

**DESIGN & CONSTRUCTION
OF GEOSYNTHETIC-
REINFORCED SOIL
STRUCTURES IN NZ:
REVIEW & DISCUSSION PAPER**

Transfund New Zealand Research Report No. 123

**DESIGN & CONSTRUCTION OF
GEOSYNTHETIC-REINFORCED
SOIL STRUCTURES IN
NEW ZEALAND:**

REVIEW & DISCUSSION PAPER

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EXECUTIVE SUMMARY

1. Background

Because of their cost-effectiveness, the application of geosynthetic-reinforced soil (GRS) to important permanent structures carrying roads and/or pedestrian traffic, such as highway embankments, bridge abutments and retaining walls, is rapidly increasing worldwide. GRS is a comparatively new technique and, therefore, the design methods for GRS are not well established. This research project was carried out in 1997-1998 to review the current state of the design practice of GRS in New Zealand and overseas, and the behaviour of such structures under static and seismic conditions. This report presents the results of the Research Review of the project.

2. Project Objectives

Objectives of the Research Review have been to:

- (a) Search international literature (with particular emphasis on that from USA and Japan) on design methods and actual behaviour of GRS structures under static and seismic conditions, and to obtain copies of relevant overseas standards and guidelines on GRS structures design (with particular emphasis on USA and Japan).
- (b) Contact New Zealand Road Controlling Authorities and Contractors for information on design details and post-construction behaviour of GRS structures in New Zealand, and to collect readily available information.
- (c) Review the above information, and compare the design assumptions and performance predictions for GRS structures based on different design methods with their actual behaviour observed under static and seismic conditions.

3. Conclusions

1. The applications of GRS to permanent structures carrying roads and/or pedestrian traffic is rapidly increasing worldwide. GRS structures can provide substantial cost savings compared with conventional type structures.
2. GRS is a comparatively new technology and, therefore, the complete set of standards and test methods for GRS have not been developed. Test procedures and interpretation of test results for GRS differ from country to country. The number of design parameters and tests required to describe properties of geosynthetics is more than that for conventional materials such as steel or reinforced concrete.
3. Test standards, design guidelines and approval procedures have been developed in some countries. The approval procedures cover a wide range of aspects, from manufacturing quality control to GRS design procedures.
4. Hundreds of GRS structures have been constructed worldwide. Recent case studies provide evidence of the satisfactory performance of GRS structures under both static and seismic conditions. At least 54 GRS structures have been constructed in New Zealand.

5. High seismic resistance of GRS structures is attributed to the following:
 - The tensile strength of geosynthetic materials which can be mobilised under seismic conditions is higher than that for static conditions. For seismic design, tensile strength tests conducted at higher than standard rates of strain would be more appropriate. These tests would often result in an allowable tensile resistance for seismic conditions appreciably higher than that for static conditions.
 - The confined (in-soil) tensile strength of geosynthetic materials is very often higher than their unconfined (in-air) strength. In-soil ultimate tensile strengths as high as four times the in-air strength have been reported.
 - The existing design methods do not take into account the ductility of GRS structures. Japanese researchers suggest that the ductility of GRS structures should be reflected in the design by using a design seismic acceleration lower than the expected peak ground acceleration.
6. Almost all existing design methods used in current engineering practice are, in essence, empirical methods. Design methods for GRS (like the test methods) differ from country to country. This research review has been conducted to compare design methodologies and the results obtained by various existing design methods. The conclusion is that there are substantial variations in the amounts of geosynthetic reinforcement required by different design methods.
7. Most of the GRS design methods for both static and seismic conditions are based on limiting equilibrium analysis. These methods do not allow the deformation and displacement of the GRS block to be assessed, but nevertheless in most cases provide safe design solutions.
8. Guidelines on the seismic design of GRS structures in different countries differ substantially. The GRS block sizes calculated by different methods vary. Designers frequently rely on the conservatism of the static design to provide seismic stability of GRS structures. However, the conservatism of the static design procedures has decreased significantly in recent years. It is, therefore, not clear whether a GRS structure, designed to resist static loads only, will have an adequate level of seismic resistance.
9. Factors of safety used by different GRS design methods are different. Because of the empirical nature of the existing design methods, factors of safety for geosynthetic strength and for different modes of instability cannot be considered separately from the design method they are used with.
10. GRS structures constructed to date (1998) have been designed by different design methods and therefore have different levels of static and seismic resistance. There is clearly a need for design and construction guidelines for GRS structures in New Zealand. These design guidelines for static conditions could be based on the existing guidelines and standards developed by UK, US, Japan and Germany. The design guidelines for seismic conditions could be based on the existing guidelines and standards developed by the US and Japan.
11. This research review summarises the existing design guidelines and forms a basis for further research on the preparation of design guidelines for constructing GRS structures in New Zealand.

ABSTRACT

The application of geosynthetic-reinforced soil (GRS) to permanent structures carrying roads and/or pedestrian traffic, such as highway embankments, bridge abutments and retaining walls, is rapidly increasing worldwide, because of its cost-effectiveness. GRS is a comparatively new technique and therefore the design methods for it are not well established.

This research, carried out in 1997-1998, reviews the current state of the design practice of GRS in New Zealand and overseas. It includes a review of the international literature on design methods and actual behaviour of GRS structures under static and seismic conditions, with particular emphasis on that from USA and Japan; information on design details and post-construction behaviour of GRS structures constructed in New Zealand; and a comparison of the design assumptions and performance predictions for GRS structures based on different design methods with their actual behaviour that was observed under static and seismic conditions. It forms a basis for further research on the preparation of design guideleines for constructing GRS structures in New Zealand.

1. INTRODUCTION

Geosynthetic-reinforced soils (GRS) have been found to have cost benefits relative to the retaining structures traditionally used in specific situations. As a consequence the application of GRS to structures that carry roads and/or pedestrian traffic is rapidly increasing worldwide. Possible applications of GRS structures on highways are shown in Figures 1.1 and 1.2. These applications include:

- Bridge abutments
- Reinforced embankments in place of viaducts
- Reinforced embankments supporting highways
- Repair of slope failures

Very often GRS can provide substantial cost savings compared with conventional type structures. Therefore, GRS retaining walls replace conventional reinforced-concrete retaining walls as well as metallic-reinforced soil retaining walls (so called *Terre Armée* or Reinforced Earth walls) in many countries. A cost comparison for GRS against conventional structures is given in Figure 1.3.

In addition to their low cost compared with conventional structures, evidence from the 1995 Kobe Earthquake (Japan) indicates that GRS structures are less prone to damage under seismic loads than conventional type structures. Also GRS structures located in strongly shaken areas of Kobe City survived with insignificant or no damage and only minor permanent displacements, but the conventional type structures were severely damaged and many of them collapsed. On the basis of this factual data many of the severely damaged conventional type structures in Kobe City have been reconstructed using GRS techniques and the use of GRS structures has been expanded for many important roading projects in Japan, in place of conventional reinforced concrete structures.

GRS is a comparatively new technique. For example, in Japan the first project involving GRS was undertaken in 1988 and the use of GRS on a wider scale started only in 1992. Therefore the design methods for GRS are not well established. New Zealand geotechnical engineers currently use several different overseas standards and design guidelines to design GRS structures.

Although static behaviour of GRS structures is comparatively well understood and design practice is well established, there are still variations between sizes of GRS block and amounts of reinforcement predicted by different design methods. Clear guidelines for the seismic design of GRS structures are also lacking.

Most of the design methods given in overseas standards do not include procedures to allow seismic loads to be taken into account. Moreover, there is disagreement between different researchers' points of view and different standards on how to take into account seismic loads in GRS design.

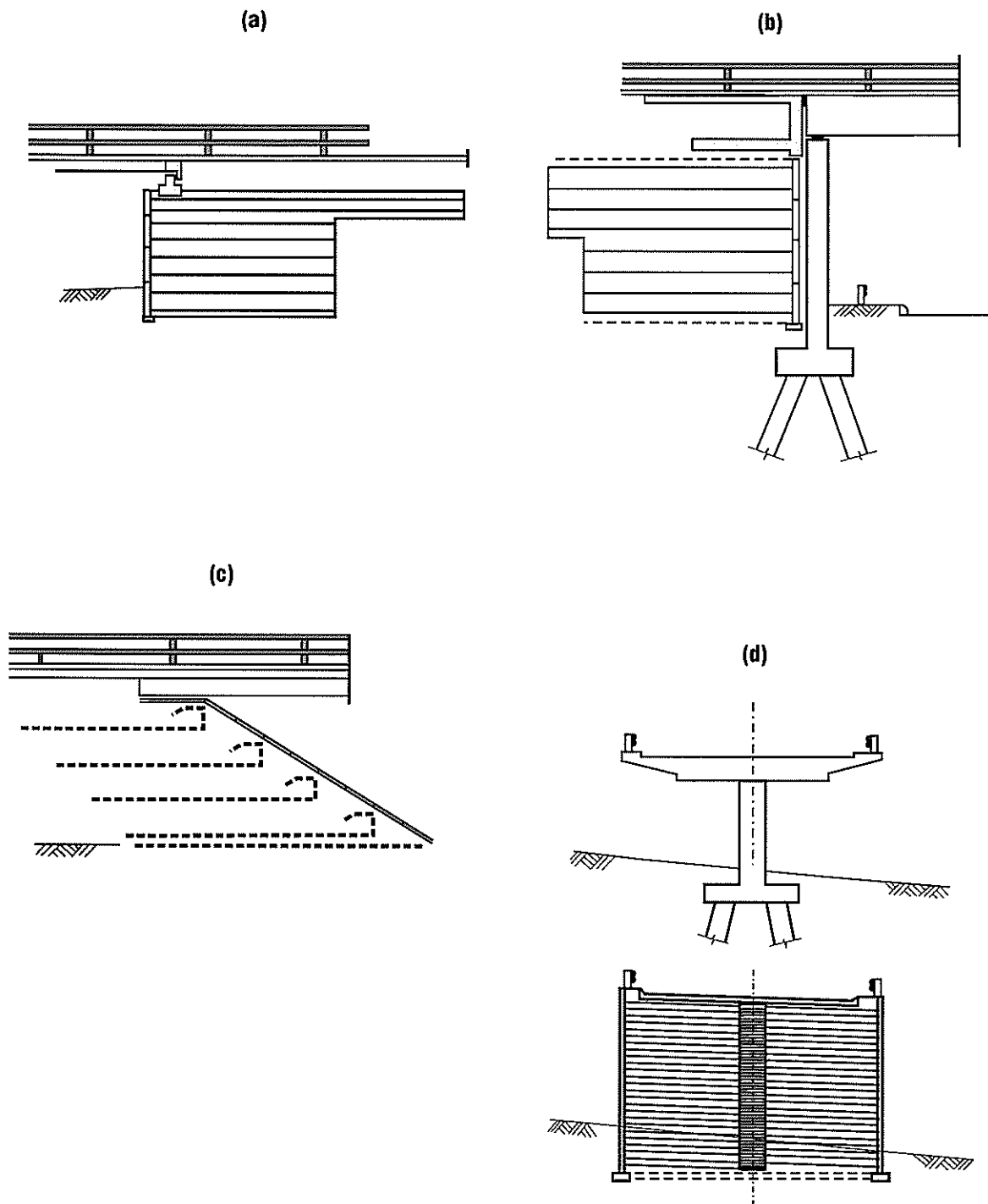


Figure I.1 Applications of GRS: (a), (b), (c) - Bridge abutments; (d) - Reinforced embankment in place of viaduct (after Jones 1996)

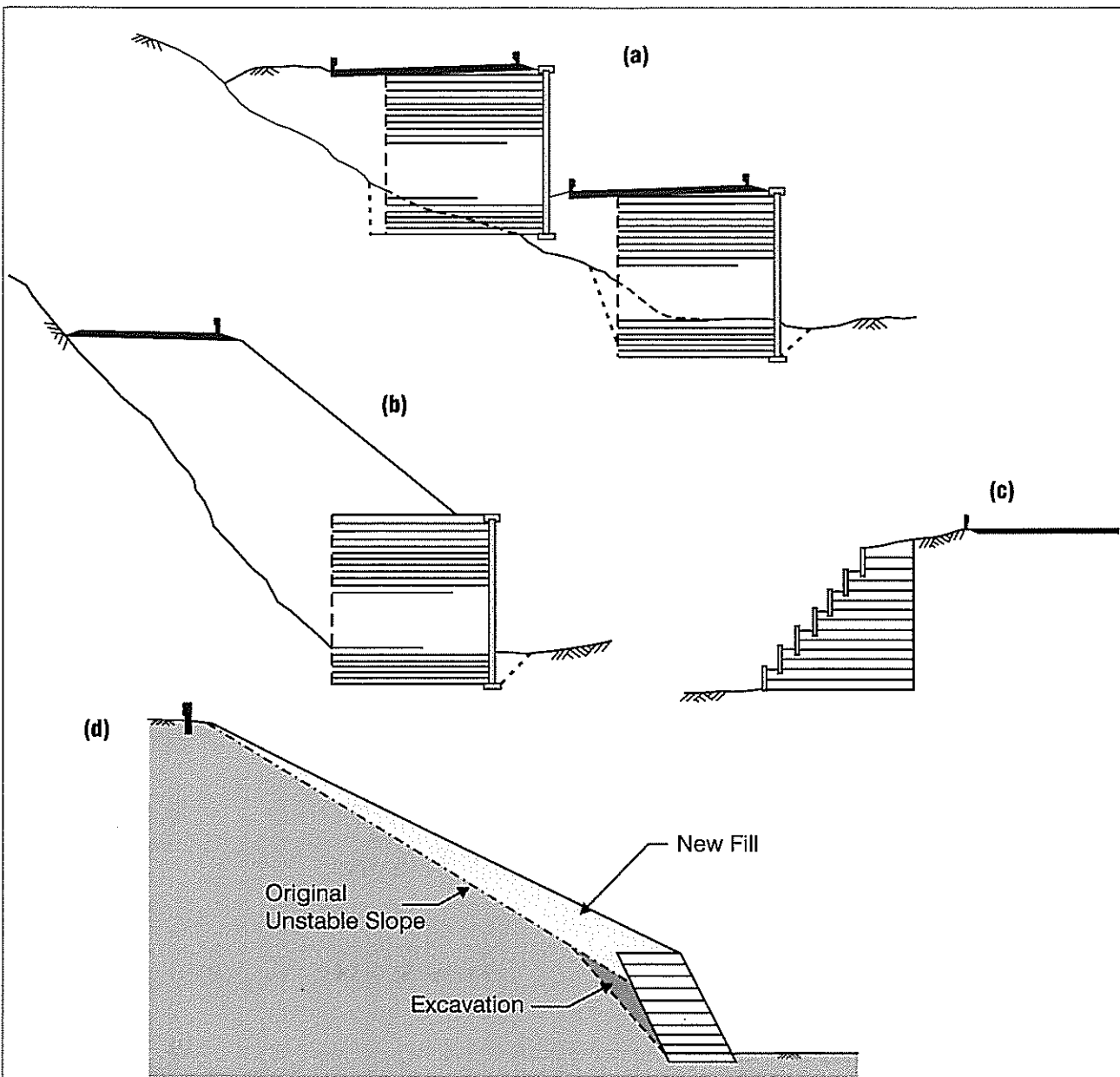


Figure 1.2 Applications of GRS: (a), (b), (c) - Reinforced embankments supporting highways; (d) - Repair of slope failure (after Jones 1996)

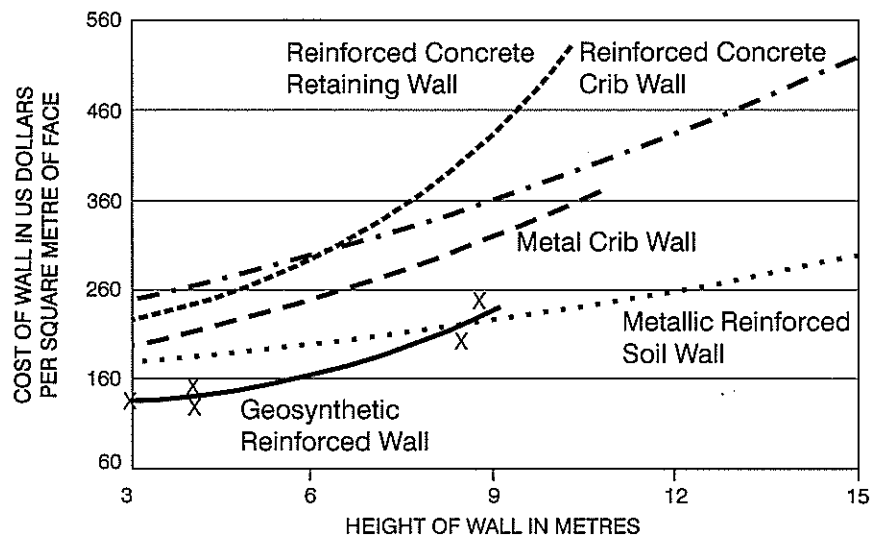


Figure 1.3 Cost comparison for retaining walls (after Holtz et al. 1995)

The amount of laboratory and field testing specified and types of soil strength parameters used by different overseas standards and design guidelines also differ. Reports have also been published of failures of GRS structures under static loads. On the other hand manufacturers of geosynthetic materials every year produce a number of booklets and design manuals stating that their products and design methods (including software products they provide) are the best available on the market. All this results in the fact that GRS structures in New Zealand are designed to different standards and, therefore, have different levels of static and seismic resistance, and different risks of failure under seismic loads.

Because of these uncertainties some New Zealand consultants still prefer to use conventional reinforced-concrete structures and Terre Armée walls rather than GRS structures. Alternatively, consultants use performance-type specifications making a contractor carry all responsibility for design, preparation of detailed specifications and drawings, and erection of GRS structures. Very often in the latter case potential contractors are forced into a situation where they do not have sufficient information on ground conditions or enough time to undertake additional geotechnical investigations required to develop cost-effective preliminary GRS designs during the tender period. Therefore, very often contractors carry high risks associated with design and construction of GRS structures. This normally results in higher cost of GRS structures to Road Controlling Authorities.

Given the fact that the use of GRS structures in New Zealand is increasing, it is obvious that the establishment of the design and construction guidelines for GRS structures in New Zealand is far behind their practical application. Therefore, this research project, which was carried out in 1997-1998, was to prepare guidelines for design and construction of GRS structures in New Zealand. This report presents the results of Stage 1 (Research Review) of the research project.

Objectives of the research review have been to:

- Search international literature (with particular emphasis on that of USA and Japan) on design methods and actual behaviour of GRS structures under static and seismic conditions. Obtain copies of relevant overseas standards and guidelines on GRS structures design (with particular emphasis on USA and Japan).
- Contact New Zealand Road Controlling Authorities and Contractors for information on design details and post-construction behaviour of GRS structures in New Zealand. Collect readily available information.
- Review the above information. Compare the design assumptions and performance predictions for GRS structures based on different design methods with their actual behaviour observed under static and seismic conditions.

2. GEOSYNTHETIC PRODUCTS AND THEIR PROPERTIES

Information on the nature and properties of geosynthetic products is one of the most important design inputs and therefore should be considered by an engineer for every project. More importantly, this information significantly influences the design outputs: very often it is a cause of discrepancies among the amounts of reinforcement required by different design methods.

2.1 Types of Geosynthetic Products

The family of geosynthetics includes a wide range of products such as geotextiles, geogrids, geonets, geomembranes, geosynthetic liners, geopipes, etc. (Koerner 1994). Only products commonly used for soil reinforcement purposes are discussed in this study. These materials include geotextiles and geogrids. Continuous polymer fibres which have recently been applied in soil reinforcement are not discussed because only very limited information on these materials and their applications is currently available.

2.1.1 Geotextiles

ASTM D4439 standard¹ defines a geotextile as "permeable geosynthetic comprised solely of textiles. The properties of a geotextile depend on the type of polymer used, the type of fibre and the fabric style. The geotextile products for soil reinforcement are commonly made from the following polymers: polypropylene, polyester, polyethylene and polyamide. The basic polymers are made into fibres or yarns by using the manufacturer's special techniques" (Ingold & Miller 1988).

Types of polymeric fibres (or yarns) used for geotextile manufacturing include: monofilament, multifilament (made by twisting monofilaments together), staple yarn (made by crimping and cutting of polymer bundles into short fibres and then spinning them into long yarns), slit-film monofilament (made from a continuous sheet of polymer by cutting), slit-film multifilament (produced by twisting slit-film monofilaments together).

The fibres or yarns are converted into fabrics by a weaving process (woven fabrics) or by processes other than weaving (non-woven fabrics).

The woven fabrics are produced using conventional textile weaving machinery. The non-woven fabrics are produced by a process which involves continuous laying of the fibres or filaments onto a moving conveyor belt to form a web and to bond the web by mechanical, thermal or chemical means.

Mechanical bonding is done by penetrating the full thickness of the web by needles to produce "needle-punched" fabrics.

¹ Standards are listed in Appendix 3, separately from Bibliography

Thermal bonding produces cohesion in the web by fusion of continuous filaments at their cross-over points to form "heat-bonded" (or thermally-bonded) fabrics.

Chemical bonding involves imparting cohesion to the web of filaments or staple fibres by addition of a chemical binder.

2.1.2 Geogrids

Geogrids are defined by ASTM Committee D-35 as a "geosynthetic used for reinforcement which is formed by a regular network of tensile elements with apertures of sufficient size to allow strike-through of surrounding soil, rock or other geotechnical material. Geogrids comprise a regular network of integrally connected elements which may be linked by extrusion, bonding or interlacing" (Holtz et al. 1995).

The materials used to produce oriented geogrids include high-density polyethylene and polypropylene.

The most widely used geogrids are Tensar grids. The manufacturing process of Tensar grids include several stages:

- feeding a sheet of polymer into a punching machine,
- punching holes on a regular grid pattern,
- heating and stretching of the punched sheet to form a uniaxial grid,
- if necessary the initial grid can be warm-drawn in the transverse direction to form a biaxially oriented grid.

The above process not only changes the initial geometry of the holes, but also orients the polymer molecules in the direction of drawing.

Another type of drawn geogrids produced by Tenax Corporation is also available.

2.2 Polymeric Materials

The polymers used to produce geotextiles and geogrids include polypropylene (PP), high density polyethylene (HDPE), low density polyethylene (LDPE), linear low density polyethylene (LLDPE), polystyrene (PS), polyvinyl chloride (PVC), acrylonitrile butadiene styrene (ABS), polyamide or nylon (PA), polyester (PET). Detailed properties of these materials are described by Koerner (1994).

High density polyethylene (HDPE), polypropylene (PP) and polyester (PET) are the polymer materials widely used to produce geosynthetics for reinforced soil applications. These materials have high strength, small creep deformations, only gradual strength reduction with time under load, and sufficient durability in the environment provided by typical GRS structures.

High density polyethylene and polyester are used more widely than polypropylene because they have higher stiffness and creep less than polypropylene, while polypropylene is often used where the loading is short term and the deformation is less critical.

2.3 Properties of Geosynthetic Products

Design of GRS structures is based on so-called "design by function" approach, or in other words by assessing the required properties of a geosynthetic product and then comparing them with allowable properties for a chosen product. It is therefore critical for the design of GRS structures to have sufficient information about properties of geosynthetic products.

Geosynthetics are comparatively new materials in the field of geotechnical engineering and therefore the complete set of standards and test methods for these materials is not yet available. Ideally, such a set of standards should cover activities of a wide range of organisations, including raw material suppliers, manufacturers, testing organisations, design engineering firms, owners, etc. At present many of the existing test methods are not completely standardised. However, approval procedures for geosynthetic soil reinforcement have been developed in UK, Germany, Australia and Hong Kong. These procedures require manufacturing process, quality control test data, creep behaviour, soil/reinforcement interaction, installation damage, durability etc. to be assessed for each type of reinforcement and, if found to be satisfactory, approval certificates are issued. Some of the existing approval procedures are listed in Appendix A.

Currently, at least 35 organisations (named in Appendix B) are involved in the process of development of appropriate standards for geosynthetics. It is beyond the scope of this project to discuss all the standards that are available worldwide. Therefore, only the most important tests with reference to standards currently in use in the US and the UK, and relating to geosynthetic materials for GRS structures, are briefly discussed below. The detailed description of the test methods can be found in appropriate standards, some of which are listed in Appendix C.

2.3.1 Geotextiles

- **Specific Gravity**

Some polymers have specific gravities of less than 1.0. The specific gravity of fibres can be determined according to ASTM D792 or D1505.

- **Mass per Unit Area**

The range of values for geotextiles is in the range of 135 to 680 g/m². The test procedure is covered by ASTM D5261.

- **Thickness**

The thickness is measured at a specified pressure. The test procedure is described in ASTM D5199.

- **Stiffness (or flex stiffness)**

This test measures a potential of a geotextile for bending under its own weight. The test method is given by ASTM D1388.

- **Tensile Strength**

This test is very important because design of GRS structures relies on this property as its primary function. Maximum tensile stress, strain at failure, toughness (work done per unit volume before failure), and modulus of elasticity are determined from the test results. The modulus of elasticity of a geotextile varies depending on a method used to measure initial slope of the stress-strain curve giving different values (initial tangent modulus, offset tangent modulus, or secant modulus).

Different standards recommend different test sample sizes which result in different test results for the same geosynthetic material. Different standards allow for a number of variations in sample size: D4632 (100 mm wide sample), D751 (25 mm), D4595 (200 mm), BS6906: Part 1 (200 mm).

- **Confined Tensile Strength**

The test proposed by McGown et al. (1982) suggests testing a geotextile with soil covering. It is noted that the needle-punched non-wovens show significantly improved stress-strain behaviour under soil confinement (Wilson-Fahmy & Koerner 1993). In-soil ultimate strength as high as four times the in-air strength has been also reported (Claybourn & Wu 1993).

- **Seam Strength**

The test procedure is given by ASTM D4884.

- **Fatigue Strength**

The ability of the geotextile to withstand repetitive loading before undergoing failure is tested. The test simulates applications of rail road loadings and wave or tidal actions to GRS structures. No standard is available at this stage.

- **Burst Strength**

The geotextile is distorted by an inflatable rubber membrane into the shape of a hemisphere until bursting occurs, when no further deformation is possible (ASTM D3786).

- **Tear Tests**

The tests assess a geotextile potential for progressive tearing. Several ASTM test procedures are available (D4533, D751 and D1424).

- **Impact Tests**

The impact resistance of geotextiles under dynamic conditions is measured. The geotextile is clamped firmly in an empty container and the amount of penetration of a cone into the geotextile is measured to assess its resistance to impact stresses. No standard is available at this stage.

- **Puncture Tests**

Same as for Impact Tests but for static conditions (ASTM D4833). A steel rod instead of a cone is used. Several correlations between the results of the puncture tests and the tensile strength are known to exist.

- **Friction Behaviour**

Soil to geotextile friction behaviour is tested according to ASTM D5321 or GRI:GS6.

- **Pullout Tests**

The ability of a geotextile to provide anchorage is tested. GRI:GT6 describes the test procedure.

It should be noted that friction behaviour tests and pullout tests yield different results for many situations (Claybourn & Wu 1993).

- **Installation Damage**

A wide range of tests on samples of geotextiles, exhumed after placement, including tensile strength, puncture resistance, burst resistance, etc. are used to assess the amount of damage caused during installation. No widely recognised standard is available.

