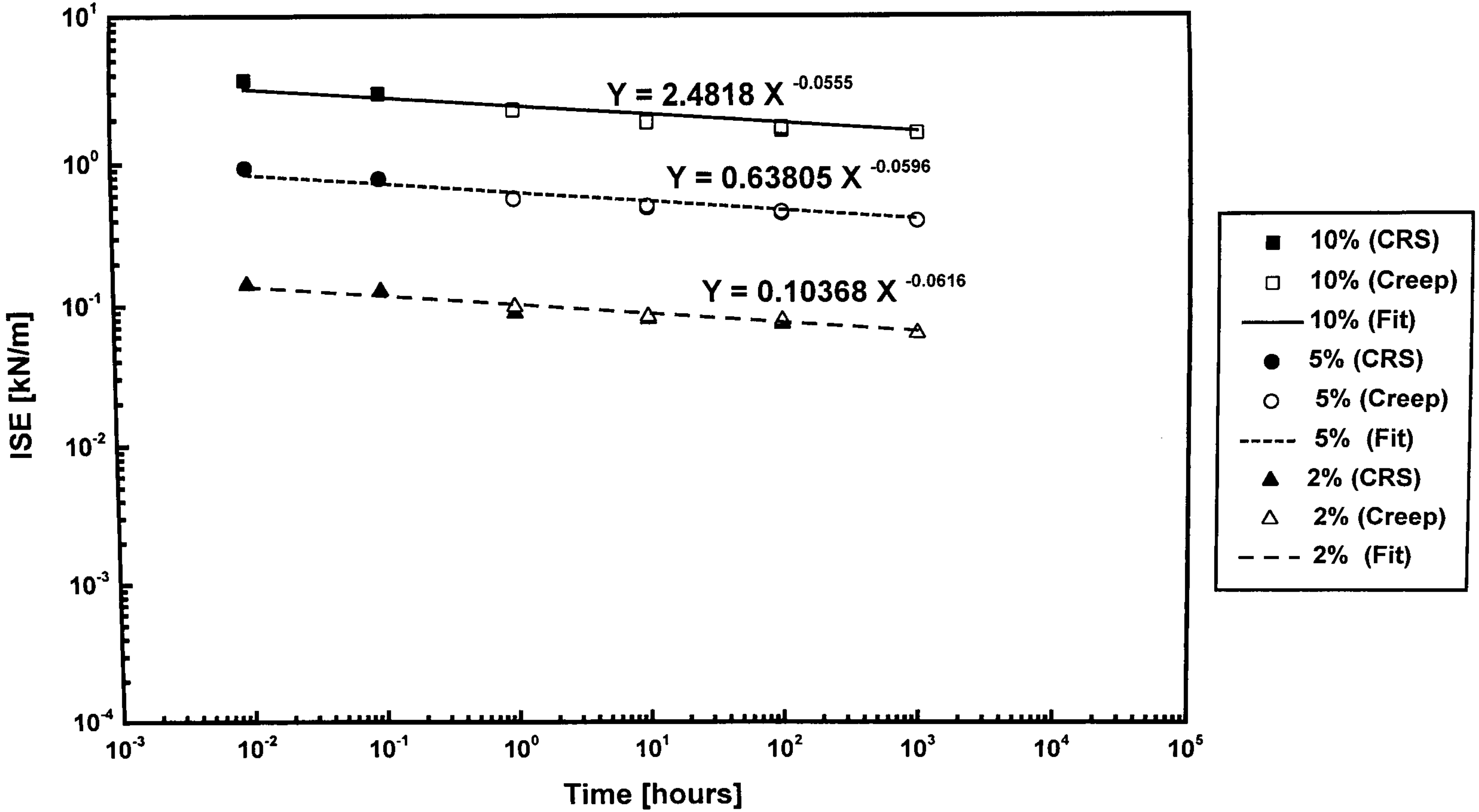
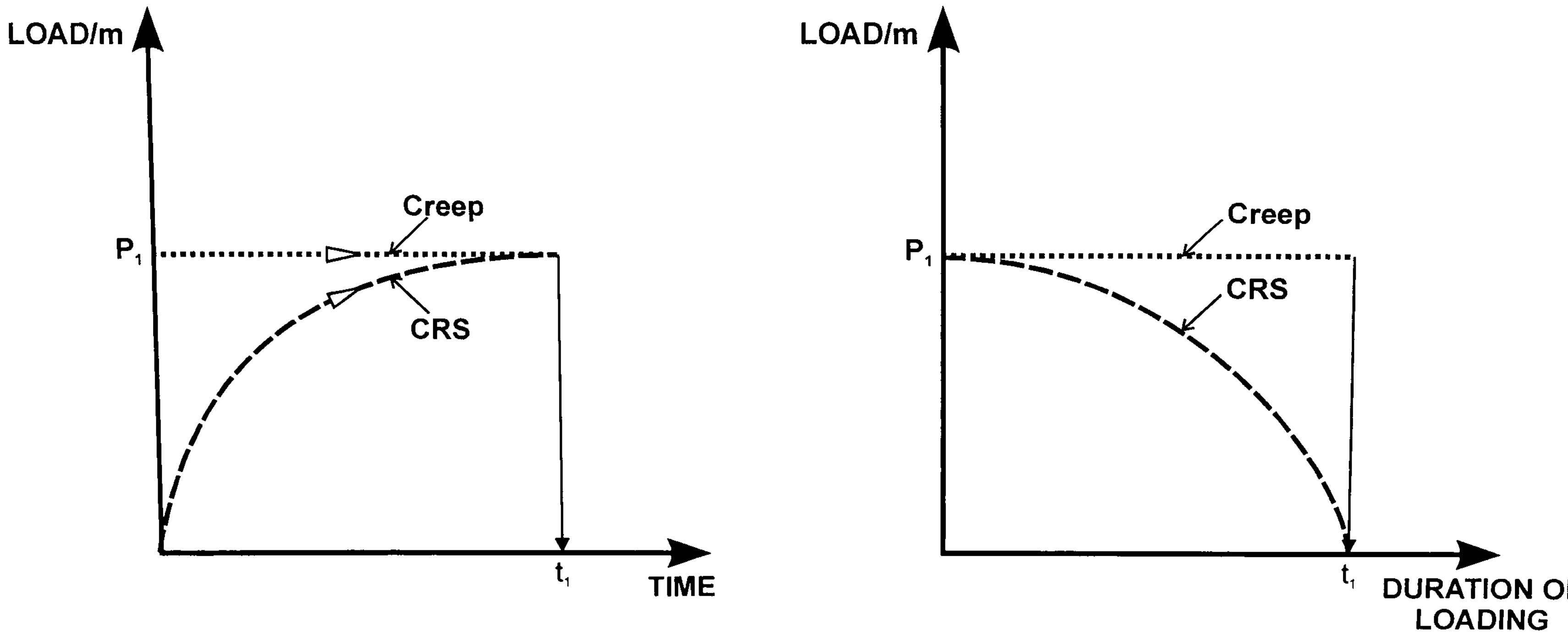


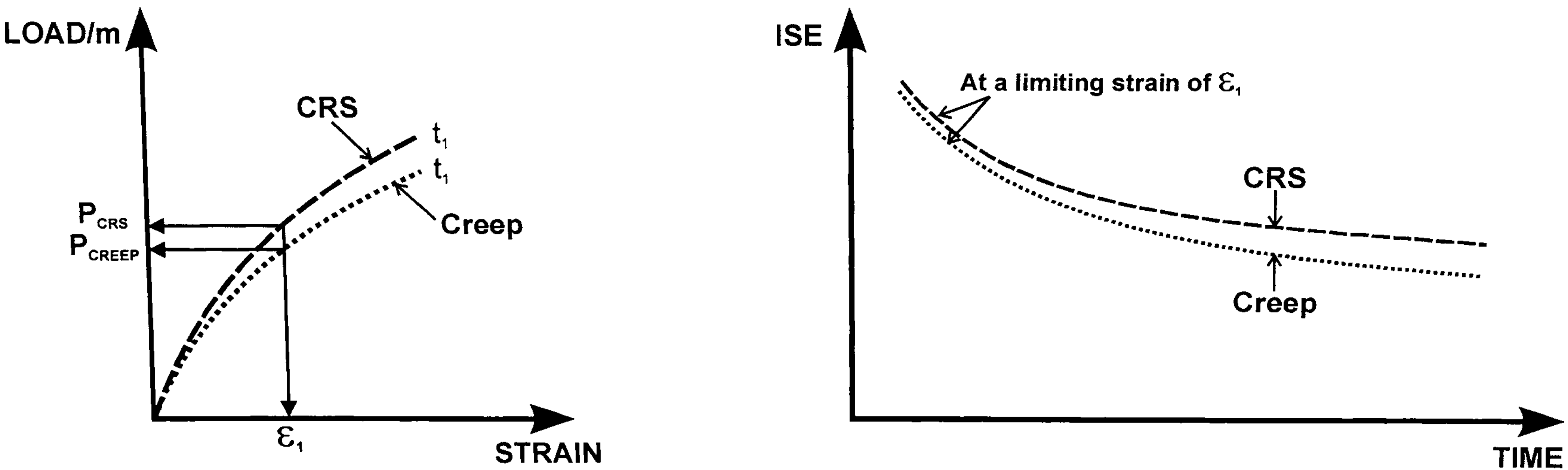
Figure 5.17(c) Best fit curves for ISE - time relationships for uniaxial geogrid A at 20°C after Khan (1999)



(a) Loading paths for CRS and creep testing

(b) Duration of loadings in CRS and creep tests

Figure 5.18 Typical load-time relationships in CRS and creep tests after Khan (1999)



(a) Isochronous load-strain curves

(b) ISE-time curves

Figure 5.19 Theoretical behaviour of geosynthetics in CRS and creep tests after Khan (1999)

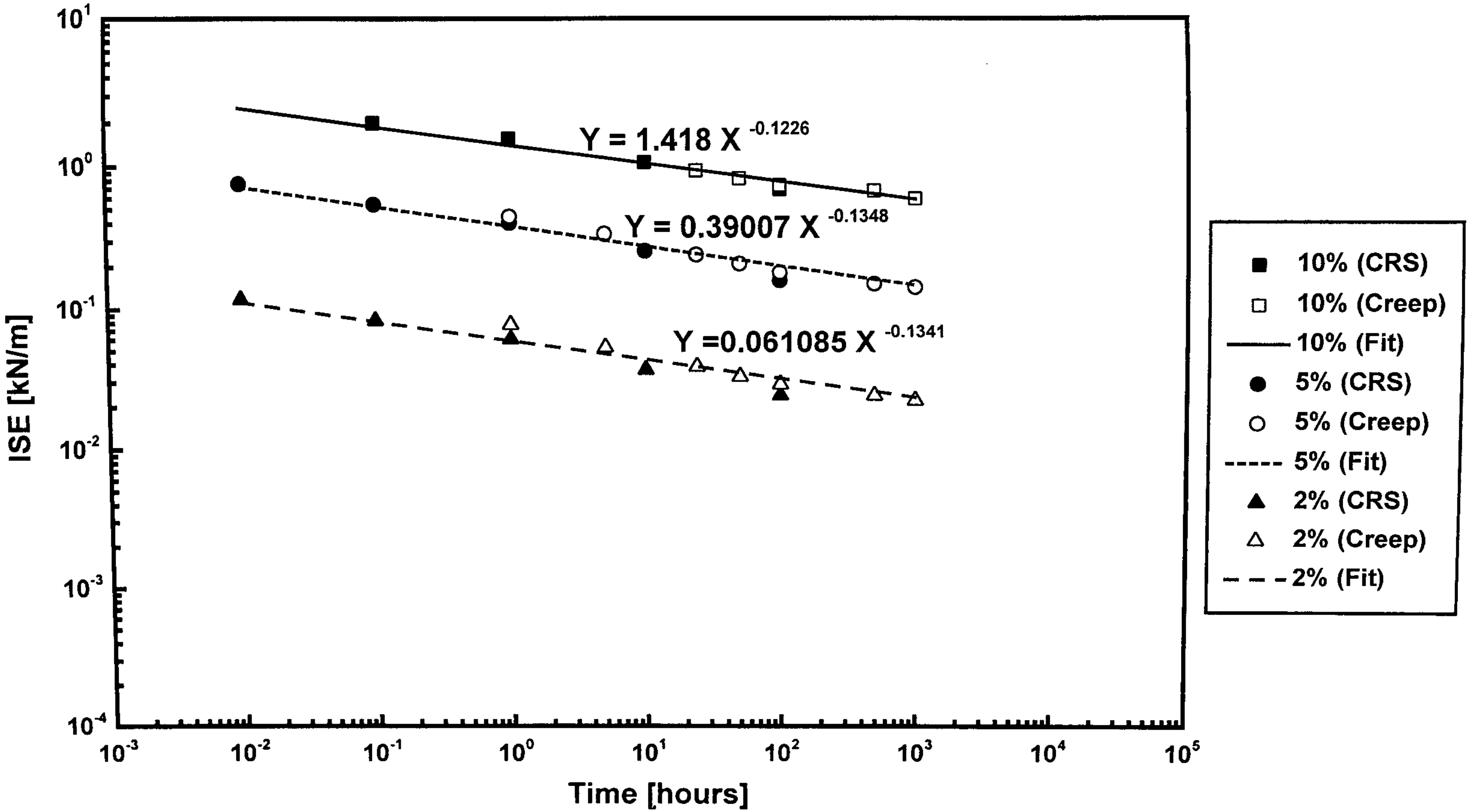


Figure 5.20 Best fit curves for ISE - time relationships for a biaxial geogrid (longitudinal direction) at 20°C after Khan (1999)

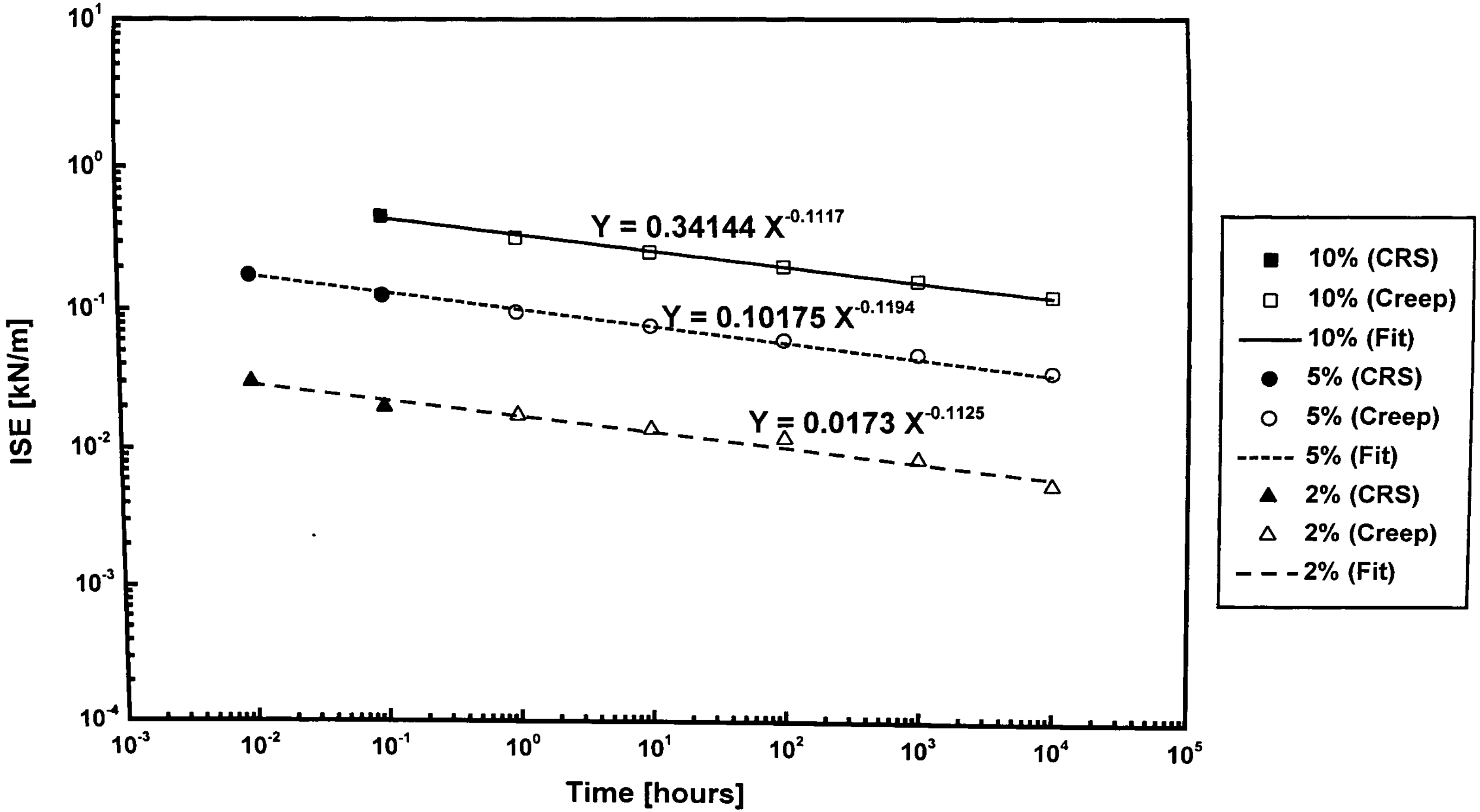


Figure 5.21 Best fit curves for ISE - time relationships for woven tapes at 20°C after Khan (1999)

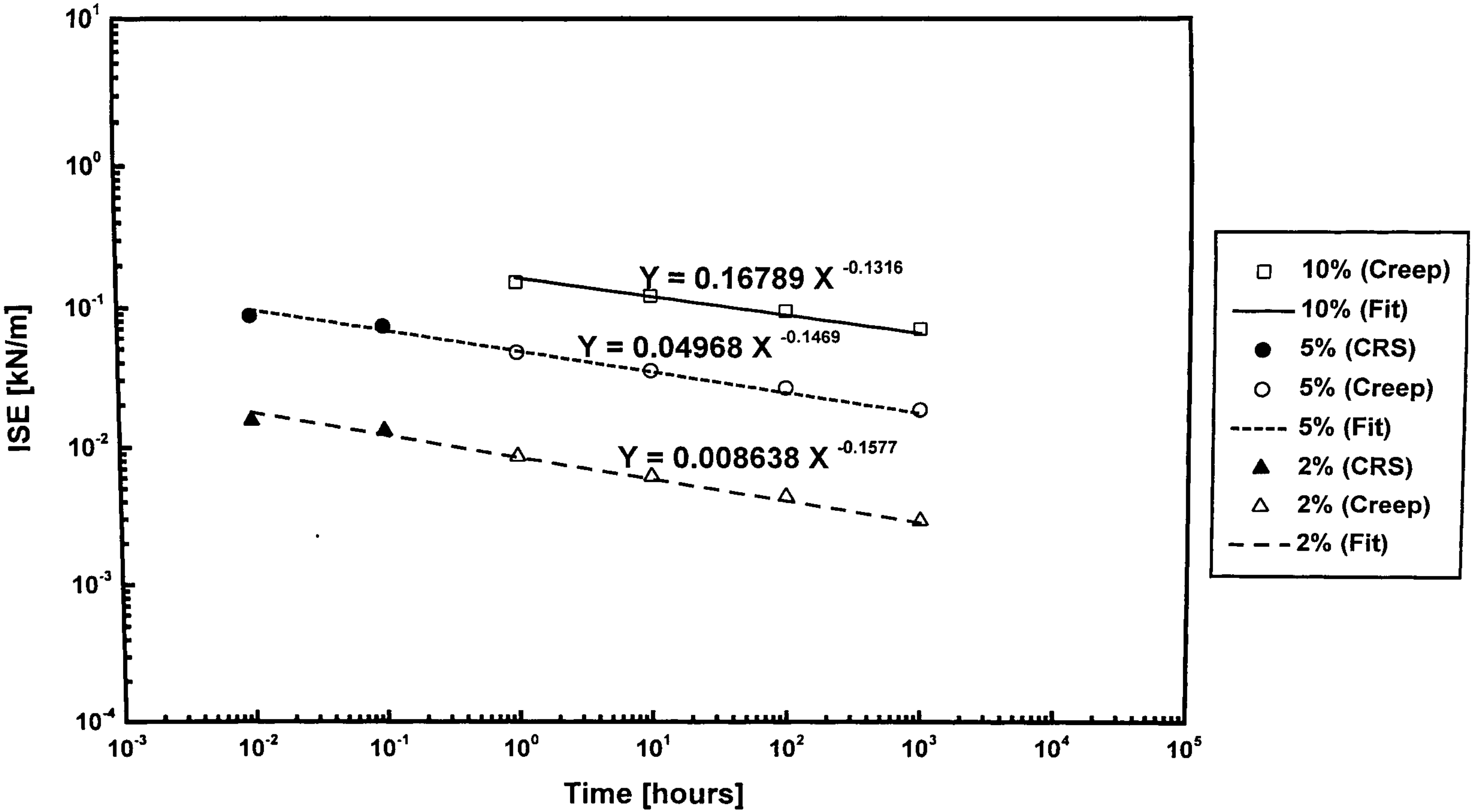


Figure 5.22 Best fit curves for ISE - time relationships for melt-bonded non-woven geotextile at 20°C after Khan (1999)

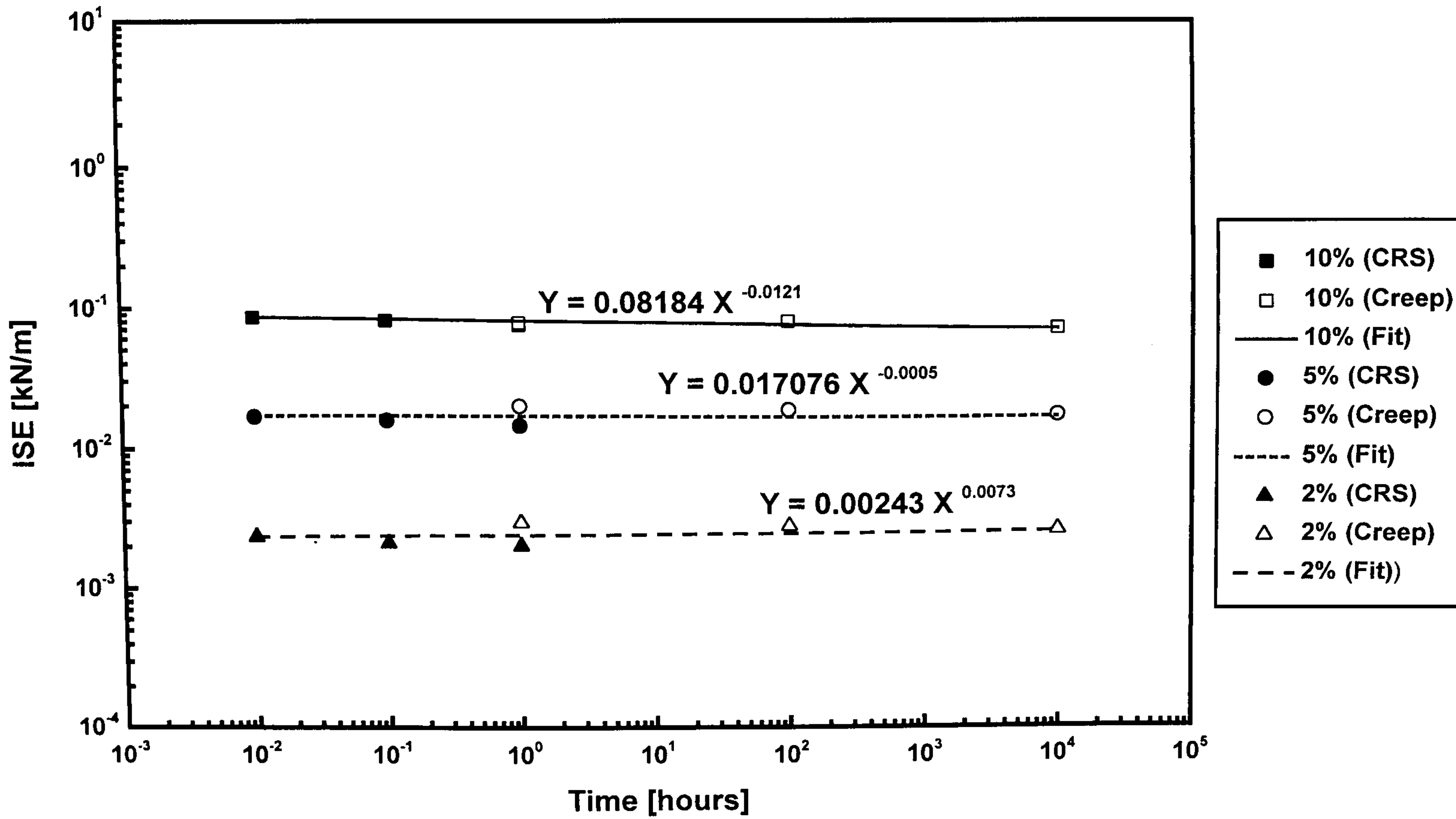


Figure 5.23 Best fit curves for ISE - time relationships for needle punched non-woven geotextile at 20°C after Khan (1999)