- **Creep Tests**

Elongation of a geotextile under constant load is determined (ASTM D5262 or GRI:GT5). Such information is very important for the design of GRS structures as creep affects the deformation and the strength of reinforcing layers of a geotextile.

- **Confined Creep Tests**

The creep behaviour of some geotextiles has been noted to improve with soil confinement. Although researchers use complicated techniques to model soil confinement in laboratory testing, no standard is currently available.

- **Temperature Degradation**

Low and high temperatures cause accelerated degradation of polymer materials. The effect of low temperatures on polymers is tested according to ASTM D746.

- **Oxidation Degradation**

ASTM D794 describes a procedure for high temperature oxidation testing for polymers. Oxygen reacts with polymers causing degradation.

- **Hydrolysis Degradation**

The effect of hydrolysis on strength loss of a geotextile at different pH levels is tested. No standard is available at this stage.

- **Chemical Degradation**

Changes in weight, dimensions, appearance and strength properties for various exposure times to up to 50 different chemical reagents are determined according to ASTM D543. ASTM D5322 and D5496 are used to assess chemical degradation of geotextiles after laboratory and field immersion respectively.

- **Radioactive Degradation**

Radioactive exposure can cause radiation degradation of geotextiles (Koerner 1994). No standard is available at this stage.

- **Biological Degradation**

Geotextiles are normally not affected significantly by biological degradation but ASTM G21 and G22 can be used to assess biological degradation.

- **Ultraviolet Degradation**

Ultraviolet degradation is caused by photons breaking the polymer's chemical bonds. The test is done according to ASTM D4355, or ASTM G53 and D5208.

2.3.2 Geogrids

- **Type of Structure, Junction Type, Aperture Size, Thickness, Mass per Unit Area, Percent Open Area**

These properties can be measured directly.

- **Stiffness (flex stiffness)**

ASTM D1388 can be used (similar to geotextiles).

- **Single Rib and Junction Strength**

The strength of a single rib and the junction strength are tested according to the test procedures given by GRI:GG1 and GRI:GG2.

- **Wide-Width Tensile Strength**

A modified form of ASTM D4595 (see Section 2.3.1, Tensile Strength) is normally used.

- **Shear Strength**

The test procedure is the same as for geotextiles (ASTM D5321).

- **Anchorage Strength**

The pullout resistance of geogrids normally exceeds their direct shear strength because of the passive soil resistance developing against transverse ribs. GRI:GG5 gives the standard test procedure.

- **Anchorage Strength from Wall Pullout**

It is important to determine the strength of the geogrid's connection to the wall facing. A wide range of facings for GRS structures is available. Therefore, there is no formalised test procedure. The issue should be considered for every type of facing.

- **Installation Damage**

Similar to geotextiles (see Section 2.3.1). Strength reduction in the order of 20-30% is possible.

- **Creep Behaviour**

Creep is a function of stress level, time, temperature and other environmental factors. Creep behaviour of geogrids is better than that of geotextiles, because of the geogrid orientation effect. The test procedures are given by ASTM D5262, GRI:GG3 and BS6906: Part 5.

- **Temperature Degradation**

High temperatures can cause creep. Therefore behaviour of geogrids at the anticipated temperatures should be tested.

- **Oxidation Degradation**

As for geotextiles.

- **Hydrolysis Degradation**

As for geotextiles.

- **Chemical Degradation**

As for geotextiles.

- **Radioactive Degradation**

Unless high-level radioactive materials are in the immediate vicinity, this should pose no problems to geogrids (Koerner 1994).

- **Biological Degradation**

Geogrids are normally not significantly affected by biological degradation (Koerner 1994).

- **Ultraviolet Degradation**

As for geotextiles.

- **Stress Cracking**

Some polymers are sensitive to brittle cracking when under stress. The test procedures are given by ASTM D1693 and ASTM D5397. Koerner (1994) states that this phenomenon is of concern only for highly crystalline polyethylene.

Some indicative strength and permeability properties of geosynthetic materials are given in Table 2.1.

2.4 Allowable Strength and Design Strength of Geosynthetics

2.4.1 Allowable Strength

As described in the following sections of this report, factors of safety on the strength of geosynthetic products affect the design of GRS structures more significantly than theoretical assumptions of the design methods used. Given that the test methods described above cannot simulate field conditions completely, it is critical to adjust the ultimate tensile strength of geosynthetic materials by using appropriate factors of safety.

The long-term allowable geosynthetic strength for GRS structures is calculated as follows:

$$T_a = \frac{T_{ult}}{FS_{SR} \times FS_{ID} \times FS_{CD} \times FS_{BD} \times FS_{JNT}} \quad (1)$$

where

- T_a is long-term allowable geosynthetic tensile strength, (kN/m);
- T_{ult} is ultimate geosynthetic tensile strength (kN/m);
- FS_{SR} is partial factor of safety for creep deformation, i.e. ratio of T_{ult} to creep limiting strength;
- FS_{ID} is partial factor of safety for installation damage;
- FS_{CD} is partial factor of safety for chemical degradation;
- FS_{BD} is partial factor of safety for biological degradation, used in environments where biological degradation potential may exist;
- FS_{JNT} is partial factor of safety for joints (seams and connections).

The partial factor of safety against ultraviolet degradation may be also included if necessary. Ultimate strength values are based upon minimum average roll values (MARV) determined in accordance with ASTM D4759. MARV is the average of a representative number of tests made on selected rolls of the lot in question that is limited in area to a particular site in question.

FS_{SR} is determined as a ratio of ultimate strength to creep limiting strength. A default factor of safety of 5 is normally permitted for preliminary design, but for detailed design an appropriate creep-reduction factor of safety based on test data should be used.

FS_{ID} is determined from the results of full scale construction damage tests. According to Holtz et al. (1995), a default factor of safety of 3 can be used if appropriate testing has not been conducted.

Soil pH testing should be conducted on all candidate reinforced and retained fill materials. If a potential for chemical and/or biological degradation is suspect, then appropriate testing should be undertaken to determine factors FS_{CD} and FS_{BD} . If pH of the soil is between 3 and 9, it is commonly accepted that chemical degradation is unlikely to develop (Elias 1990). Default factors $FS_{CD} = 2$ and $FS_{BD} = 1.3$ can be used for preliminary design.

FS_{JNT} is determined as the ratio of the unjointed specimen strength to the jointed specimen strength. Appropriate testing should be undertaken to determine the factor of safety, but a default factor of 2 can be used for preliminary design. All default factors of safety and the documents which specify the factors are listed in Table 2.2. The strength of the connection of the geosynthetic to the wall facing must be addressed in the design of every particular GRS structure (if appropriate).

Table 2.1 General range of strength and permeability properties^{1,2} for representative types of geotextiles and geogrids (after Holtz et al. 1995)

Geotextile Type	Weight ³ (g/m ²)	Ultimate Tensile ⁴ Strength (kN/m)	Strain at Ultimate ⁴ Tensile Strength (%)	Secant ⁴ Modulus at 10% Strain (kN/m)	Grab ⁵ Strength (N)	Puncture ⁶ Strength (N)	Burst ⁷ Strength (kPa)	Tear ⁸ Strength (N)	Equivalent ⁹ Darcy Permeability (m/sec)
Woven									
Monofilament-Polypropylene	120-240	16-70	20-40	70-260	700-2300	320-700	2700-4800	200-440	10 ⁴ - 10 ⁻²
Slit Film	50-170	12-45	20-40	50-260	320-1600	80-600	1400-4800	200-1600	10 ⁴ - 10 ⁻³
Fibrillated Tape and Multifilament Polypropylene	240-760	35-210	15-40	175-700	700-6200	700-1100	4100-10400*	440-1800	10 ⁴ - 10 ⁻³
Multifilament-Polyester	140-710	25-350*	10-30	175-1050*	700-9000*	200-1400	3400-10400*	360-2300	10 ⁴ - 10 ⁻³
Nonwoven									
Continuous Filament-Melt Bonded	50-240	4-35	30-100	18-90	180-1800	80-440	550-3500	120-900	10 ⁴ -10 ⁻²
Needlepunched (lightweight)	70-240	4-18	40-150	2-25	180-1100	200-550	1000-2700	120-700	10 ³ -10 ⁻²
Needlepunched (heavyweight)	240-850	8-35	40-150	9-55	700-2300	440-1100	2000-6900	320-900	10 ⁴ -10 ⁻²
Geogrid									
Polypropylene	140-240	8-35	10-20	90-230	n/a	n/a	n/a	n/a	>10
High-Density Polyethylene	240-710	8-90	10-20	55-70	n/a	n/a	n/a	n/a	>10
Polyester	240-710	35-140	5-15	350-2600	n/a	n/a	n/a	n/a	>10

Notes:

1. Data was obtained from numerous sources, in some cases estimated, and represents an average range. There may be products outside this range. No relation should be inferred between maximum and minimum limits for different tests.
2. Both directions
3. Method I.1.84, Appendix B, FHWA Geotextile Engineering Manual
4. Wide Width Method, ASTM D4595
5. ASTM D4632
6. ASTM D4833
7. ASTM D3786
8. ASTM D4533
9. ASTM D4491
- * Limited by test machine

Table 2.2 Default Factors of Safety (after AASHTO 1990; GRI:GG4a 1990; GRI:GG4b 1991; GRI:GT7 1992)

FS_{ID}	FS_{SR}	FS_{CD}	FS_{BD}	FS_{JNT}
3.0	5.0	2.0	1.3	2.0
Notes:				
1. A default FS_{SR} may be used for preliminary design only; actual test data are required for final design.				
2. A default FS_{CD} should not be used for acid-sulphate, organic, salt-affected, ferruginous, calcareous or modified soils (where $9 \leq pH \leq 3$ can be expected).				

Testing standards for determination of the partial factors of safety described above are not fully developed and, therefore, test procedures and interpretation of test results vary. It is, therefore, very often a difficult task to evaluate the test data supplied by manufacturers. While an assessment of the long-term allowable strength (or a review of values supplied by a manufacturer) is very important for the design of GRS structures, many consultants and contractors do not have in-house expertise to undertake the assessment.

A number of other diverse guidelines exist for selecting allowable strengths of geosynthetics, which differ from those described above. Some of these guidelines are discussed in Sections 4.2.3 and 4.3 of this report.

2.4.2 Design Strength

The long-term design strength (T_d) is normally taken as a portion of the long-term allowable strength to account for variations from design assumptions and to provide reasonable assurance against failure:

$$T_d = \frac{T_a}{FS} \quad (2)$$

A factor of safety of 1.5 is used by most of the existing standards with the exception of the AASHTO Bridge Manual (1992) which requires that a minimum factor of safety of 1.78 be used.

Some of the existing design methods use the factor of safety of 1.0, to calculate the long-term design strength but use other factors of safety incorporated in the design (e.g. a factor of safety for soil shear strength). Some of these recommendations are described in Section 4.2.3 of this report.

2.5 Soil–Geosynthetic Interaction

Pullout resistance and direct shear resistance of a geosynthetic are important parameters for the design of GRS structures.

2.5.1 Pullout Resistance

According to Christopher et al. (1989), the long-time pullout performance should be evaluated with respect to three basic criteria:

1. Pullout capacity (the pullout resistance should be adequate to resist the design working tensile force with a specified factor of safety);
2. Allowable displacement (displacement required to mobilise the design tensile force should be smaller than allowable displacement);
3. Long-term displacement (the pullout load should be smaller than the critical creep load).

According to Holtz et al. (1995), pullout resistance of geosynthetic reinforcement is defined by the lower value of the ultimate tensile load required to generate outward sliding of the reinforcement through the soil mass or the tensile load which produces a 38 mm displacement.

A normalised definition of the ultimate pullout resistance (P_r) of the reinforcement per unit width of reinforcement is given by Christopher et al. (1989):

$$P_r = F^* \times \alpha \times \sigma'_v \times 2 \times L_e \quad (3)$$

where:

L_e is the embedment or adherence length in the resisting zone behind the failure surface;

F^* is the pullout resistance factor;

α is a scale effect correction factor to account for a non-linear stress reduction over the embedded length of extensible reinforcement;

σ'_v is the effective vertical stress at the soil-reinforcement interfaces.

Ideally, the pullout resistance factor F^* (for long-term conditions) should be obtained from a pullout test performed on backfill material to be used on the project. Pullout test procedures are given by GRI:GG5 and GRI:GT6.

A number of other diverse guidelines for selecting the alternate pullout resistance are described in the literature. Some of the guidelines are discussed in Sections 4.2.3 and 4.3 of this report.

2.5.2 Direct Shear Strength

Soil–geosynthetic direct shear resistance can be determined in accordance with GRI:GS6 or ASTM D5321.

3. BEHAVIOUR OF GRS STRUCTURES

3.1 Static Behaviour

Over 200 published GRS case histories have been reviewed as part of this research. While analysis of every case study is beyond the scope of this report, the reviewed articles on the case histories are listed in the Bibliography of this report.

One of the reviewed articles (Yako & Christopher 1988) presents a summary of 54 GRS projects in North America, with specific emphasis placed on projects that have been instrumented and monitored in such ways that performance assessment could be made. The article provides information on GRS structures with different facing systems and various geosynthetic reinforcement brands (including geotextiles and geogrids) used in the 54 projects. All the GRS structures described had performed satisfactorily. Yako & Christopher estimated that over 200 GRS structures had been constructed in North America by 1988.

As part of this research project Beca Carter Hollings & Ferner Ltd (BCHF) has collected information on 54 recent GRS case studies in New Zealand. This information is given in Table 3.1 and the typical geometry of the GRS structures is shown on Figure 3.1. The information was obtained through a review of published case histories and interviews with the New Zealand traffic controlling authorities, representatives of manufacturers and design engineers. General comments received were that performance had been satisfactory and no advice was received of any significant problems.

Although information about failures of GRS is not always published, a review of published case histories undertaken by BCHF indicates that there have been cases of unsatisfactory performance of GRS structures under static loads.

For example, Burwash & Frost (1991) described unsatisfactory performance of a wrap-around 9 m-high geogrid-reinforced wall supporting an asphalt-surfaced parking area. The wall settled during 22 months to 900 mm and rotated outwards about the toe.

Goehring (1991) reported that a thin tension crack developed between the reinforced block and the backfill of a 12 m-high wrap-around geogrid-reinforced wall supporting a 4-lane roadway.

Chandler & Kirkland (1991) reported that some settlement, causing cracking of pavement overlying a 3 m-high wrap-around geotextile-reinforced wall, occurred 9 months after construction.

However, most of the reviewed case histories prove that generally GRS structures perform well under static conditions.

Table 3.1 Geosynthetic-reinforced soil structures: New Zealand case histories

No	Year Constructed	Location	Function	Wall Length (m)	Geometry (Figure 3.1)					Type of Facing	Reinforcement make and grade	Design Method Used	Performance of Completed Structure or Other Comments
					H (m)	Z (m)	B (Deg)	W (Deg)	l (m)				
1	1997	Rural Wanganui Makitoro Rd	Slip Repair	12	2.5		90		3	0.3	Ground engineering Nelson Ltd		
2	1997	Rural Wanganui N° 2 Line	Slip Repair	12	4.5		40		3	0.5	Winwall Bautechnik method		
3	1997	Rural Wanganui Kaurapapa Rd	Slip Repair	20	3.5		80		3.4	0.3	Winwall Bautechnik method		
4	1996	Rural Wanganui River Rd	Slip Repair	20	5						Winwall Bautechnik method		
5	1996	Rural Wanganui Mangamahū Rd	Slip Repair	80	4	4	45	30	6	1	Winwall Bautechnik method		
6	1993	Okehu	Slip Repair	18	5.4	0	76	0		0.4			
7	1993	Manawapou	Road Realignment	120	10	0	76	0	2.4 to 6.9	0.4, 0.6			
8	1997	SH4 RP208/8.52	Road embankment	20	2	0.5	76.4	20	2	0.03	MaccaFeri Fortrac 35		
9	1993	SH45 Herekawe	Road embankment	40	8.4		75	0	5.5	0.6	Tensar SR55 14 layers 5.5m long. SSI geogrids 1m long centrally each layer	Soil-bagged front face very successful. Now has vertical grass face. All very satisfactory	
10	1997	Anzac Parade, Wanganui	Footpath Stabilising Slip Repair	12	2	0	85		1.6	0.3	Winwall typically Bautechnik method		
11	1992	Albany Highway, Auckland	Slip Repair Road embankment	100	14	0.5	30	5	5 to 7	3	Utexas 2 (computer programme) and Bautechnik method	Satisfactory	
12	1994	SH 58, Wellington	Road embankment	60	5	0.5	45	10	8	0.4	Utexas 2, slope stability method	Satisfactory	
13	1991	Taupo/Ponui Point	Support of the upgraded access road		4.5	0	75	0			Tensar SR55, Tensar SS1, Bodkin joints, Xcel Erosion blanket	Satisfactory	
14	1991	Gisborne/Riverside Rd	Slope Stabilisation	2.5-5.9 15-6.5	5.9 6.5	0	45	0	4.5	0.5/1.0	Tensar SR55 1.5m long as secondary reinforcement and Tensar SR80 4.5m long as primary reinforcement	Tensar wall and slope design program/Bautechnik Method	Satisfactory
15	1991	North Shore Grove Slip	Slope Stabilisation		4.2	0.5	45	22.5	6	0.45, 0.6, 0.75, 0.9	Tensar SR55 2.0m long as secondary reinforcement and 6m long as primary reinforcement, Tensar Mat	Tensar wall and slope design program/Bautechnik Method	Satisfactory

Table 3.1 Geosynthetic-reinforced soil structures: New Zealand case histories (continued)

N ^o	Year Constructed	Location	Function	Wall Length (m)	Geometry (Figure 3.1)					Type of Facing	Reinforcement make and grade	Design Method Used	Performance of Completed Structure or Other Comments
					H (m)	Z (m)	B (Deg)	W (Deg)	L (m)				
16	1992	Wanganui/SH 52 Weber Slip Reinforcement	Slope Stabilisation and road widening		7	0	45-60	0	5.5	0.4, 0.6, 0.8, 1.0	Tensor SR55, Tensor SS2, Bodkin joints, Xcel erosion blanket	Tensor wall and slope design program/Bautechnik Method	Satisfactory
17	1993	Wanganui/Flood damage south of Okehu Stream Bridge	Support of the highway	15	4	0	75	0	4.2	0.4, 0.7, 0.9, 1.0	Tensor SR55, Tensor SS1, Bodkin joints, Xcel Erosion Blanket	Tensor wall and slope design program/Bautechnik Method	Satisfactory
18	1993	Wanganui/Flood damage Ore Dropout SH4	Road Reinstatement	35	6 to 7	0	75	0	4.6	0.6, 0.7, 0.9, 1.0	Tensor SR55, Tensor SS1, Bodkin joints, Xcel Erosion Blanket	Tensor wall and slope design program/Bautechnik Method	Satisfactory
19	1993	Wanganui/SH4 Flood damage Burrells Road	Reinforced Road embankment	50	10	0	60	0	9, 10	0.4, 0.5, 0.7, 1.0	Tensor SR55, Tensor SS1, sandbags	Tensor wall and slope design program/Bautechnik Method	Satisfactory
20	1993	Wanganui/South of Raukawa Falls Slip Reinstatement	Support of SH4	20	5.5	0	75	0	3.7	0.4, 0.6, 0.8, 1.0	Tensor SR55, Tensor SS1, Bodkin joints, Xcel erosion blanket	Tensor wall and slope design program/Bautechnik Method	Satisfactory
21	1993	Apiti Area N ^o 3 Flood Damage Repairs	Edge of the road support	15	5.5	0	60	0	4.55	0.4, 0.45, 0.5	Tensor SR55, Tensor SS1, Bodkin joints	Tensor wall and slope design program/Bautechnik Method	Satisfactory
22	1993	North Shore/Attwood Rd Slip Greenhithe	Support to the access road	25	8	0	45	0	4.5	0.4, 0.6, 0.8, 1.0	Tensor SR44, Tensor SS2, Tensor Mat	Tensor wall and slope design program/Bautechnik Method	Satisfactory
23	1994	Wanganui/Manawapou Reconstruction/SH3	Road realignment	160	9.3	0	75	0	6	0.4, 0.8	Tensor SR55, Tensor SR80, SS1, Bodkin joints, topsoil filled bags	Tensor wall and slope design program/Bautechnik Method	Satisfactory
24	1994	Hot Water Beach Rd	Support of the road	70	8	0	75	0	2.8, 3.8, 4.8	0.3, 0.6	Tensor SR80, Tensor SR55, SS20	Tensor wall and slope design program/Bautechnik Method	Some movement behind wall, lack of drainage
25	1994	Huntly/Renown Mine Haul Road overbridge Abutment	Bridge support		5.9	0	90	0	4.5	0.4, 0.6	Tensor SR80, Tensor SR110	Tensor wall and slope design program/Bautechnik Method	Satisfactory
26	1994	BP Waterview, Auckland	Support of service station platform	135	5.5	0	90	0	3.6	0.4, 0.6	Tensor SR55	Tensor wall and slope design program/Bautechnik Method	Satisfactory
27	1994	Gulf Harbour Marine Village	Canal system	2000	6.5	0	90	0	5	0.4, 0.6	Tensor SR55	Tensor wall and slope design program/Bautechnik Method	Satisfactory
28	1995	High Point Shopping Centre, Highbury, Auckland	Support of the carparking area		4	0	90	0			Tensor SR55	Tensor wall and slope design program/Bautechnik Method	Satisfactory

Table 3.1 Geosynthetic-reinforced soil structures: New Zealand case histories (continued)

N ^o	Year Constructed	Location	Function	Wall Length (m)	Geometry (Figure 3.1)					Type of Facing	Reinforcement make and grade	Design Method Used	Performance of Completed Structure or Other Comments
					H (m)	Z (m)	B (Deg)	W (Deg)	I (m)				
29	1995	Green Bridge, Waipiata, Central Otago	Bridge approach support							Gabion wall	Tensar wall and slope design program/Bautechnik Method	Satisfactory	
30	1996	Westfield Park Reinforced Slope	Slope Stabilisation	95	13	0	45	0	8, 9	0.9	Tensar 55RE, Tensar SS20, Tensamat 200	Satisfactory	
31	1997	Nihotupu Landfill Stabilisation	Slope Stabilisation		7	0	25	0	7	1	Tensar RE55, Tensar RE120	Satisfactory	
32	1996	Lakefront Hotel, Queenstown	Support of the Lakefront Hotel grounds		3.6	0	90	0	1.5	0.6	Tensar SR55	Satisfactory	
33	1997	Milburn, Milburn Quarry	Platform for Primary Crusher	21	11.6	0	87.95	0	6.0 and 7.0	0.4, 0.6	Tensar SR80, Tensar SR55	Satisfactory	
34	1997	South Eastern Arterial Route, Auckland	Support of approaches to rail overbridge	90m northern 150m southern	8						Tensar	Satisfactory	
35	1997	Karangahake Gorge	Slope stabilisation	225	5	0.25-0.5	63	12	4, 5.5	0.5, 1.0	Tensar SR55, Tensar SR80	Satisfactory	
36	1997/98	Mangawhai Road	Slip Repair	58 (max)	13	-	65	-	6.5 6.5	0.4 0.6	Tensar Geogrid 120RE, 80RE, 55RE	Satisfactory performance. Long construction period due to wet confined nature of client-nominated borrow area.	
37	1992	SH2 361/7.6	Slip Repair	25	7	0	45		0.6		Tensar SR	Satisfactory	
38	1996	SH35 124/2.2	Slip Repair	25	6	0	40		6	0.6	Slope PC/Tensar/Fortrac	Satisfactory	
39	1996	SH35 132/1.52	Slip Repair	25	5	0	40		5	0.6	Slope PC/Tensar/Fortrac	Satisfactory	
40	1996	SH35 132/1.84	Slip Repair	25	5	0	40		5	0.6	Slope PC/Tensar/Fortrac	Satisfactory	
41	1996	SH35 172/5.00	Slip Repair	20	5	0	40		5	0.6	Slope PC/Tensar/Fortrac	Satisfactory	
42	1996	SH2 483/10.2	Slip Repair	30	6	0	45		4	0.8	Slope PC/Tensar/Fortrac	Satisfactory	
43	1996	SH2 544/7.65	Slip Repair	29	10	0	39		7	0.6	Slope PC/Tensar/Fortrac	Satisfactory	

Table 3.1 Geosynthetic-reinforced soil structures: New Zealand case histories (continued)

N ^o	Year Constructed	Location	Function	Wall Length (m)	Geometry (Figure 3.1)						Type of Facing	Reinforcement make and grade	Design Method Used	Performance of Completed Structure or Other Comments
					H (m)	Z (m)	B (Deg)	W (Deg)	L (m)	S (m)				
44	1998	SH2 562/3.3	Slip Repair	46	11	0	70		7	0.75	Bayfacing and precast	Tensar RE	Win Slope, Win Wall	Satisfactory
45	1997	SH2 361/4.0	Slip Repair	11.5	11	0	42		5.5	1	Wrap around	Tensar RE	Win Slope, Win Wall	Satisfactory
46	1996	SH38 161/10.72	Slip Repair	26	6	0	45		5.5	0.5	Wrap around	Tensar SR	Win Slope, Win Wall	Satisfactory
47	1997	SH2 608/16.5	Slip Repair	25	2	0	90		4	0.5	Precast concrete	Tensar SR	Win Slope, Win Wall	Satisfactory
48	1997	SH38 131/12.95	Slip Repair	20	2	0	90		3.5	0.5	Precast concrete	Tensar SR	Win Slope, Win Wall	Satisfactory
49	1998	SH38 147/4.02	Slip Repair	71	8	0	53		5	1	Wrap around	Tensar RE	Win Slope, Win Wall	To be constructed
50	1996	Blacks Beach	Slip Repair	15	6.5	0	37		5.5	0.75	Wrap around	Fortrac 35	Win Slope, Win Wall	Minor settlement
51	1996	Blacks Beach	Slip Repair	15	4	0	51		5	0.75	Wrap around	Fortrac 35	Win Slope, Win Wall	Minor settlement
52	1996	Approx 2109 East Coast Bays Road, Silverdale	Slip Repair	35 m	4m	0	53		4m	0.5m	Wrap and grip, with erosion control blanket, then hydroseeded.	Tensar SR80 RE Geogrid		Satisfactory
53	1996	SH4 - RP 0/14.10 Waterfall Hill	Slip Repair	23 m	10	0	57	0	18.3.5 to 8 m	0.6	Wraparound	Tenax TT 060 & Tenax TT 090	Bautechnick Method	A difficult site to construct as country was very unstable and wet
54	1994	SH3 PR 76/8.35 MacLachlans Hill	Slip Repair	15	9	0	45	0	18.3.5	0.5	Wraparound	Tensar SR 55		

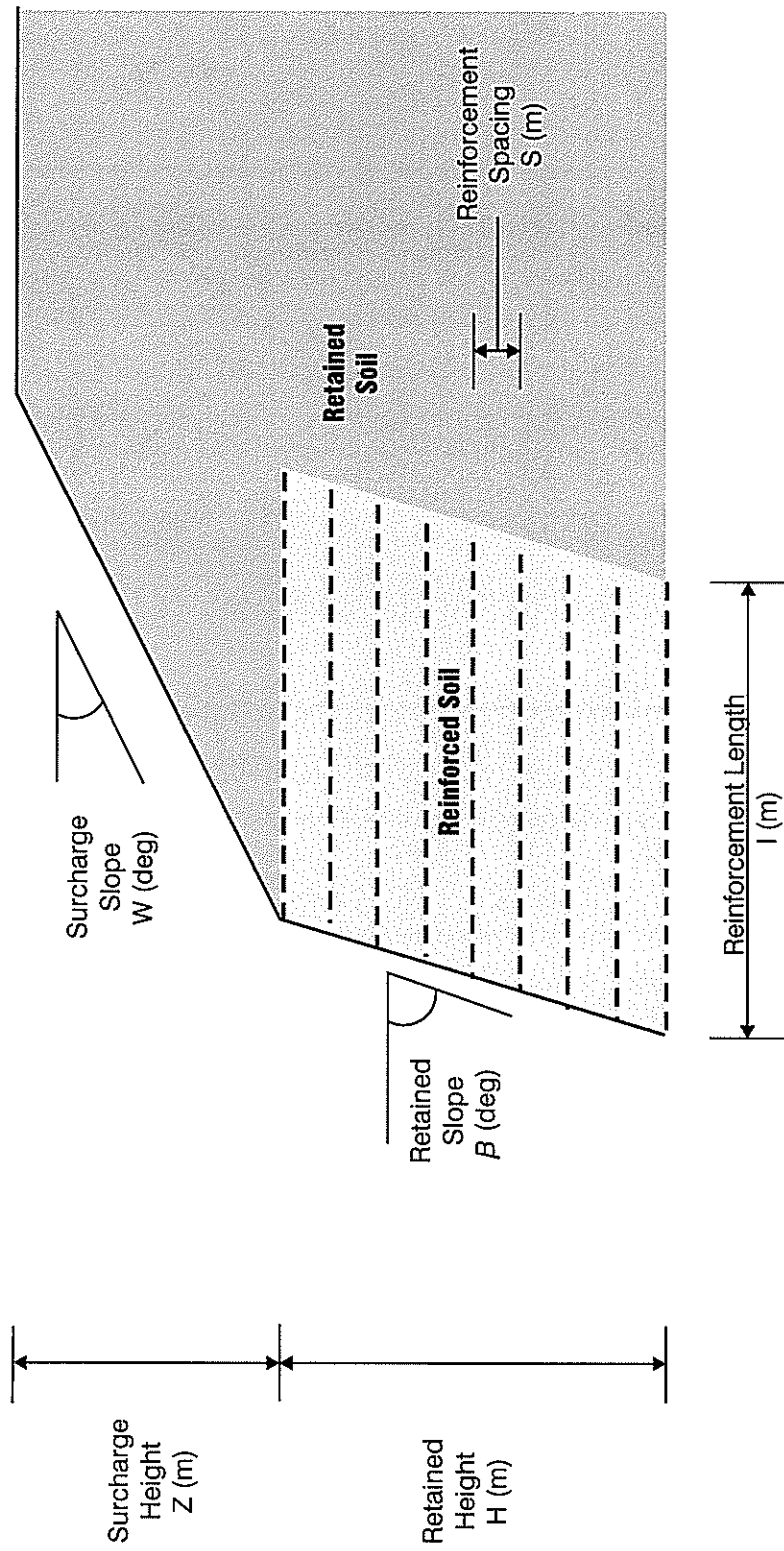


Figure 3.1 Typical geometry of a GRS structure.

3.2 Seismic Behaviour

The 1995 Kobe earthquake (Japan) had its epicentre on the northern tip of Awaji Island. The earthquake caused severe damage of gravity-type retaining walls. The Japanese Geogrid Research Board (GRB) and the Public Works Research Institute (PWRI) inspected sixteen GRS structures near the hypocentre where a seismic intensity of MMVII had been recorded (Nishimura et al. 1996). The inspected structures were geogrid-reinforced walls supporting highways, railway lines and parking areas. No signs of damage were observed during the inspections after the earthquake.

For ten of the inspected sixteen GRS structures a detailed verification of their seismic resistance was undertaken. The committee on Earthquake Observation and Research in Kansai reported peak ground accelerations between 0.4 g and 0.8 g for the area where the ten structures were located. The information about the inspected GRS structures is given in Table 3.2 (walls 1 to 10).

The authors suspect that some of the structures had not been designed to withstand any seismic loads. Nevertheless all the structures demonstrated high seismic resistance and stayed in serviceable condition after the earthquake. For example, the detailed analysis of the seismic stability of wall No. 5, assuming the horizontal ground acceleration to be as low as 0.1 g, indicated that the structure's factor of safety was less than 1.0. However, the wall stayed in serviceable condition despite having undergone ground accelerations between 0.5 g and 0.7 g, and despite the ground rupture that resulted in a 0.3 m-wide crack on the ground surface in front of the wall.

The predicted critical horizontal seismic coefficients (i.e. the seismic coefficient which, according to the current design methods, would be expected to initiate permanent movement of the structure) for all analysed walls are given in Table 3.2. These coefficients appear to be significantly lower than the actual recorded seismic coefficients. On the basis of the analysis, Nishimura et al. (1996) suggest that the existing seismic design methods for GRS structures are conservative.

Similar observations and conclusions were reported by Tatsuoka et al. (1996) for walls No. 11 to 15 (Table 3.2).

Information on the performance of five GRS structures that experienced the 1989 Loma Prieta earthquake (USA) was reported by Collin et al. (1992). None of the five structures were damaged during the earthquake.

3. Behaviour of GRS Structures

The satisfactory dynamic performance of the GRS structures can be explained by the following:

- The actual tensile strength of geosynthetic materials under seismic conditions is higher than that for static conditions. For seismic design, tensile strength tests conducted at higher than standard rates of strain would be more appropriate. These tests would often result in an allowable tensile resistance for seismic conditions appreciably higher than that for static conditions.
- The confined (in-soil) tensile strength of geosynthetic materials is very often higher than their unconfined (in-air) strength. In-soil ultimate tensile strength as high as four times the in-air strength has been reported (Claybourn & Wu 1993).
- The existing design methods do not take into account the ductility of GRS structures. For example, Tatsuoka et al. (1996) suggest that the ductility of GRS structures could be reflected in the design by using a design seismic acceleration lower than the expected peak ground acceleration.

Table 3.2 Behaviour of GRS structures during 1995 Kobe earthquake (after Nishimura et al. 1996; Tatsuoka et al. 1996).

No.	Completion of Construction	Facing	Reinforcement Type	Reinforcement Length (m)	Height of Wall (m)	Face Slope	Assumed Internal Friction Angle (°)	Critical Seismic Coefficient of Structure	Reported Seismic Coefficient in E'quake	Observations
1	June '90	Steel frame	Tensar SR55	4.8	6.5	1V:0.1H	30	0.2	≥0.4	No damage
2	Feb '91	Concrete block	Tensar SR55	3.0	4.0	1V:0.3H	30	0.1	≥0.4	No damage
3	Aug '91	Sandbag	Tensar SR55	5.5	6.2	1V:0.5H	30	0.1	≥0.4	No damage
4	Feb '92	Sandbag	Tensar SR55	5.0	6.6	1V:0.3H	30	0.3	≥0.4	No damage
5	March '93	Concrete block	Tensar SR55	4.5	5.25	1V:0.5H	30	0.1	0.5-0.7	Small gaps between concrete blocks; a few cm settlement occurred; wall stayed in serviceable condition
6	Dec '93	Sandbag	Tensar SR80	7.5	11.0	1V:0.3H	30	0.2	≥0.4	No damage
7	July '94	Steel frame	Tensar SR55	2.0	5.0	1V:0.5H	35	0.1	≥0.4	No damage
8	Aug '94	Sandbag	Tensar SR35	4.0	5.5	1V:0.2H	30	0.2	0.4	Crack ran parallel to wall face on crest, because of unstable foundation
9	Sept '94	Sandbag	Tensar SR35	3.5	5.0	1V:0.0H	30	0.1	≥0.4	No damage
10	Dec '94	Sandbag	Tensar SR80	5.5	5.0	1V:0.3H	30	0.4	≥0.4	No damage
11	Sept '93	L-shaped steel wire frame	Tensar SR55	2.0	4.0	1V:0.05H	Unknown	0.0	0.4-0.7	Minor deformations (in order of 20mm)
12	April '92	Full height rigid facing (concrete)	Vinylon grid	2.5-4.0	4.9	1V:0.5H	Unknown	0.2	0.2	No damage or deformation

3. *Behaviour of GRS Structures*

No.	Completion of Construction	Facing	Reinforcement Type	Reinforcement Length (m)	Height of Wall (m)	Face Slope	Assumed Internal Friction Angle (°)	Critical Seismic Coefficient of Structure	Reported Seismic Coefficient in E'quake	Observations
13	1993	Full height rigid facing (concrete)	Vinylon grid	2.5	5.0	1V:0.05H	Unknown	0.2	0.5-0.6	Wall moved outwards c.20mm at top of facing
14	March '94	Full height rigid facing (concrete)	Vinylon grid	1.8	3.5	1V:0.05H	Unknown	0.2	0.2	No damage or deformation
15	Feb '92	Full height rigid facing (concrete)	Vinylon grid	2.5-3.0	4.5	1V:0.05H	Unknown	0.2	0.6-0.7 or higher	Wall moved outward: 260mm at top, 100mm at ground surface level

4. STATIC DESIGN OF GRS STRUCTURES

4.1 Design Philosophy

A wide range of methods for the design of geosynthetic-reinforced structures are available. The different methods yield widely varying results.

4.1.1 Empirical Design Methods

Empirical design methods commonly used in engineering practice are based on simple analytical models. Such models normally adopt a number of assumptions which are not totally correct and, as a result, over-simplify the complex stress and strain distributions that actually occur in GRS structures. Different design methods recommend not only different models, but also different factors of safety which, in turn, increase the diversity of results among the various methods.

Most of the design methods commonly used in routine design, apply limiting equilibrium analysis to determine factors of safety against failure. These methods do not take into account the displacement and deformation behaviour of GRS structures and therefore are not able to model the behaviour of such structures with high accuracy. Strain compatibility between soil and geotextile for the limit equilibrium methods is not satisfied. They are also unable to model stress distribution of a surface load and the interaction between soil and reinforcement elements. These methods are constantly refined on the basis of new test data and information on actual behaviour of GRS structures and therefore, in essence, are empirical methods.

4.1.2 Finite Element Analysis

Empirically based design procedures for GRS structures consider overall structure stability, and the rupture and pullout capacities of the reinforcement, without analysis of deformations of the structure. However, in some cases the empirical methods may not provide accurate predictions of GRS behaviour and therefore more complex methods such as finite element modelling are applied. Such cases include:

- Sites with heterogeneous stratigraphy of foundation ground or with non-uniform backfill;
- Pre-established surfaces of discontinuity behind the structure (for example, existing slopes in the case of highway enlargements);
- Structures with non-standard geometry (for example, cases where reinforcement length is shorter than that required by empirical methods);
- Structures with complex load conditions;
- Structures where deformations must be predicted with high accuracy;
- Detailed analysis of the stress-strain state of GRS structures for major research projects.

The finite element analysis allows more accurate modelling of the stress-strain behaviour of GRS structures (Chew et al. 1990; Thamm et al. 1990). A wide variety of finite elements modelling soil–reinforcement interaction have been proposed. Elastic or more complex non-linear elasto-plastic type models are used to model the behaviour of soil and reinforcement. The interactions of soil and reinforcement are normally modelled by zero-thickness interface elements. A number of successful applications of the finite element method have been described in the literature (Fishman & Desai 1991; Christopher et al. 1989).

However, finite element modelling is very time-consuming and requires more parameters, compared with empirical methods, to describe soil and reinforcement behaviour. Very often such parameters are not readily available. Therefore, empirical methods are currently used for routine design and the research recorded in this report concentrates on the empirical design methods only.

4.2 Empirical Design Methods

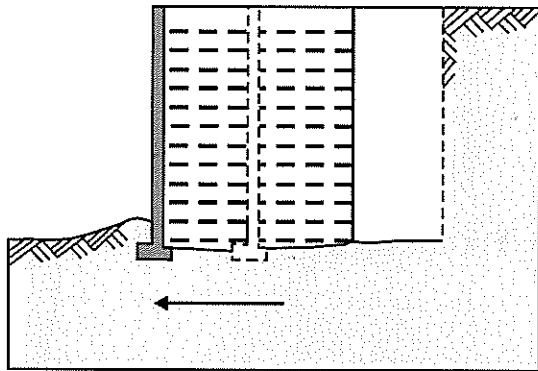
4.2.1 General

All empirical methods analyse external and internal stability of GRS structures. In the external stability analysis the structure itself is considered to be a rigid body and the stability of the structure with respect to sliding, overturning, bearing capacity or global failure (beneath or behind the structure), is checked by common methods of soil mechanics and foundation engineering. There is a great deal of similarity in the external stability analyses for different design methods. Therefore, the external stability analyses for several different design methods are discussed jointly rather than separately, in Section 4.2.2 of this report.

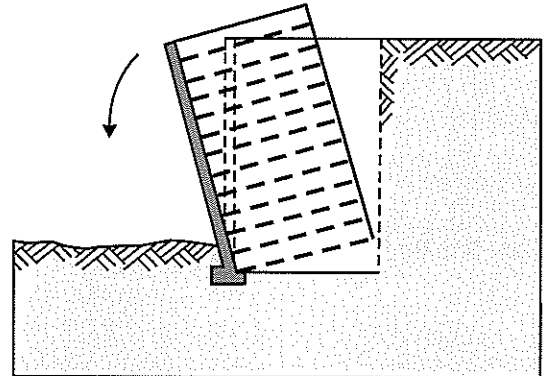
The internal stability analysis addresses the structural integrity of GRS structures. A number of different methods for the internal stability assessment have been developed. Most of them are based on limiting equilibrium analyses of destabilising horizontal forces resulting from earth pressures and stabilising horizontal forces from reinforcement. Most of the methods do not assess deformations of GRS structures, caused by the flexibility of the body of the structure itself. Several typical design methods most frequently used to assure internal stability of GRS are discussed in Section 4.2.3 of this report.

Design methods for flat GRS slopes (flatter than 45°) normally use conventional analytical methods for unreinforced soil slopes, and the methods are modified to account for the presence of geosynthetic reinforcement. A discussion of such design methods is beyond the scope of this study.

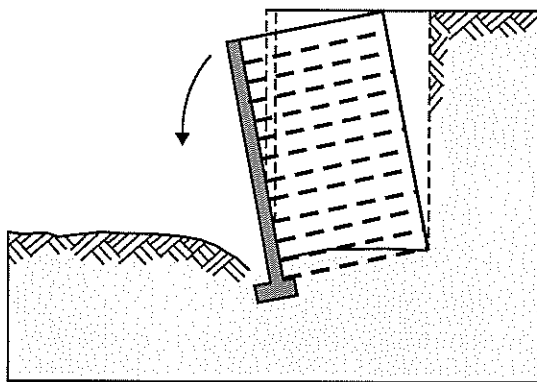
Some uncertainty with respect to the behaviour of GRS structures with cohesive backfills still exists. Therefore only cohesionless, free-draining, non-liquefiable soils are considered in this research project.



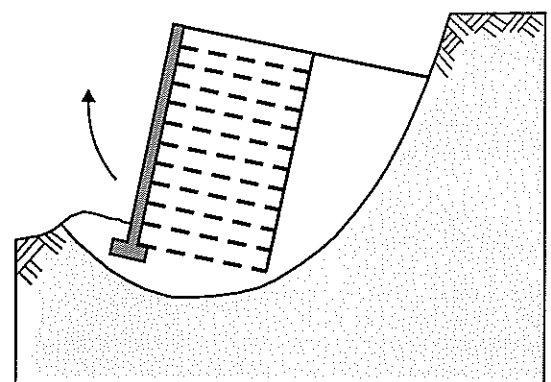
(a) Sliding



(b) Overturning



(c) Bearing Capacity



(d) Deep Seated Failure

Figure 4.1 External failure modes

4.2.2 Design for External Stability

The following failure modes are normally checked in the external stability analysis (Figure 4.1):

- (a) Forward sliding
- (b) Overturning
- (c) Bearing capacity
- (d) Deep seated failure

The failure modes are common to all retaining wall types and, therefore, are treated in a similar way to those of gravity or crib walls. A detailed description of the methods for the external stability analysis can be found in many existing guidelines, e.g. BS8006 (BSI 1995).

Because of the flexibility of GRS structures and their satisfactory field performance, the factors of safety used in their external stability analysis are, in some cases, lower than those used for gravity-type retaining walls. Likewise, the geometry and flexibility of GRS structures makes the potential for overturning failure very low. Nevertheless, most of the existing guidelines require overturning criteria to be checked. Elias & Christopher (1997) believe that the overturning failure criteria aid in controlling lateral deformation by limiting tilting and, as such, should be satisfied.

Most of the existing guidelines also require that the total settlement of a GRS structure must be checked. The total settlement is considered to be the combined effect of the settlement of the foundation soil under the influence of the pressures imposed by the GRS structure and the internal compression of the reinforced backfill.

No methods for the assessment of the internal compression settlement are given in the existing design guidelines. However, BS8006 provides some general comments on the factors that should be considered in the settlement calculation.

The settlement of the foundation soil is normally assessed on the basis of conventional settlement analyses for shallow foundations.

If foundation settlement (total or differential) is excessive, then improvement of the foundation soils should be considered.

4.2.3 Design for Internal Stability

Internal failure of a GRS structure occurs in two different ways:

- Failure by excessive deformation or breakage of geosynthetic reinforcement;
- Failure by pullout of geosynthetic reinforcement from the soil mass.

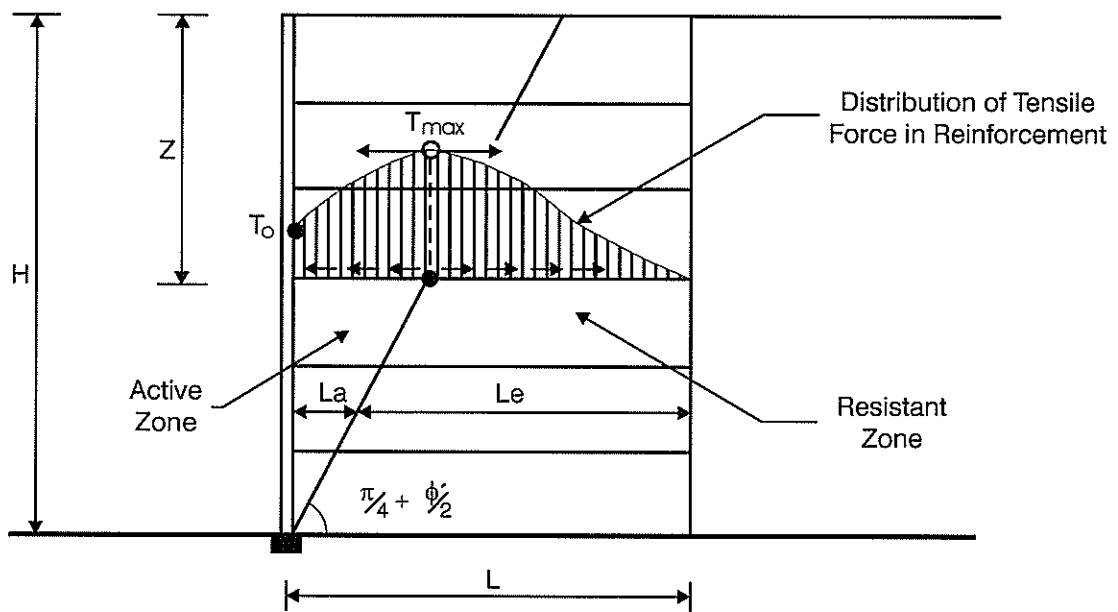


Figure 4.2 Tensile forces in reinforcement

The design process comprises determining maximum tension forces in the geosynthetic reinforcement and comparing them with the available pullout capacity and the tensile strength.

Almost all empirical methods assume that an active zone and a resistant zone are present within a GRS structure (Figure 4.2). These zones are separated by an hypothetical maximum tensile force line. In an active zone the shear stresses on the reinforcements are directed towards the wall face. In a resistant zone the shear stresses are directed away from the wall face. Christopher et al. (1989) have found that the location of the maximum tensile force line is influenced by the extensibility of the reinforcement. For geosynthetic reinforcement most of the existing design methods assume (on the basis of test data) that the maximum tensile force line coincides with the Coulomb or Rankine active failure plane.

A generalised design procedure is described by Elias & Christopher (1997). This procedure comprises the following:

- Select a type of geosynthetic reinforcement;
- Select the location of the critical failure surface;
- Select a reinforcement spacing compatible with the facing;
- Calculate the maximum tensile force at each reinforcement level;
- Calculate the maximum tensile force at the connection to the facing;
- Calculate the pullout capacity at each reinforcement level.

Descriptions of several most frequently used design methods are given below. This report is not intended to be used as design guidelines and, therefore, only the most essential features of each method are described and references are given to documents containing detailed method descriptions. As this report concentrates on methods of determining geosynthetic reinforcement length and spacings, the design of facing details is not discussed.

There is a great deal of similarity between mathematical symbols used by different design methods. Therefore the symbols used for each method are explained where they appear the first time in the text, and they are the symbols that were used by the authors of the methods. Where required, additional clarifications and explanations of already defined symbols are given for each method.

(a) Forest Service Method (USA)

This design approach was first proposed by Steward et al. (1977) of the US Forest Service and has been subsequently incorporated (with some amendments) in a number of US design guidelines for GRS structures (AASHTO 1992; Elias & Christopher 1997; Christopher et al. 1990b).

To determine the geosynthetic layer separation distances in the Forest Service method, earth pressures are assumed to be linearly distributed with depth based on active (K_a) conditions for the soil backfill and at-rest (K_o) conditions for the surcharge. However, Jarrett & McGown (1988) showed later that K_a rather than K_o conditions should be used for surcharge.

The internal stability check is given by the following procedure:

Horizontal earth pressure:

$$\sigma_h = K_a \times (\gamma Z + q) \quad (4)$$

where σ_h is horizontal earth pressure;

γ is unit weight of soil;

Z is depth measured from the top of the structure;

q is uniformly distributed, vertical surcharge.

Maximum tension (T_{max}) in each reinforcement layer:

$$T_{max} = \sigma_h \times S_v \quad (5)$$

where S_v is the vertical spacing between layers of geosynthetic.

Resistance to breakage:

$$T_{max} \leq T_d \quad (6)$$

where T_d is long-term design strength of a geosynthetic material (Section 2.4.2).

Resistance to pullout failure:

$$T_{max} \leq \frac{1}{FS} F^* \times \alpha \times \sigma'_v \times 2 \times L_e \quad (7)$$

where $F^* \times \alpha \times \sigma'_v \times 2 \times L_e$ is ultimate pullout resistance (Section 2.5.1);

$FS = 1.5$ and is safety factor against pullout.

If the above criteria are not satisfied for all layers of geosynthetic reinforcement, the reinforcement length must be increased and/or reinforcement with higher tensile strength must be used.

Total length of reinforcement:

$$L = L_a + L_e \quad (8)$$

where L_e is required embedment or adherence length;

$L_a = (H-Z) \tan (45^\circ - \phi'/2)$ (Figure 4.2);

Z is the depth to the reinforcement level;

Base sliding:

$$FS_{\text{sliding}} = \frac{\Sigma \text{ horizontal resisting forces}}{\Sigma \text{ horizontal driving forces}} \geq 1.5 \quad (9)$$

where FS_{sliding} is factor of safety against sliding.

For the ease of construction a final uniform length is normally chosen, based on the maximum length required. However, if necessary the length can be non-uniform.

(b) Broms Method (Sweden)

This method was proposed by Ben B. Broms, Professor at the Royal Institute of Technology, Stockholm, Sweden (Broms 1978) and has been widely used for the design of GRS structures in Sweden since then. Broms proposed to use the same uniform lateral earth pressure distribution (Figure 4.3) as that suggested by Terzaghi and Peck for anchored sheet pile walls:

$$\sigma_h = 0.65 K_a (1.5q + \gamma H) \quad (10)$$

The total lateral earth pressure $\sigma_h \times H$, in this case, is approximately 30% higher than the active earth pressure to take into account the natural variations of the lateral earth pressure. The vertical spacing (S_v) between the layers of geosynthetic is given by:

$$S_v = \frac{T_a}{0.65 K_a (1.5q + \gamma H)} \quad (11)$$

where T_a is long-term allowable strength of geosynthetic reinforcement.

To calculate the required adhesion length, Broms assumed that if sliding failure occurs, involving a wedge with the width at the bottom of L_n (where L_n is the length at the n -th reinforcement layer), the height of Z_n and the back face extending up at the angle of $45^\circ + \phi'/2$, then the sliding resistance for layer $n-1$ developed beyond the back face of the wedge should be sufficient to mobilise allowable tensile strength in reinforcement (increased by an appropriate factor of safety). This assumption results in a stepped failure surface and stepped reinforcement for GRS structures (Figure 4.3). The adhesion length for layer L_{n-1} is:

$$L_{n-1} = \frac{FS \times T_a}{\gamma S_v (n-1) \tan \phi_{SG}} \quad (12)$$

where $FS = 1.3$ and is factor of safety;

$\tan \phi_{SG}$ is soil-geosynthetic friction coefficient.

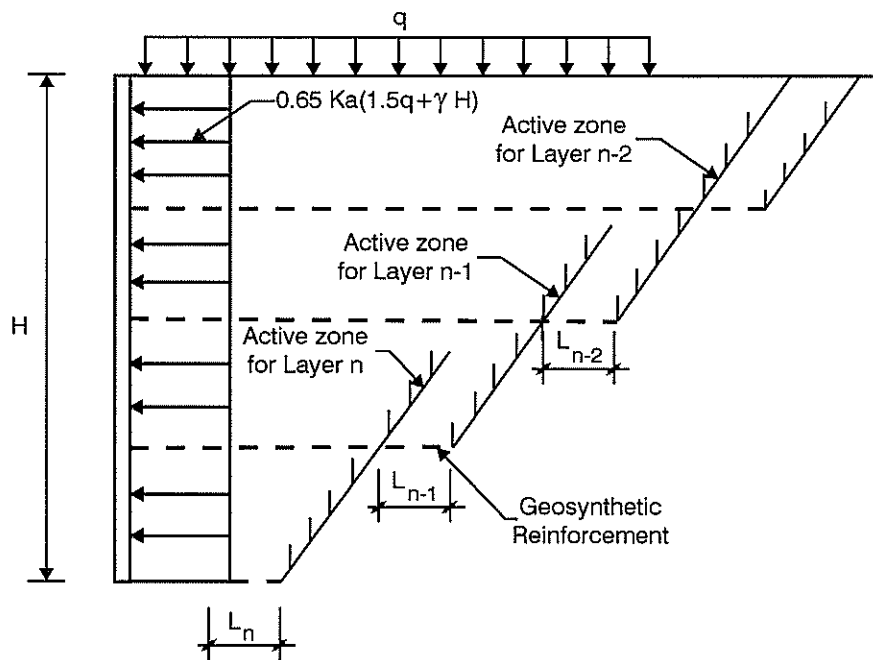


Figure 4.3 Broms method

(c) Leshchinsky Method (USA)

Leshchinsky & Perry (1987) proposed a design method based on the variational limit equilibrium approach developed by Baker & Garber (1977). This method with some amendments is described in detail in the US Army "Design Procedure for Geosynthetic-Reinforced Steep Slopes" (Leshchinsky 1995). It is assumed that the soil within the GRS structure obeys Mohr-Coulomb's failure criterion and that each geosynthetic sheet can develop tensile resistance t_j . The mobilised strength of each component resisting failure is:

$$\tau_m = \sigma \Psi / FS = \sigma \Psi_m \quad (13)$$

$$t_{mj} = t_j / FS \quad (14)$$

where FS is the factor of safety of the GRS structure against collapse;

$\Psi = \tan \phi'$ (only cohesionless materials are considered, $c' = 0$);

ϕ' is angle of internal friction;

$$\Psi_m = \frac{\tan \phi'}{FS}$$

σ and τ are the stresses normal and tangent to the potential slip surface shown on Figure 4.4a;

t_j is the allowable tensile resistance of geosynthetic layer j ;

m symbolises a mobilised strength component.

t_j is determined, taking into account the following three aspects:

- Pullout resistance developing over the reinforcement length beyond the slip surface.
- A prescribed elongation of the geosynthetic reinforcement to minimise incompatibility with soil (a 10% elongation criteria is proposed).
- Possible creep elongation.