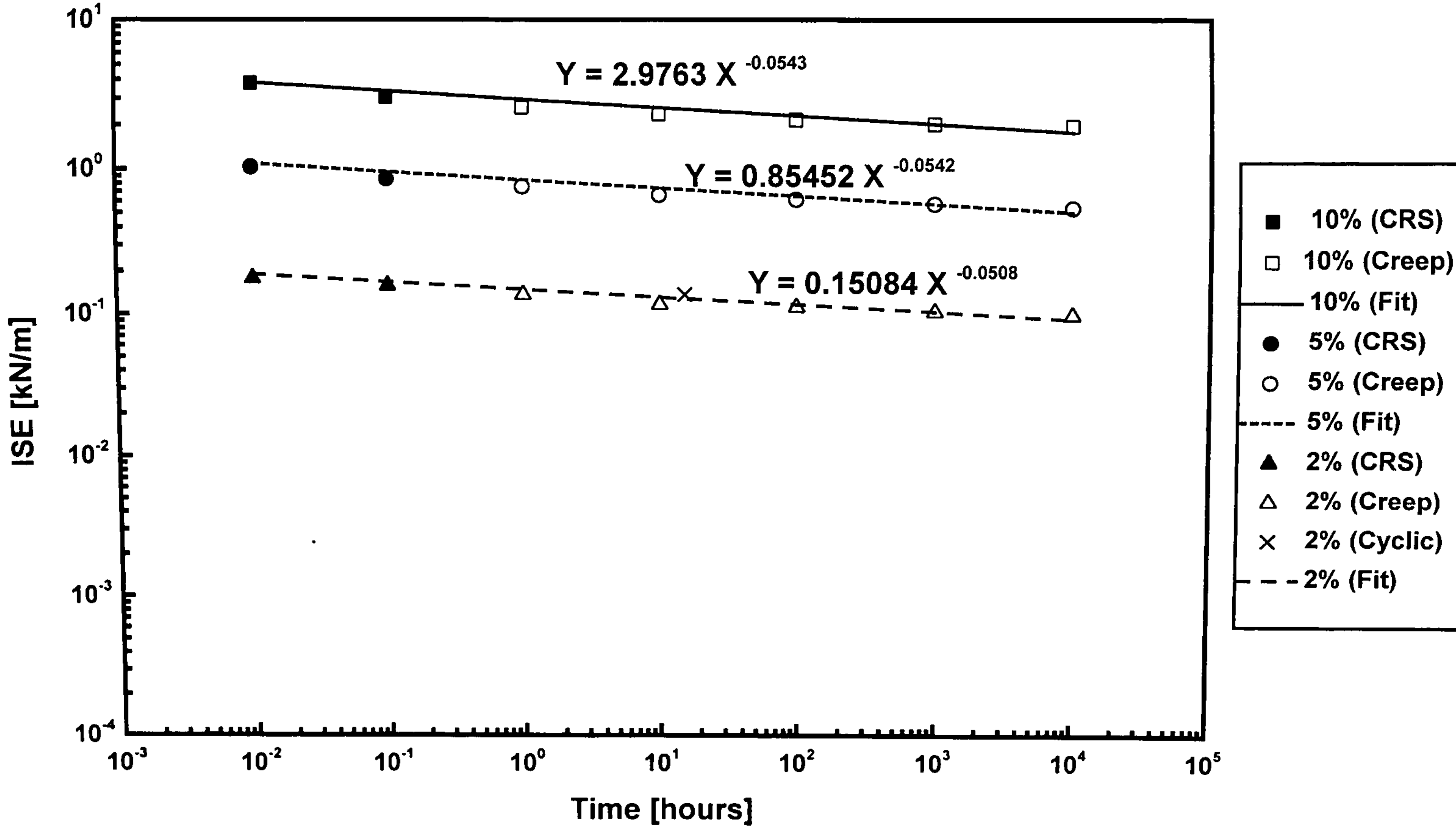
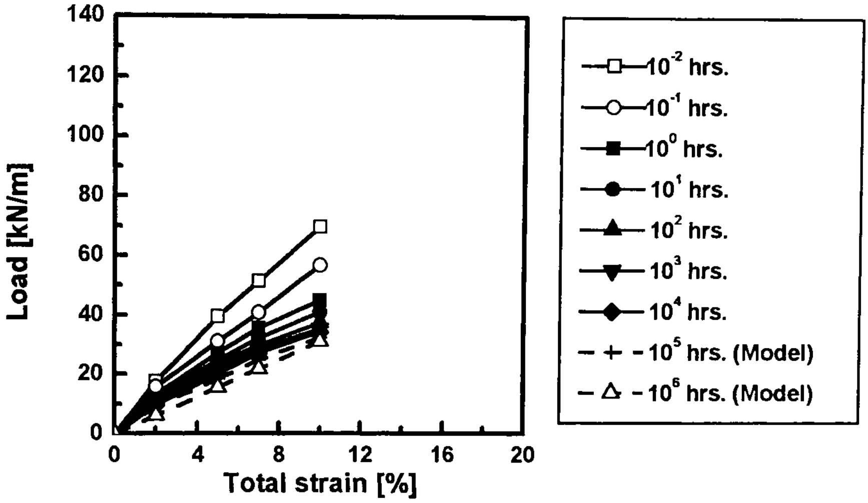
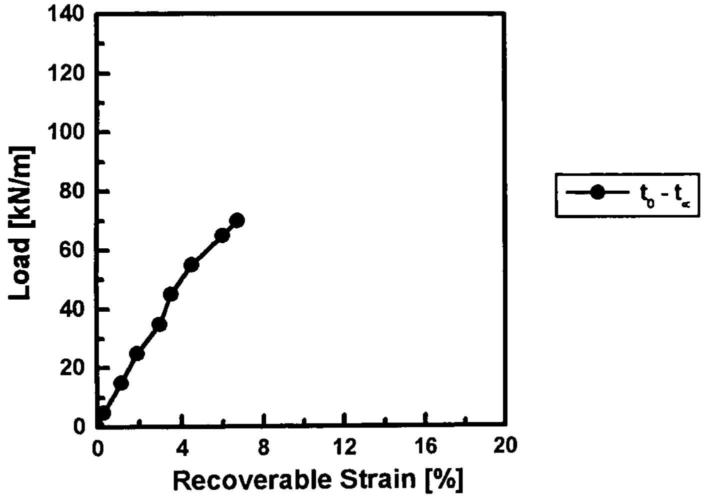


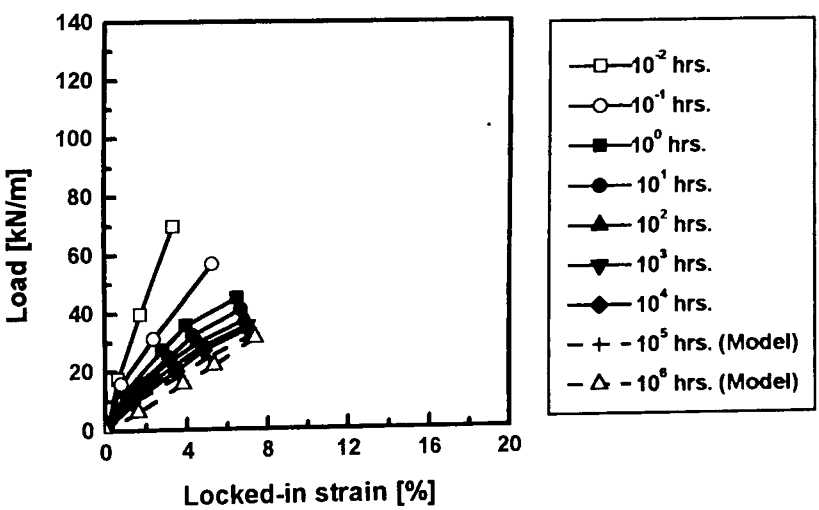
Figure 5.24 Best fit curves for ISE - time relationships for uniaxial geogrid B at 20°C after Khan (1999)



(a) Load vs. Total Strain



(b) Load vs. Recoverable Strain



(c) Load vs. Locked-in Strain

Figure 5.25 Isochronous load-strain curves for uniaxial geogrid B at 20°C after Khan (1999)

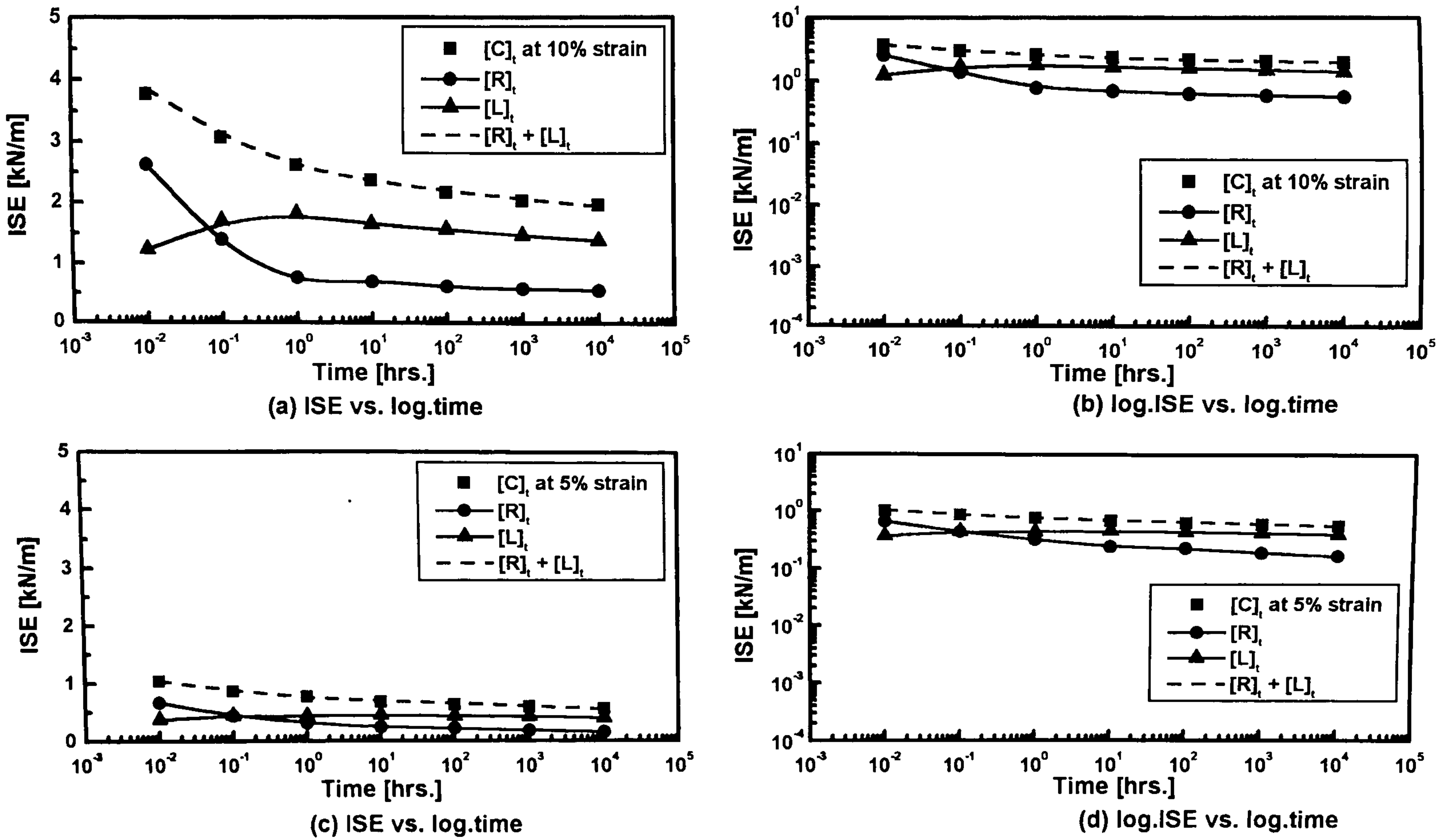


Figure 5.26 Variation of ISE capacity and its components with time for uniaxial geogrid B at 20°C after Khan (1999)

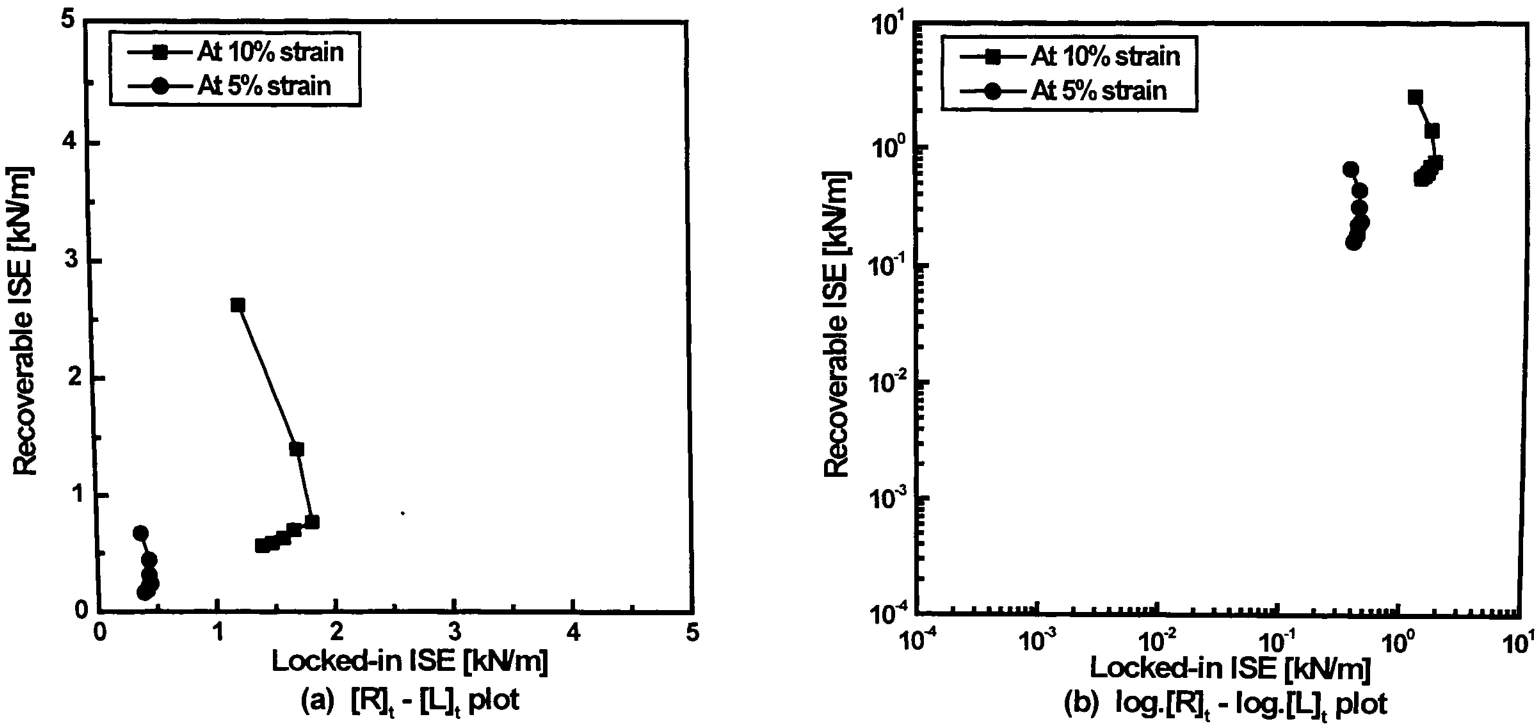
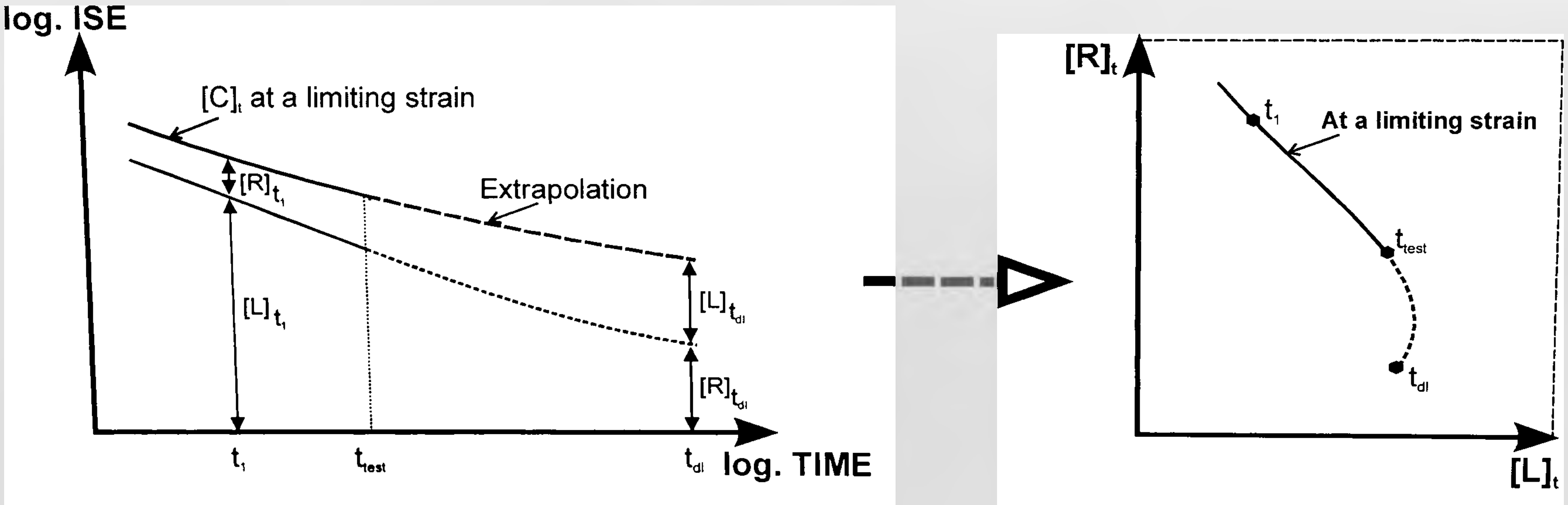


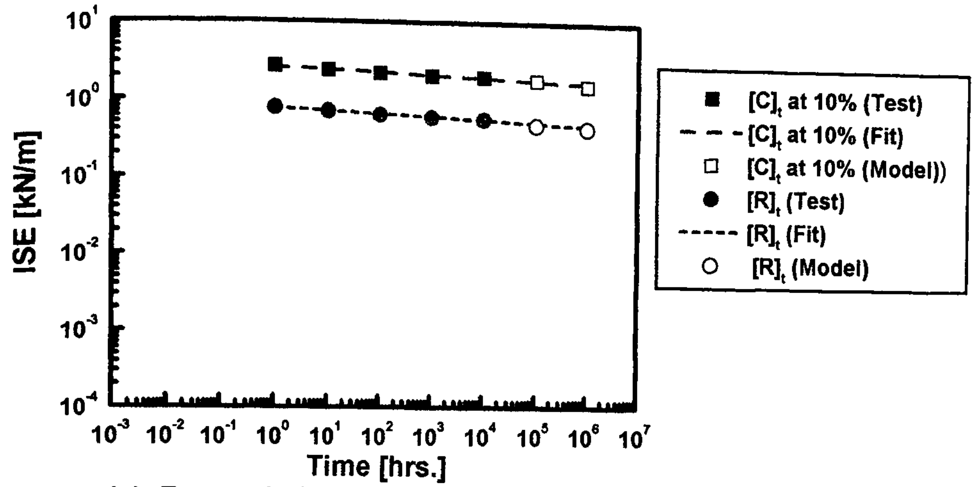
Figure 5.27 Combination of $[R]_t$ and $[L]_t$ at different limiting strains for uniaxial geogrid B at 20°C after Khan (1999)



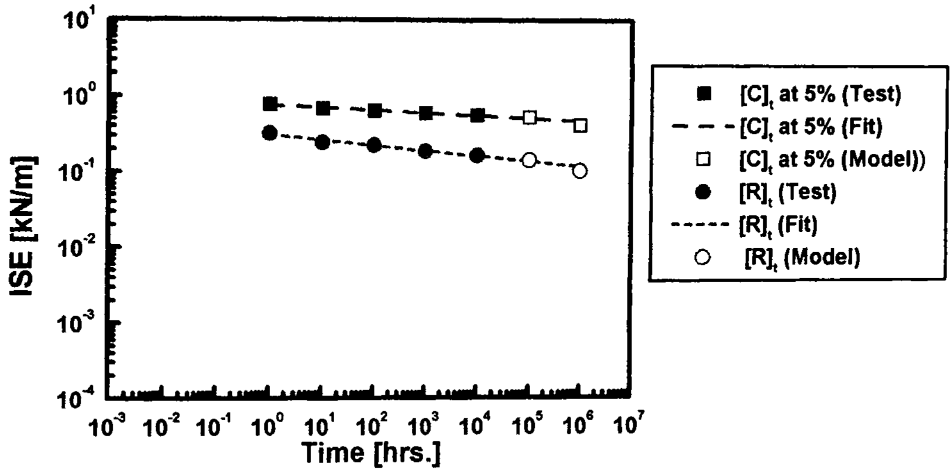
(a) log.ISE - log.time plot

(b) [R]$_t$ - [L]$_t$ plot

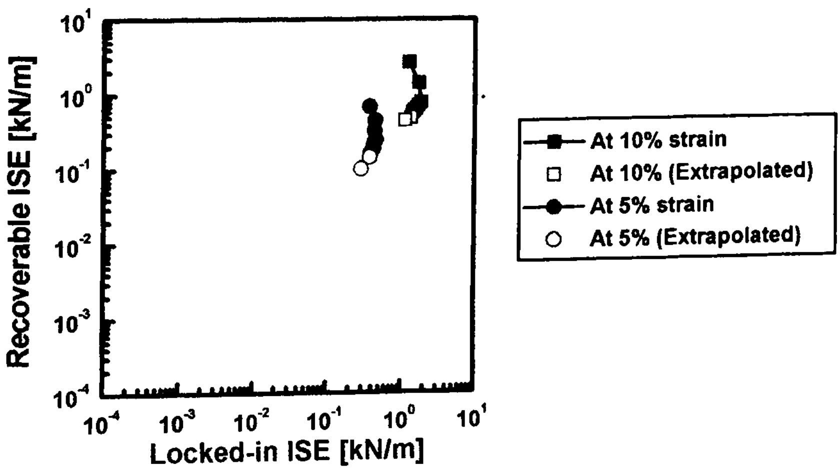
Figure 5.28 Extrapolation of test data after Khan (1999)



(a) Extrapolation at 10% limiting strain



(b) Extrapolation at 5% limiting strain



(c) Extrapolation of $[R]_t$ and $[L]_t$

Figure 5.29 Extrapolation of test data for uniaxial geogrid B at 20°C after Khan (1999)

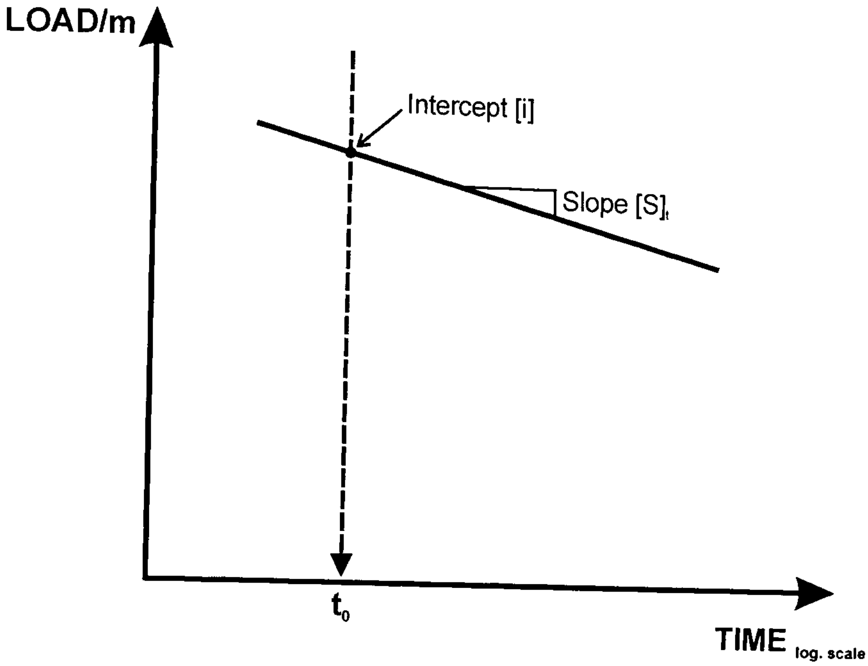


Figure 5.30 Definition of intercept and slope value after Khan (1999)