Detailed recommendations on the assessment of allowable tensile resistance are given by Leshchinsky & Perry (1987).

Using variational extremisation, Leshchinsky showed that there are two possible failure modes in homogeneous soil: rotational or translational. The slip surface geometry corresponding to the first mode is log-spiral and to the second mode is planar. The failure mechanism with the lowest factor of safety is likely to develop. Rotational failure mechanism is shown on Figure 4.4b. The log-spiral failure surface is expressed as:

$$R = Ae^{-\Psi m \beta} \quad (15)$$

where R , β are co-ordinates of the log-spiral failure surface, relative to a polar co-ordinate system which has its origin at a point with normalised co-ordinates $X_c = x_c/H$, $Y_c = y_c/H$ in the cartesian system XOY;

A is an unknown constant.

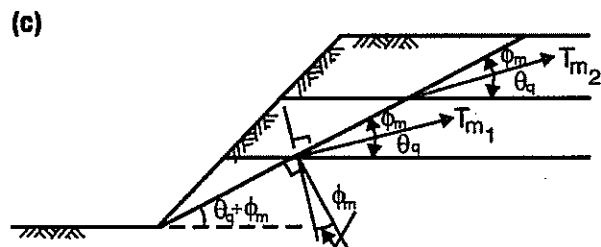
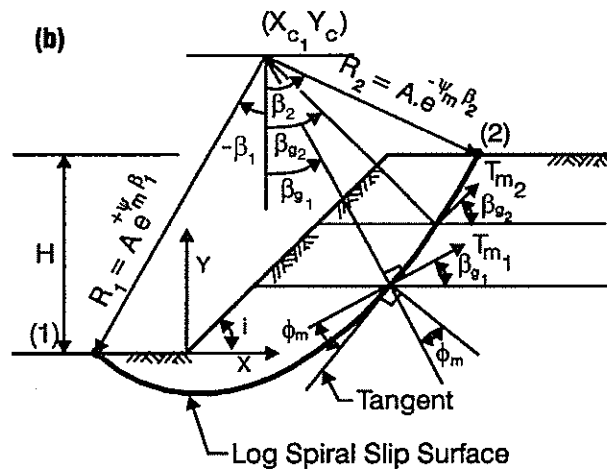
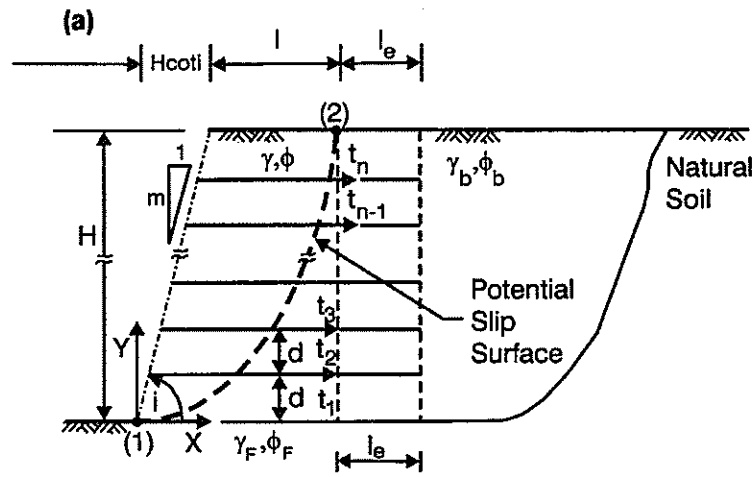


Figure 4.4 Leshchinsky method

It is assumed that, when the failure of a GRS structure occurs, t_j (or the geosynthetic reinforcement layer) at the slip surface will be inclined so as to contribute its most resistance, i.e. orthogonal to the radius of the log-spiral (Figure 4.4b):

This translational failure mechanism is shown on Figure 4.4c. In the case of translational failure, the geotextile is inclined at ϕ'_m to the failure plane:

$$\phi'_m = \tan^{-1} [(\tan \phi)/FS] \quad (16)$$

The determination of the required t_j value is coupled with the pullout resistance check. It is therefore assumed that t_j is a function of the overburden pressure:

$$t_j = t_1 (H-y_j) H \quad (17)$$

where t_1 is the pullout resistance of the reinforcement layer at $y_1 = 0$.

It is also assumed that all layers of reinforcement extend to the same vertical plane, $x = H \cot i + l + l_e$ (Figure 4.4a), and that the geotextile pullout resistance is developed only along l_e , i.e. pullout resistance developing between the slip surface and $x = H \cot i + l$ is neglected.

The angle of friction between the retained soil and geotextile is assumed to be $2\phi'/3$.

A closed-form solution for each failure mode based on variational approach was used to develop detailed design charts. The charts account for a transition from a translational failure surface (for near-vertical slopes) to a rotational failure surface (for flatter slopes).

(d) Schmertmann Method (Design Charts) (USA)

Schmertmann et al. (1987) developed design charts for GRS slopes with angles ranging from 30° to 80° for the Tensar Corporation. Schmertmann's method is a further development and improvement of the design charts published earlier by Ingold (1982), Murray (1984), Jewell et al. (1984), Leshchinsky & Reinschmidt (1985), Schneider & Holtz (1986). All these methods are based on limit equilibrium models to investigate the amount of reinforcement required to maintain slope stability.

The method is based on the following assumptions:

- The reinforcement layers are truncated along a plane parallel to the slope face.
- The soil is modelled as a purely frictional material ($c' = 0$) complying with the Mohr-Coulomb failure criterion.
- Slope foundation is stable.
- Slope face and crest are flat.
- There are no pore pressures within the slope.
- Only uniform surcharge is considered.

- Soil–reinforcement interaction coefficient equals 0.9.
- The desired overall soil factor of safety is accounted for by using factored soil shear strength:

$$\phi'_f = \tan^{-1} (\tan \phi' / FS) \quad (18)$$

where ϕ' is the soil friction angle.

- No overall factor of safety is applied to the allowable reinforcement strength but the amount of strain required to mobilise the long-term allowable strength should not be more than the GRS structure limit strain.
- All failure surfaces enter behind the slope and exit through the toe of the structure.

The two part wedge model shown on Figure 4.5a was used to predict the required total reinforcement force, T_t . The ratio between the magnitudes of the two reinforcement forces T_1 and T_2 was determined based on an assumed triangular reinforcement tension distribution which increases proportionally with depth below the slope crest.

A search was then performed by Schmertmann for the critical combination of node location and wedge angle θ , producing the largest total required reinforcement force, T_t . In the design charts, T_t is presented non-dimensionally as horizontal reinforcement force coefficient K :

$$K = 2T_t / \gamma (H')^2 \quad (19)$$

where γ is unit weight of soil fill; and H' is modified slope height:

$$H' = H + (q/\gamma) \quad (20)$$

where H is slope height; and q is uniform surcharge on top of slope.

It was also assumed that the reinforcement length at the bottom of the slope, L_B (Figure 4.5c), would be controlled by external stability requirements and that the reinforcement length at the top of the slope, L_T , would be controlled by internal stability requirements (Figure 4.5b). The straight-line wedge model shown on Figure 4.5b was used to predict L_T .

According to the method, geosynthetic layers should be spaced inversely proportional to the depth measured from the crest of the slope. The maximum vertical spacing (S_{vmax}) between the layers of geosynthetic is given by:

$$S_{vmax} = \frac{T_a}{K \gamma Z} \quad (21)$$

The results were generated for a range of slope angles and soil strengths, and compared to results obtained from the more detailed limit equilibrium analysis methods such as Bishop's Modified Method (with allowance for reinforcement effect) and Spencer's Method.

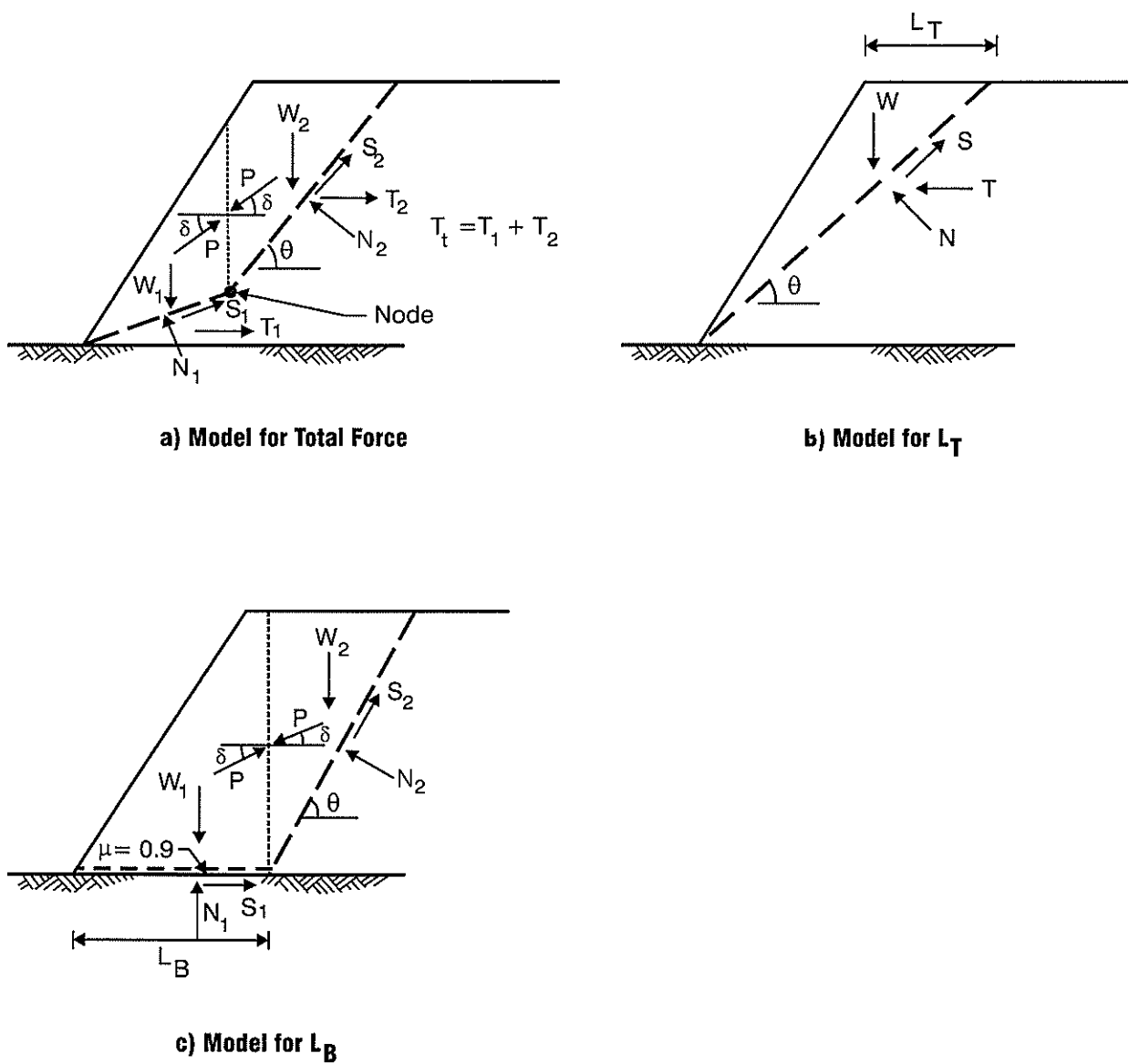


Figure 4.5 Schmertmann method

In some cases K , L_B and L_T predicted by Bishop's Modified Method and Spencer's Method proved to be more conservative and, therefore, the results of simple wedge analysis were adjusted to be more consistent with these more conservative values and with other published results.

(e) Bonaparte Method (USA)

Bonaparte et al. (1985) suggested a design method which is very similar to all other tieback wedge methods. The only difference is in the definition of the lateral pressures.

The method assumes mobilisation of the full shear strength of the reinforced fill and generation of active earth pressures. The classical Rankine failure surface is assumed to be the locus of maximum reinforcement tensile forces. As opposed to other tieback wedge methods, earth pressures within the reinforced soil mass in the Bonaparte method include a vertical component of the earth pressure thrust from the retained fill (Figure 4.6).

The maximum vertical stresses, σ_z , induced by gravity, uniform vertical surcharges, and the active thrust from the retained fill (at any depth z) are calculated assuming Meyerhof's internal vertical stress distribution beneath and within the reinforced fill:

$$\sigma_z = \frac{\gamma z + q}{1 - \left[\frac{K_{ar} (\gamma_r + 3q) (z/L)^2}{3 (\gamma_r z + q)} \right]} \quad (22)$$

- where K_{ar} is coefficient of active earth pressure of retained fill;
- γ, γ_r are unit weights of reinforced soil and retained fill respectively;
- q is uniformly distributed normal surcharge;
- L is reinforcement length, obtained from the external stability analysis.

The above equation is based on the assumption that active thrust from the retained fill is horizontal. This assumption is conservative.

The maximum horizontal stress at depth z from gravity, normal surcharge and the thrust of the retained fill is calculated by:

$$\sigma_h = K_a \sigma_z \quad (23)$$

where K_a is coefficient of active pressure of reinforced soil.

The required maximum tensile force per unit width in the reinforcement at depth z is finally given by:

$$T_{max} = \sigma_h S_v \quad (24)$$

where S_v is reinforcement vertical spacing.

For internal stability calculations it is recommended that a factor of safety be applied to the soil shear strength.

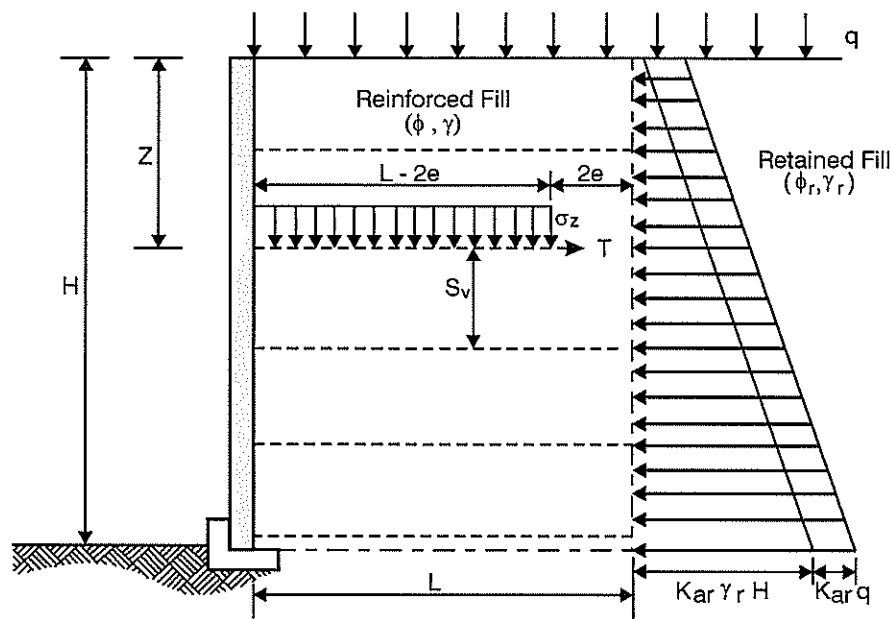


Figure 4.6 Bonaparte method

$$\tan \phi'_f = \frac{\tan \phi'}{FS} \quad (25)$$

where ϕ' is angle of internal friction of soil;

ϕ'_f is factored angle of internal friction;

FS is factor of safety.

It is also suggested that a factor of safety of 1 is used to calculate the design tensile strength.

(f) Collin Method (USA)

An extensive research on design methods for GRS structures was undertaken by Collin (1986). In his doctoral thesis he recommended two design methods: one for geotextiles and the other for geogrids.

Collin used finite element analysis to develop recommended lateral pressure distributions within the reinforced block. The proposed pressure distributions were then verified by test data from three field-instrumented walls. Therefore, as opposed to the other methods discussed in this report, Collin's pressure distributions account for the interaction between soil and reinforcement in the reinforced block.

A coefficient of interface friction for geogrids and geotextiles of $(2/3) \phi'$ is used. Recommended pressure distributions are shown on Figure 4.7.

Although the lateral pressures in the Collin method depend on the height of a GRS structure only, good agreement between the predictions based on the method and the field test data has been reported by the author. All calculations are similar to those in the Forest Service and Broms methods.

(g) Displacement Method (France)

The displacement method was proposed by Gourc et al. (1986) and has been adopted by French standard NF G 38064. The CARTAGE software has been developed on the basis of the displacement method and is currently sold and widely used in France. A thorough validation of the design method and CARTAGE software on full scale GRS structures has been undertaken as part of a joint Franco-Japanese project (Yoshioka et al. 1990).

As in all limiting equilibrium methods, the block is divided into active and passive zone, by a potential slip surface. The slip surface may be flat or curved. It is assumed that the active wedge slips without any overall deformation $\Delta z_j = \Delta z$ (Figure 4.8a). Mobilisation of shear strength along the sliding surface is assumed to be a function of the displacement.

The geosynthetic is considered to act as an anchored membrane with the active $L^a a_j$ and passive $L^p a_j$ zones.

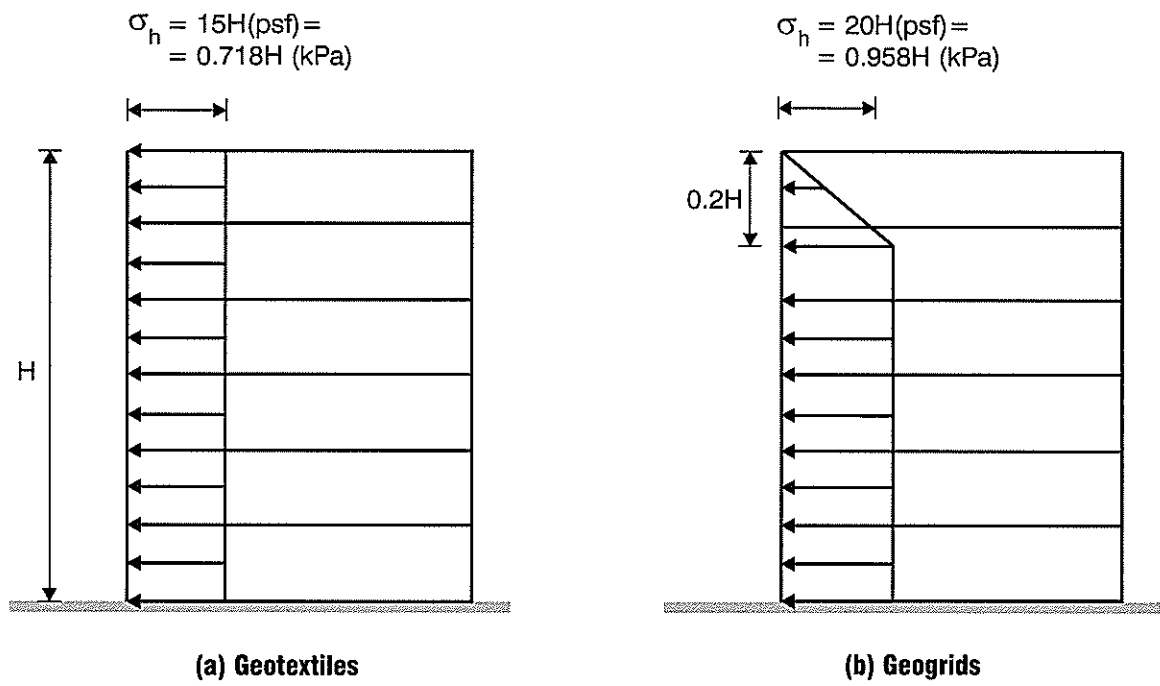


Figure 4.7 Lateral earth pressure distributions in Collin method

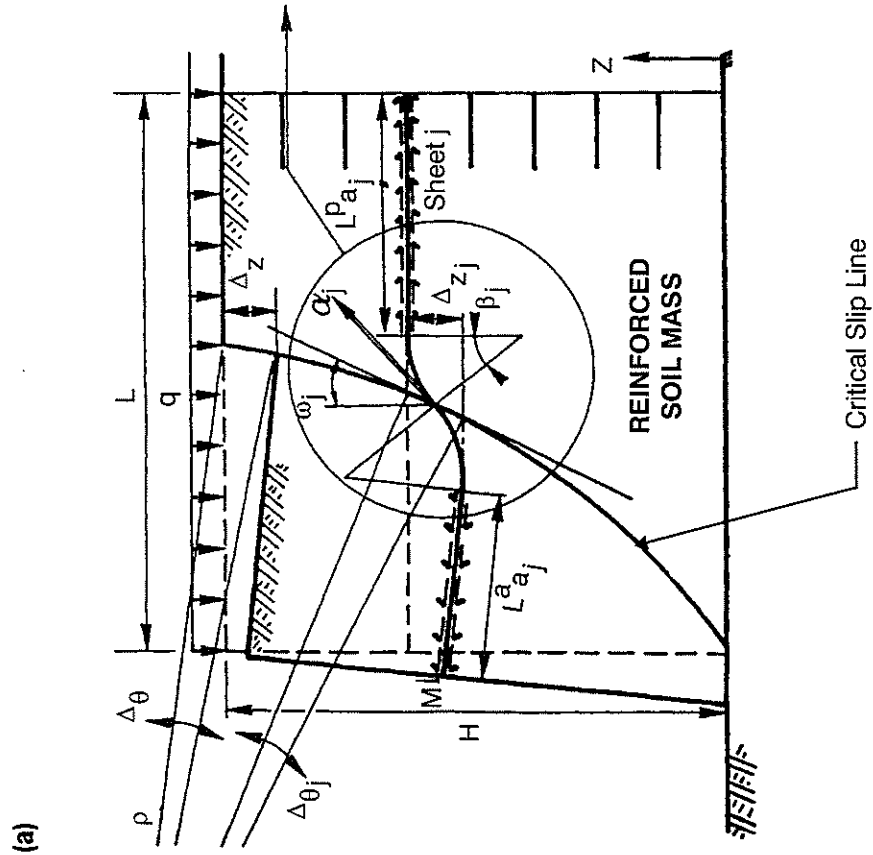
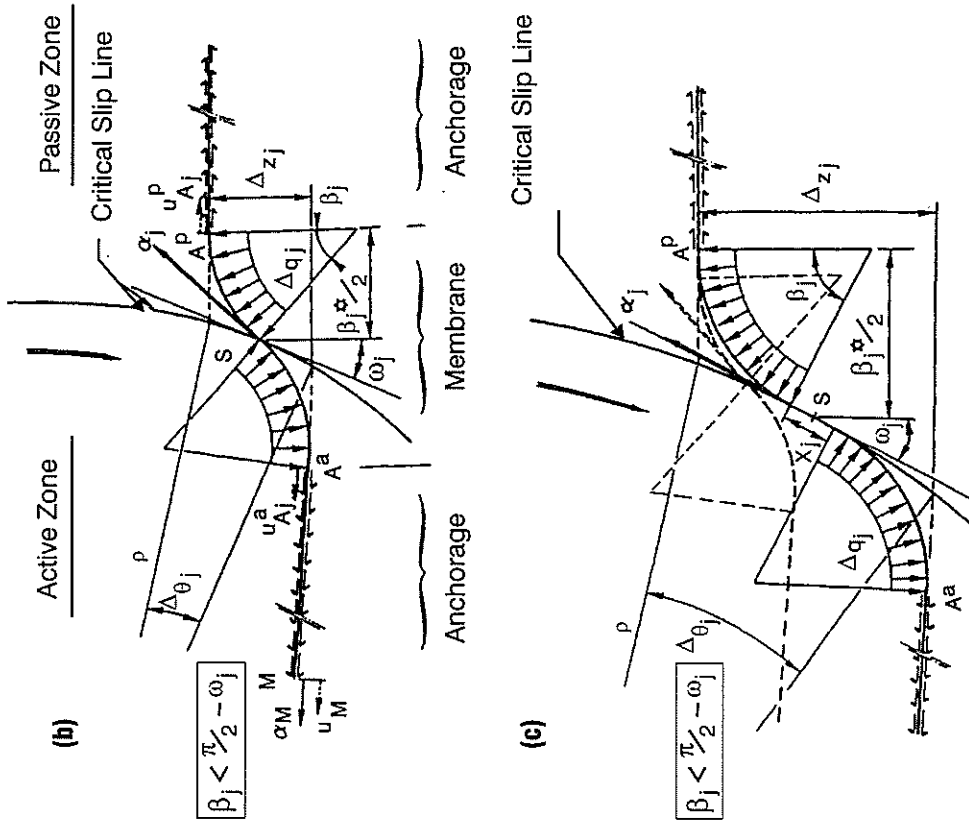


Figure 4.8 Displacement method (after Gourc et al. 1986)

Tensile force-strain relationship for a geosynthetic is assumed to be linear:

$$\alpha = J \times \epsilon \quad (26)$$

where α is tensile force;

J is geosynthetic stiffness modulus;

ϵ is geosynthetic strain.

The tensile force in the geosynthetic is assumed to be dependent on the geosynthetic stiffness modulus, the anchorage length, the friction law, and the displacement at the point of the force application on the slip surface:

$$\alpha_j = f(u_{Aj}, J, \sigma_{zj}, L_{aj}) \quad (27)$$

where u_{Aj} is the displacement at the head of the anchorage.

It is considered that a geosynthetic which behaves as a membrane is loaded by stresses, Δq_j , normal to the deformed geosynthetic layer. As long as Δz_j remains small the membrane is assumed to be bi-circular (Figure 4.8b).

For large displacements of Δz_j , the membrane is assumed to be tangent to the slip surface (Figure 4.8c).

The method allows calculation of the amount of geosynthetic reinforcement required to satisfy stability requirements and also to assess the deformations of a GRS structure. If the required factor of safety is obtained within the specified displacements limit, then the calculation stops. If not, then the required quantity of reinforcement is identified by interpolation and iteration. Calculations are carried out on an array of potential failure surfaces to identify the critical surface. Constant volume soil friction angles (Section 4.3.3) are used.

(h) Deutsches Institut für Bautechnik (DIBt) Method (Germany)

The DIBt design method is described in the Deutsches Institut für Bautechnik (DIBt) Certificate (1990) and is based on appropriate German DIN standards. The method requires that the constant volume soil friction angle (Section 4.3.3 of this report) for soils ϕ'_{cv} is used.

Internal stability calculations are based on a two part wedge analysis of the reinforced soil block. A series of two part wedges (such as wedges 1 to 4 on Figure 4.9a) are examined with the lower part of the slip surface originating at the structure face and passing through the block, and the upper part of the slip surface passing up the back face of the reinforced soil block. The active pressure E_a above that point where the slip surface cuts the back face of the reinforced soil block is added to the disturbing forces acting on the two part wedge to give the total disturbing force. The disturbing force on any wedge is (Figure 4.9b):

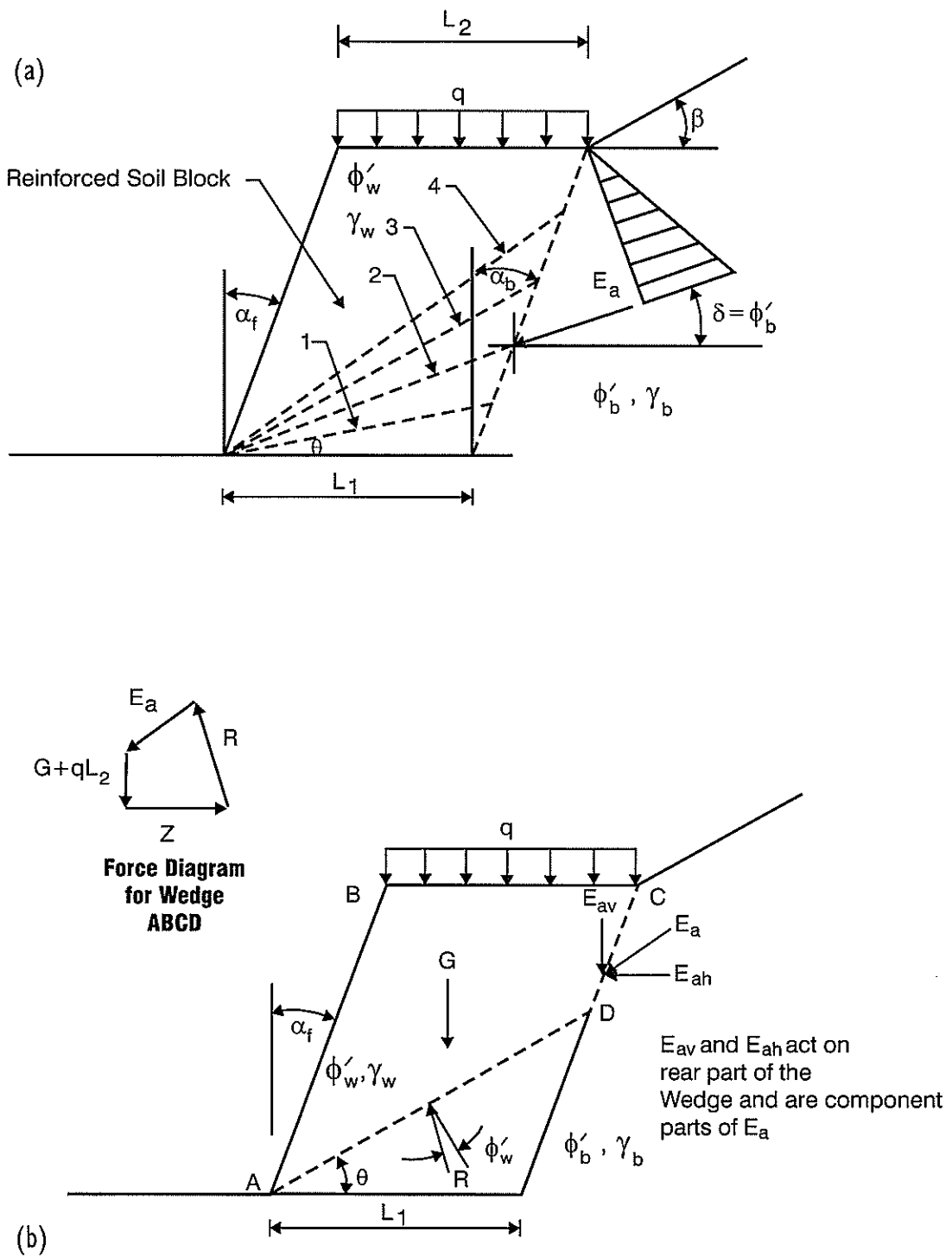


Figure 4.9 Bautechnik method

4. *Static Design of GRS Structures*

$$Z = (G + qL_2 + E_{av}) \times \tan(\theta_i - \phi'_w) + E_{ah} \quad (28)$$

where G is the self weight of the reinforced block;

E_{av} , E_{ah} are vertical and horizontal components of the active soil pressure E_a ;

ϕ'_w is the angle of internal friction of reinforced soil.

The sets of wedges, inclined at a range of angles θ_i , are checked at the toe of the structure, at the bottom grid layer (if at a different level), at levels where the reinforcement spacing alters, and at every level where the geosynthetic type alters.

Reinforcement must be provided to resist the disturbing force for each two part wedge. The design strength of the geosynthetic reinforcement is specified in the DIBt Certificate.

The method has been incorporated into the Winwall computer programme developed by Netlon Limited.

(i) HA68/94 Method (UK)

The details of the method are given in the UK Department of Transport Advice Note HA68/94 (1994). The method deals with steep slopes (up to 70°) on stable foundations and assumes a two part wedge mechanism.

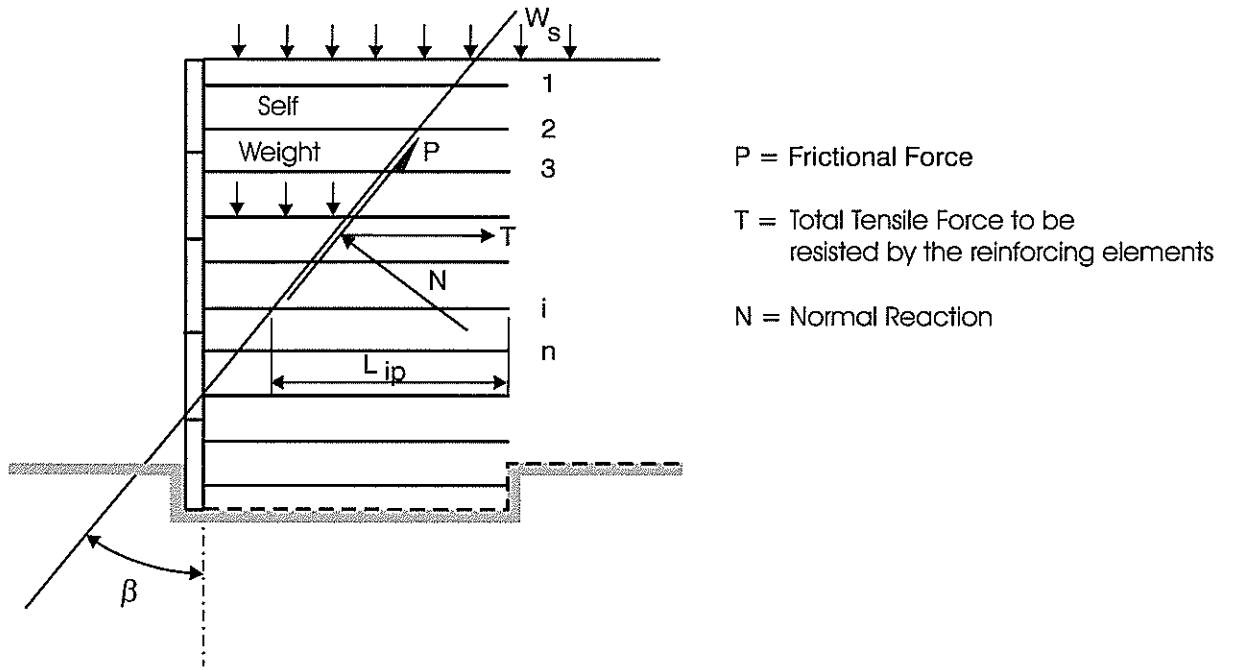
Surcharges are presented as uniformly distributed loads which can be modelled as an extra height of the GRS structure. Constant volume soil strength (ϕ'_{cv}) is used. The method is based on a two part wedge mechanism of failure which, while being a simplified approach, enables a conservative solution to be obtained.

In essence, this method is very similar to the Schmertmann method described in Section 4.2.3(d). The only difference between HA68/94 and the Schmertmann method is that interwedge friction is ignored in HA68/94, but in the Schmertmann method the interwedge friction angle is assumed to be equal to the factored soil friction angle. HA68/94 gives design charts similar to the Schmertmann method.

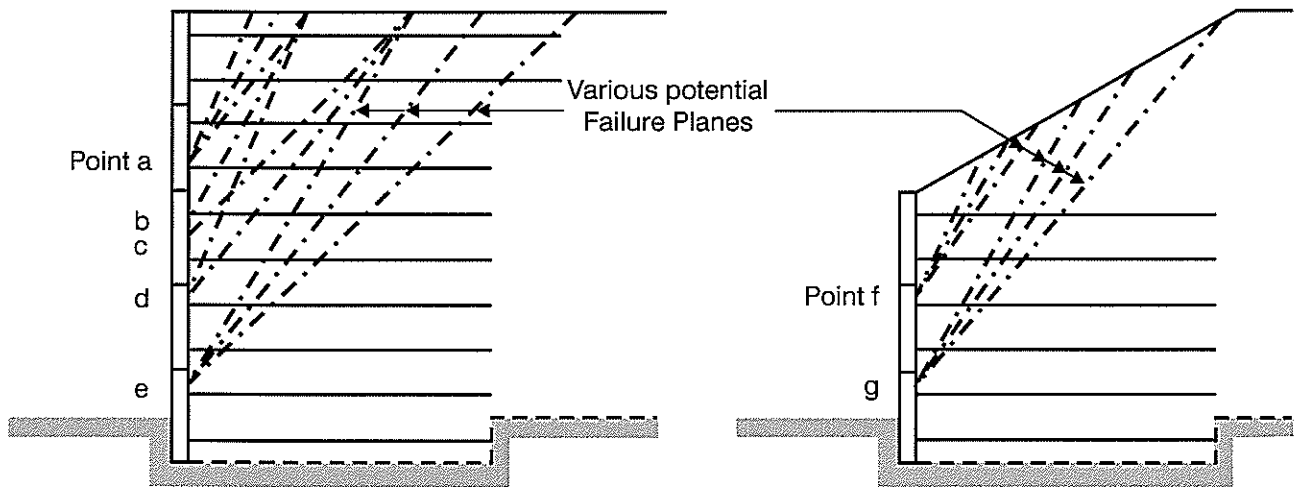
(j) BE 3/78 Method (UK)

The details of the method are given in the UK Department of Transport Technical Memorandum BE3/78 (1987) which was first published in 1978 and updated in 1987. Reinforcement is designed at working loads with built-in safety factors to produce permissible stresses. The peak friction angle of the soil is used in design, and lateral earth pressure distribution is based on the active pressure coefficient.

The depth of the reinforced soil block and hence the length of reinforcement is determined from the external stability analysis while the quantity and layout of reinforcement is determined from the internal stability analysis.



(a) Forces to be considered



(b) Various Potential Failure Planes

Figure 4.10 BE 3/78 method

4. *Static Design of GRS Structures*

In the analysis of the external stability the effect of friction on the back of the wall is ignored. The minimum length of reinforcement is specified as the greater of 5 m or 80% of the retained height.

The maximum tensile force T_i to be resisted by the i -th layer of reinforcement at a depth h_i is:

$$T_i = V K_a (\gamma h_i + w_s) \quad (29)$$

where V is vertical spacing between layers of geosynthetic;

w_s is uniformly distributed surcharge on the top of the GRS structure.

Concentrated horizontal and vertical surcharges can also be allowed for, if required. Resistance to breakage and to pullout failure is checked as follows:

$$T_i \leq T_{at} \quad (30)$$

where T_{at} is the permissible tensile strength of the geosynthetic (recommendations for assessing T_{at} are given in BE3/78);

$$T_i \leq \frac{1}{FS} T_{max} \quad (31)$$

where T_{max} is ultimate pullout resistance;

FS is factor of safety against pullout taken as 2.0.

The method requires that sliding of the upper portions of a GRS structure on any horizontal plane is checked. The frictional resistance at an interface should be not less than twice the total horizontal load that is tending to cause instability.

The following interfaces are considered:

- fill on fill within any layer,
- reinforcement on any layer of fill.

The method also requires that the stability of a series of wedges, each of which is bounded by the front face of the wall, the top face and an inclined potential failure plane, is checked as shown on Figure 4.10a. According to the proposed analytical procedure a fan of wedges at a number of vertical positions (Figure 4.10b) are checked to ensure that pullout is not a problem at any point in the structure.

(k) BS 8006 Method (UK)

BS8006 (BSI 1995) has been developed as guide for good current practice in the UK. The document has been written in the limit state format, using partial factors of safety.

The BS8006 method for reinforced walls is a tie-back wedge method similar to the Bonaparte method described in Section 4.2.3(e) of this report, in that the earth pressures within the reinforced soil mass include a vertical component of the active

thrust from the retained fill, and that the Meyerhof internal vertical stress distribution is used to calculate the vertical stresses induced by gravity, vertical surcharges and the active thrust.

The maximum ultimate limit state tensile force, T_j , to be resisted by the j -th level of reinforcement according to the Meyerhof distribution, at a depth of h_j , below the top of the structure is obtained from:

$$T_j = K_a \sigma_{vj} S_{vj} \quad (32)$$

where σ_{vj} is the factored vertical stress acting on the j -th level of reinforcement according to the Meyerhof distribution.

The rupture stability and the pullout capacity checks are done in a manner similar to all other methods:

Rupture

$$\frac{T_D}{f_n} \geq T_j \quad (33)$$

where T_D is the design strength of the geosynthetic reinforcement calculated using partial factors of safety recommended by BS8006;

f_n is the partial factor of safety for economic ramification of failure.

Pullout

$$P_j \geq \frac{T_j}{(\mu L_{ej} \sigma'_{vj}) / (f_p f_n)} \quad (\text{for cohesionless soils}) \quad (34)$$

where P_j is the total horizontal width of the top and bottom faces of the geosynthetic reinforcement;

L_{ej} is the length of reinforcement in the resistant zone (L_e on Figure 4.2);

μ is soil-geosynthetic friction coefficient;

σ'_{vj} vertical pressure induced by gravity and uniform normal surcharges only;

f_p the partial factor for reinforcement pullout resistance;

f_n the partial factor for economic ramification of failure.

BS8006 also requires to check the stability of a series of wedges in a manner similar to that of BE 3/78 (Figure 4.10).

BS8006 is probably the only document which provides recommendations on the acceptable post-construction strain limits for the reinforcement. According to BS8006 the post-construction strains should be considered separately from both the external and the internal stability analyses.

4.3 Comparison of Design Methods

Several papers have been identified which compare different design methods. These show that different design methods yield wide ranging results.

A detailed comparison is given for the first six design methods described in Section 4.2.3 (a, b, c, d, e, f) of this report. These methods are used worldwide. Nearly all GRS structures constructed in North America were designed by these methods (Claybourn & Wu 1993).

While no detailed design comparisons have been undertaken for other methods, it is, however, shown in Section 4.3.3 below that they yield varying results similar to those for the first six methods discussed.

4.3.1 Detailed Comparison

All the six design methods discussed in this Section 4.3 use the allowable reinforcement tensile strength as a design input. Most of the methods describe allowable tensile strength as a factored ultimate tensile strength determined by the wide-width tensile test (ASTM D4595).

Reduction factors for construction damage, time-dependent deterioration and creep are used. For some of the methods the reduction factors are a function of reinforcement polymer type and manufacturing process. Depending on the method used, the reinforcement design strength may range from 10% to about 50% of the ultimate tensile strength.

Only drained granular backfill soils are considered, and therefore their strength is described by means of the effective angle of internal friction ϕ' .

The interaction between the soil and geosynthetic reinforcement is described by a coefficient of friction $\tan \delta$ between the reinforcement and the soil. Some methods use frictional efficiency, defined as $\tan \delta / \tan \phi'$, where δ is the soil/reinforcement friction angle, and ϕ' is the internal friction angle of the soil.

The methods discussed use two different approaches to define factors of safety. The first is the overall factor of safety approach, where the factor of safety is defined as the ratio of resisting forces to disturbing forces or moments. The second is the partial factor of safety approach, where partial factors of safety are used to reduce the strength parameters and to factor the loads.

Three different reinforcement materials were selected (Table 4.1). Design parameters and factors of safety used by each of the six design methods are given in Table 4.2.

The information on the design comparison has been extracted from a paper published by Claybourn & Wu (1993). Designs were conducted for GRS wall heights of 3.6 m and 9.1 m to compare the results for low and high walls.

Figures 4.11 and 4.12 present design comparisons for a 3.6 m high wall using Reinforcement 1 and for a 9.1 m high wall using Reinforcement 3 respectively. The figures clearly illustrate the differences in the results obtained by the six design methods. The Leshchinsky method uses low allowable tensile strength and therefore results in a substantially more extensive reinforcement than the other methods.

Table 4.1 Properties of three reinforcement materials (after Claybourn & Wu 1993).

Reinforcement Type No.	Description	Tensile Strength	Assumed Frictional Efficiency
1	Non-woven polypropylene (PP) geotextile	Wide width tensile strength of 26.3 kN/m	90% unless otherwise specified by a design method
2	Woven polypropylene (PP) geotextile	Wide width tensile strength of 52.6 kN/m	80% unless otherwise specified by a design method
3	High-density polyethylene (HDPE) Tensar geogrid	Tensile strength of 43.8 kN/m (recommended by Tensar company)	90% unless otherwise specified by a design method

The differences are similar for the two assumed wall heights. Tables 4.3 and 4.4 present reinforcement quantities in square metres per metre length of GRS wall for each reinforcement type and design case. The tables show that, consistent with Figures 4.11 and 4.12, there are significant differences in reinforcement requirements.

Table 4.3 Summary of reinforcement quantity comparisons for a 3.6 m high wall (after Claybourn & Wu 1993).

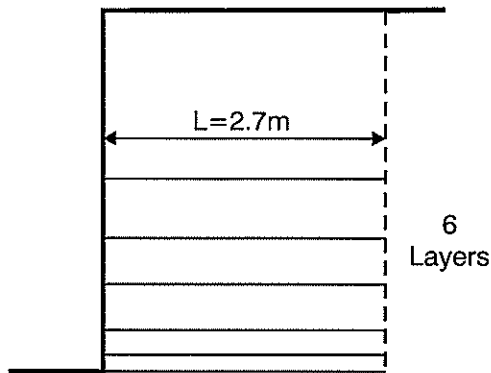
Design Method	Reinforcement 1 ($T_{ult} - 26.3 \text{ kN/m}$)*	Reinforcement 2 ($T_{ult} - 52.6 \text{ kN/m}$)	Reinforcement 3 ($T_{ult} - 109.5 \text{ kN/m}$)
	Calculated quantity (m^2/m)**	Calculated quantity (m^2/m)	Calculated quantity (m^2/m)
Forest Service	16	16	5
Broms	22	10	6
Collin	9	3	1
Bonaparte	15	8	3
Leshchinsky	130	67	34
Schmertmann	15	8	2

* T_{ult} = ultimate reinforcement tensile resistance

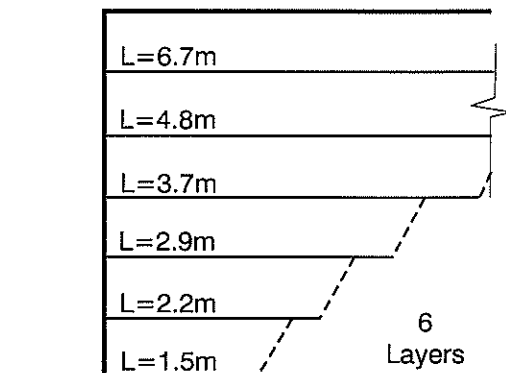
** m^2/m = square metres of reinforcement required per metre length of wall

Table 4.2 Summary of design parameters (after Claybourn & Wu 1993)

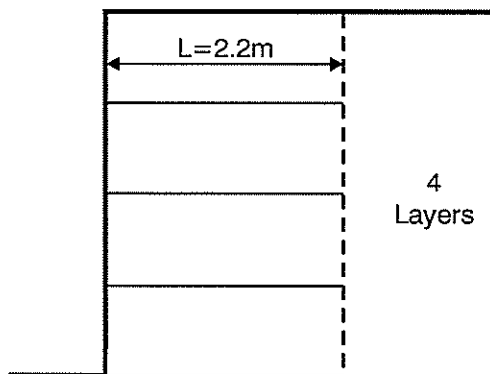
Design Method	Allowable Reinforcement Strength	Soil/Reinforcement Interface Friction	Factor of Safety
Forest Service	Reinf. 1, 55% ultimate Reinf. 2, 55% ultimate Reinf. 3, 1/3 ultimate	$\delta = (2/3)\phi'$	Rupture - 1.2 (method suggests 1.2-1.5) Sliding - 1.5 Pullout - methods recommendations not clear Use either 1.2-1.5 or 1.5-1.75 (For cases evaluated, all are above 2.0 by using FS = 1.5 for sliding)
Broms	1/3 ultimate strength for all reinforcement types.	Frictional efficiency of 90% for non-woven geotextile and geogrid and 80% for woven geotextile	1.3 for embedment length beyond failure surface of underlying layer.
Collin	1/3 ultimate strength for all reinforcement types.	$\delta = (2/3)\phi'$	Rupture - 1.0 (allowable strength is working value) Pullout - 1.5 Sliding - 1.5
Bonaparte	1/3 ultimate strength for all reinforcement types (20%-40% suggested)	Frictional efficiency of 90% for non-woven geotextile and geogrid and 80% for woven geotextile.	Method recommends safety factor be applied to soil strength and that allowable value be used for reinforcement strength. FS = 1.5 (Used to factor soil strength and to evaluate sliding)
Leshchinsky	Reinf. 1, 12.5% ultimate Reinf. 2, 12.5% ultimate Reinf. 3, 12.5% ultimate	$\delta = (2/3)\phi'$	Composite structure - 1.5 Reinforcement tensile resistance - 2.0
Schmertmann	Reinf. 1, 1/3 ultimate Reinf. 2, 1/3 ultimate Reinf. 3, 43.8 kN/m (recommended by Tensar)	Method has frictional efficiency of 90% built in (assumed appropriate for Reinf.'s 1 and 3). 80% efficiency used for Reinf. 2	1.5 (applied to soil strength)



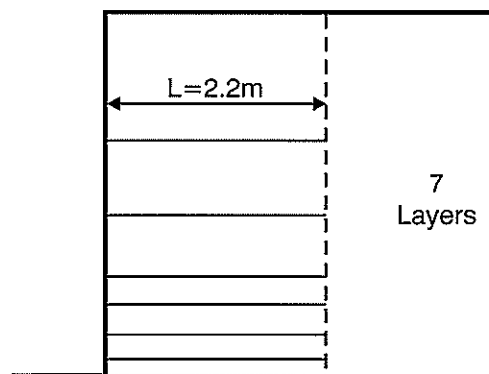
Forest Service Method



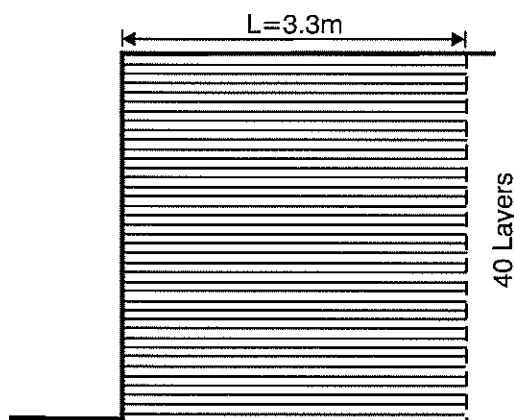
Broms Method



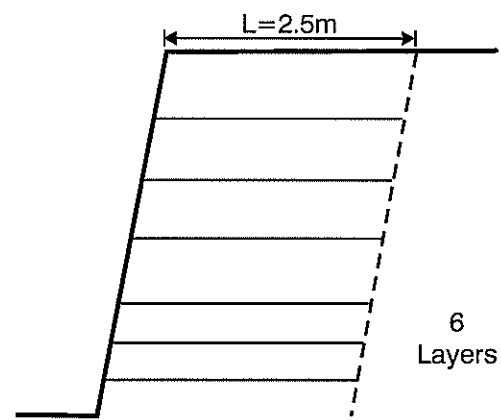
Collin Method



Bonaparte Method



Leshchinsky Method



Schmertmann Method

Figure 4.11 Design comparisons: 3.6m high wall, Reinforcement I (after Claybourn & Wu 1993)

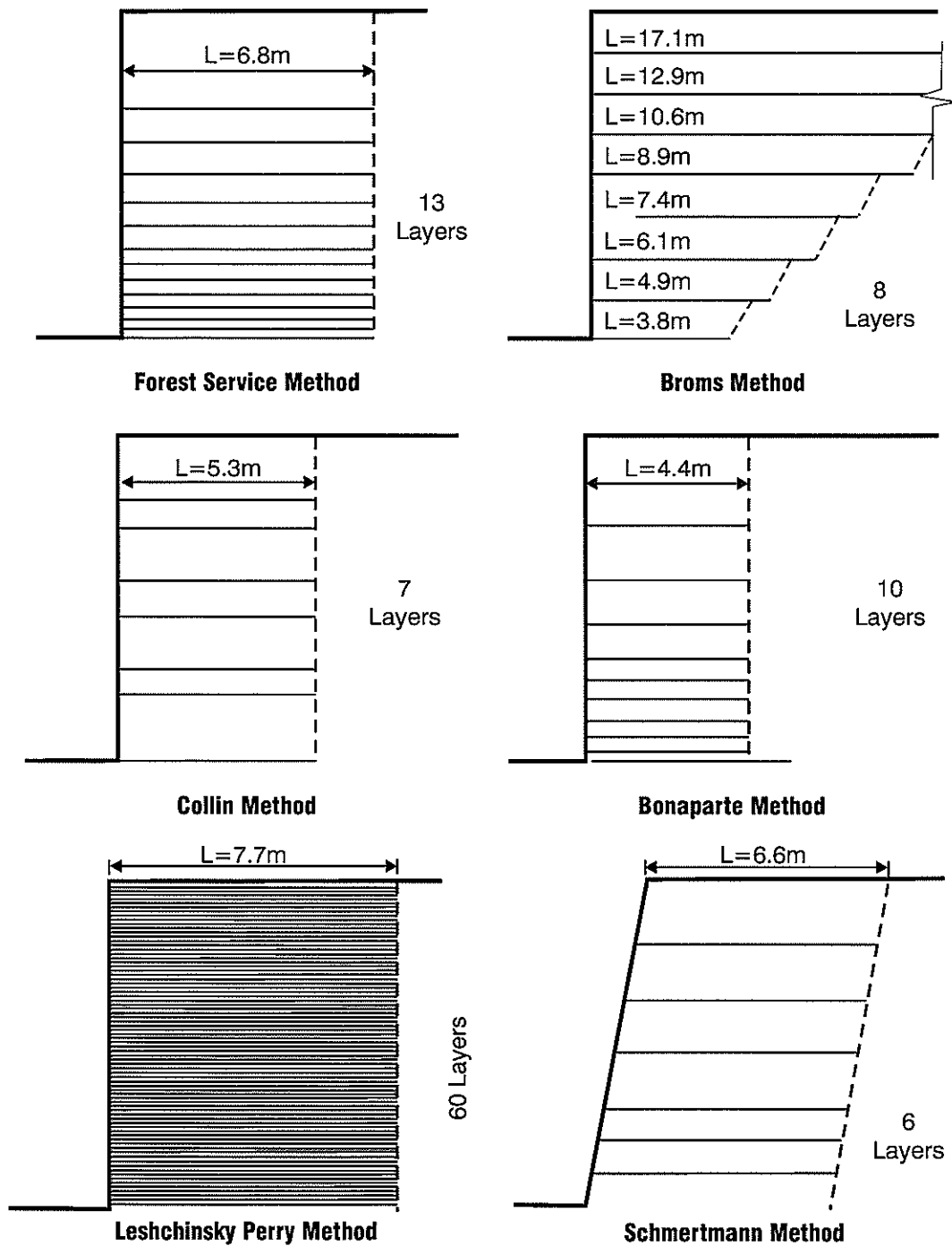


Figure 4.12 Design comparisons: 9.1m high wall, Reinforcement 3 (after Claybourn & Wu 1993)

Table 4.4 Summary of reinforcement quantity comparisons for a 9.1 m high wall (after Claybourn & Wu 1993).