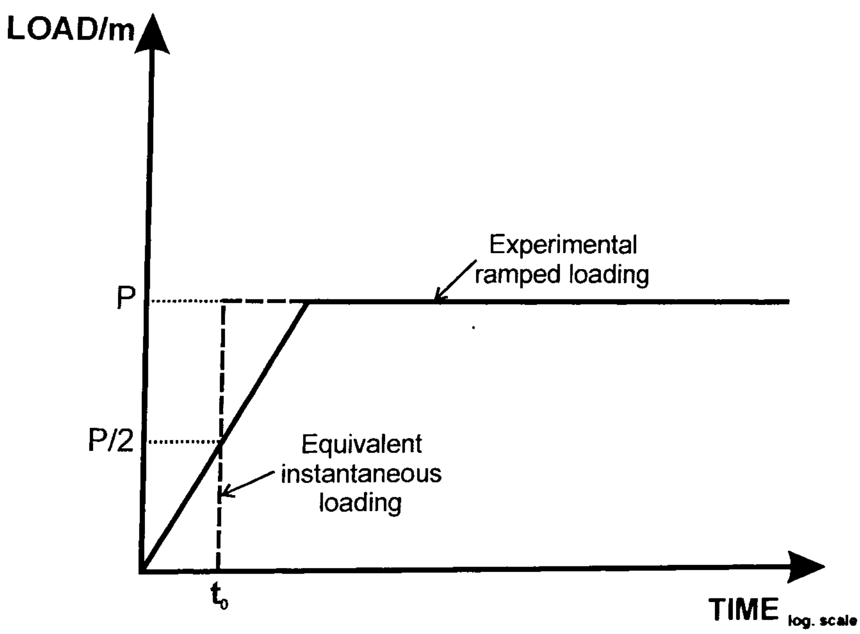


Figure 5.31 Method of obtaining equivalent instantaneous loading time after Khan (1999)

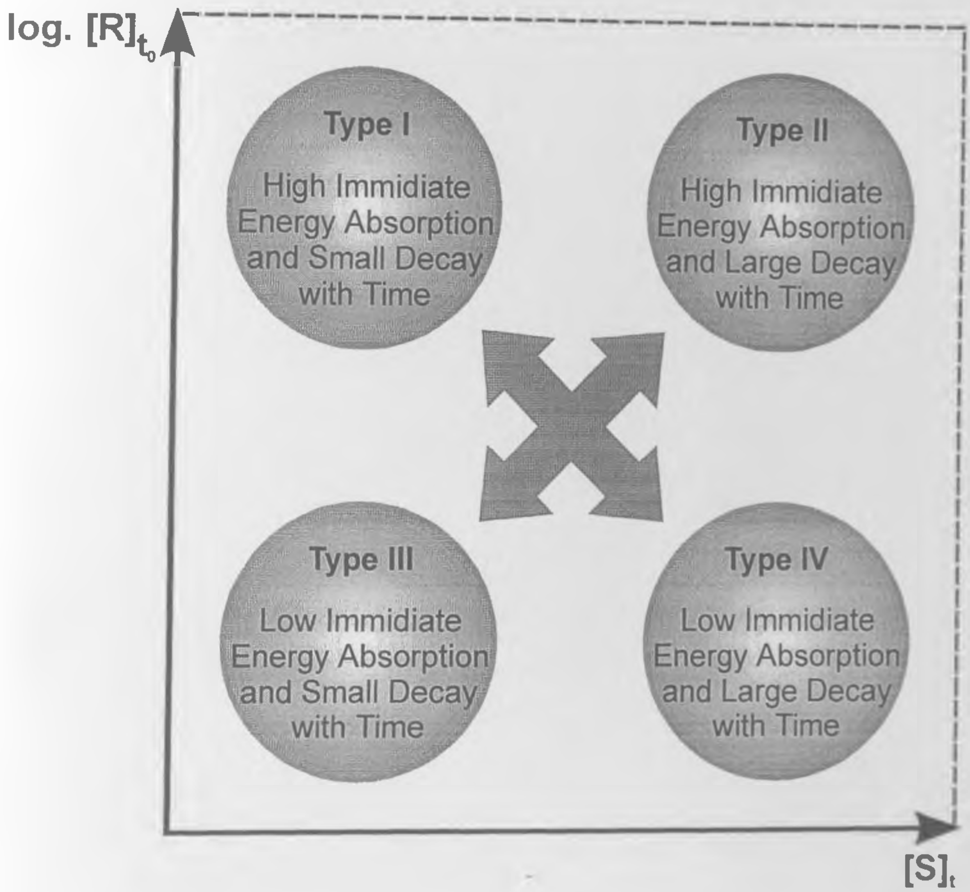


Figure 5.32 Combinations for intercept and slope values after Khan (1999)

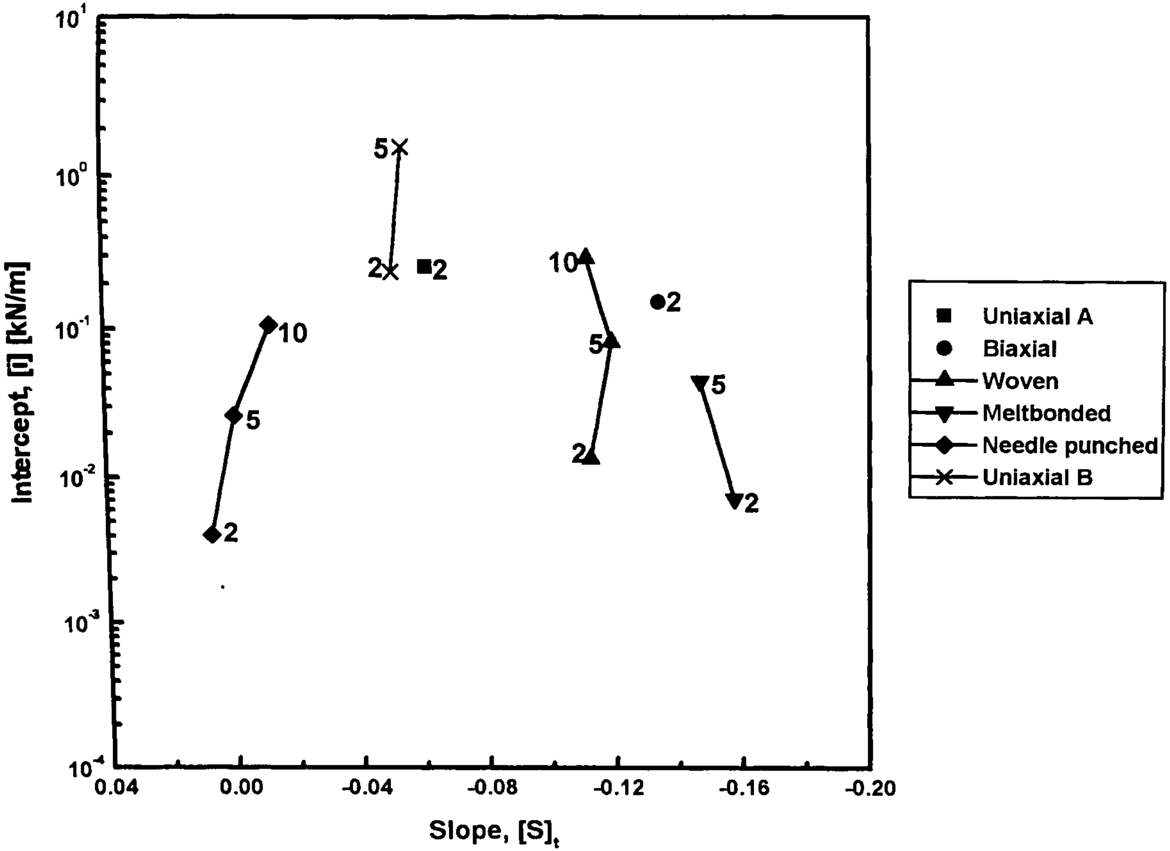
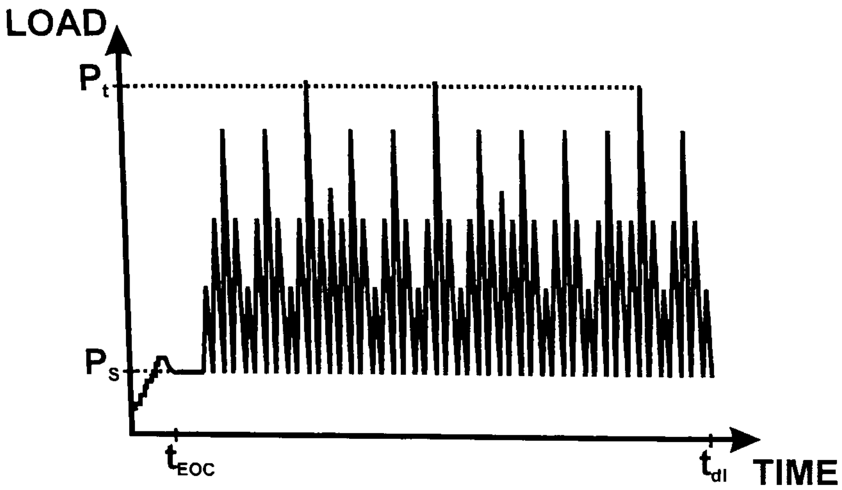
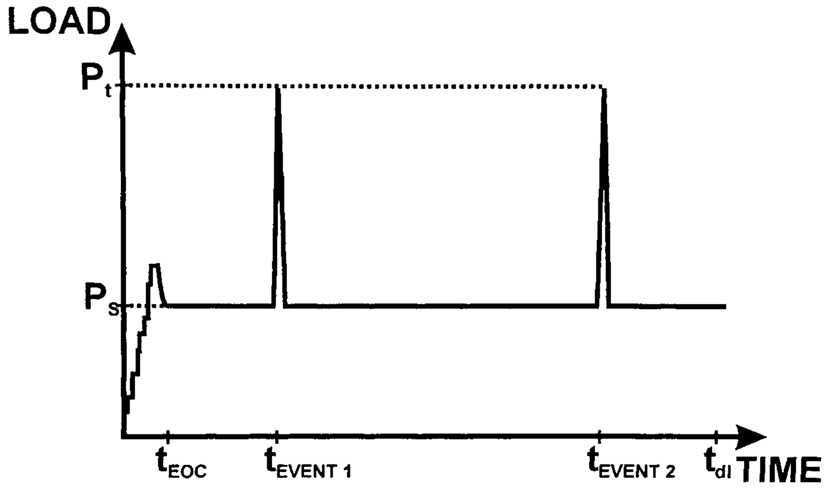


Figure 5.33 Intercept versus slope values at different limiting strains for various geosynthetics at 20°C after Khan (1999)

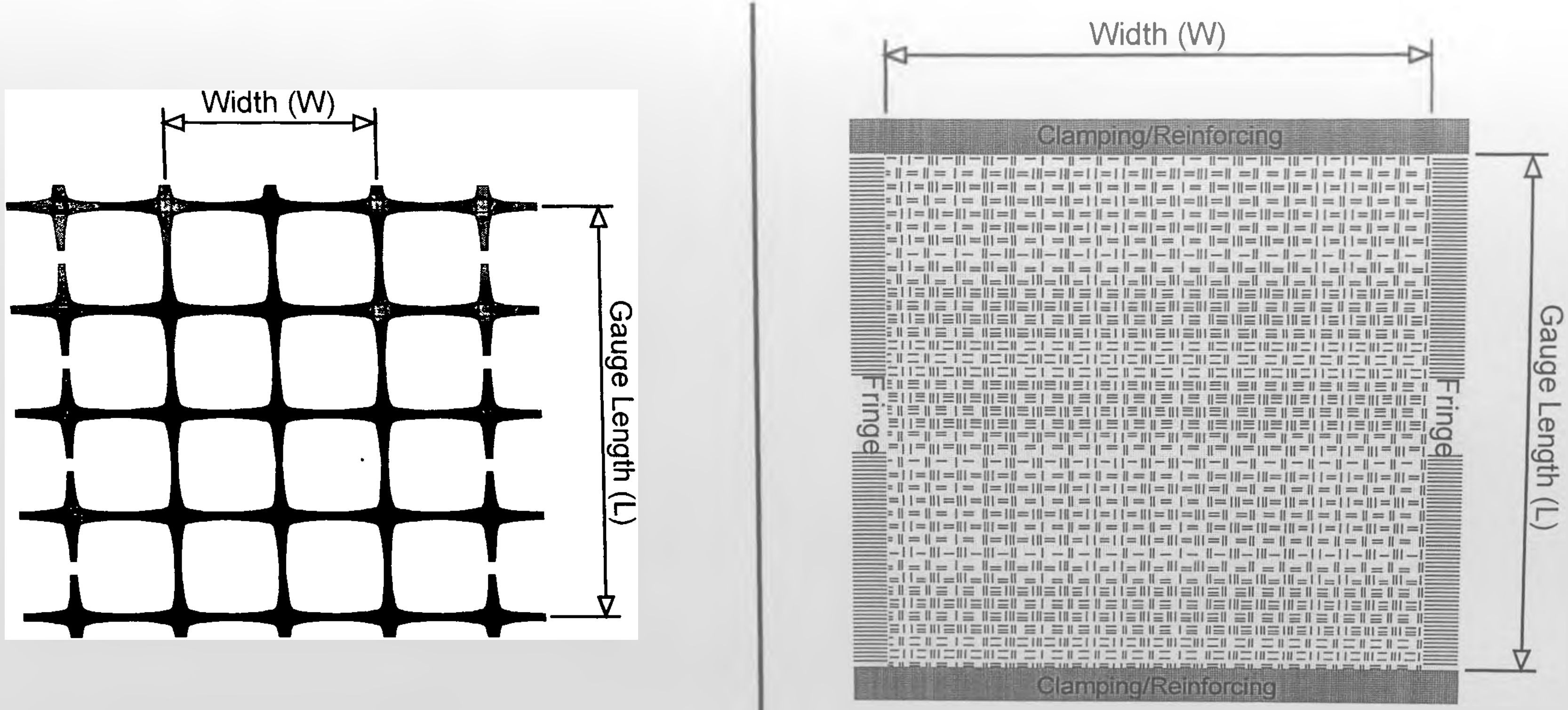


(a) Multi-Stage Actions for Load Supporting Structures



(b) Multi-Stage Actions for Earth Structures

Figure 6.1 Multi-Stage Actions for various Civil Engineering structures



(a) Geogrid specimen

(b) Geotextile specimen

Figure 6.2 Specimen dimensions

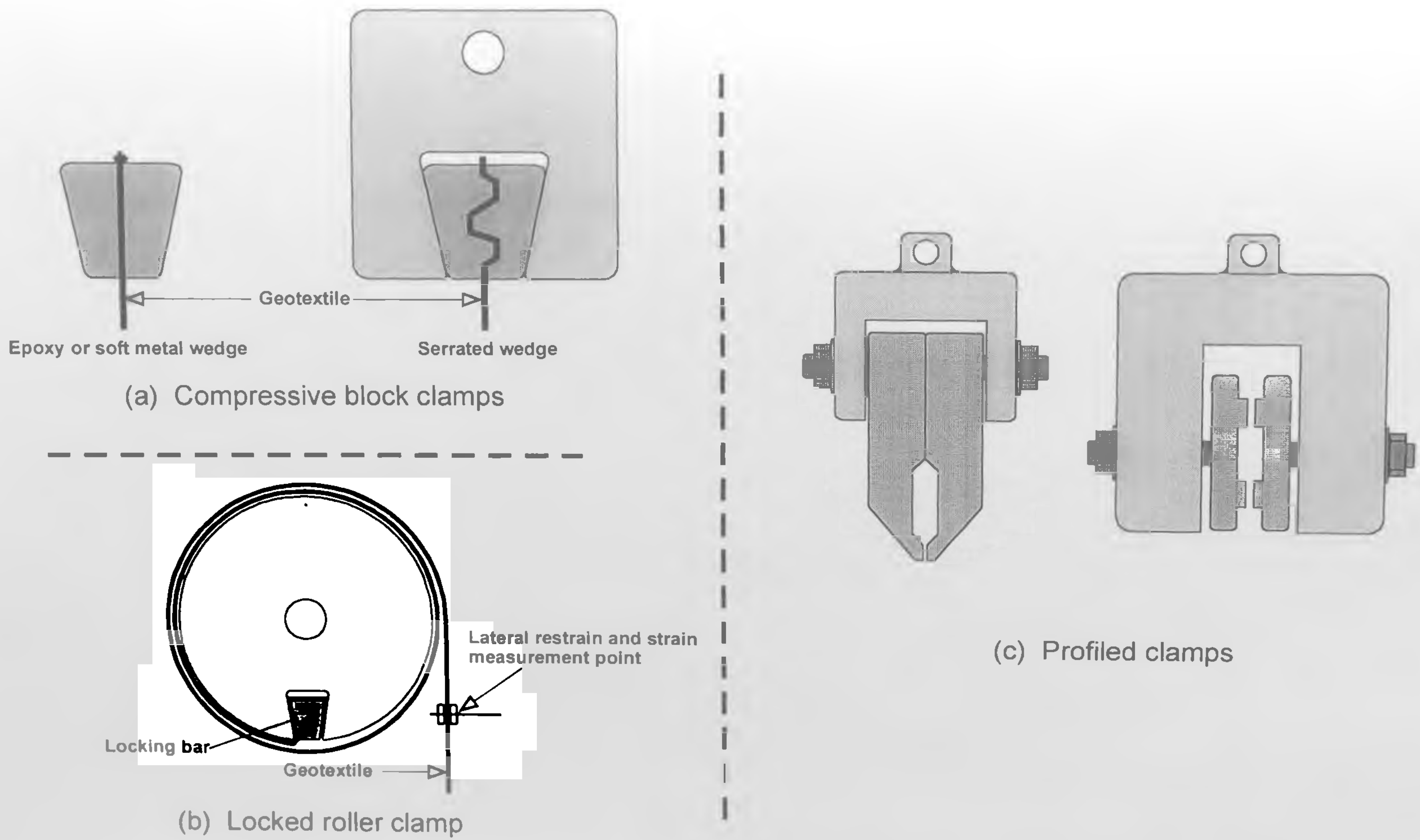


Figure 6.3 Clamps for Multi-Stage Loading tests

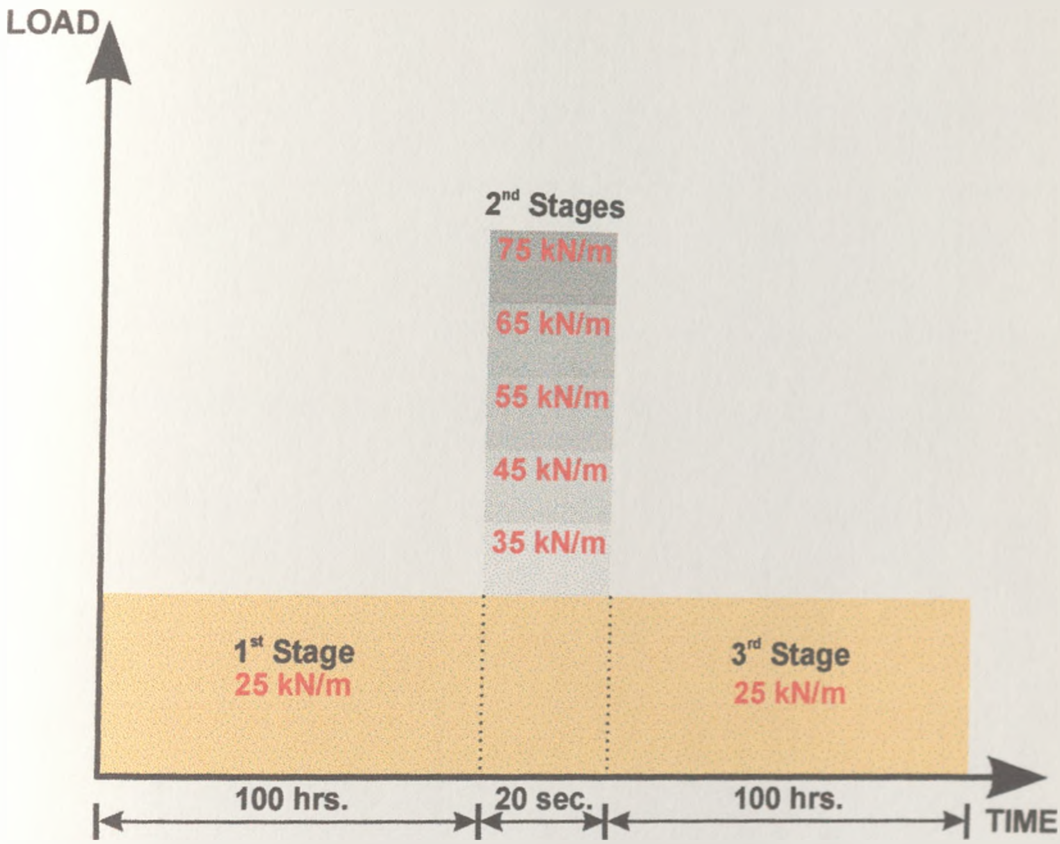


Figure 7.1 Combined Sustained-Short Term Loading test sequence

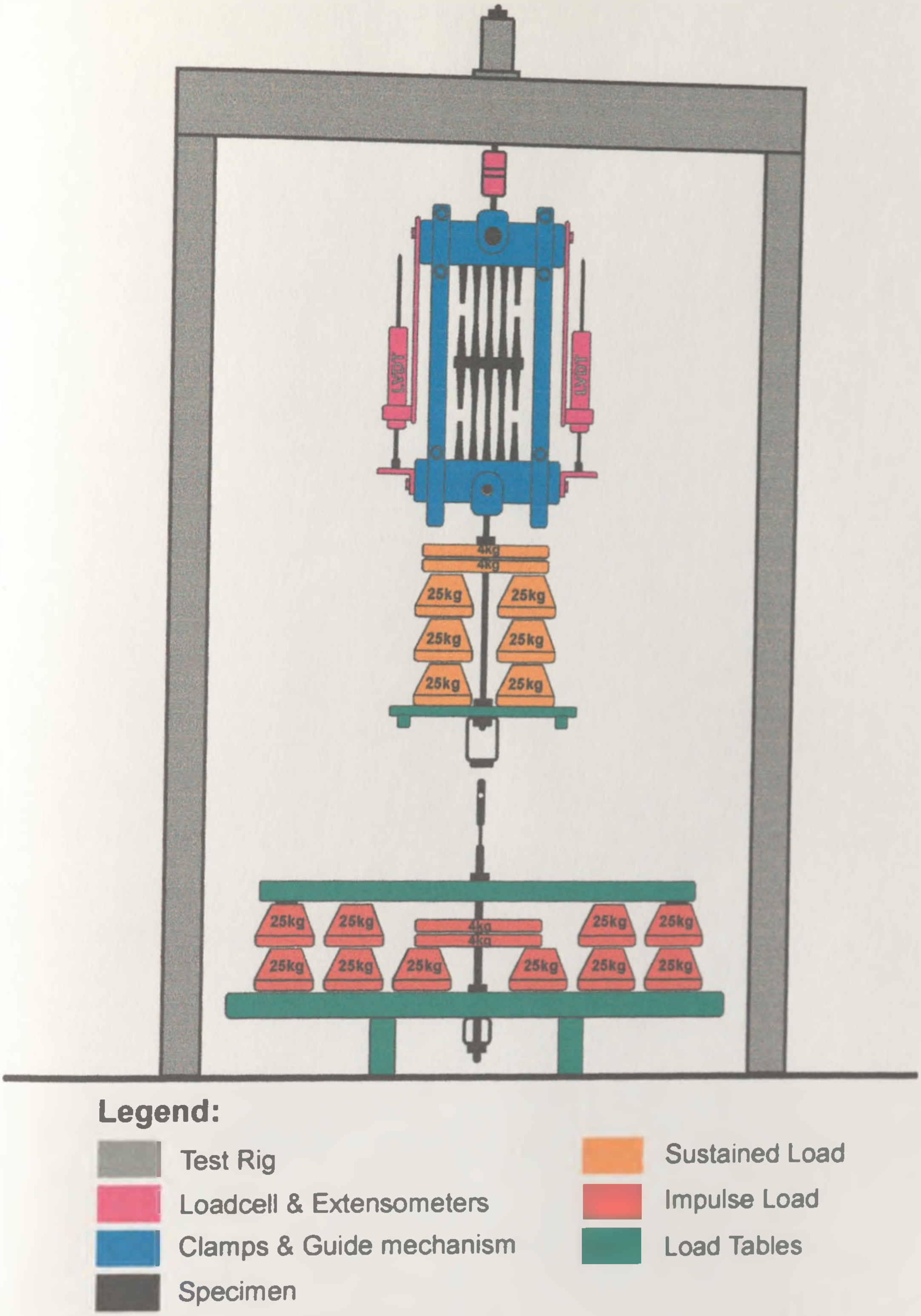
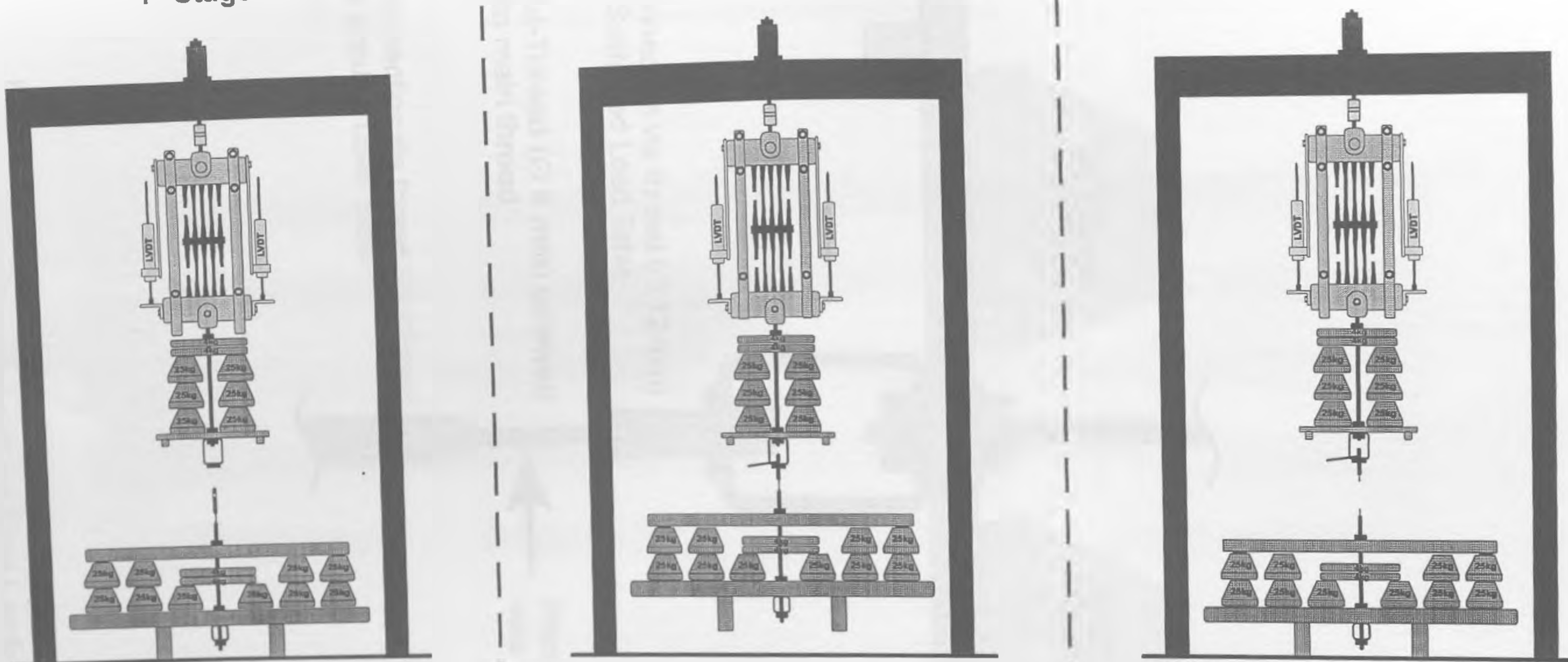


Figure 7.2 Scheme of Test Apparatus used in Combined Sustained-Short Term Loading test

1st Stage

2nd Stage

3rd Stage



(i) Sustained loading regime

(ii) Sustained and additional loading regime

(iii) Sustained loading regime
(after removal of additional loading)

Figure 7.3 Combined Sustained-Short Term Loading test apparatus during various Loading Regimes

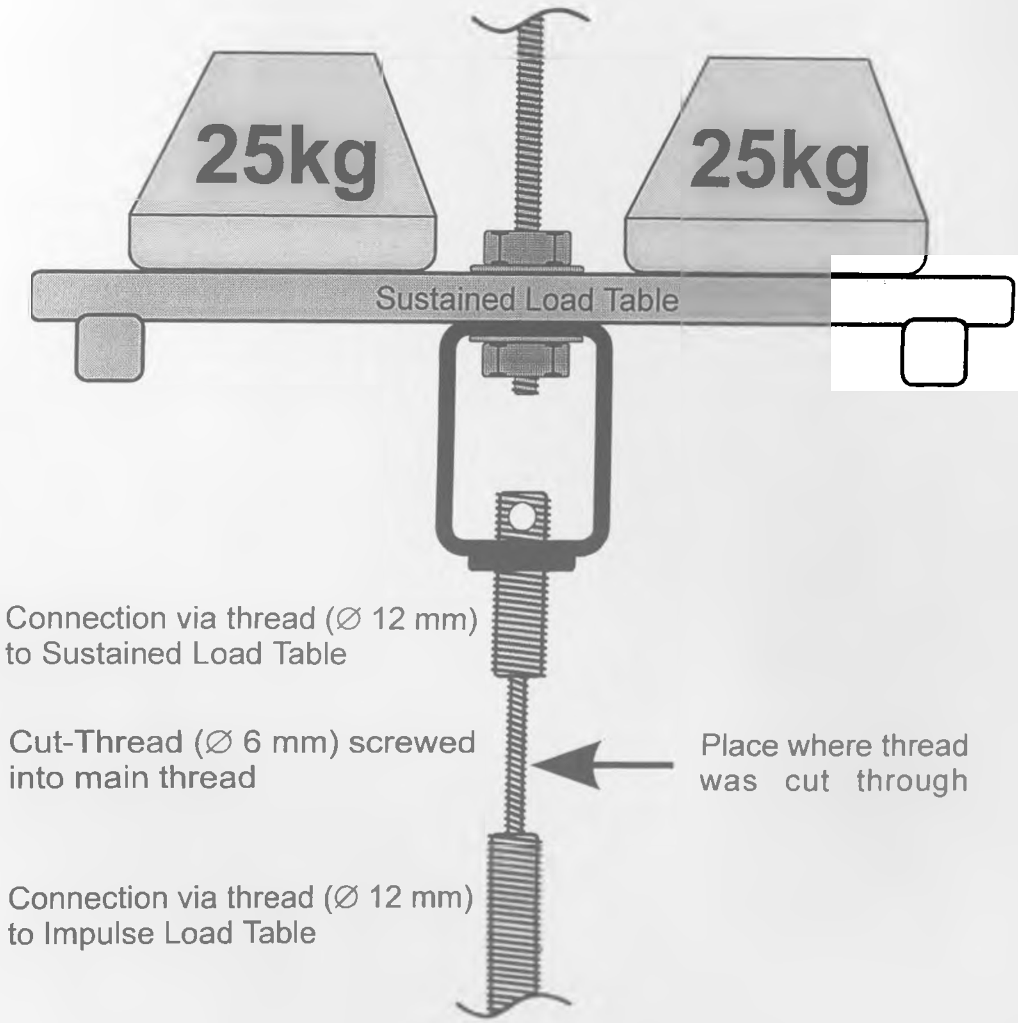


Figure 7.4 Scheme of Release Mechanism used in the Combined Sustained-Short Term Loading test

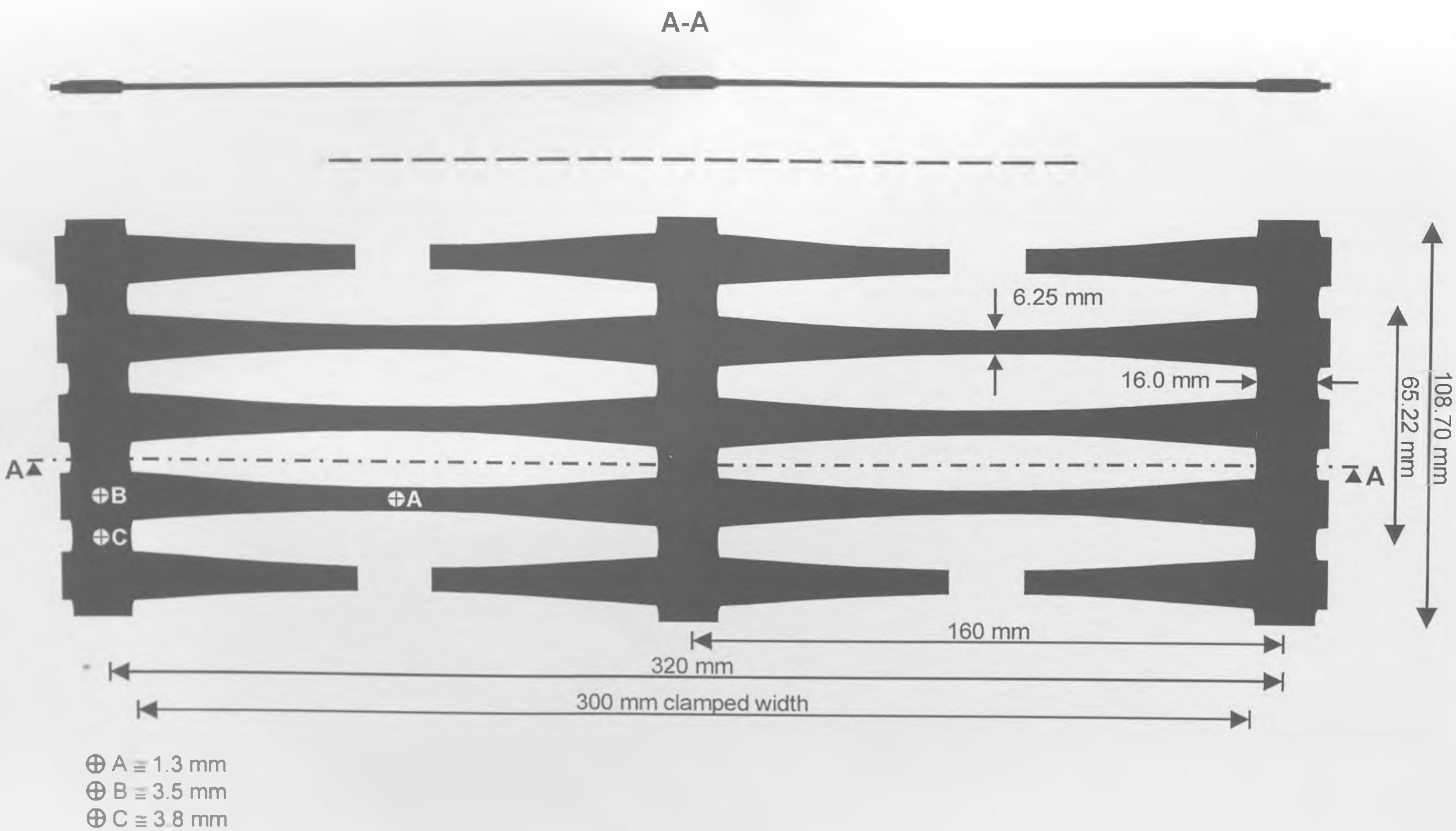
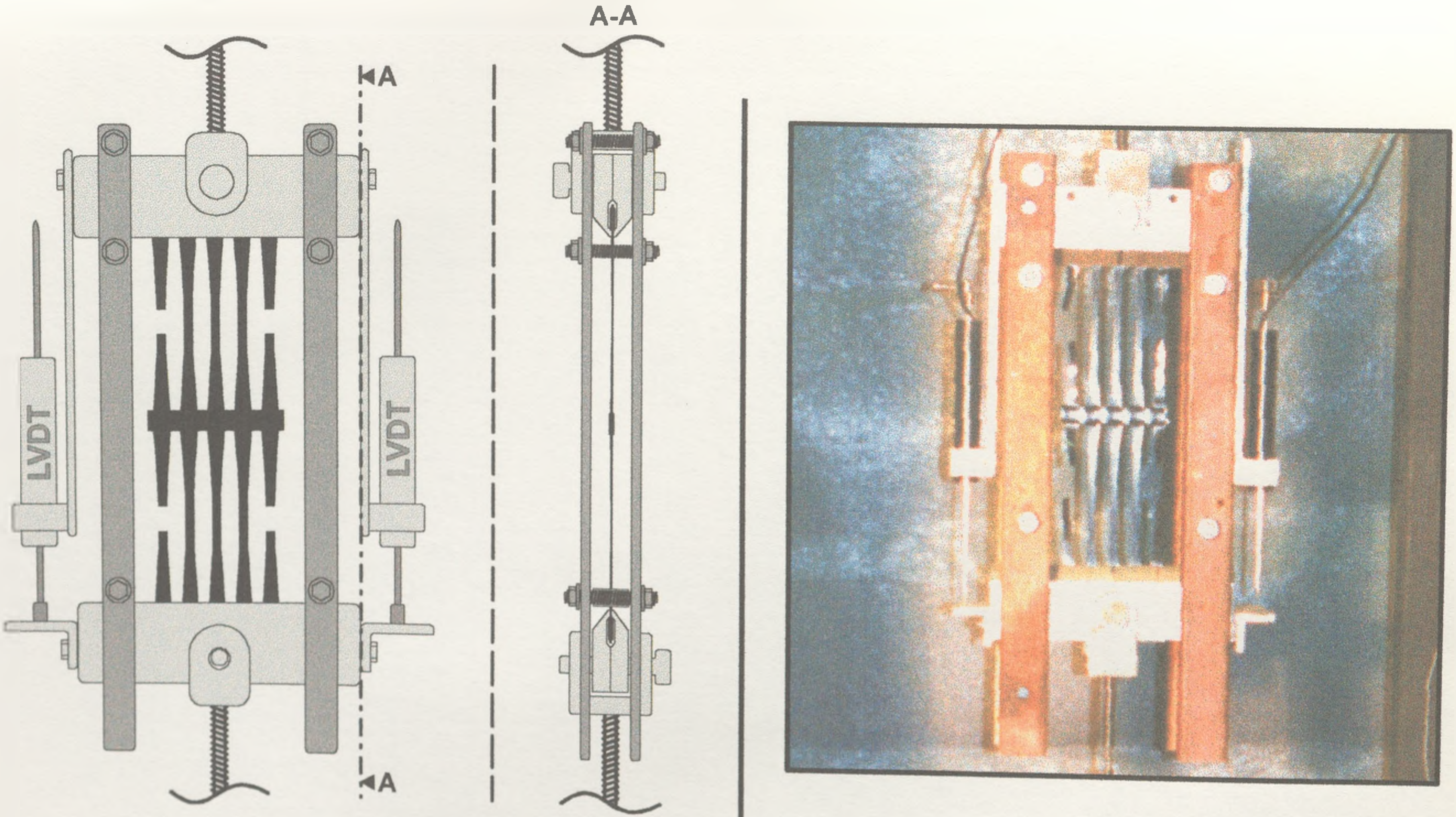


Figure 7.5 Prepared Geogrid B specimen



(a) Scheme

(b) Plate

Figure 7.6 Unloading Guide mechanism

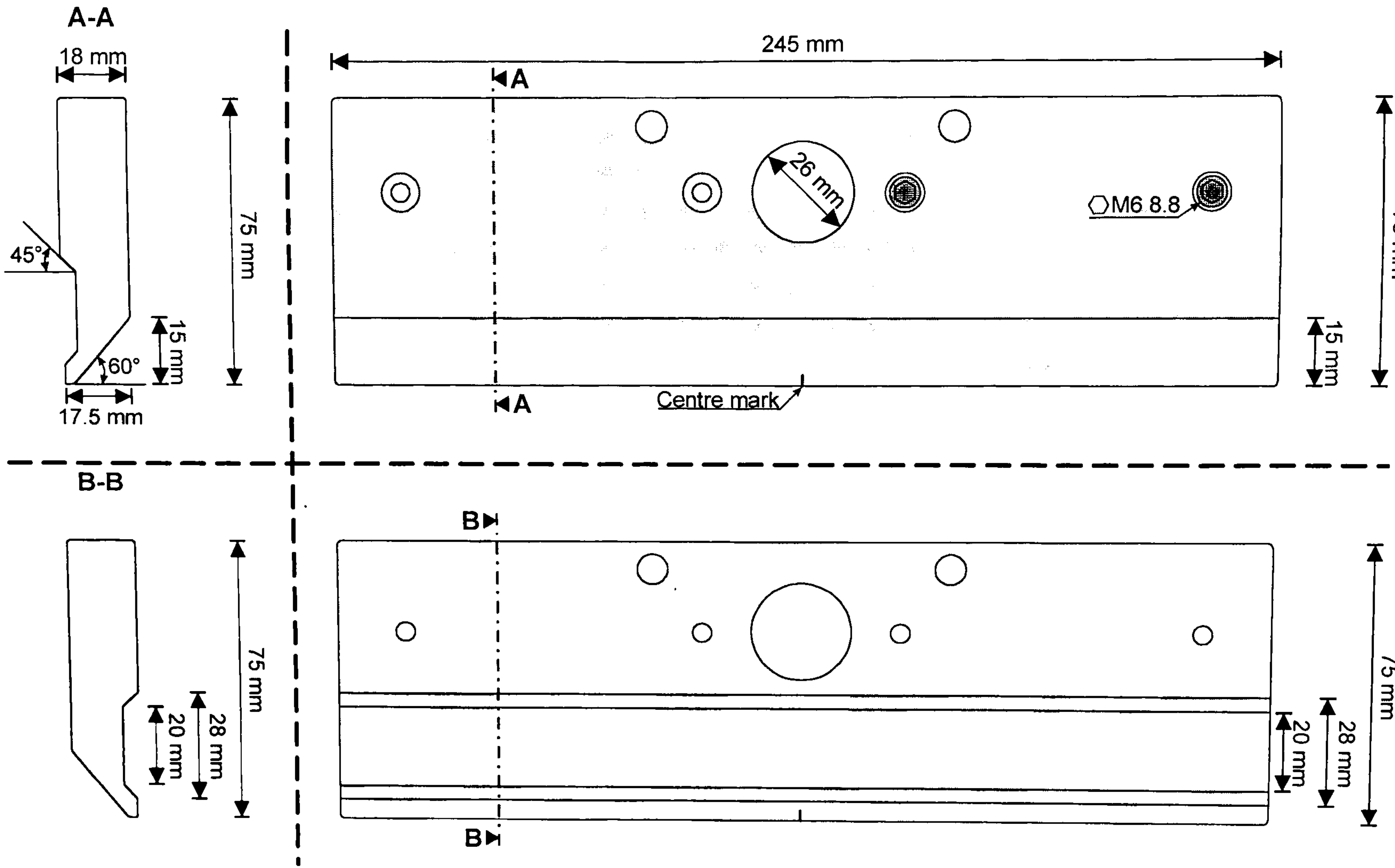


Figure 7.7 Profiled Clamps (recommended by Manufacturer)

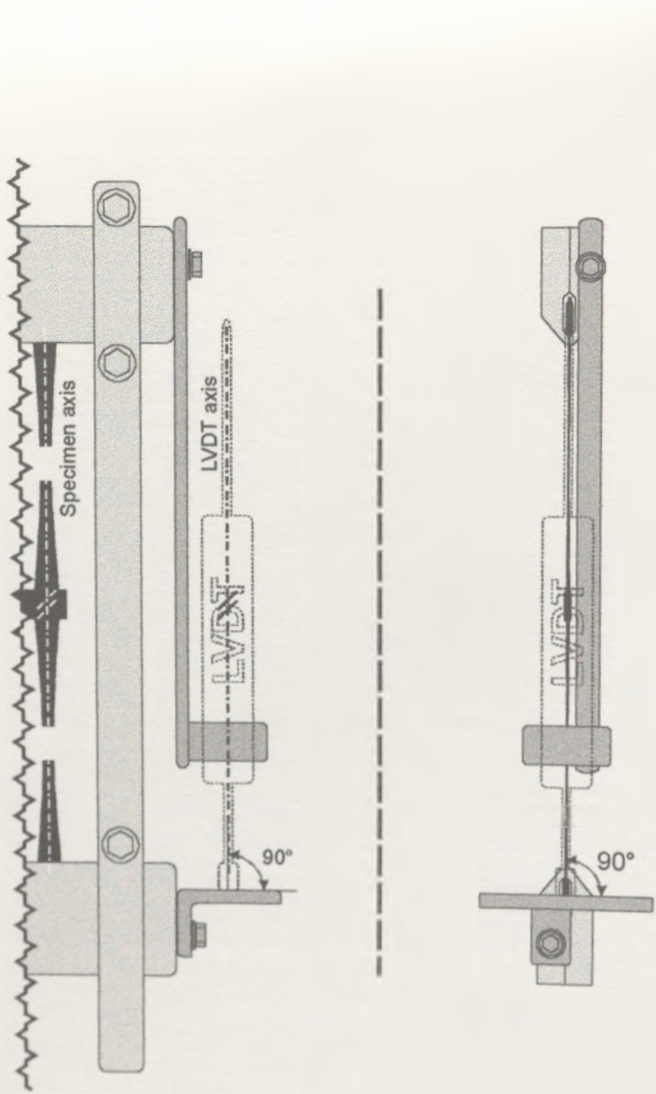


Figure 7.8 Universal Couplings

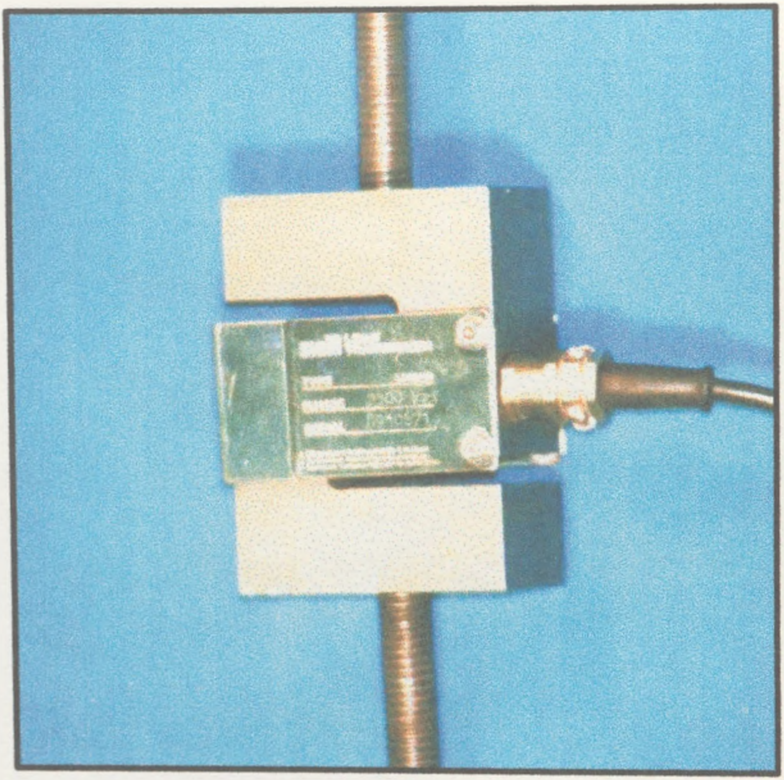


Figure 7.9 Load cell

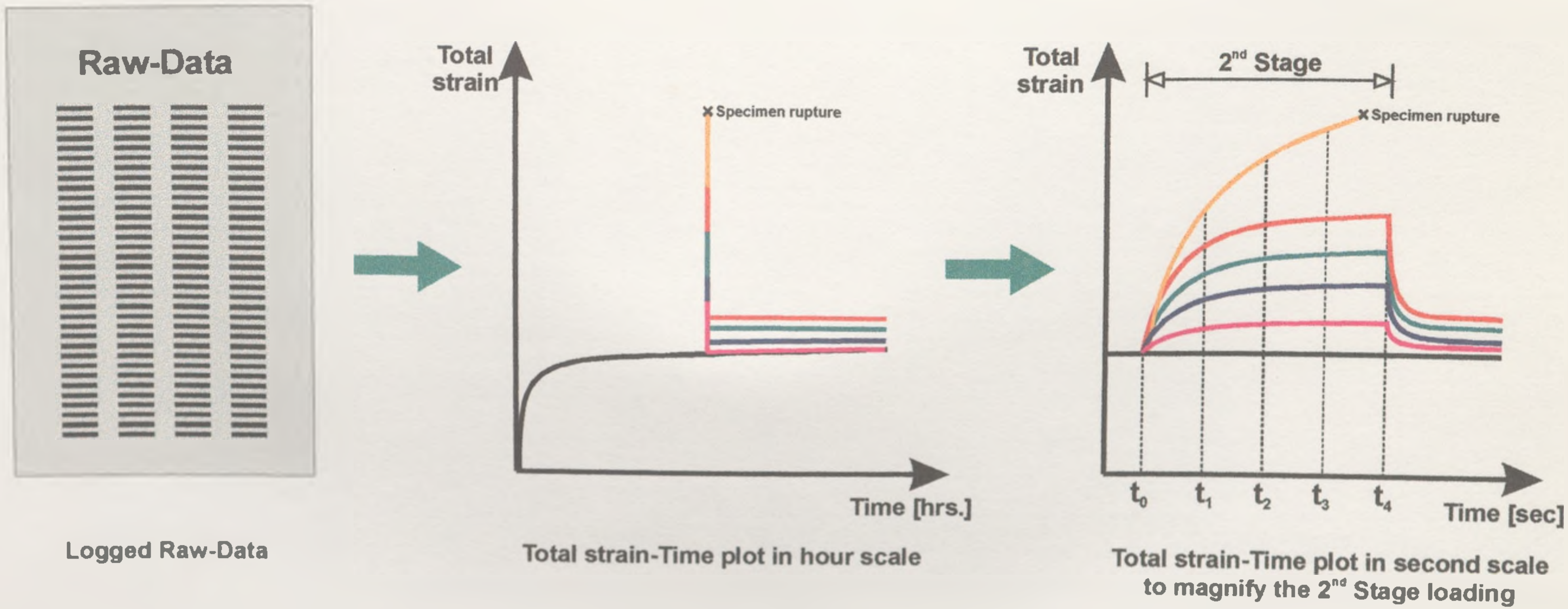


Figure 7.10 Strain-time plots for short-term loading (2nd Stage)