Design Method	Reinforcement 1 ($T_{ult} - 26.3 \text{ kN/m}$)*	Reinforcement 2 ($T_{ult} - 52.6 \text{ kN/m}$)	Reinforcement 3 ($T_{ult} - 109.5 \text{ kN/m}$)
	Calculated quantity (m^2/m)**	Calculated quantity (m^2/m)	Calculated quantity (m^2/m)
Forest Service	250	250	88
Broms	300	161	72
Collin	150	77	38
Bonaparte	170	95	44
Leshchinsky	1900	957	460
Schmertmann	220	116	44

* T_{ult} = ultimate reinforcement tensile resistance

** m^2/m = square metres of reinforcement required per metre length of wall

Table 4.5 presents so-called safety ratios. The safety ratio is the ratio of reinforcement quantity determined using all of the prescribed safety factors to the reinforcement quantity determined by following the methods with all safety factors set equal to 1.0 and using ultimate strength parameters. (Note that only safety ratios for the case of 9.1 m high wall, Reinforcement 2 are given.)

Table 4.5 Safety Ratios, the design reinforcement quantity versus quantity for FS = 1.0 for a vertical 9.1 m high wall reinforced with Reinforcement 2 (after Claybourn & Wu 1993).

Design Method	Design Quantity (m^2/m)	Quantity determined for FS = 1.0 (m^2/m)	Safety Ratio
Forest Service	250	25	10.0
Broms	161	39	4.1
Collin	77	18	4.3
Bonaparte	95	30	3.2
Leshchinsky	957	41	23.3
Schmertmann	116	16	7.3

The table shows that the largest design quantity is approximately 12.4 times the smallest design quantity. However, if all factors of safety are assumed to be equal to 1.0, then the ratio of largest to smallest quantity is approximately 2.4.

It is therefore obvious that the most substantial differences among the design methods appear to result from discrepancies in recommendations on defining allowable strength and safety factors for the considered design methods.

4.3.2 Comparisons with Test Data

Claybourn & Wu also compared the results of tests conducted on two GRS test walls that were loaded to failure, with the results predicted by each of the six design methods, without using any of the prescribed factors of safety. The details of the two test walls (Wall A is a 6.1 m-high wrap-around geotextile wall and Wall B is a 3 m-high geogrid-reinforced wall with timber facing panels) are shown on Figure 4.13.

The effective reinforcement lengths for both walls are also shown on Figure 4.13. Claybourn & Wu define the effective reinforcement length as the length beyond which the performance of the wall is presumed to be unaffected by further lengthening of the reinforcement for the loading conditions at failure, i.e. no stresses are mobilised along the portion of reinforcement beyond the effective length.

Wall A (Figure 4.13a) was constructed up to a height at which failure occurred. Wall B (Figure 4.13b) was surcharged in increments to a maximum 50 kPa at which point failure occurred. The mode of failure for both walls was creep with large displacements but no brittle failure.

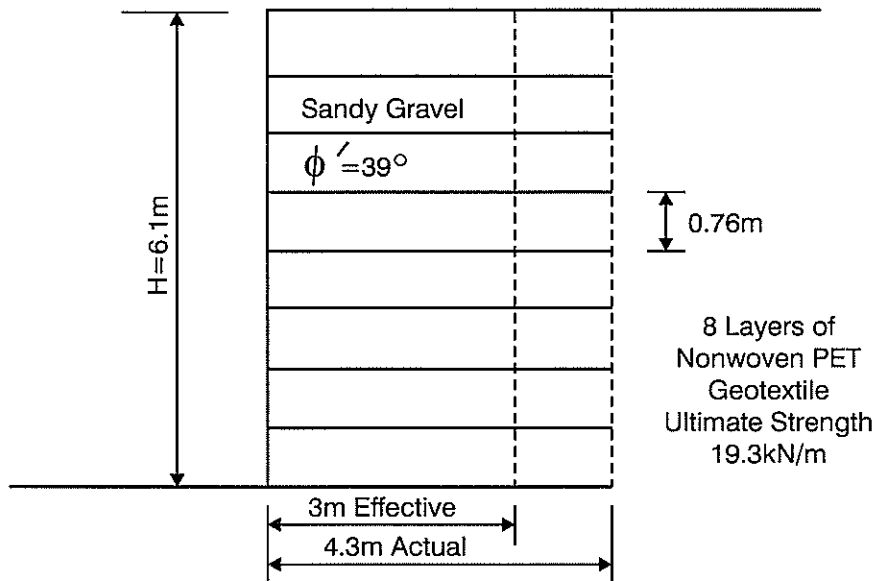
To predict the behaviour of the test walls on the basis of the design methods, all factors of safety in the calculations were assumed to be equal to 1.0.

A reinforcement strength equal to the ultimate strength was used to predict the behaviour of Wall A. An effective strength (a stress in reinforcement at which failure occurred) was used in the calculations for Wall B.

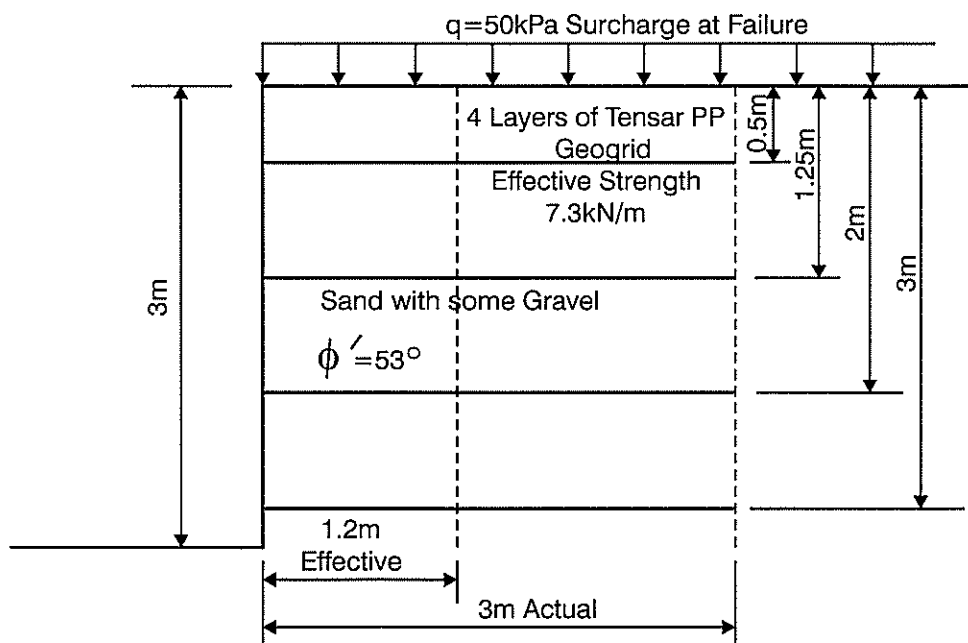
The predicted tensile forces in the reinforcements are compared with the ultimate tensile strength for Wall A and with the measured tensile stresses and the effective strength for Wall B on Figures 4.14 and 4.15 respectively.

Each of the six methods was also used to determine reinforcement lengths and spacings for the test wall heights and reinforcement strengths as shown on Figure 4.13 using factors of safety of 1.0.

Comparison of Figure 4.16 with Figure 4.13a shows that all of the six methods result in either the same number (8) or fewer layers of reinforcement, than used for test Wall A. Comparison of Figure 4.17 with Figure 4.13b indicates that the design methods result in either the same (4) or more layers of reinforcement than used for test Wall B.



(a) Wall A



(b) Wall B

Figure 4.13 Geometry and details of two test walls (after Claybourn & Wu 1993)

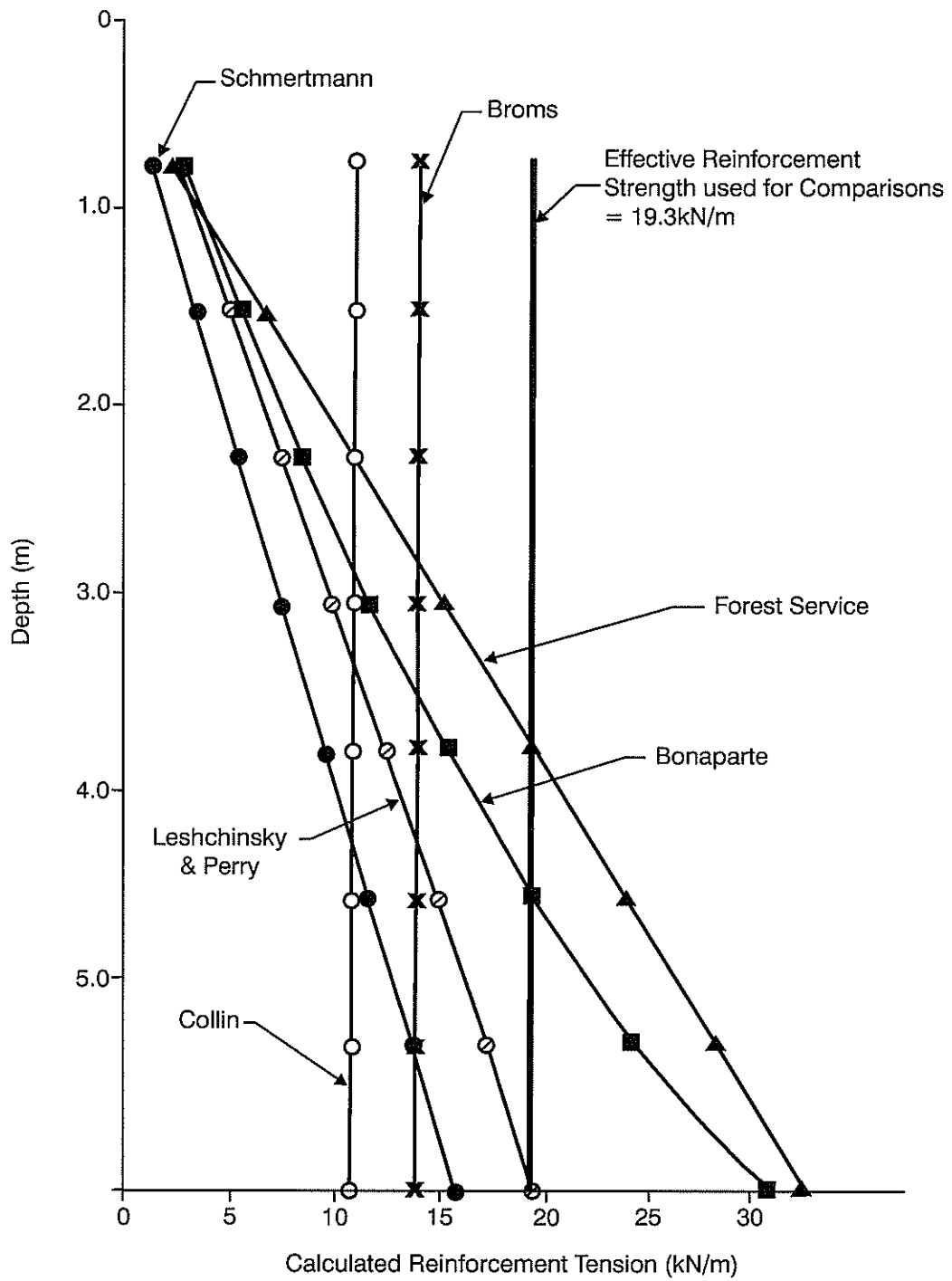


Figure 4.14 Reinforced tension calculated for Wall A (after Claybourn & Wu 1993)

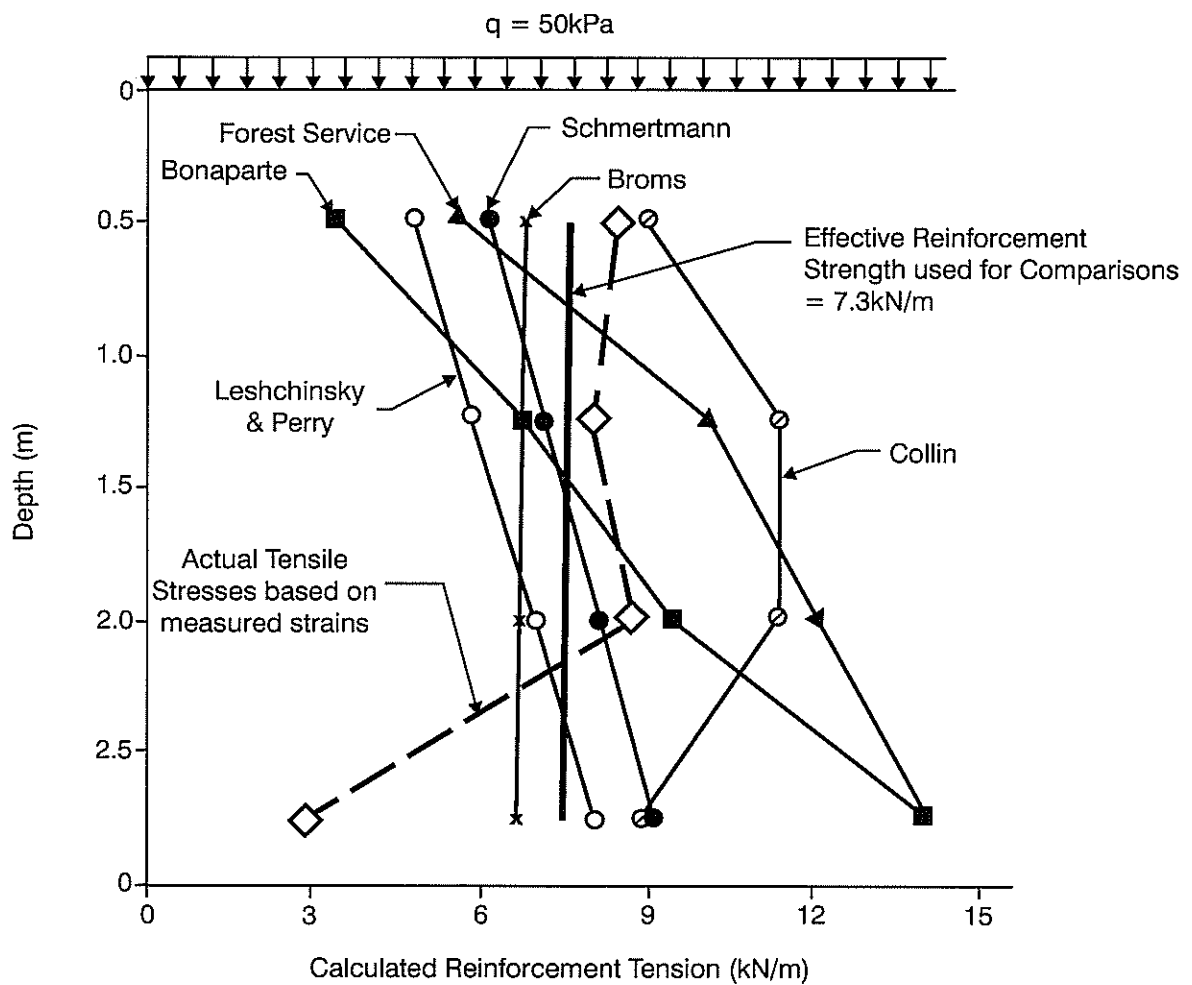
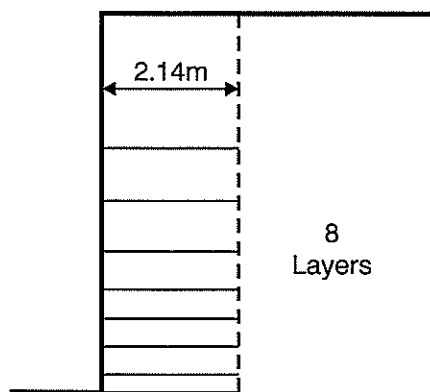
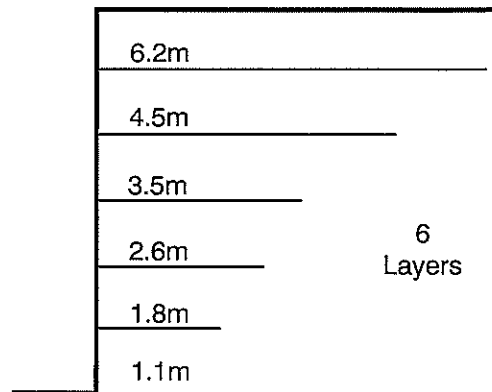


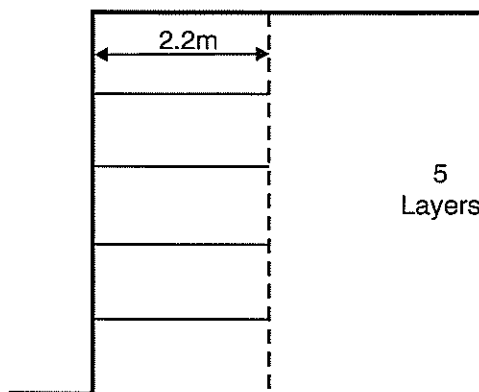
Figure 4.15 Reinforced tension calculated for Wall B (after Claybourn & Wu 1993)



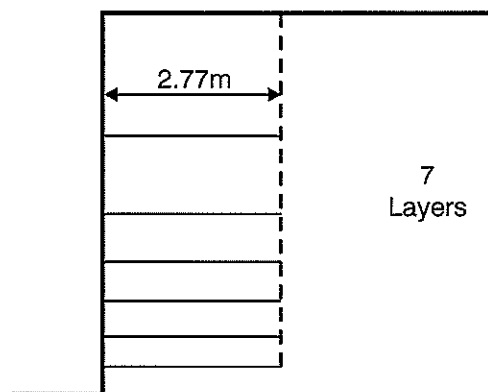
Forest Service Method



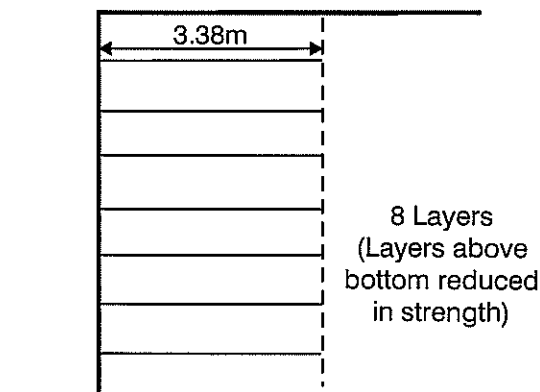
Broms Method



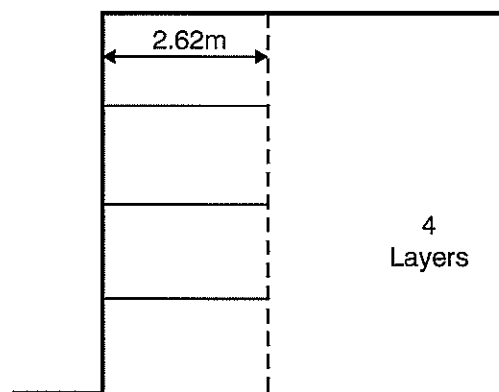
Collin Method



Bonaparte Method



Leshchinsky Method



Schmertmann Method

Figure 4.16 Comparison of methods for FS = 1.0, Wall A (after Claybourn & Wu 1993)

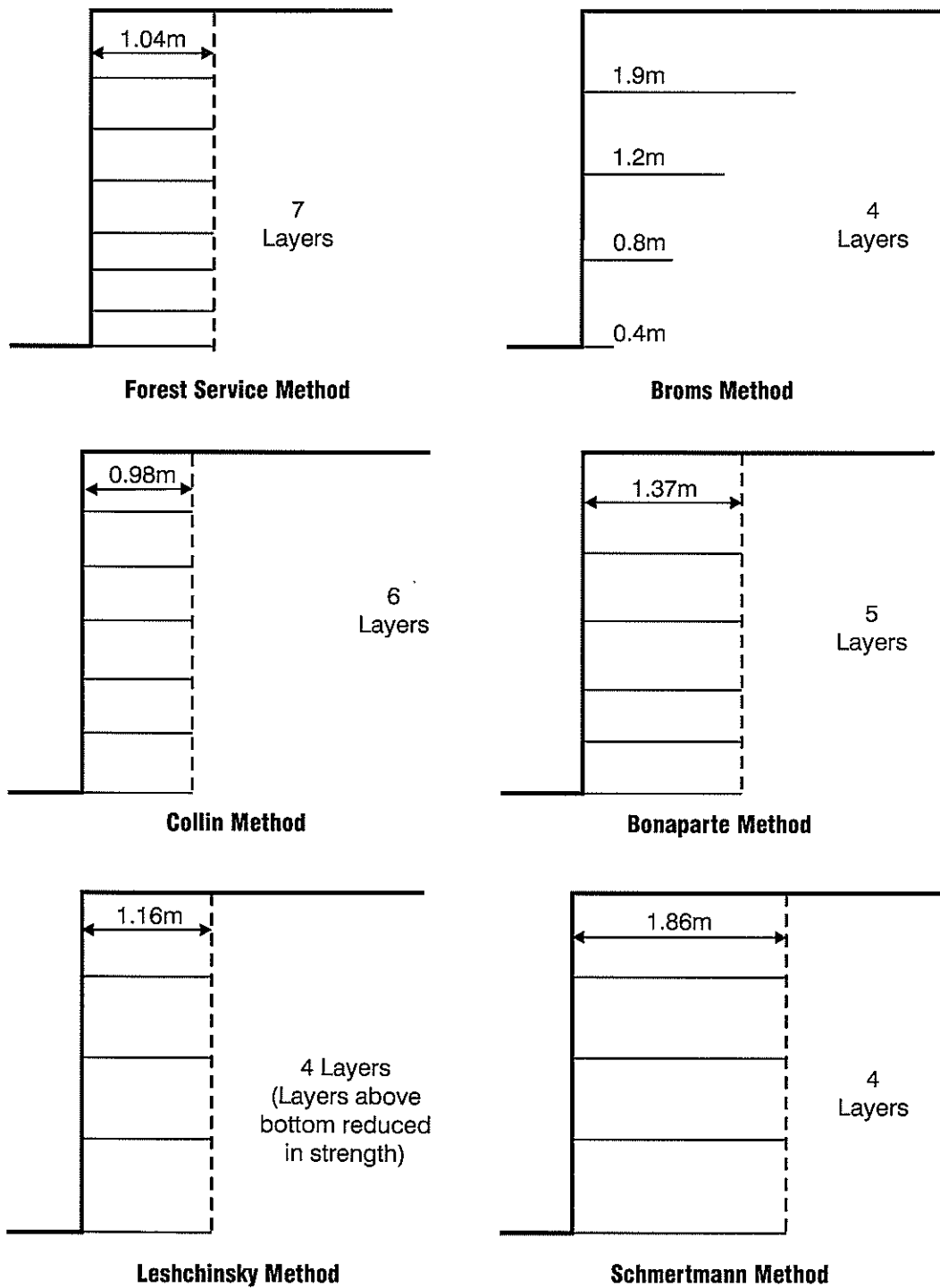


Figure 4.17 Comparison of methods for $FS = 1.0$, Wall B
(after Claybourn & Wu 1993)

The comparison indicates that the tensile forces in reinforcement predicted by different design methods vary and can be higher or lower than the ultimate or effective tensile strength. The comparison also indicates that some of the design methods may underestimate the number of layers of reinforcement required for relatively high walls (6 m and higher).

4.3.3 Comments on Other Methods

The layout of reinforcements obtained by methods other than those discussed in Section 4.3.1 of this report for a geogrid-reinforced wall with vertical front face is given in Table 4.6 assuming the following design parameters (Paul 1996):

Wall height	$H = 5 \text{ m}$
Peak angle of soil-shearing resistance	$\phi'_p = 37^\circ$
Angle of constant volume soil shearing resistance	$\phi'_{cv} = 30^\circ$
Unit weight of soil	$\gamma = 20 \text{ kN/m}^3$
Uniform surcharge on top of the wall	$q = 10 \text{ kN/m}_2$

Some discrepancies among the amount of reinforcement and the reinforcement lengths required by different design methods are evident from Table 4.6. These discrepancies can be explained by different design assumptions adopted by different design methods. Some of the design assumptions are also compared in Table 4.6.

The methods discussed in this Section 4.3 do not give detailed recommendations on whether the peak value (ϕ'_p) or the constant volume value (ϕ'_{cv}) of the soil friction angle should be used in the GRS design. Therefore, the peak value is normally adopted for the design. However, some of the methods compared in Table 4.6 consider this issue in detail. The logic behind this consideration is that, in soil subject to a compressive loading, mobilised shear strength increases with increasing axial and lateral strain until the peak shear strength (ϕ'_p) is mobilised. After the peak value is achieved the shear strength continues to be mobilised at the peak level (for so-called elasto-plastic soils) or decreases, and at large strains achieves a constant minimum value (ϕ'_{cv}) independent of strain magnitude (for so-called strain-softening soils) as strains exceed those necessary to mobilise peak strength.

The strains required to mobilise ϕ'_p are normally assumed to be small (less than 1.3% according to BS8006). On the other hand, available information on strength properties of geosynthetics suggests that the complete mobilisation of their tensile resistance requires large deformations and therefore cannot be mobilised simultaneously with the maximum shear strength of soil.

Given the empirical nature of the existing design methods and a number of uncertainties with respect to design models and other design parameters, the suggestion to use ϕ'_{cv} seems to be an attempt to introduce an additional factor of safety into the design. This factor of safety, however, has already been included in many methods as a partial factor of safety for the soil strength and, if the factored value of the peak soil strength does not exceed the constant volume strength (which is very often the case), then a safe design can be obtained.