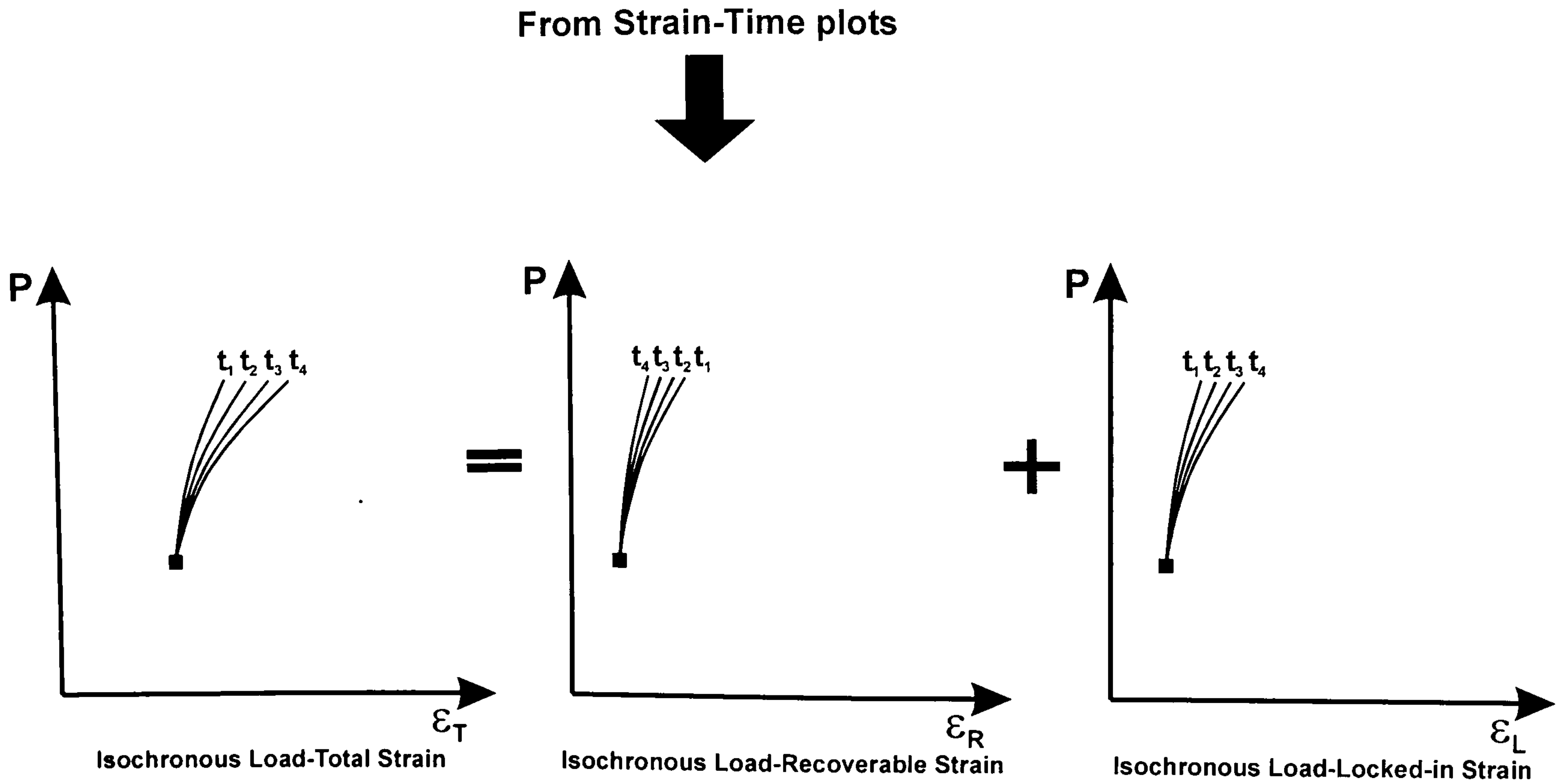


Figure 7.11 Isochronous Load-strain curves for seismic loading (2nd Stage)

$$\Delta[A]_{t_2} = \Delta[R]_{t_2} + \Delta[L]_{t_2}$$

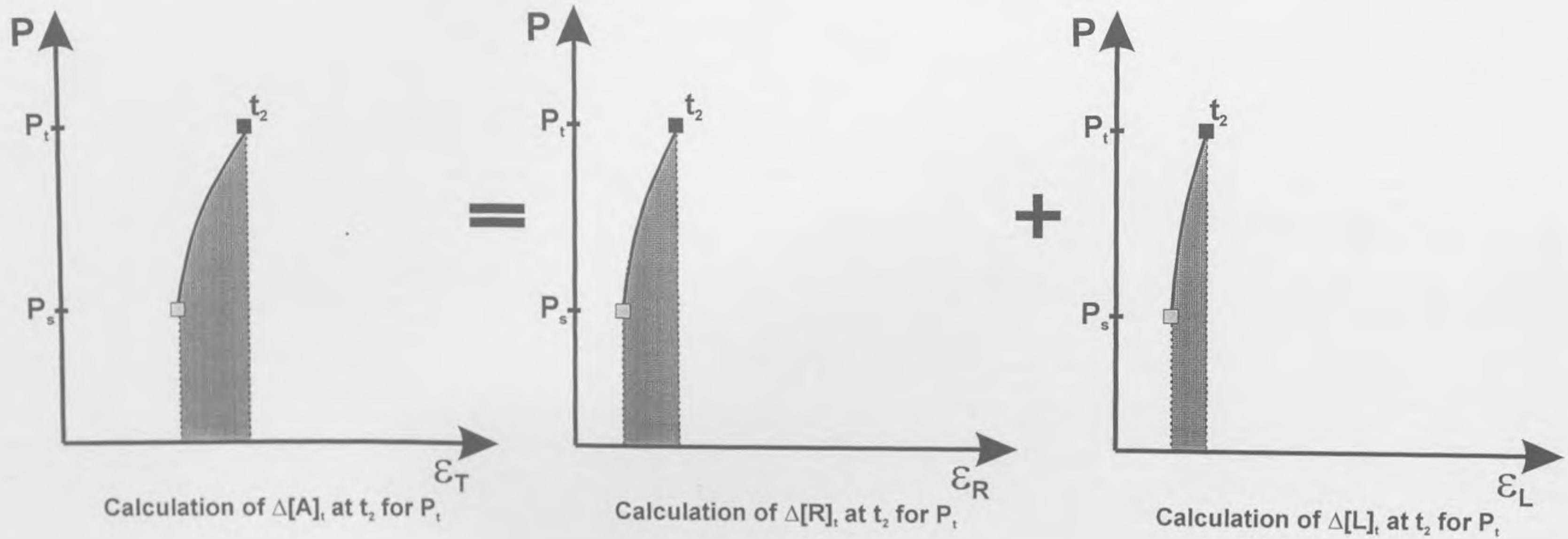
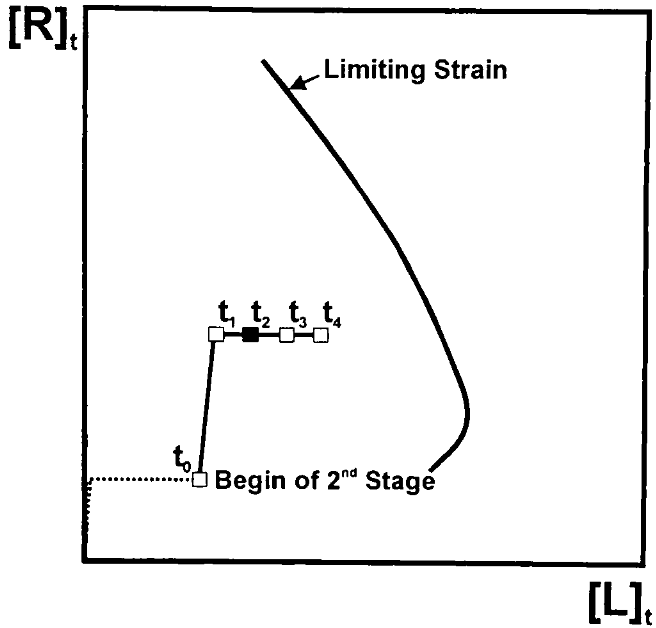


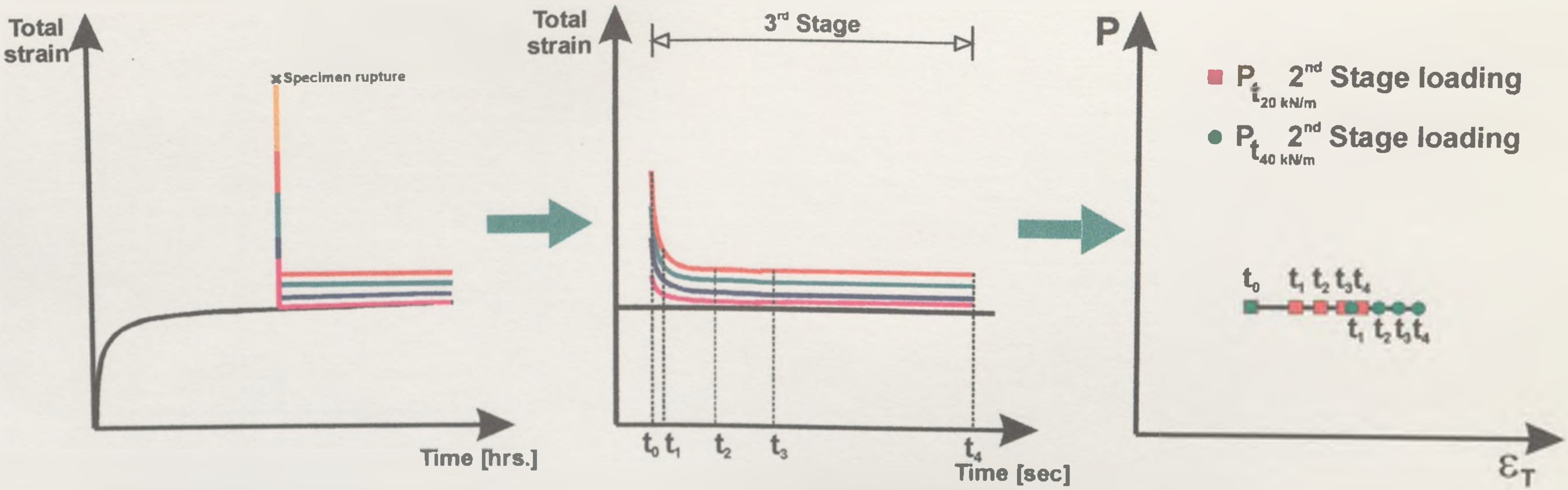
Figure 7.12 ISE components calculation for seismic loading (2nd Stage)

$[R]_{t_2}$ and $[L]_{t_2}$
from calculated area under IS curves.
(Same methodology was used to obtain values for other times)



$[R]_t$ and $[L]_t$ plot for 2nd Stage loading

Figure 7.13 $[R]_t$ - $[L]_t$ plot for seismic loading (2nd Stage)



Total Strain-Time plot in hour scale

Total Strain-Time plot in second scale for the 3rd Stage loading

Isochronous Load-Total Strain curves for certain 2nd Stage load levels

Figure 7.14 Strain-time plots for further sustained loading (3rd Stage)

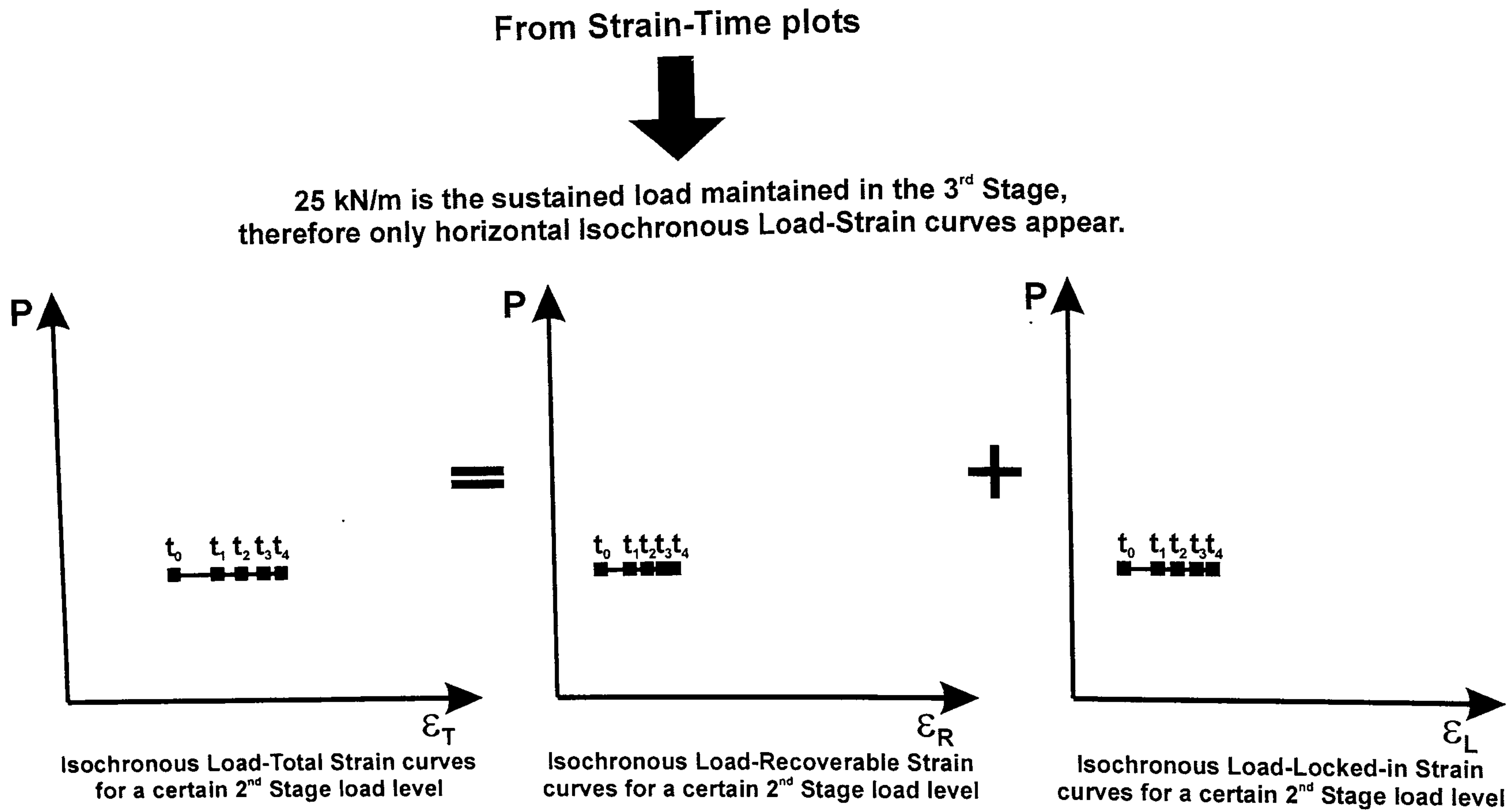


Figure 7.15 Isochronous Load-strain curves for further sustained loading (3rd Stage)

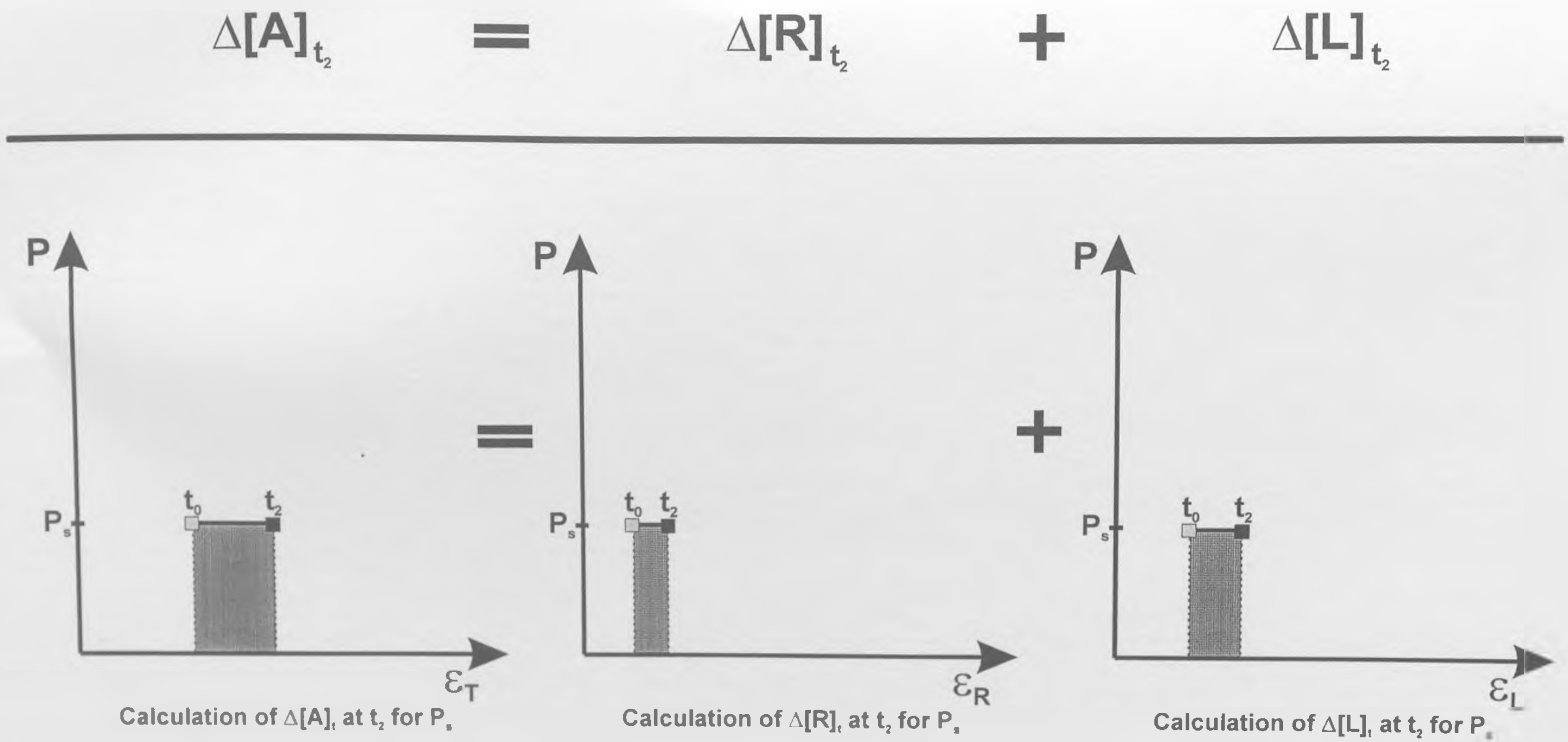
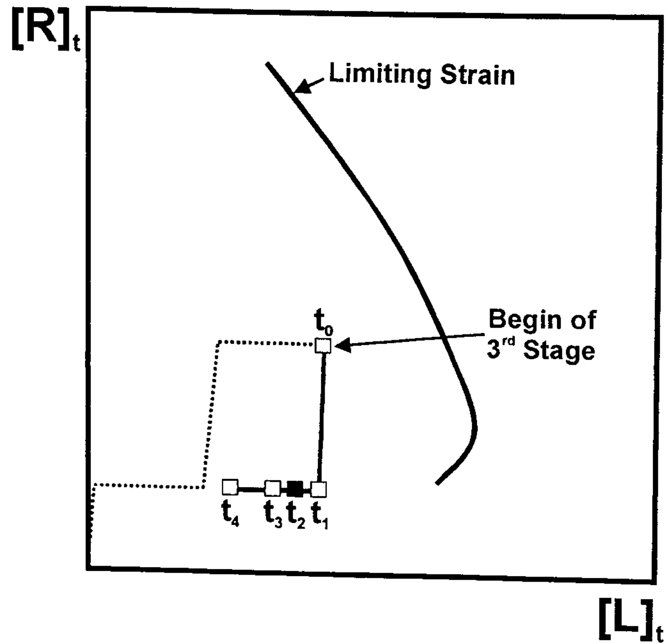


Figure 7.16 ISE components calculation for further sustained loading (3[rd] Stage)

$[R]_{t_2}$ and $[L]_{t_2}$

from calculated area under IS curves.