Table 4.6 Design comparison (after Paul 1996)

Design Method	Design comparison for a vertical 5 m high geogrid reinforced wall ($\phi'_p = 37^\circ$, $\phi'_{ev} = 30^\circ$, $\gamma = 20 \text{ kN/m}^3$) loaded with uniform surcharge of 10 kN/m^2		Design Assumptions					
	Required number of layers of reinforcement		Length of reinforcement (m)	Recommended soil strength	Friction on back of wall	Soil Pressures	Foundation Bearing Pressures	Minimum Length of Reinforcement
	Tensar SR80	Tensar SR55						
Bautechnik Method (Germany)	6	3	3.0	ϕ'_{ev}	Yes	Coulomb	Meyerhof	-
BE 3/78 Method (UK)	7	3	4.0	ϕ'_p	No	Rankine	Trapezoidal	0.8H or 5 m
BS 8006 Method (UK)	5	4	3.5	ϕ'_p	No	Rankine	Meyerhof	0.7H or 3 m
Coherent Gravity Method (France)	4	4	3.5	ϕ'_p	No	Rankine	Meyerhof	0.7H
Displacement Method (France)	-	-	-	ϕ'_{ev}	-	-	-	-

5. SEISMIC DESIGN OF GRS STRUCTURES

5.1 Design Philosophy

A number of different methods for seismic design of GRS structures are currently available. The various methods yield widely varying results and the philosophies of the methods are substantially different.

Only a few GRS structures have been instrumented for seismic performance to date. Some factual information on the behaviour of GRS structures during earthquakes has been collected (refer Section 3.2 of this report). For most GRS case studies, detailed information on soil and reinforcement properties and on earthquake characteristics is not available.

Most of the GRS structures described in Section 3.2 have performed satisfactorily during strong earthquakes and no catastrophic failures of GRS structures have been reported. The observed satisfactory seismic performance is attributed to the conservatism of the static design procedures and large static factors of safety employed (Ling et al. 1997). However, the conservatism of the static design procedures has decreased significantly in recent years and therefore designers cannot rely on it to guarantee adequate seismic resistance of GRS structures.

The existing design methods described below are based on some theoretical assumptions rather than on good understanding of the dynamic behaviour of GRS. More research is required worldwide to improve the predictions of the GRS behaviour under seismic loads.

Currently there are two different groups of design methods. The first group is based on a pseudo-static approach. These methods assume that the failure occurs along a certain slip surface. The internal stability, i.e. the ability of the reinforced soil mass to behave as a coherent mass and be self-supporting under the action of its own weight, reinforcement tension force, surcharges and inertia forces (caused by the seismic excitation of the reinforced fill), is checked. The reinforcement is selected and placed to prevent tension rupture and pullout from the soil mass beyond the assumed failure plane. These methods address the seismic stability of GRS structures but do not address their deformation or displacement.

The second group of methods is based on the consideration of permanent displacement of GRS structures. These methods allow GRS structures to be designed for a critical acceleration less than the anticipated maximum seismic acceleration. The permanent displacement is assessed on the basis of the Newmark type analysis.

Several methods commonly used in the current design practice are described below.

5.2 Design Methods Based on Pseudo-static Approach

5.2.1 Public Works Research Institute (PWRI) Method (Japan)

The method was developed as a result of an extensive research undertaken by PWRI and Japanese consulting companies (PWRI 1992, Koga & Washida 1992).

The method assumes that tensile forces in geosynthetic reinforcement under static conditions increase linearly with depth (Figure 5.1). It is also assumed that tensile forces in reinforcement caused by seismic loads are distributed uniformly.

The maximum total tensile force in the reinforcement is then obtained by the equilibrium analysis using circular slip surfaces, where a pseudo-static force equal to the seismic coefficient (k_c) times the weight (w) is added at the centroid of each slice. The factor of safety of 1.2 is required for static conditions, and the factor of safety of 1.0 is considered sufficient for seismic conditions.

The tensile forces in each reinforcement layer are then calculated (using the assumed tensile force distribution), and geosynthetic type, length and spacing are determined. Finally, the overall external stability under seismic conditions is checked assuming that GRS block behaves as a pseudo-retaining wall. Factors of safety of 1.2 and 2.0 are required for sliding and bearing capacity respectively. The eccentricity of the resulting force at the base of the structure also must not exceed $B/3$, where B is the width of the structure. GRS structures designed by this method have performed satisfactorily. Nishimura et al. (1996) suggest that the method is conservative.

5.2.2 Geogrid Research Board (GRB) Method (Japan)

The Japanese Geogrid Research Board (GRB) developed and published design and construction manual for GRS structures (Nishimura et al. 1996). The shape of the assumed slip surfaces in the GRB method is different from that in the PWRI method (Figure 5.2).

A two part wedge failure mechanism is considered to obtain the maximum tensile force in reinforcement (P_{h2}). The required reinforcement length is then assessed from the reinforcement pullout analysis. The external stability analysis is done in a manner similar to that for the PWRI method. The final reinforcement length is the maximum of those obtained from the two part wedge stability analysis and the external stability analysis (including sliding and overturning failure modes).

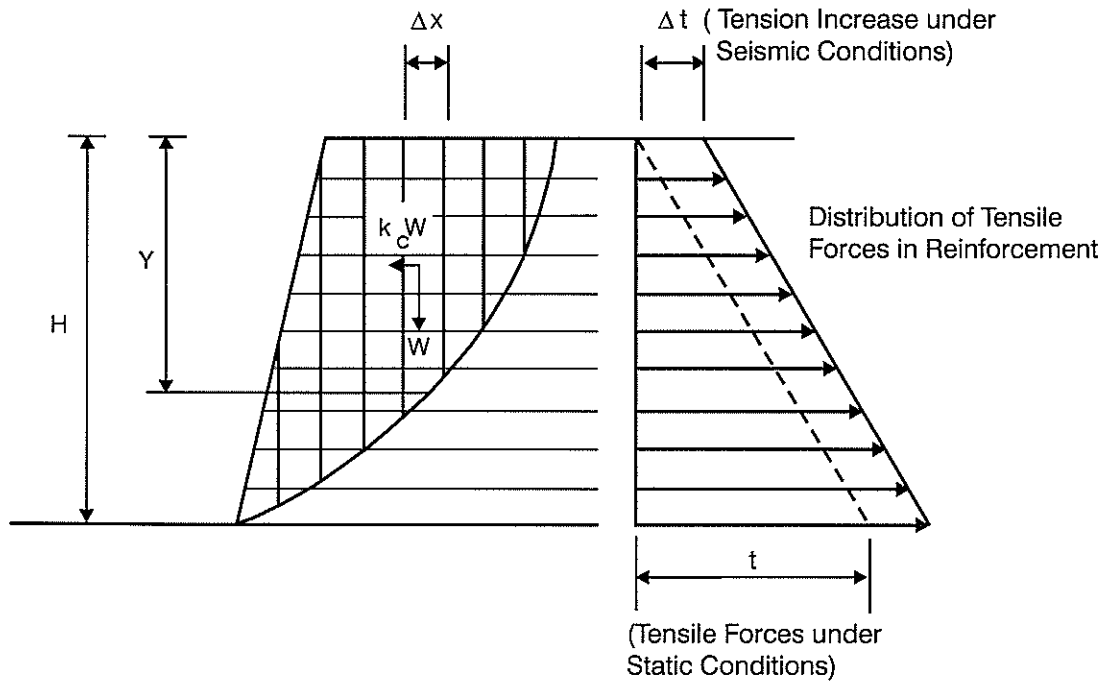


Figure 5.1 PWRI method

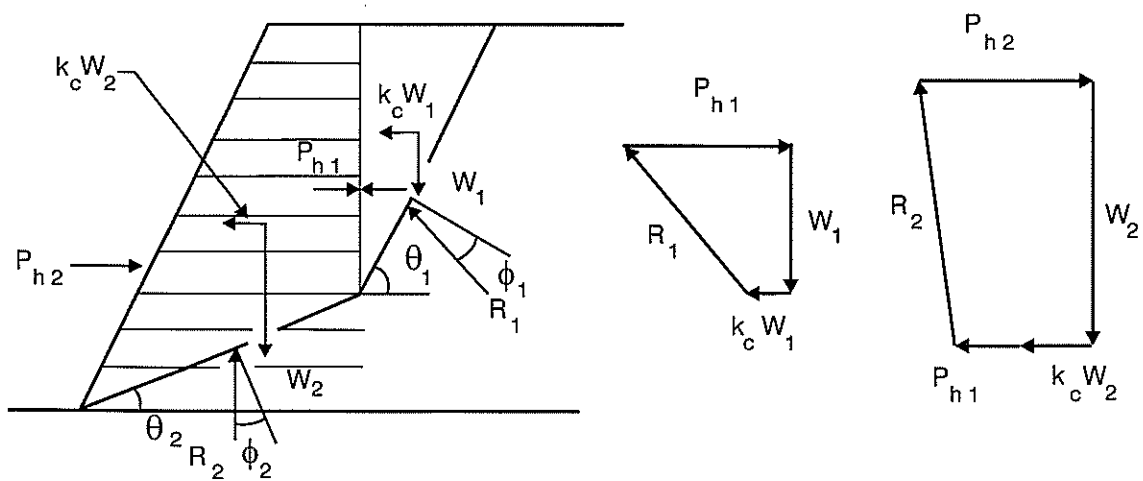


Figure 5.2 GRB method

5.2.3 Bonaparte Method (USA)

Bonaparte et al. (1986) used a simple pseudo-static, rigid-body analytical model to develop design charts for the seismic design of GRS slopes. Bonaparte et al. recommends the use of the constant volume shear strength (ϕ'_{cv}) rather than the peak strength (ϕ'_p).

The total required reinforcement force (T) is determined through a limit equilibrium analysis assuming a Coulomb failure wedge (Figure 5.3a). It is known that the use of this simple wedge analysis usually results in under-estimation of the total reinforcement tensile force (T), compared to the force obtained from analyses using more complex failure mechanisms (such as circle, two part wedge or logarithmic spiral as described in Sections 5.2.1, 5.2.2 and 5.2.5).

However, when ratios $R_T = T_e / T_s$ (where T_e is the total reinforcement tensile force to maintain equilibrium for the seismic loading case, and T_s is the total reinforcement tensile force to maintain equilibrium for the static loading case) for different design approaches are compared, the error introduced into the ratio R_T through use of the Coulomb wedge analysis is normally found to be small. Therefore, the charts developed by Bonaparte et al. give the ratio R_T (for different seismic coefficients and slope angles) rather than the total force T_e .

The increase in the required length of reinforcement due to the horizontal inertia force is evaluated based on two criteria:

- (1) reinforcement pullout behind the critical wedge (Figure 5.3b)
- (2) sliding of reinforcement soil mass over a layer of reinforcement at the elevation of the toe of the slope (Figure 5.3c).

The soil–reinforcement interaction coefficient is assumed to be 0.8. Bonaparte et al. developed charts showing the ratio $R_L = L_e / L_s$ for different seismic accelerations (where L_e is the reinforcement length to maintain equilibrium for the seismic loading case, L_s is the reinforcement length to maintain equilibrium for the static loading case). The reinforcement length L_e was evaluated using both criteria illustrated on Figure 5.3b and c.

As in the PWRI method, the dynamic force increment ($T_e - T_s$) is assumed to act at the midpoint of the slope and, as a result, to be uniformly distributed through the height of the slope.

The Bonaparte method suggests the use of 85% of the peak ground acceleration, because the horizontal inertia force associated with the peak ground acceleration is applied for only a very short period of time. It also suggests using 90% of the allowable reinforcement tensile strength and to apply factors of safety 1.1 to 1.5 to the constant volume soil shear strength.

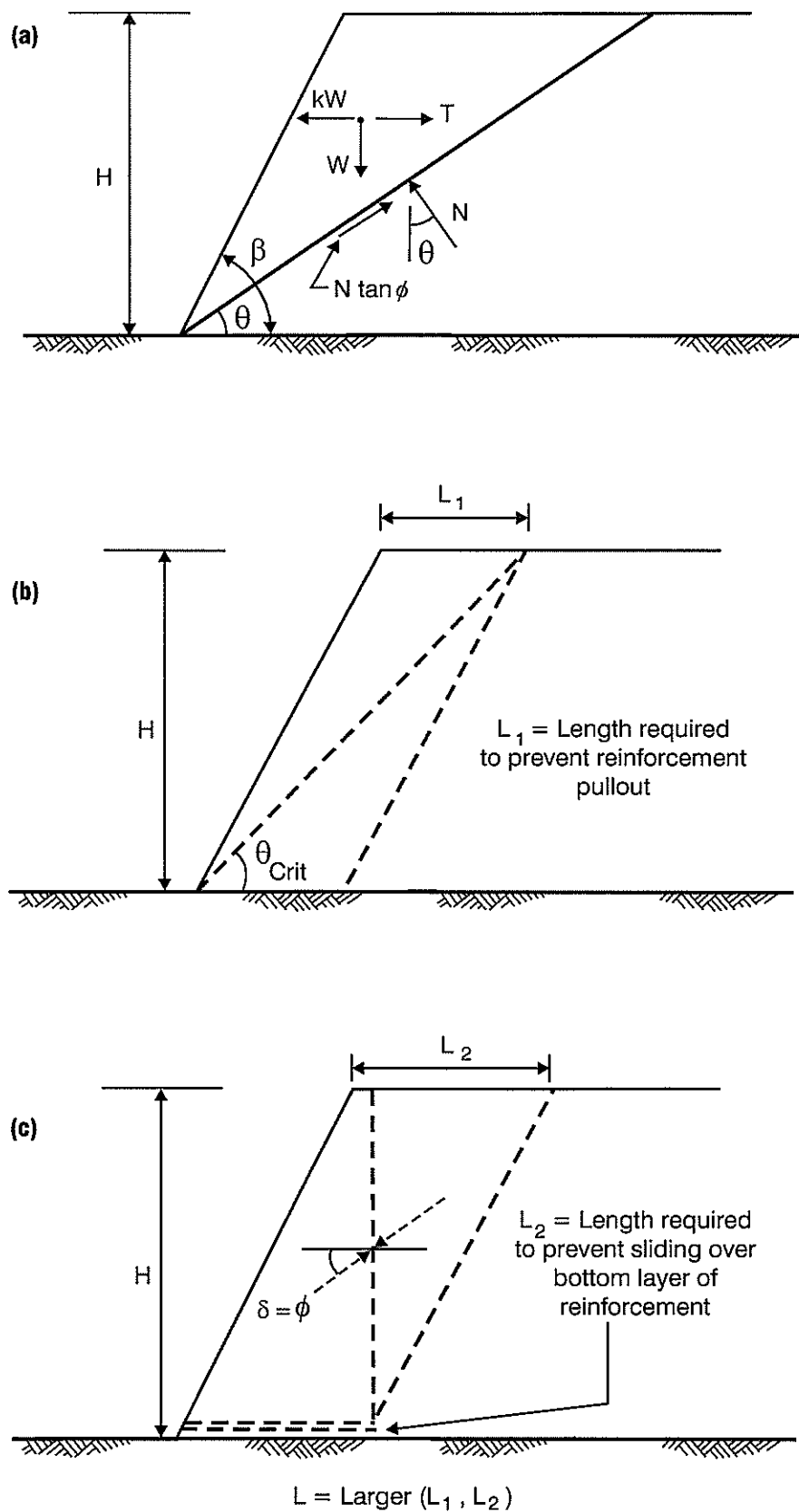


Figure 5.3 Bonaparte method: (a) Failure wedge; (b) Length required to prevent reinforcement pullout; (c) Length required to prevent sliding

5.2.4 AASHTO Method (USA)

The AASHTO method was developed by Christopher et al. (1990b). The method is based on a modified pseudo-static design approach.

A dynamic horizontal thrust (P_{AE}) exerted by the retained fill is evaluated by the pseudo-static Monanabe-Okabe analysis as shown on Figure 5.4a:

$$P_{AE} = 0.375 A_m \gamma_f H_2 \quad (\text{for horizontal back slope}) \quad (35)$$

The horizontal inertia force acting in the reinforced soil mass (P_{IR}) is calculated assuming that only a portion of the total reinforced block with a base width of $0.5H$ is affected:

$$P_{IR} = 0.5 A_m \gamma_r H^2 \quad (36)$$

In both formulas A_m is the maximum acceleration coefficient at the centroid of the reinforced mass, given by:

$$A_m = (1.45 - A) A \quad (37)$$

where A is the maximum ground acceleration coefficient.

In the external stability analysis the equilibrium of the reinforced block under static forces (lateral thrust related to static soil pressures and surcharges), 50% of the seismic thrust (P_{AE}), and the full internal force (P_{IR}), is checked. The reduced P_{AE} is used because P_{AE} and P_{IR} are unlikely to peak simultaneously.

The method requires factors of safety for the external stability analysis to be equal to or greater than 75% of the minimum static factors of safety (described in Section 4.2.3(a)). It is also required that the eccentricity falls within $B/3$ where B is the width of a GRS structure.

The internal stability is checked with respect to the elongation (or breakage) of the reinforcement and the pullout failure. The design for internal stability, therefore, consists of determining the maximum developed tension forces, their location along a critical slip surface, and the resistance provided by the reinforcement both in pullout and tensile strength. The assumed critical slip surface is shown on Figure 5.4b. The inertia force affecting the active wedge is:

$$P_I = A_m W_A \quad (38)$$

where W_A is the weight of the active wedge.

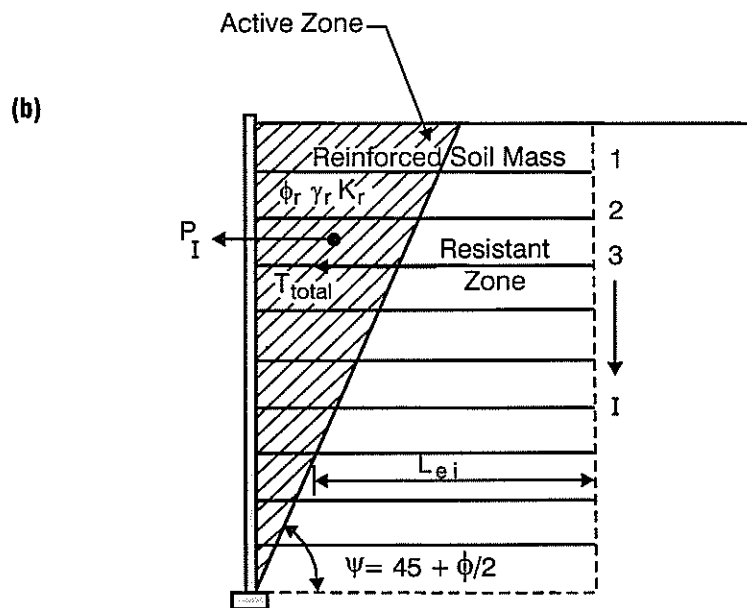
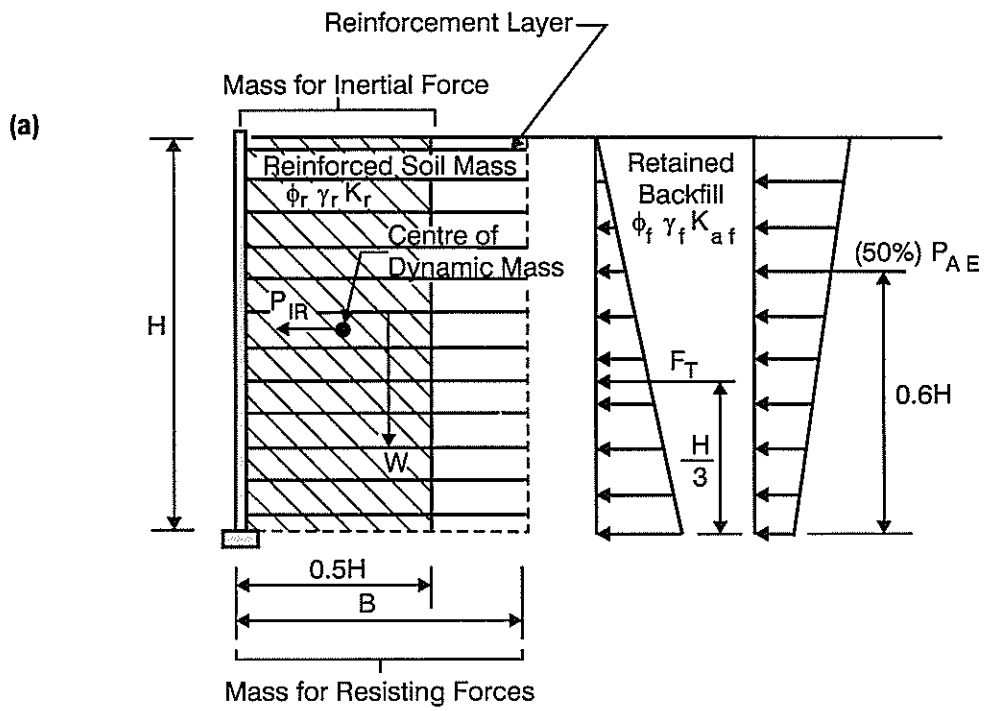


Figure 5.4 AASHTO method: (a) Seismic external stability;
(b) Seismic internal stability

P_1 is distributed in the reinforcement layers in proportion to the resistant length (L_{ei}) of each reinforcement layer:

$$T_{mdi} = P_1 \frac{L_{ei}}{\sum L_{ei}} \quad (39)$$

The total tensile force in the layers of geosynthetic reinforcement is calculated as a sum of the maximum static tension T_{max} (Section 4.2.3(a)) and the dynamic increment T_{md} :

$$T_{total} = T_{max} + T_{md} \quad (40)$$

To assess the pullout resistance, this method recommends to use 80% of the static soil/reinforcement interaction coefficient.

5.2.5 Ling Method (USA)

The detailed description of this method is given by Ling et al. (1997). The design procedure is an extension of the static design method proposed by Leshchinsky and described in Section 4.2.3(c) of this report. The seismic inertia force is considered to be pseudo-static and acting horizontally at the centre of gravity of the potential failure soil mass.

Ling proposed to use maximum earthquake acceleration $C_s g$ (where g is the gravitational acceleration, C_s is acceleration coefficient) to describe seismic loading, because amplification in GRS structures is still not well understood. The vertical component of seismic acceleration is not considered. The design internal friction angle (ϕ_d) of soil is obtained by reducing the available internal friction angle (ϕ_a) with an appropriate factor of safety:

$$\phi_d = \tan^{-1} \left(\frac{\tan \phi_a}{FS} \right) \quad (41)$$

In the internal stability analysis, the required strength for each layer, t_j , is calculated as shown on Figure 5.5a. The equilibrium of a potential failure soil mass, extending from layer n (or from the top of structure for the first step) to layer $n-1$ is considered and all maximum geosynthetic forces ($t_1, t_2, \dots, t_{n-1}, t_n$) required to stabilise the soil mass are determined.

The outermost log-spiral surface obtained from this analysis defines the active soil mass.

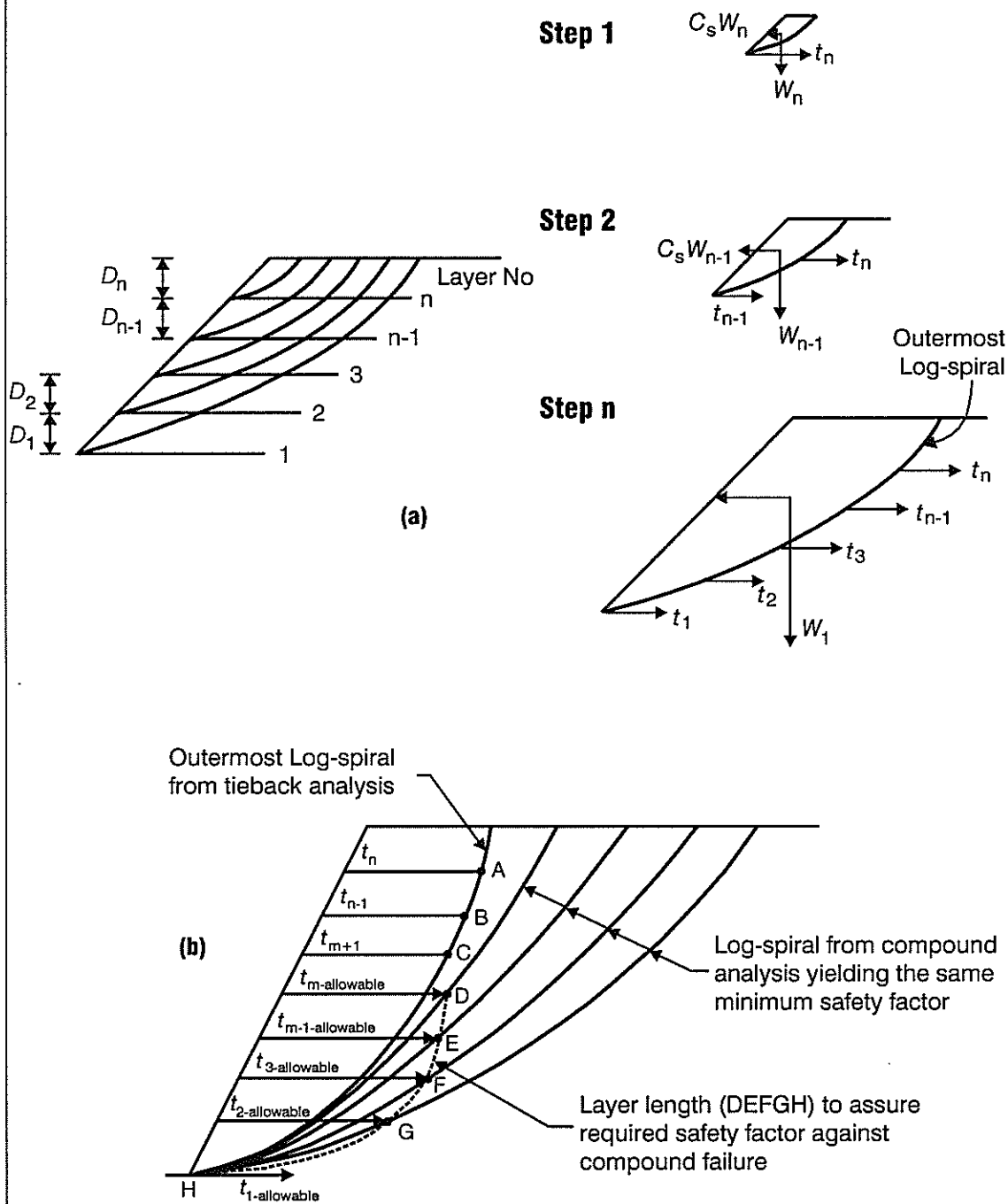


Figure 5.5 (a) Tieback failure analysis, and (b) Compound failure analysis (after Ling et al. 1997)

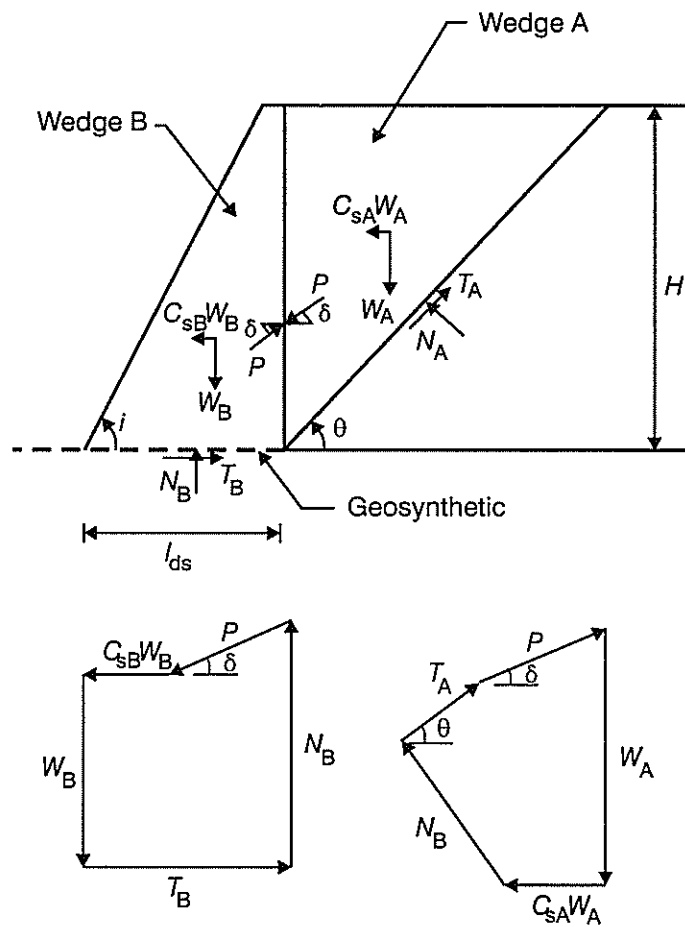


Figure 5.6 Direct sliding analysis (after Ling et al. 1997)

Compound stability analysis (considering potential slip surfaces emerging beyond the outermost log-spiral failure surface either outside or within the effective anchorage length) is then undertaken to determine the required anchored length of geosynthetic. If necessary, the minimal lengths calculated from the internal stability analysis are increased to achieve a required factor of safety (see layers l to m on Figure 5.5b).