(Same methodology was used to obtain values for other times and other load levels)



3rd Stage $[R]_t$ - $[L]_t$ plot for a certain previously applied 2nd Stage load

Figure 7.17 $[R]_t$ - $[L]_t$ plot for further sustained loading (3rd Stage)

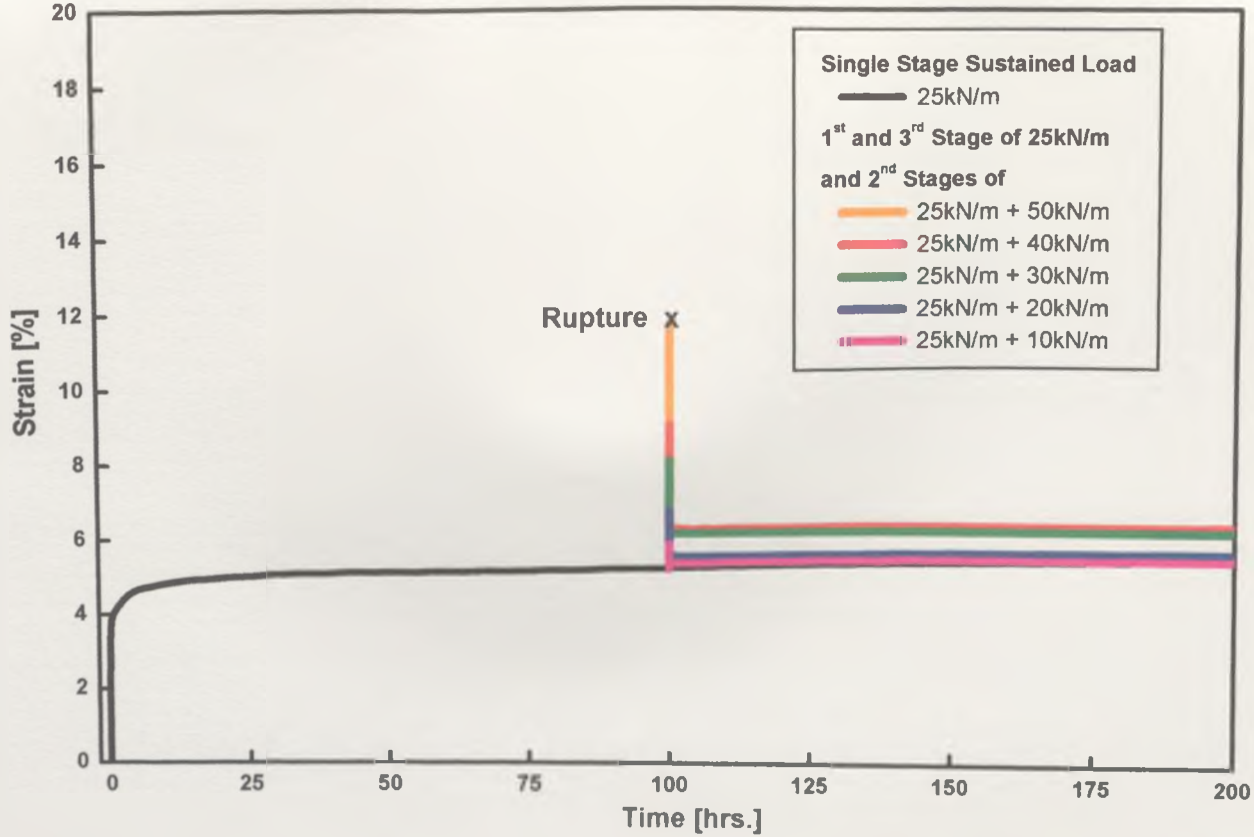


Figure 7.18 Combined Sustained-Short Term Loading test results in hour scale at 20°C

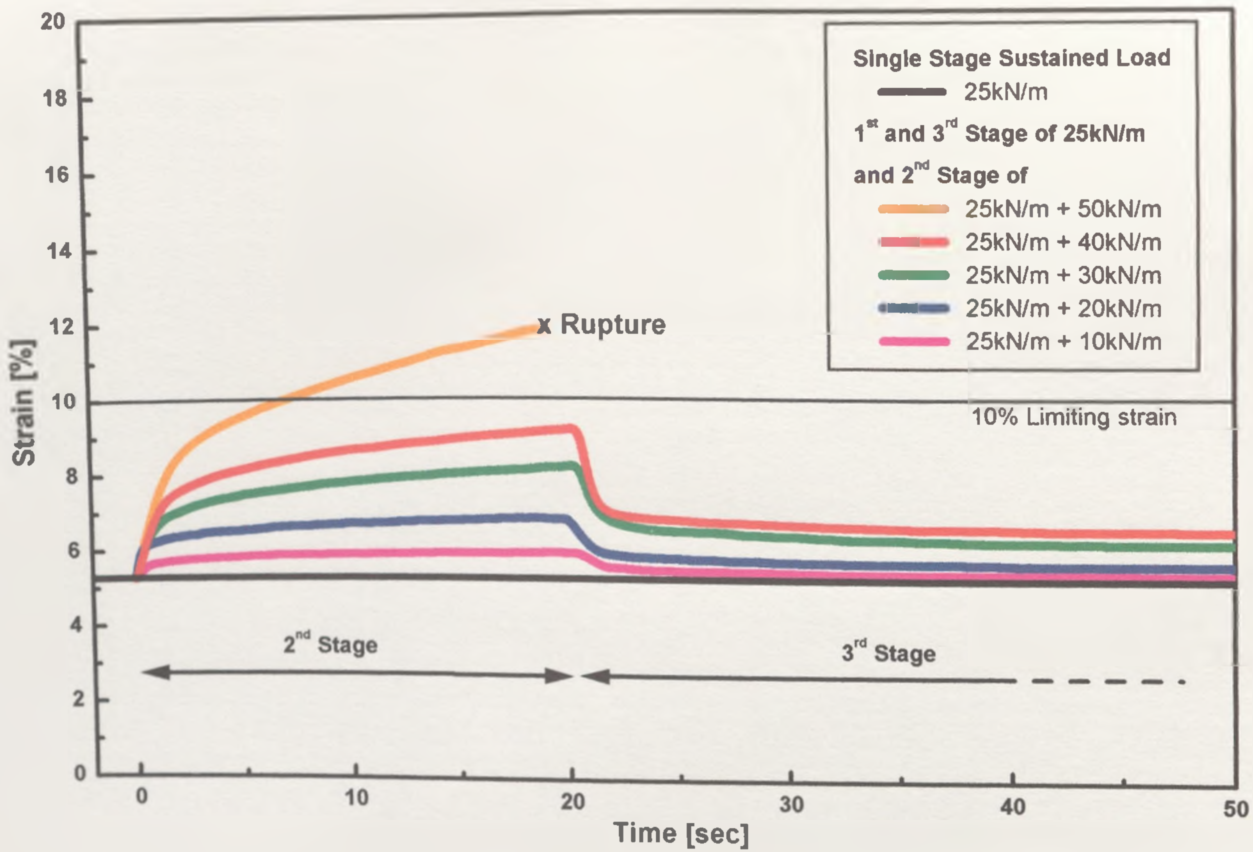


Figure 7.19 Combined Sustained-Short Term Loading test results for the 2nd Stage in second scale at 20°C

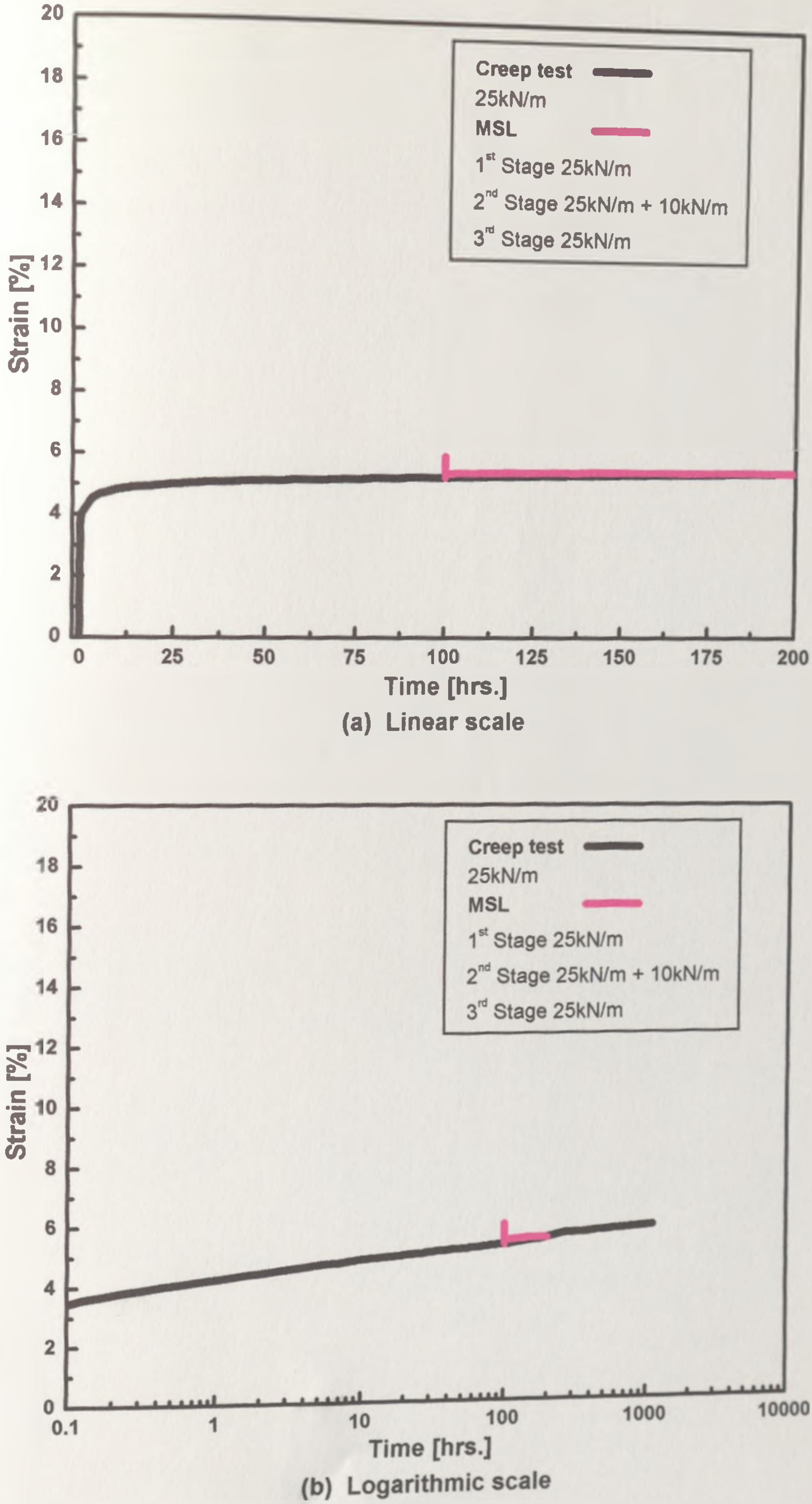
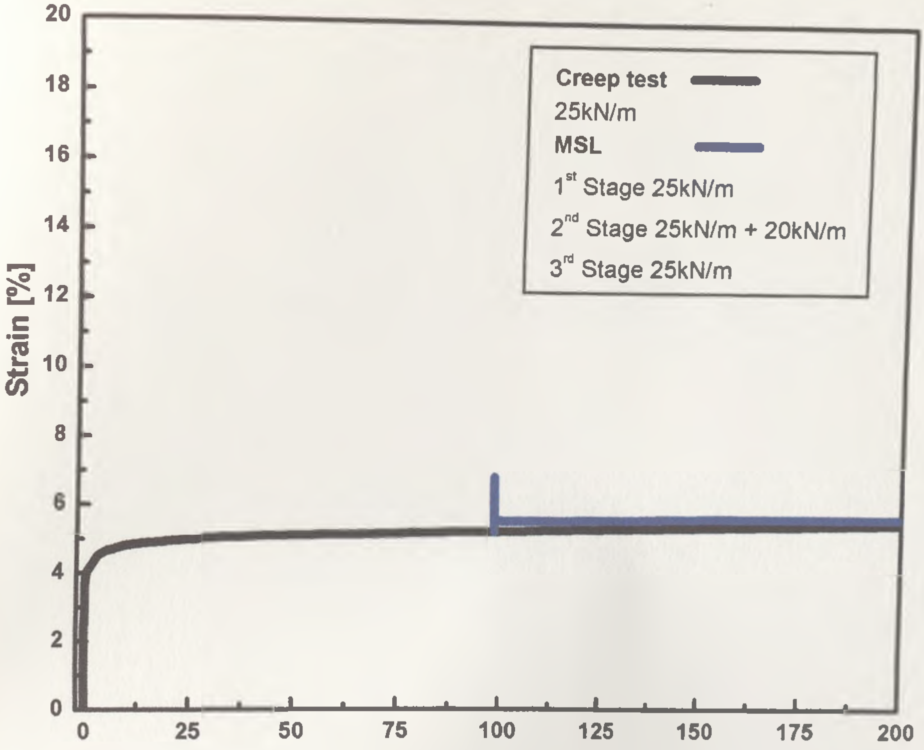
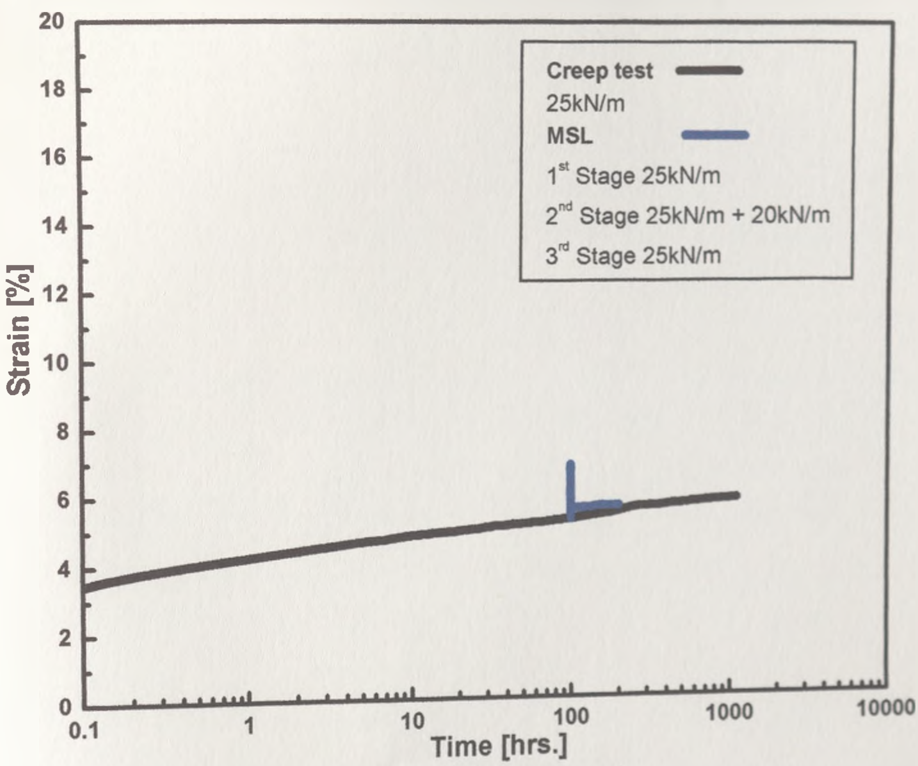


Figure 7.20 10 kN/m 2nd Stage loading at 20°C

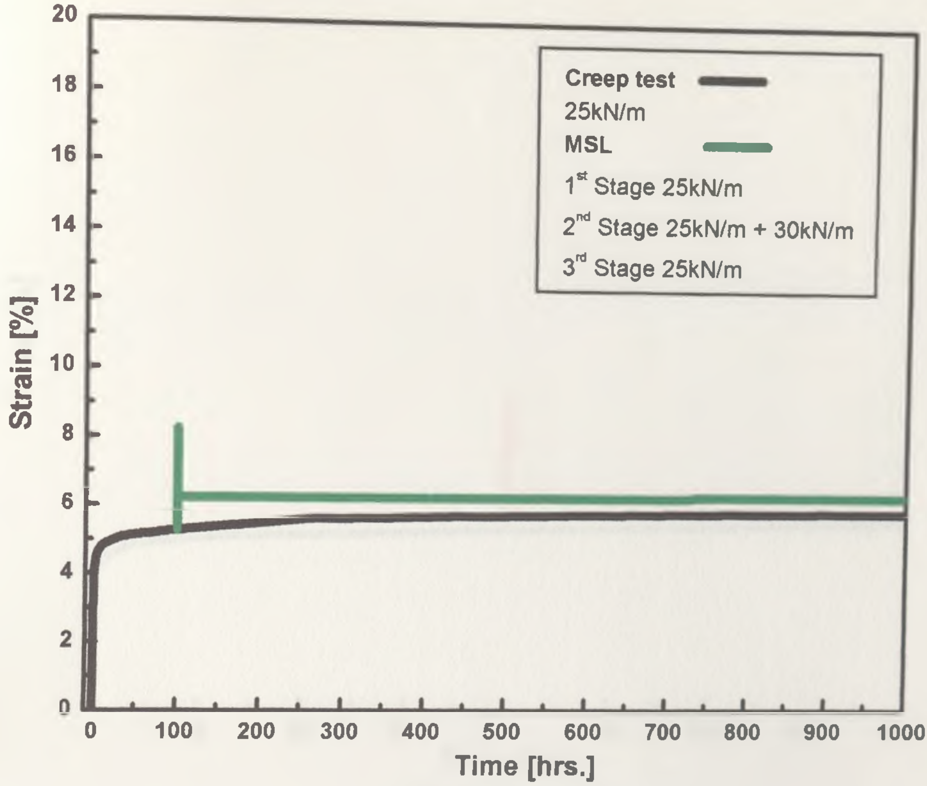


(a) Linear scale

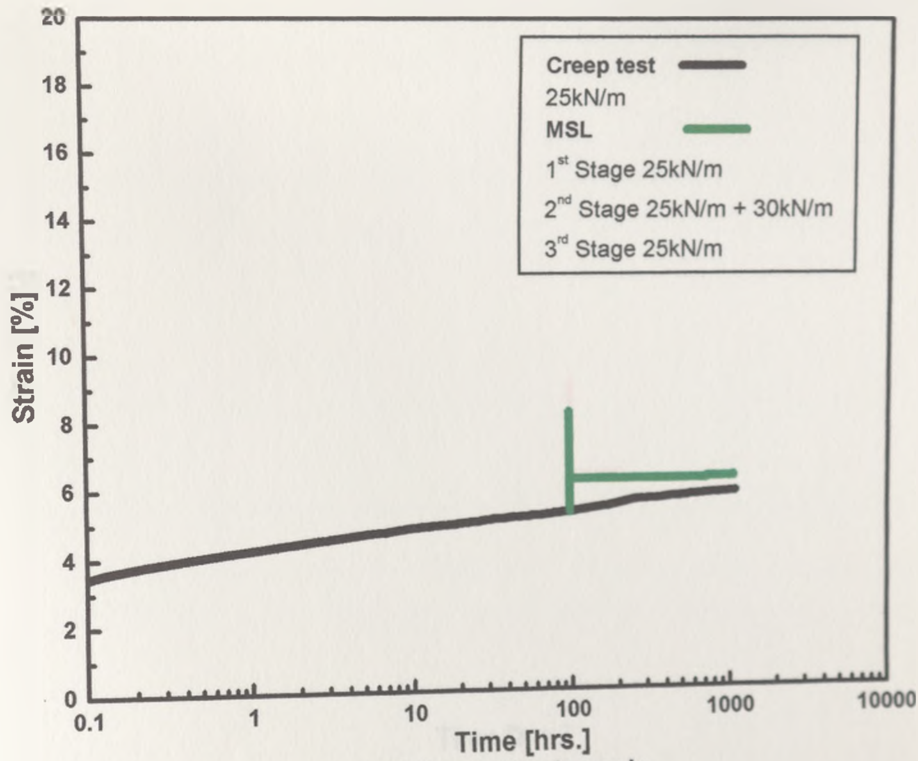


(b) Logarithmic scale

Figure 7.21 20 kN/m 2nd Stage loading at 20°C

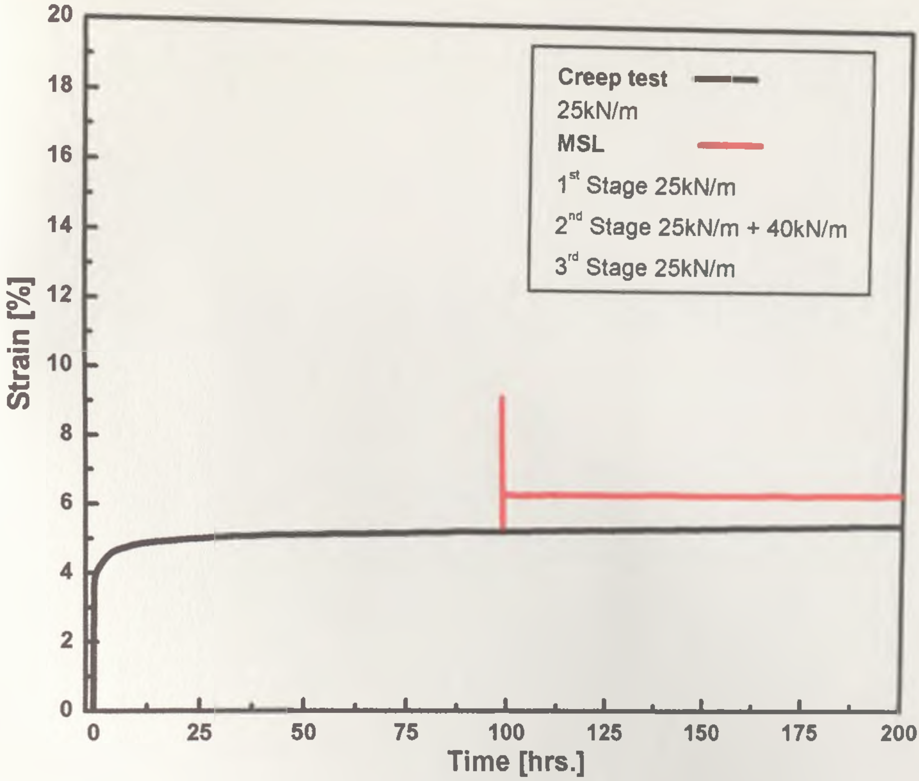


(a) Linear scale

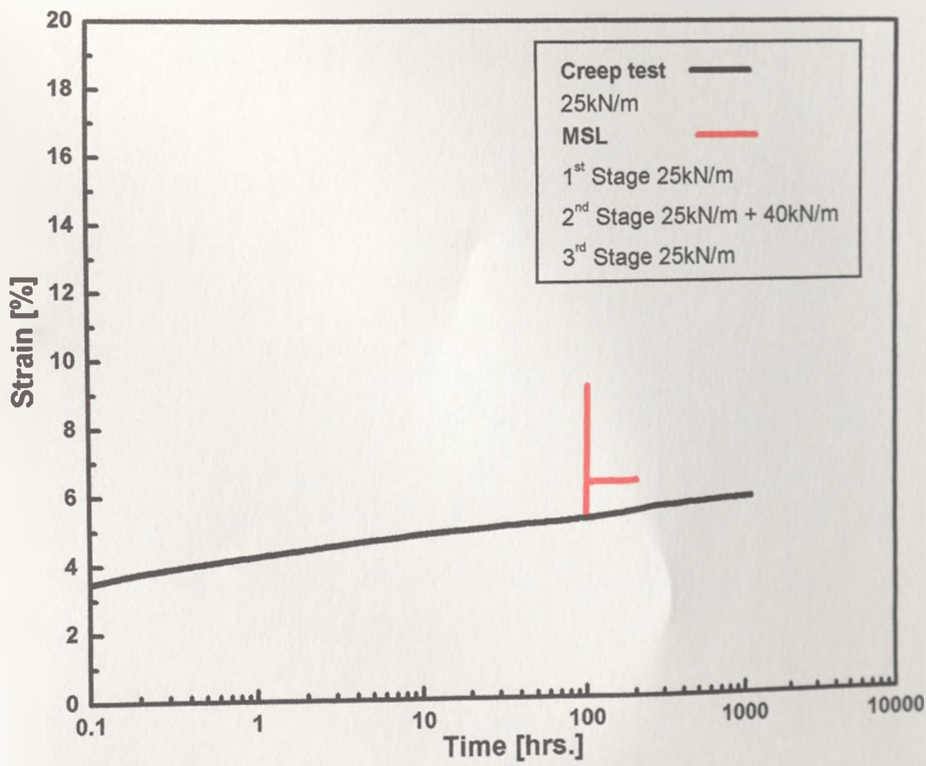


(b) Logarithmic scale

Figure 7.22 30 kN/m 2nd Stage loading at 20°C

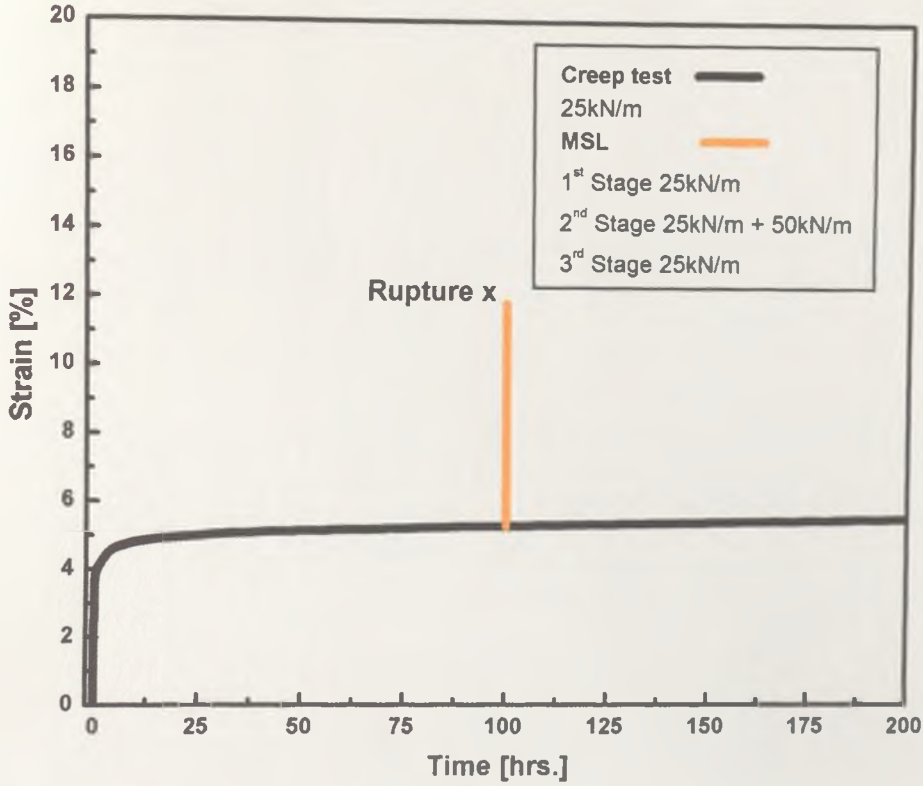


(a) Linear scale

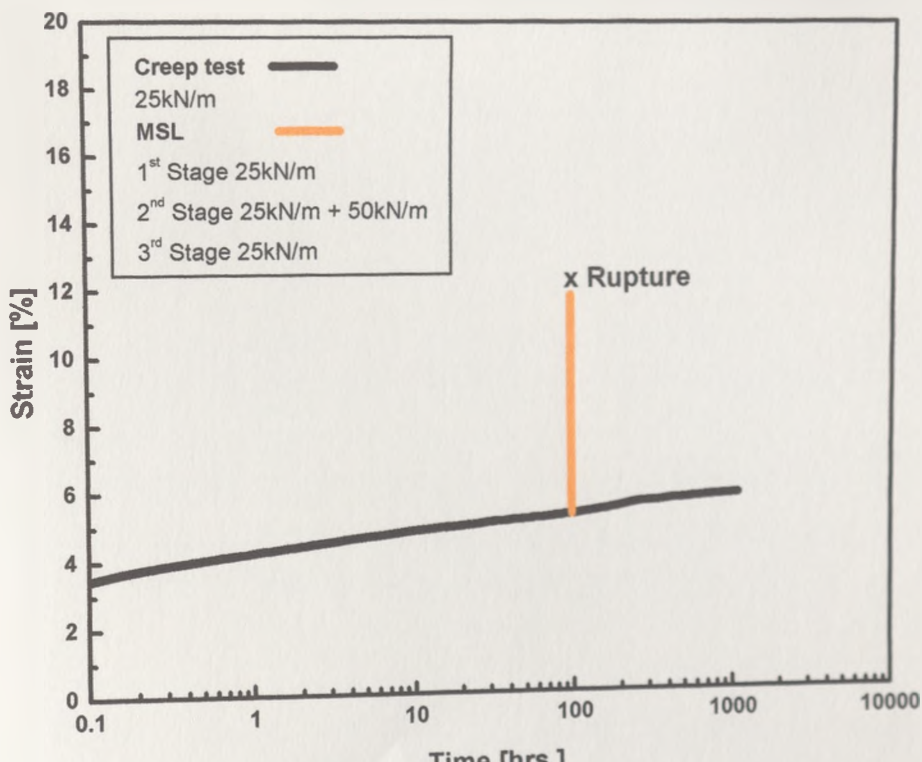


(b) Logarithmic scale

Figure 7.23 40 kN/m 2nd Stage loading at 20°C

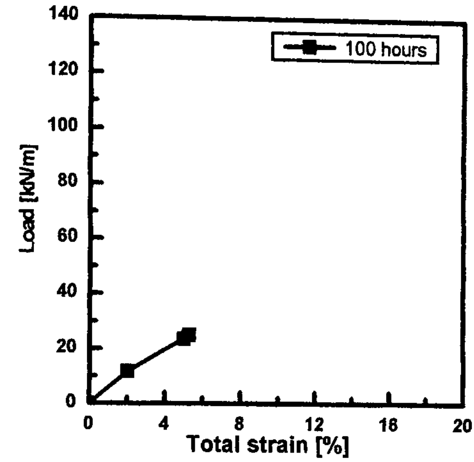


(a) Linear scale

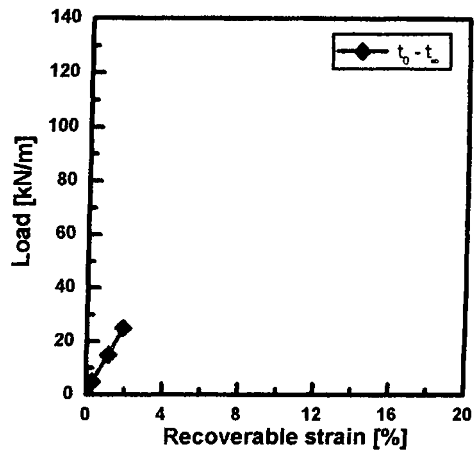


(b) Logarithmic scale

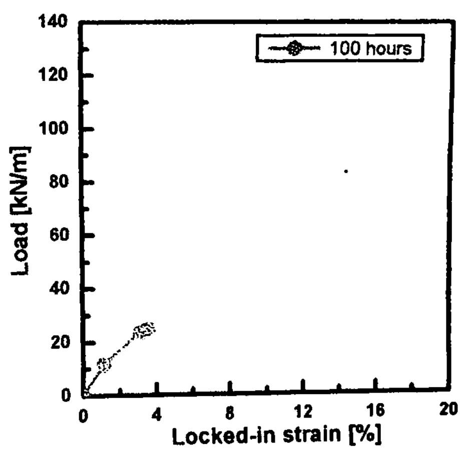
Figure 7.24 50 kN/m 2nd Stage loading at 20°C



(a) Isochronous Load Total-strain curve



(b) Isochronous Load Recoverable-strain curve



(c) Isochronous Load Locked-in strain curve

Figure 7.25 Isochronous Load-strain curves for 1[st] Stage loading at 20°C

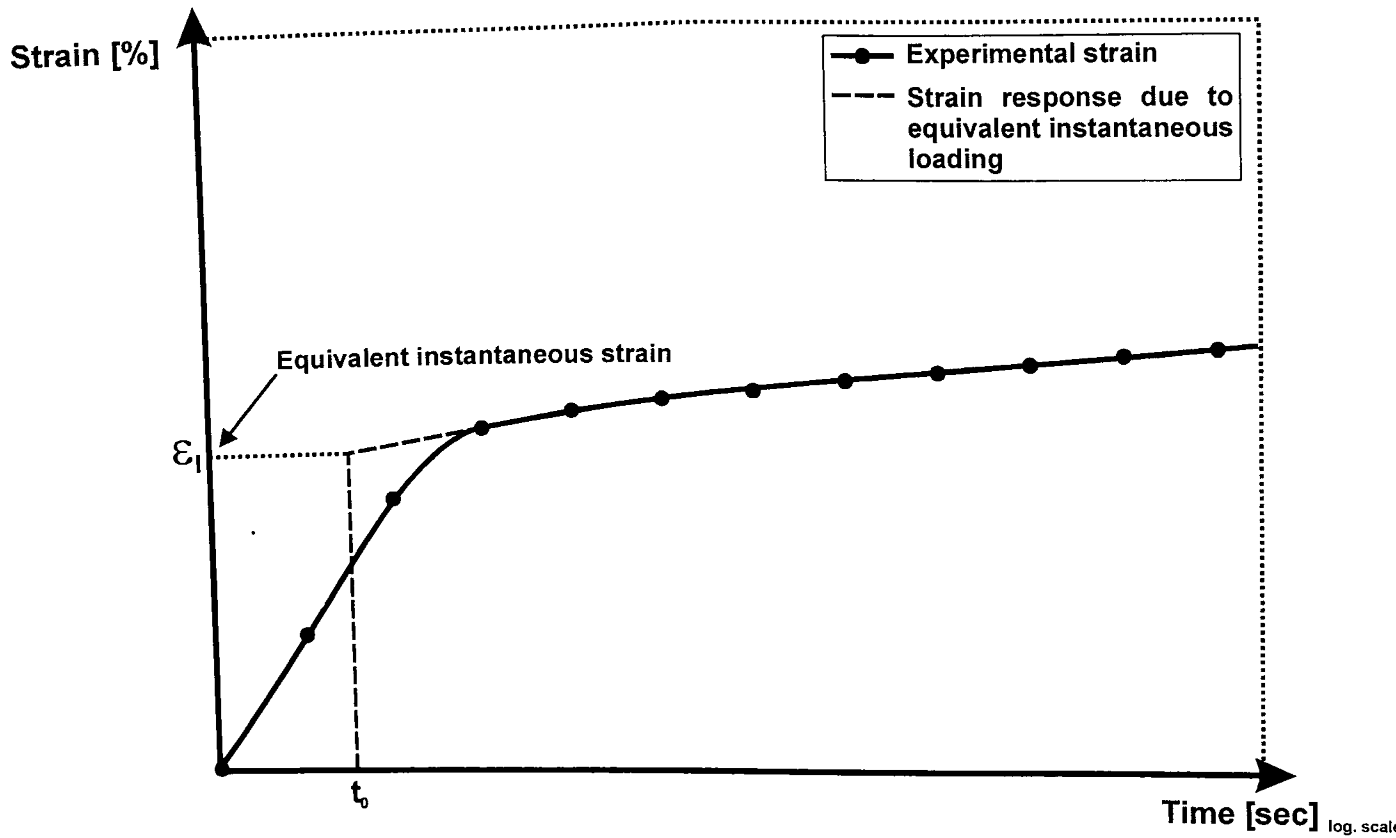
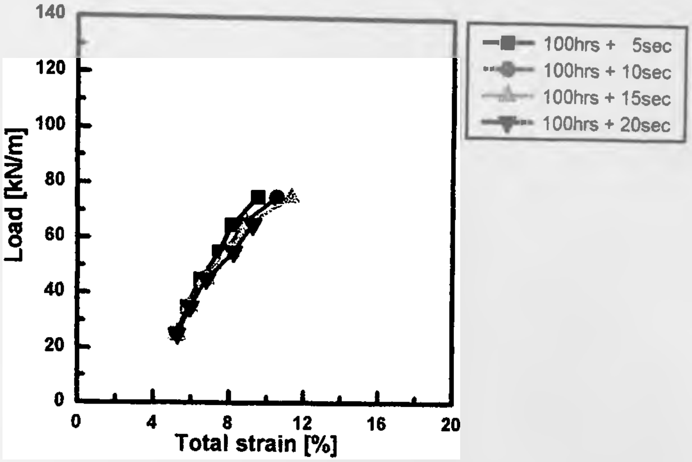
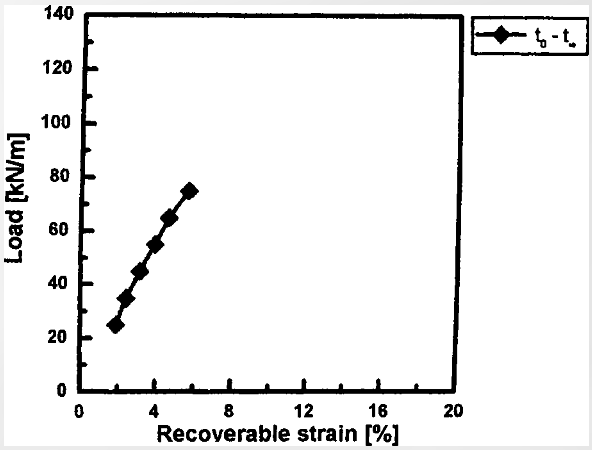


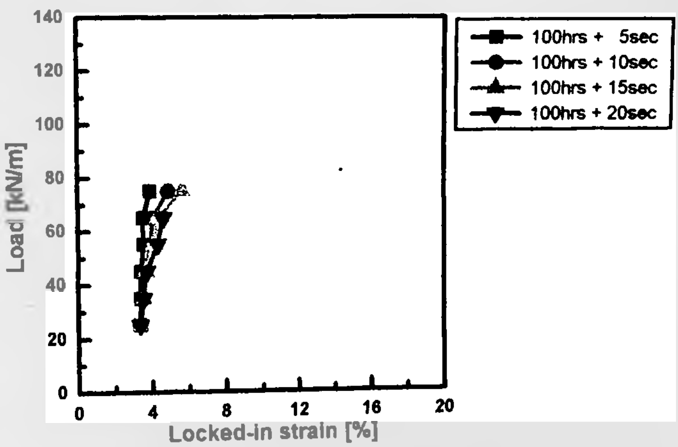
Figure 7.26 Method of obtaining equivalent instantaneous loading strain



(a) Isochronous Load Total-strain curves

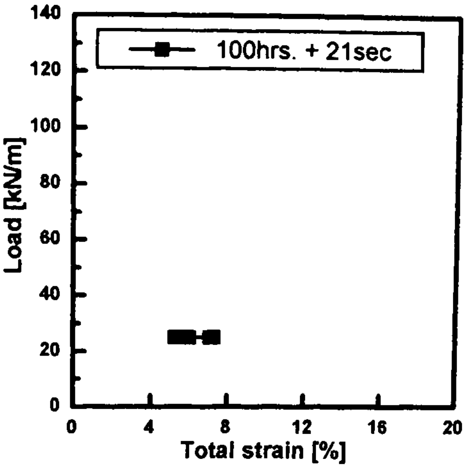


(b) Isochronous Load Recoverable-strain curve

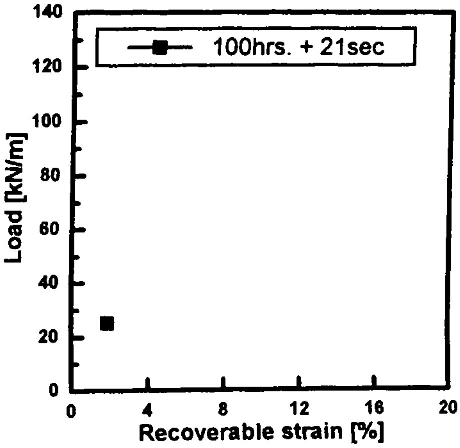


(c) Isochronous Load Locked-in strain curves

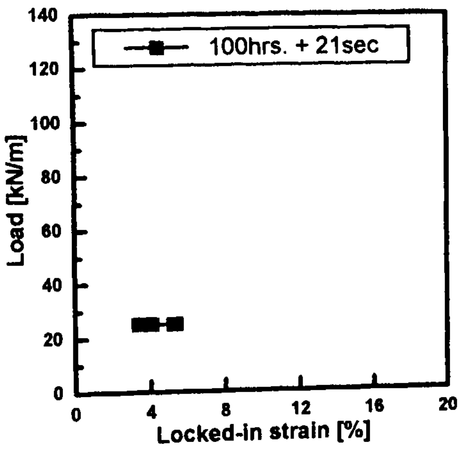
Figure 7.27 Isochronous Load-strain curves for 2nd Stage loading at 20°C



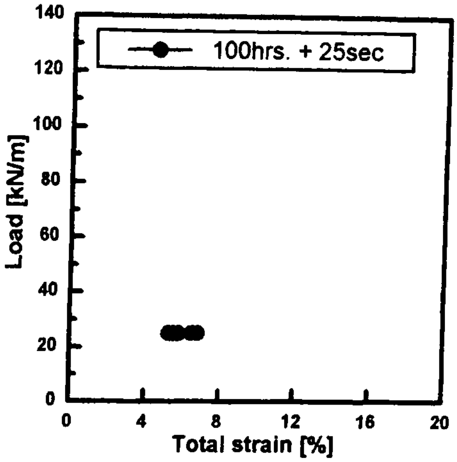
(a) Load vs. Total strain



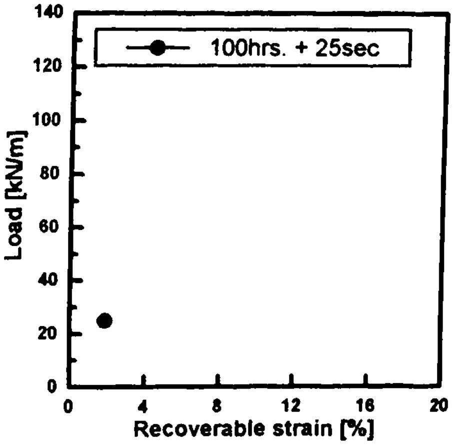
(b) Load vs. Recoverable strain



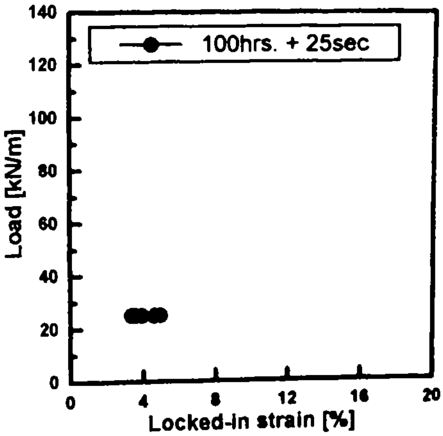
(c) Load vs. Locked-in strain



(d) Load vs. Total strain



(e) Load vs. Recoverable strain



(f) Load vs. Locked-in strain

Figure 7.28 Isochronous Load-strain curves for 3rd Stage loading at 100 hours + 21 and 25 seconds at 20°C

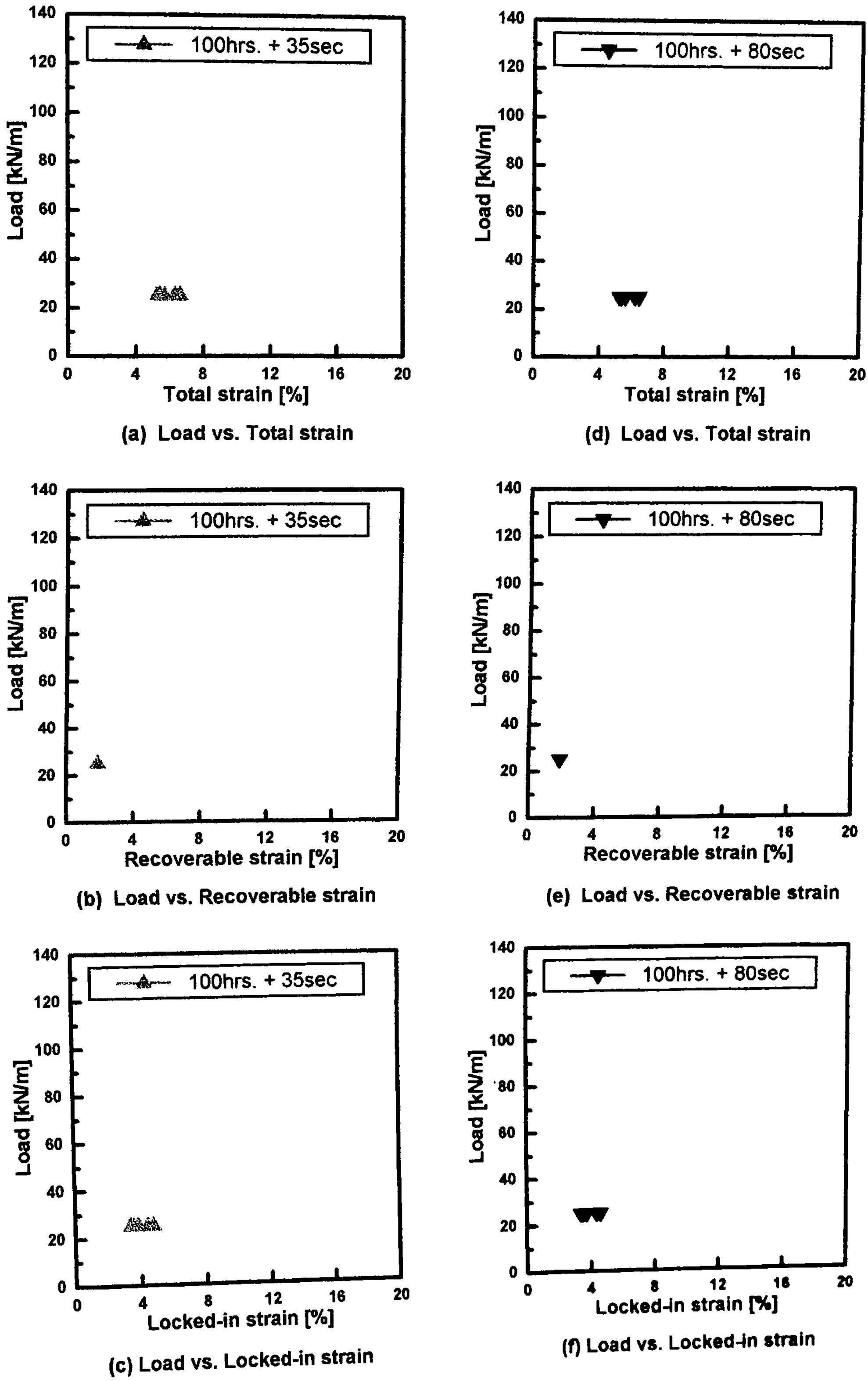
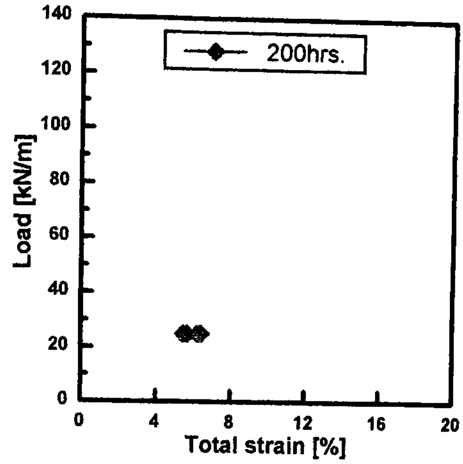
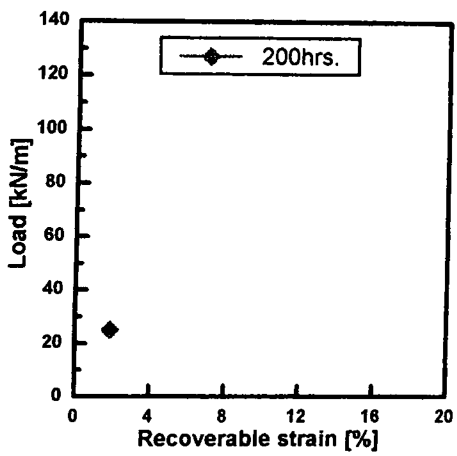


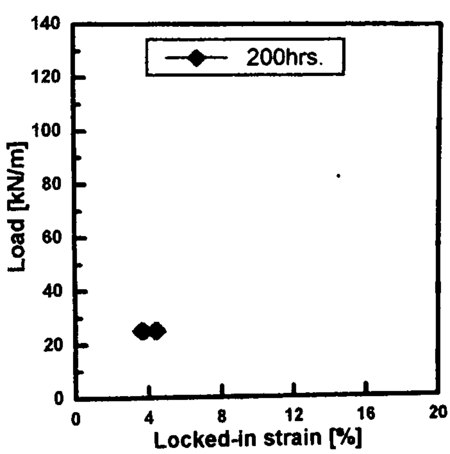
Figure 7.29 Isochronous Load-strain curves for 3rd Stage loading at 100 hours + 35 and 80 seconds at 20°C



(a) Load vs. Total strain

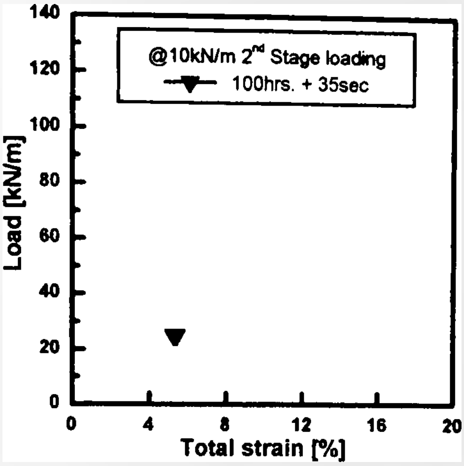


(b) Load vs. Recoverable strain

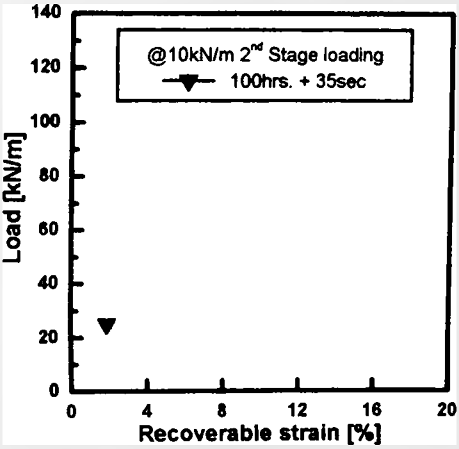


(c) Load vs. Locked-in strain

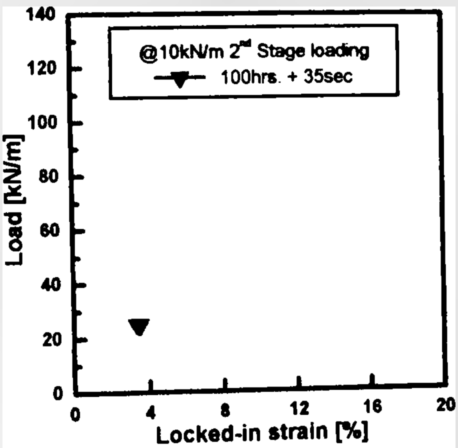
Figure 7.30 Isochronous Load-strain curves for 3rd Stage loading at 200 hours at 20°C



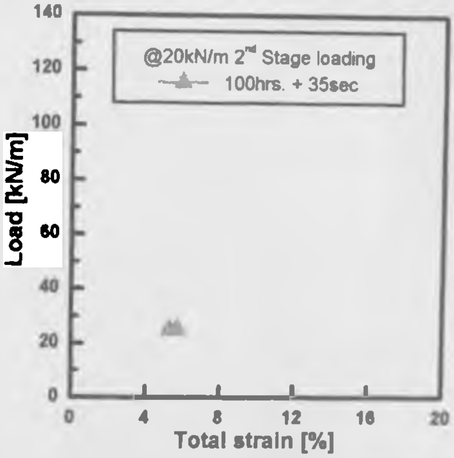
(a) Load vs. Total strain



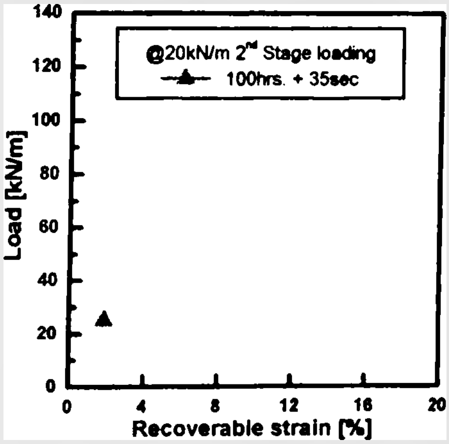
(b) Load vs. Recoverable strain



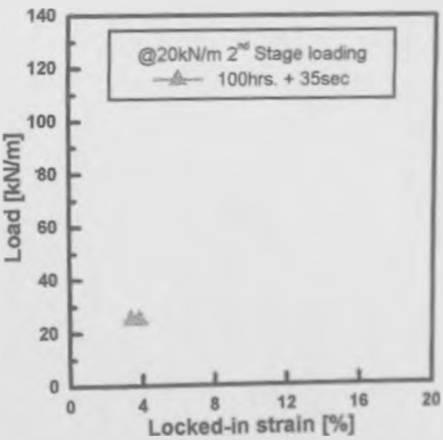
(c) Load vs. Locked-in strain



(d) Load vs. Total strain

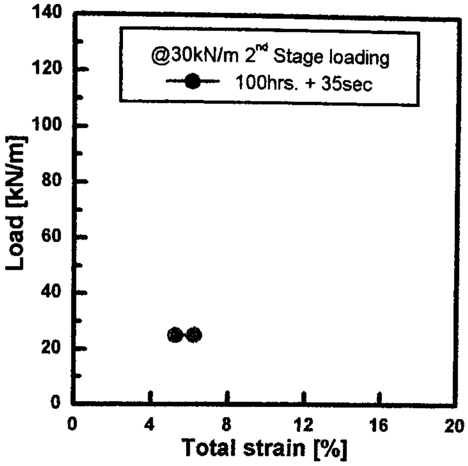


(e) Load vs. Recoverable strain

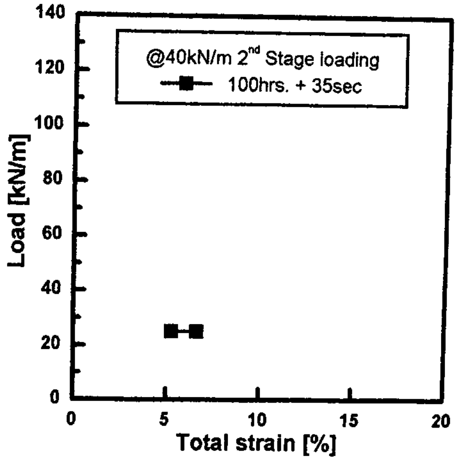


(f) Load vs. Locked-in strain

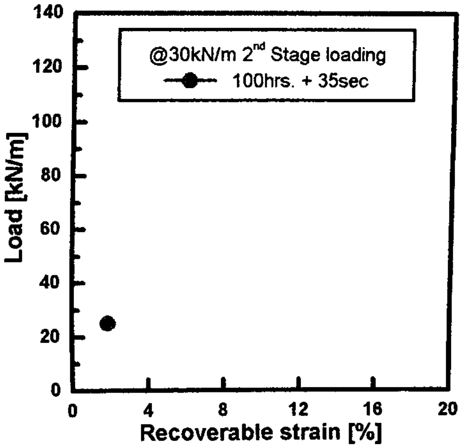
Figure 7.31 Isochronous Load-strain curves for 3rd Stage loading at 100 hours plus 35 seconds for 10 kN/m and 20 kN/m 2nd Stage loading at 20°C



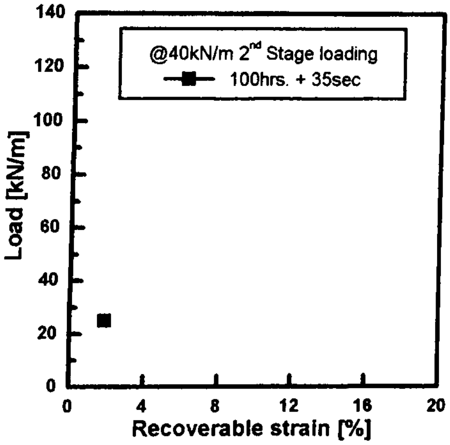
(a) Load vs. Total strain



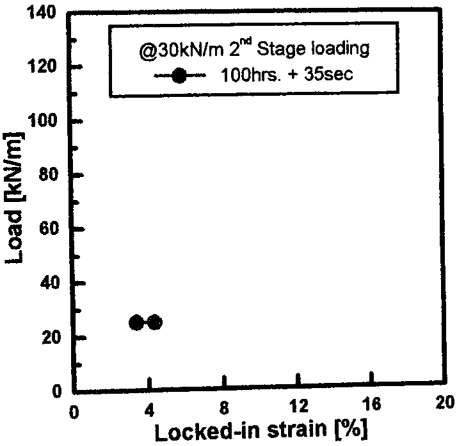
(d) Load vs. Total strain



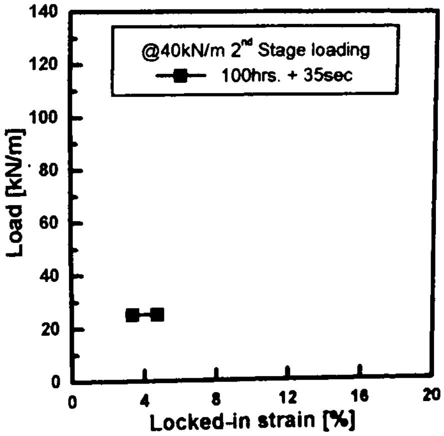
(b) Load vs. Recoverable strain



(e) Load vs. Recoverable strain



(c) Load vs. Locked-in strain



(f) Load vs. Locked-in strain

Figure 7.32 Isochronous Load-strain curves for 3rd Stage loading at 100 hours plus 35 seconds for 30 kN/m and 40 kN/m 2nd Stage loading at 20°C

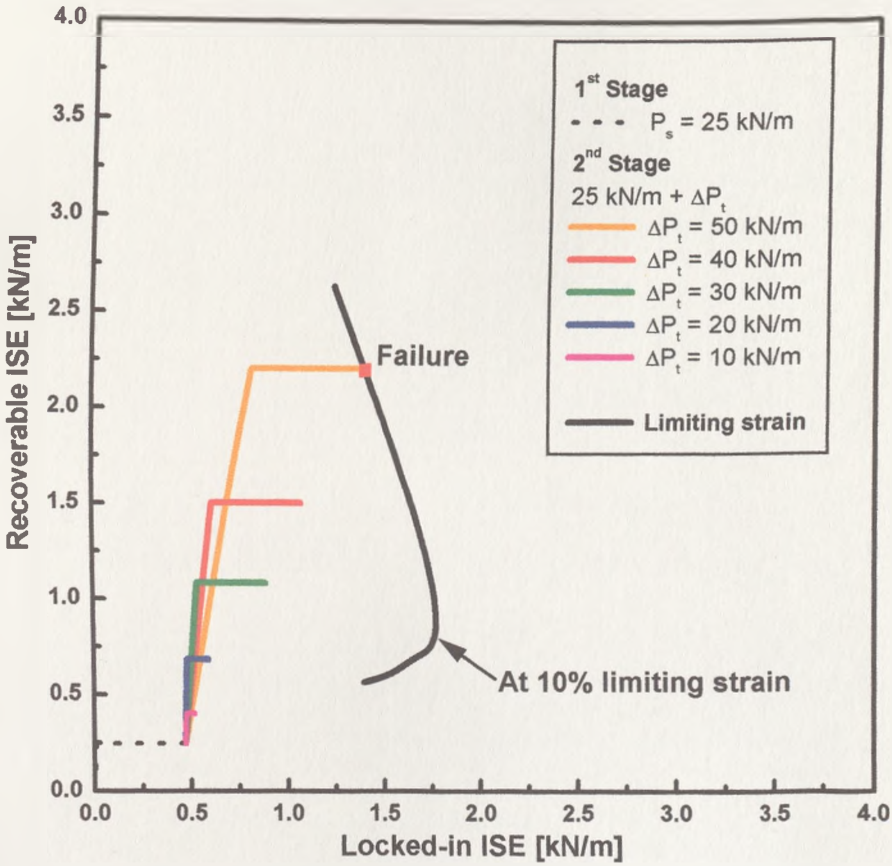


Figure 7.33 $[R]_t$ - $[L]_t$ plot for the 2nd Stage

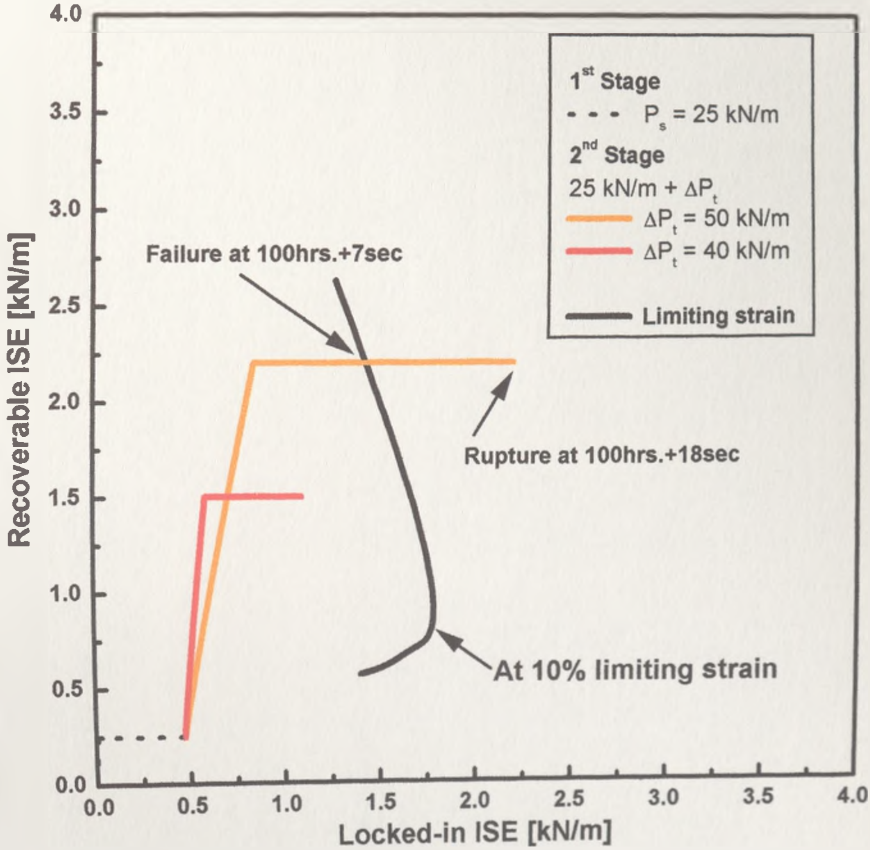


Figure 7.34 $[R]_t$ - $[L]_t$ plot for 40 kN/m and 50 kN/m 2nd Stage loads

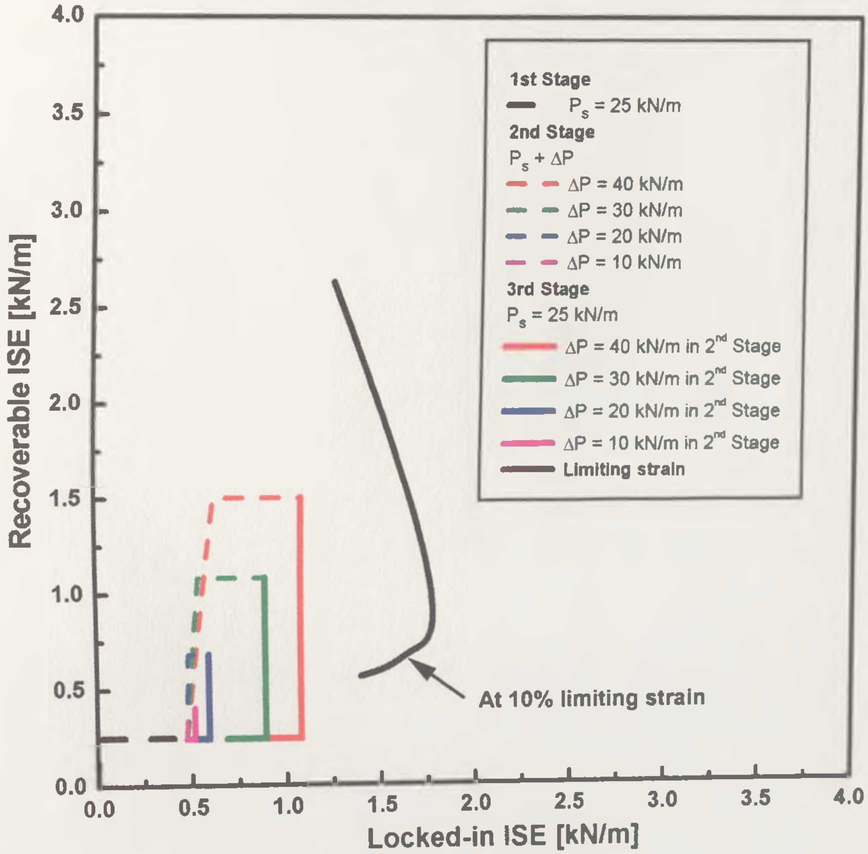
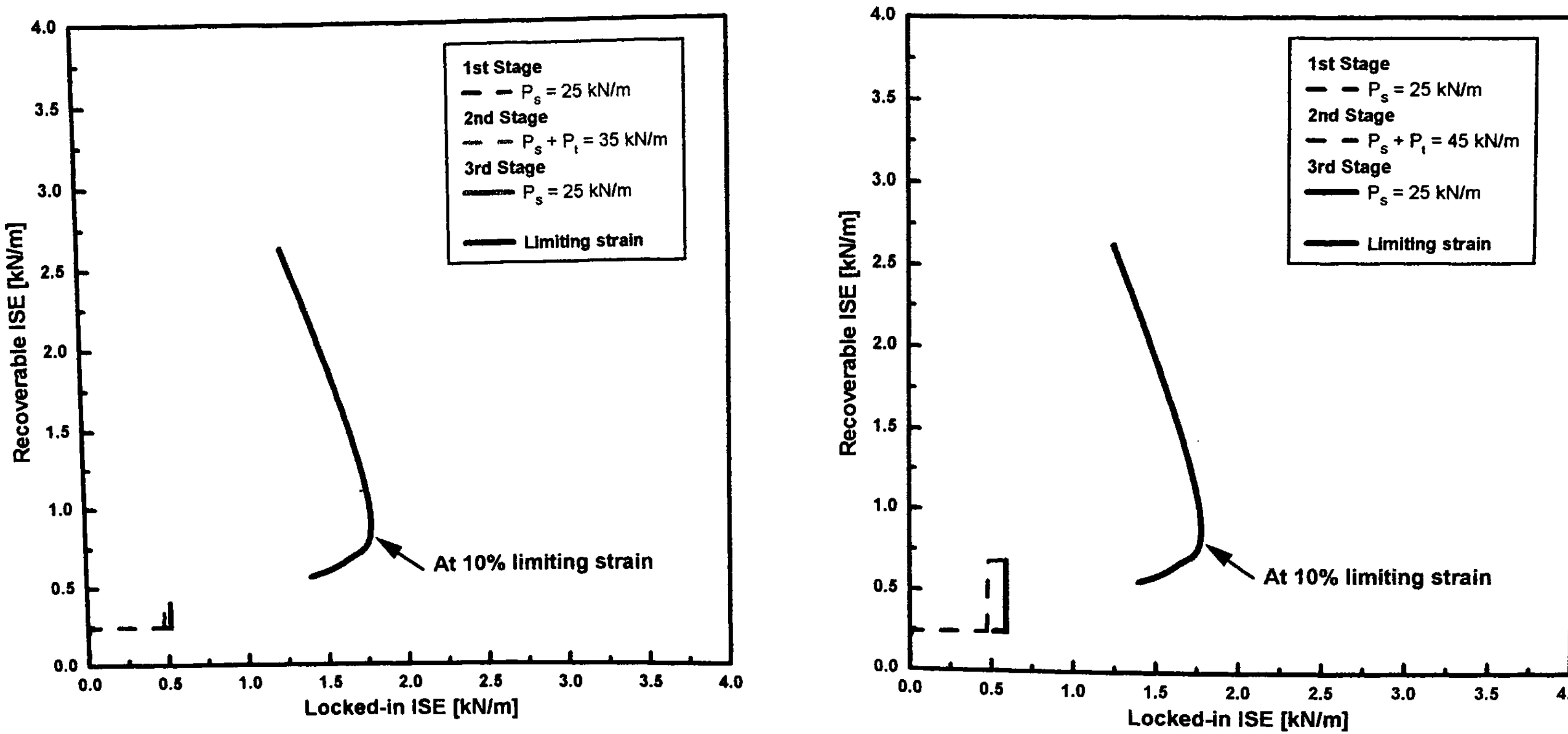


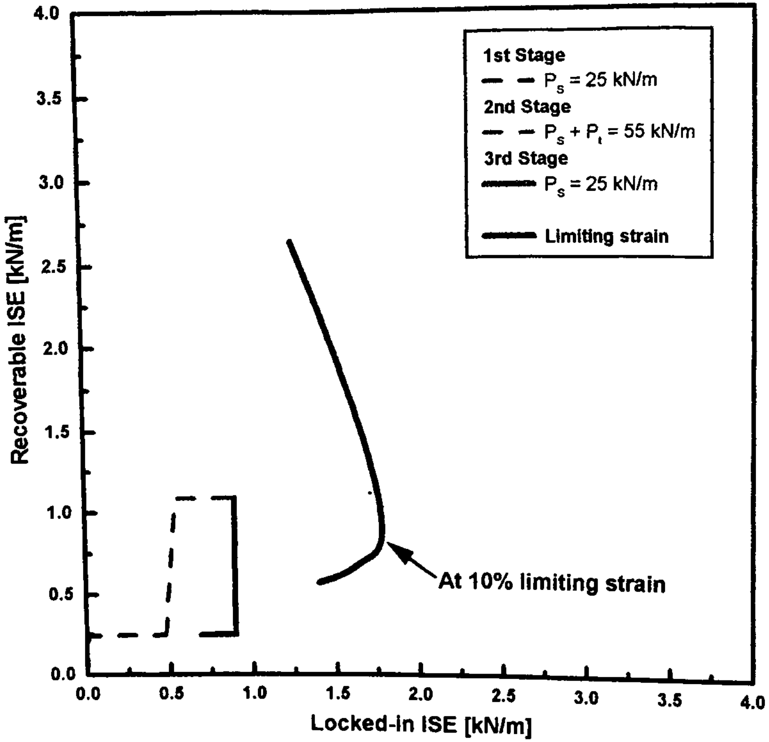
Figure 7.35 $[R]_t$ - $[L]_t$ plot for the 3rd Stage



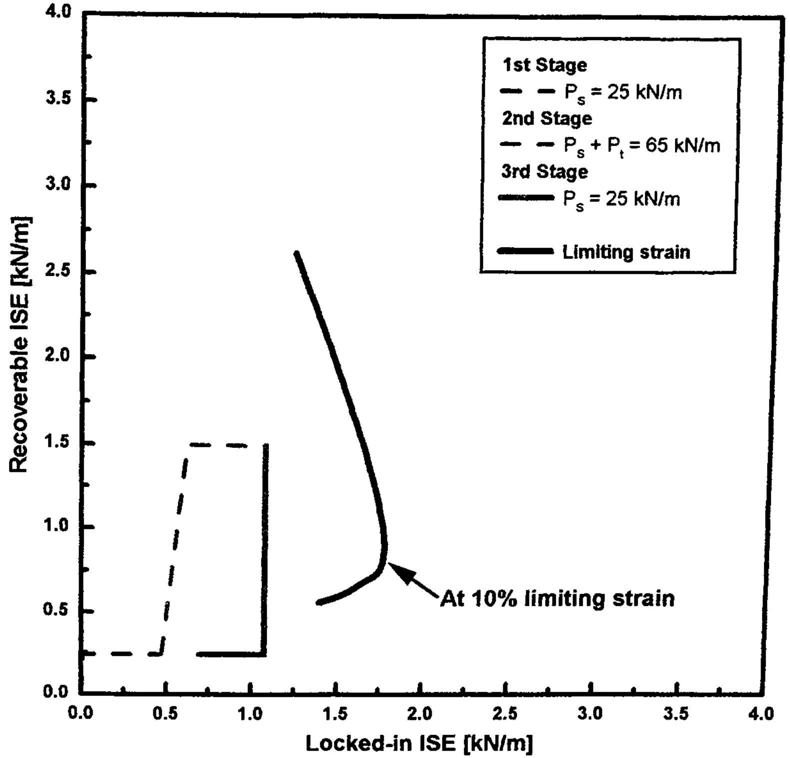
(a) 10kN/m 2nd Stage loading

(b) 20kN/m 2nd Stage loading

Figure 7.36 $[R]_t$ - $[L]_t$ plot for 10 kN/m and 20 kN/m loading

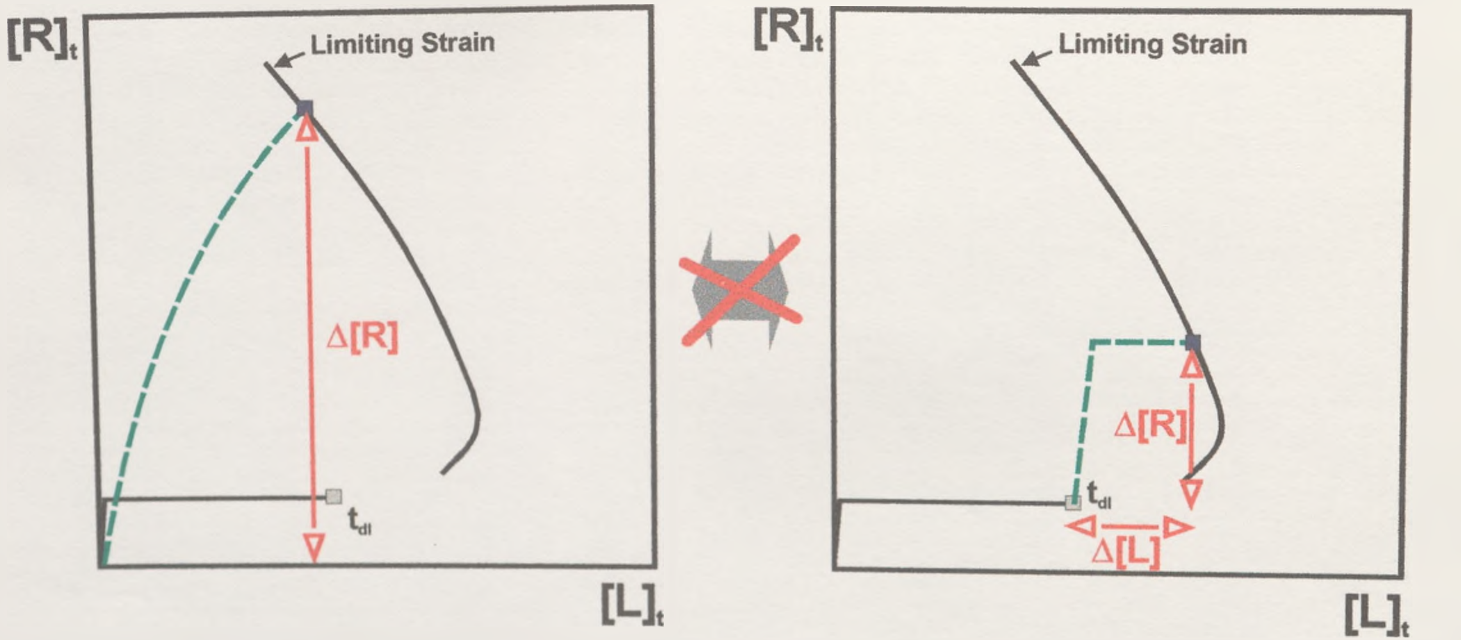


(a) 30kN/m 2nd Stage loading



(b) 40kN/m 2nd Stage loading

Figure 7.37 $[R]_t$ - $[L]_t$ plot for 30 kN/m and 40 kN/m loading



(a) Present design approach
AASHTO (1997), NCMA (1997) and DIBt (1998)

(b) Test results interpreted using the ISE approach

Figure 7.38 Comparison between present design methodology and obtained test results