The length required to resist direct sliding of a reinforced soil block, l_{ds} , is then determined on the basis of the two-part wedge mechanism (Figure 5.6). The horizontal seismic coefficients for wedges A and B can be different (C_{SA} and C_{SB} respectively), as recommended by Christopher et al. (1990).

A uniform design layout of the geosynthetic reinforcement is recommended, based on the longest of the lengths required to resist compound or direct sliding.

5.3 Design Method Based on Consideration of Permanent Displacement

Pseudo-static analysis of GRS structures under earthquake loading normally indicates that excessively long reinforcement (very often longer than $1.5H$, and in some cases longer than $3H$, where H is the height of the GRS structure) is required to provide stability against sliding. However, recent publications give information about satisfactory behaviour of GRS structures with substantially shorter reinforcements under seismic loads significantly higher than those the structures were designed to withstand (Section 3.2 of this report), so this design approach seems to be very conservative. Also installation of long geosynthetics is very often impossible because of space constraints or high cost of excavation works.

It would be, therefore, more reasonable to use a design approach which considers permanent displacement of a GRS structure under seismic loads higher than those providing ultimate equilibrium. This approach would allow a GRS structure to be designed for a critical acceleration (i.e. the acceleration level that initiates permanent movement of a GRS structure) less than the anticipated maximum seismic acceleration. In this design approach the permanent displacement is limited by serviceability requirements.

To the author's knowledge there are no GRS design methods based on permanent displacement which currently can be used in practice. However, the design approach based on permanent displacement has been used for reinforced concrete walls and retaining walls with steel reinforcement (Wood & Elms 1990).

The first attempt, known to the author, to develop a similar method for GRS structures has been recently undertaken by Ling et al. (1997). The proposed procedure is an extension of a Newmark type sliding block analysis. In direct sliding analysis the reinforced soil zone is treated as a rigid-plastic block in which displacement is induced only when the critical acceleration is exceeded.

The method also assumes that the critical acceleration in the reverse direction is large enough not to cause any permanent displacement. The critical acceleration against sliding is obtained by considering the equilibrium of wedge B (Figure 5.6).

The horizontal permanent displacement is obtained by double-integrating the equation of motion ignoring the damping and stiffness terms:

$$\ddot{x} = (C_s - C_{sy}) g \quad (42)$$

where x is horizontal displacement;

C_s is design acceleration;

C_{sy} is yield acceleration;

g is gravitational acceleration.

The double-integrating is conducted numerically for a random earthquake motion representative of site conditions. It is critical for the method that a representative strong ground motion and displacement chart is selected for the design.

Ling's comparison of predicted permanent displacements with factual monitored displacement (based on a representative ground motion chart) for two GRS structures indicated satisfactory agreement.

5.4 Comparison of Design Methods

5.4.1 Effect of Seismic Force on Required Geosynthetic Lengths

As described in Sections 5.2 and 5.3 above, different methods recommend different design procedures for seismic conditions and the design procedures differ in many ways. However, in this author's view, the main difference is the difference in the recommendations of each method on the magnitude of seismic acceleration that should be used in the design. According to such recommendations, the methods can be divided into three categories:

- (A) Methods which require the maximum magnitude of seismic acceleration to be used for the whole GRS structure including backfill.
- (B) Methods which recommend the use of reduced magnitudes of seismic accelerations but assume that permanent displacement of a GRS structure will not take place.
- (C) Methods which allow permanent displacement of GRS structures to be assessed. GRS structures, designed by these methods, can have critical seismic accelerations significantly lower than expected maximum seismic accelerations.

A detailed comparison of three methods (one from each of these three categories) has been undertaken by Ling et al. (1997):

5. Seismic Design of GRS Structures

Method 1 (Category A) Ling et al. method, as described in Section 5.2.5 of this report.

Method 2 (Category B) As above, but with reduced seismic accelerations for the reinforced soil block according to the recommendations of Christopher et al. (1990; described in Section 5.2.4 of this report).

Method 3 (Category C) Method based on consideration of permanent displacement, as described in Section 5.3 of this report.

Ling et al. conducted a computer parametric study for 5 m-high GRS structures with 20 equally spaced geosynthetic layers to illustrate the effect of the seismic inertia force on the required geosynthetic strength and length. The results of the parametric study for different seismic coefficients (C_s) and front face slope steepness (i) are shown on Figures 5.7, 5.8, and 5.9 using the following variables:

$$K = \frac{\sum t_j}{\frac{1}{2}\gamma H^2} \quad (43)$$

$$L_c = l_c/H \quad (44)$$

$$L_{ds} = l_{ds}/H \quad (45)$$

$$C_{ds} = \tan \phi_{sG} / \tan \phi_s \quad (46)$$

where H is the height of the GRS structure;

$\sum t_j$ is required total force in geosynthetic reinforcement;

l_c is the largest value of required geosynthetic lengths determined from internal stability and compound stability analyses;

l_{ds} is geosynthetic length required to resist direct sliding;

ϕ_{sG} is soil-geosynthetic friction angle;

ϕ_s is soil friction angle.

The results for Method 1 are shown on Figures 5.7 and 5.8. It can be seen from Figure 5.7 that, for the seismic coefficient $C_s = 0.3$, significantly longer lengths of geosynthetic reinforcement are required to resist tieback/compound failures compared with the static design case ($C_s = 0$). Figure 5.7 also demonstrates that, for ϕ' slightly less than 30° and seismic coefficients C_s exceeding 0.3, pseudo-static equilibrium cannot be achieved regardless of the widths of the GRS structure (L_{ds}).

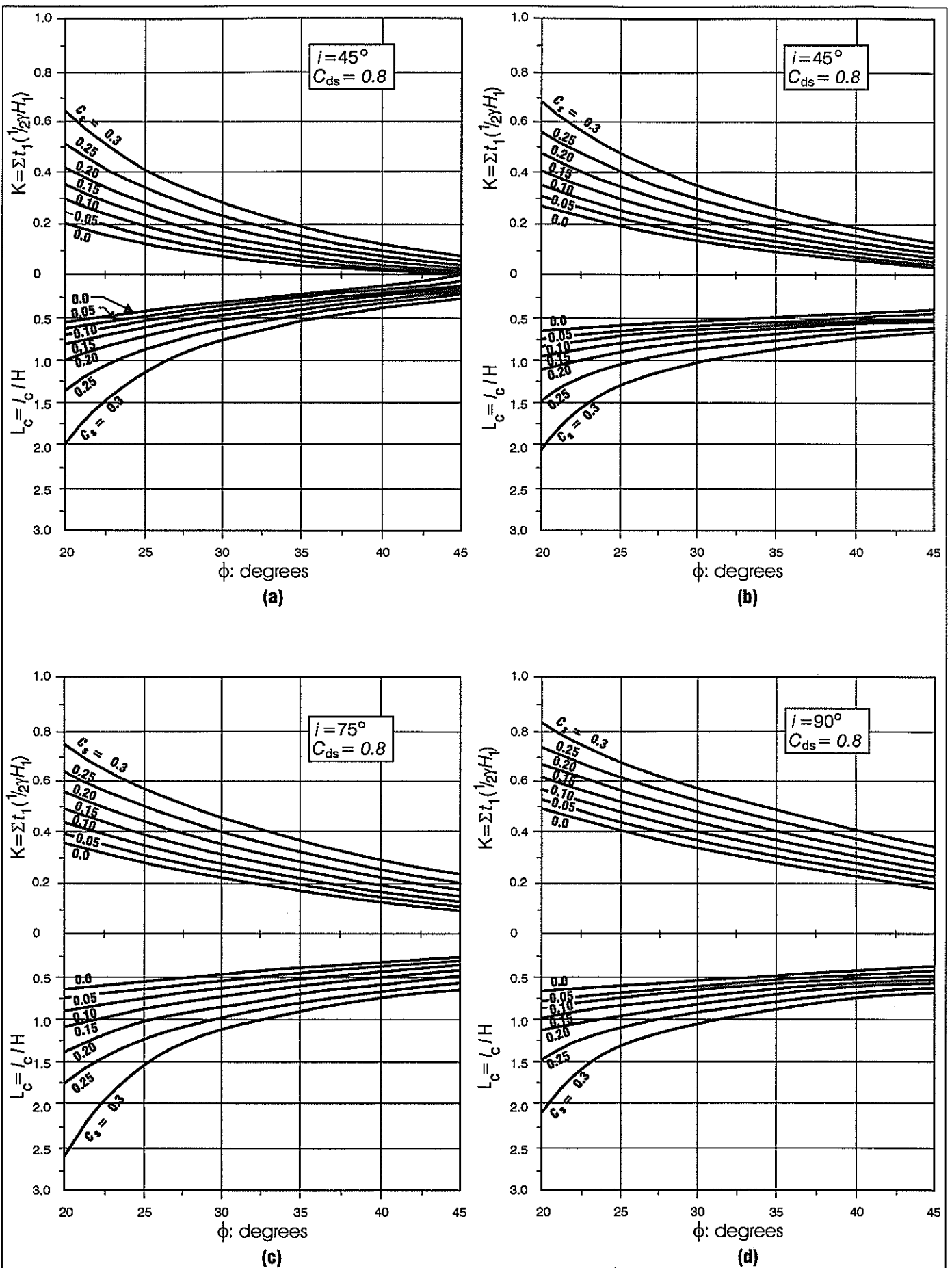


Figure 5.7 Required geosynthetic force and length (tieback/compound failure):
 (a) $i = 45^\circ$; (b) $i = 60^\circ$; (c) $i = 75^\circ$; (d) $i = 90^\circ$
 (after Ling et al. 1997)

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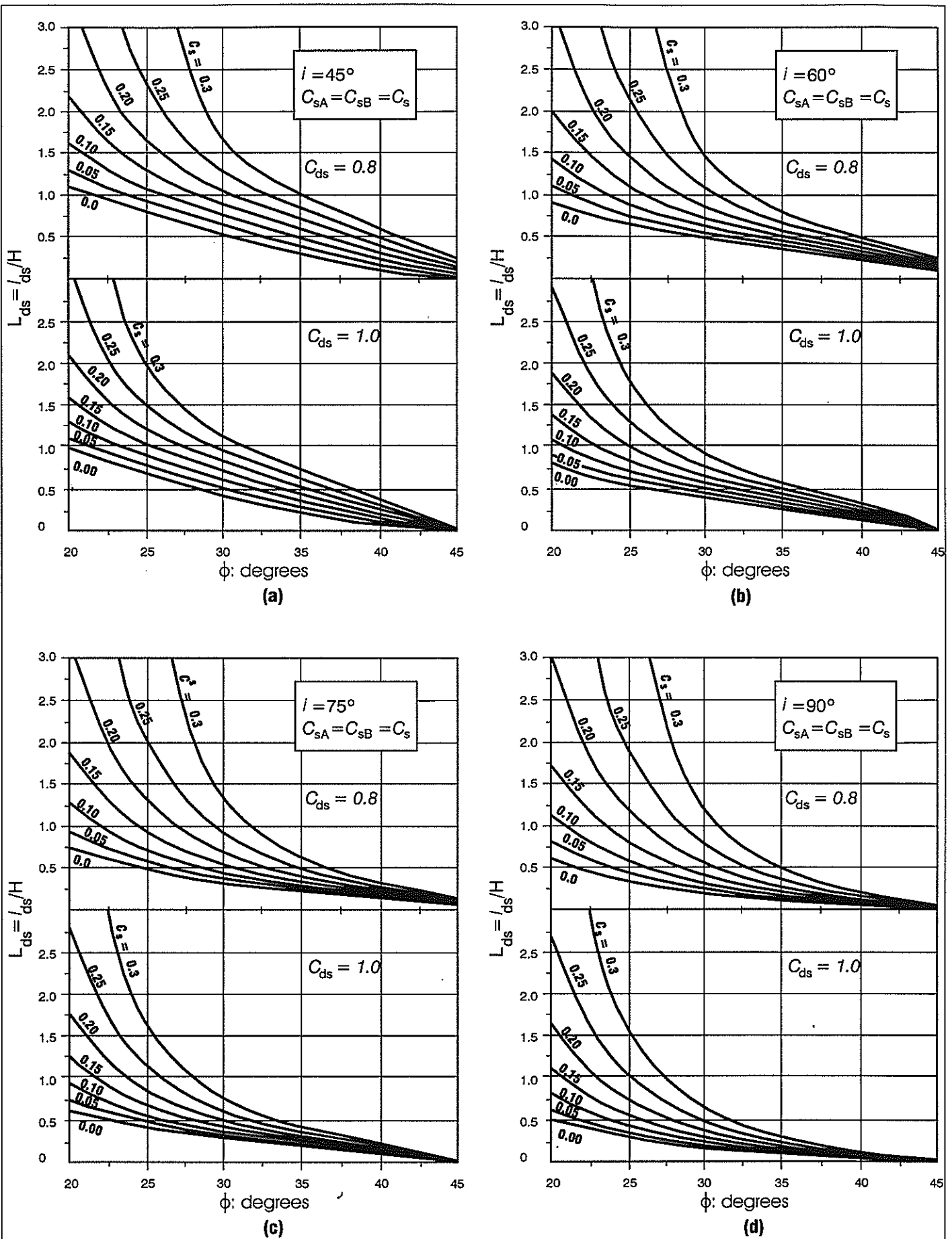


Figure 5.8 Required geosynthetic length to resist direct sliding with a reduced coefficient: (a) $i = 45^\circ$; (b) $i = 60^\circ$; (c) $i = 75^\circ$; (d) $i = 90^\circ$ (after Ling et al. 1997)

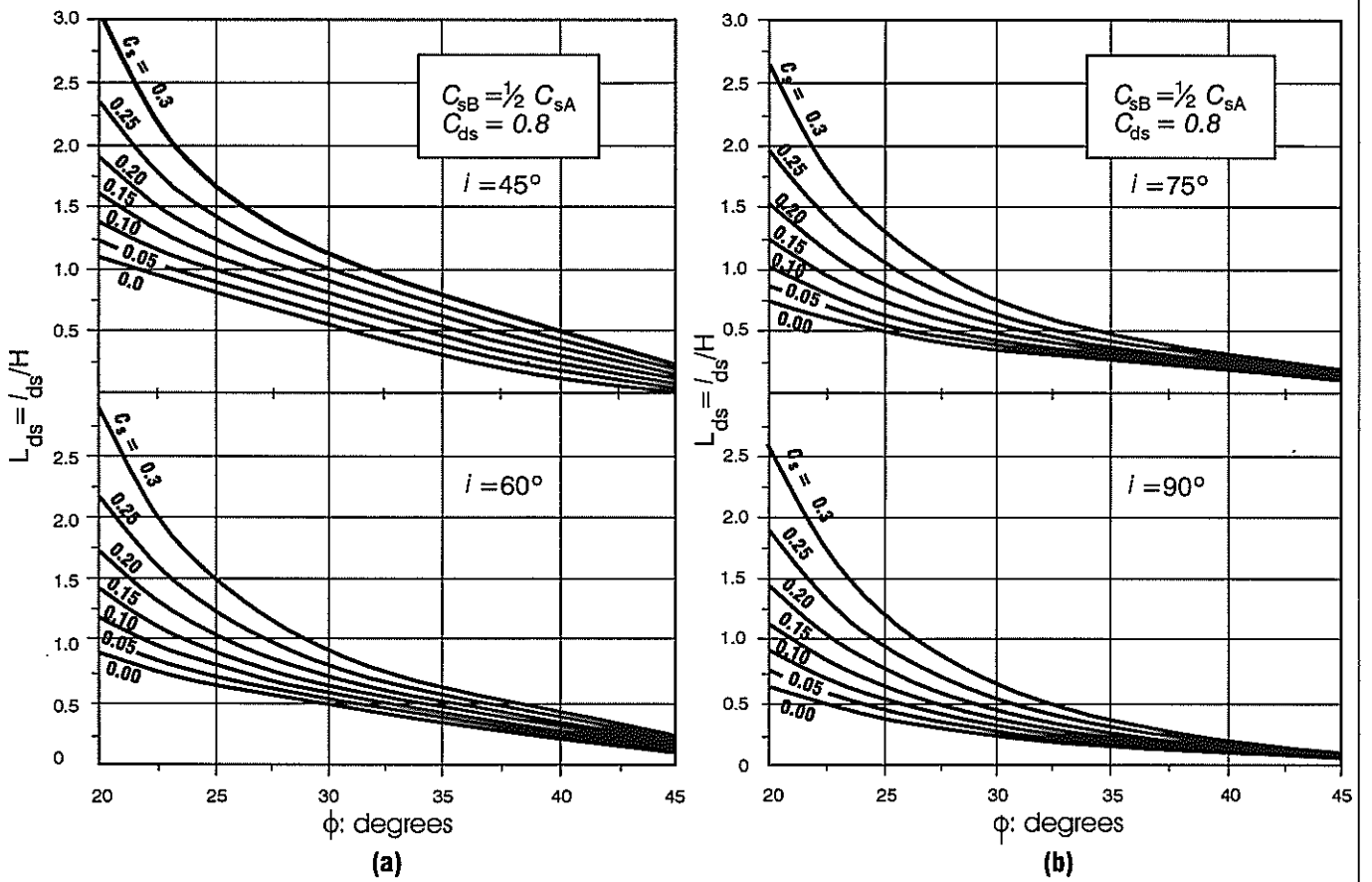


Figure 5.9 Required geosynthetic length to resist direct sliding with a reduced coefficient: (a) $i = 45^\circ$ and 60° ; (b) $i = 75^\circ$ and 90° (after Ling et al. 1997)

The results for Method 2 are shown on Figure 5.9. A reduced seismic coefficient ($C_{SB} = \frac{1}{2} C_S$), was used for the reinforced soil block, while the maximum seismic coefficient ($C_{SA} = C_S$) was used to calculate seismic thrust. It can be seen from Figure 5.9 that the required geosynthetic lengths reduced for all slope angles compared with those obtained by Method 1. For example, for a GRS structure with a vertical face ($i = 90^\circ$), backfill soil with $\phi' = 30^\circ$ and assuming $C_{SA} = 0.3$ and $C_{SB} = 0.15$, the reinforcement length required to resist direct sliding becomes $0.62H$ instead of $1.2H$ as required by Method 1.

The required reinforcement length to resist sliding can be reduced further if Method 3 is used. According to the charts developed by Ling et al. (note that the charts are not included in this report), the length of the structure (at $i = 90^\circ$, $\phi' = 30^\circ$, $C_{SA} = 0.3$, $C_{SB} = 0.15$) can be reduced to $0.5H$ and $0.4H$ if permanent displacements of 15 mm and 60 mm respectively are assumed to be acceptable.

In summary, for the GRS structure with parameters described above, the required reinforcement length ranges from $0.4H$ for Method 3, assuming a permanent displacement of 60 mm, to $1.2H$ for Method 1 assuming no permanent displacement. In some instances, described by Ling et al., the upper limit of the required reinforcement length for Method 1 was as high as $14H$.

It is, therefore, obvious that in many cases methods of Categories A and B require excessively long geosynthetics. Very often space constraints and the high cost involved render installation of long geosynthetics impractical. As currently there are no commonly accepted methods for Category C, designers need to rely on the conservatism of the static design procedures in these cases.

5.4.2 Comparison with Factual Data

Only a few GRS structures have been appropriately instrumented and tested under seismic or dynamic loads to date. Detailed information on the behaviour of these structures is not readily available. For most of the recent case studies there are uncertainties with respect to the soil properties, earthquake characteristics and a lack of accurate information about deformation of GRS structures after an earthquake. Therefore, a qualitative rather than a detailed quantitative comparison has been undertaken. This comparison is described below.

(a) Methods Based on Pseudo-Static Approach

It has been shown by Nishimura et al. (1996), Tatsuoka et al. (1996) and other researchers that the design methods based on the pseudo-static approach are conservative. As described in Section 3.2 of this report and shown in Table 3.2, critical seismic accelerations predicted on the basis of these methods are normally significantly lower than those that GRS can withstand without any signs of deformation and damage. For the fifteen GRS walls included in Table 3.2 the difference between the predicted critical seismic accelerations and the recorded accelerations which the structures experienced without any signs of damage or deformation ranges between 33% and 400 %.

Similar data has been reported by Sitar et al. (1997). Therefore most researchers conclude that the design methods based on pseudo-static approach are conservative.

(b) Methods Based on Consideration of Permanent Displacement

To the author's knowledge, currently only one design method (proposed by Ling et al. 1997) allows the assessment of permanent displacement of GRS structures. This method is described in Section 5.3 of this report. Ling et al. collected several relatively well documented case histories from recent major earthquakes to validate the method. It was difficult to assess permanent displacements of the GRS structures for the case studies, because the structures had not been instrumented for seismic performance. Therefore, cracking was used as an indication that permanent displacement has occurred.

Information about the case histories is given in Table 5.1. From this table for GRS structures, where the critical seismic acceleration was higher than the estimated maximum acceleration, no cracking has been reported. It was, therefore, assumed that no permanent displacement had occurred.

For the GRS structures that experienced accelerations higher than their critical seismic accelerations some cracking on the crest, just behind the reinforced soil block, was evident. This data indicated that the predictions of critical accelerations by the Ling et al. method are probably accurate. The magnitudes of the predicted permanent displacements has been compared for three GRS walls (Table 5.1).

These predictions are not always correct. For example, for the Valencia Wall the measured crack width is 6 mm, but the predicted permanent displacement is 160 mm. It is therefore obvious that further research is required to refine and validate the method.

Table 5.1 Comparison of performance of geosynthetic-reinforced soil structures during major earthquakes (after Ling et al. 1997)

Structure	Amagasaki Wall	Gould Wall	Valencia Wall	Valencia-top Wall	Kushiro Wall	Seiken Wall
Slope Geometry						
Height H : m	4.7	4.6	6.4	2.7	4.5	5.5
Slope i : degrees	90	86.4	86.4	86.4	73.3	78.7
Facing type:	Rigid cast-in-place concrete	Segmental blocks	Segmental blocks	Segmental blocks	Wire cages	Concrete panel, shotcrete
Backfill Soil						
Friction angle ϕ : °	35	33	33	33	35	37
Unit weight γ : kN/m ³	20	20	20	20	18	18
Geosynthetic: Type	Polyester geogrid, coated with PVC	Polyester geogrid	Polyester geogrid	Polyester geogrid	HDPE geogrid	Non-woven polypropylene geotextile
Number of layers:	16	12	12	4	11	12
Length: m	2.5	3.6	1.0-5.5	1.8-5.5	5.6	2.5-5.5
Earthquake (Mag)	Kobe (7.3)	Northridge (6.7)	Northridge (6.7)	Northridge (6.7)	Kushiro-Oki (7.8)	Chiba-ken Toho-Oki (6.7)
Date	17 Jan 1995	17 Jan 1994	17 Jan 1994	17 Jan 1994	15 Jan 1993	17 Dec 1987
Distance from epicentre: km	35	35	23	23	33	40
Estimated maximum acceleration: g	0.27	0.3	0.5	0.5	0.3	0.326
Permanent Displacement						
Calculated critical acceleration C_{sy}	0.31	0.312	0.324	0.318	0.357	0.427
Crack width: mm	0.0	6	6	60	0.0	0.0
Calculated width: mm	0.0	at verge of failure 0.0	160	60	0.0	0.0

6. CONCLUSIONS

1. The applications of GRS to permanent structures carrying roads and/or pedestrian traffic is rapidly increasing worldwide. GRS structures can provide substantial cost savings compared with conventional type structures.
2. GRS is a comparatively new technology and, therefore, the complete set of standards and test methods for GRS have not been developed. Test procedures and interpretation of test results for GRS differ from country to country. The number of design parameters and tests required to describe properties of geosynthetics is more than that for conventional materials such as steel or reinforced concrete.
3. Test standards, design guidelines and approval procedures have been developed in some countries. The approval procedures cover a wide range of aspects, from manufacturing quality control to GRS design procedures.
4. Hundreds of GRS structures have been constructed worldwide. Recent case studies provide evidence of the satisfactory performance of GRS structures under both static and seismic conditions. At least 54 GRS structures have been constructed in New Zealand.
5. High seismic resistance of GRS structures is attributed to the following:
 - The tensile strength of geosynthetic materials which can be mobilised under seismic conditions is higher than that for static conditions. For seismic design, tensile strength tests conducted at higher than standard rates of strain would be more appropriate. These tests would often result in an allowable tensile resistance for seismic conditions appreciably higher than that for static conditions.
 - The confined (in-soil) tensile strength of geosynthetic materials is very often higher than their unconfined (in-air) strength. In-soil ultimate tensile strengths as high as four times the in-air strength have been reported.
 - The existing design methods do not take into account the ductility of GRS structures. Japanese researchers suggest that the ductility of GRS structures should be reflected in the design by applying a design seismic acceleration that is lower than the expected peak ground acceleration.
6. Almost all existing design methods used in current engineering practice are, in essence, empirical methods. Design methods for GRS (as for the test methods) differ from country to country. This research review has been conducted to compare design methodologies and the results obtained by various existing design methods. The conclusion is that there are substantial variations in the amounts of geosynthetic reinforcement required by different design methods.

6. *Conclusions*

7. Most of the GRS design methods for both static and seismic conditions are based on limiting equilibrium analysis. These methods do not allow the deformation and displacement of the GRS block to be assessed, but nevertheless in most cases provide safe design solutions.
8. Guidelines on the seismic design of GRS structures in different countries differ substantially. The GRS block sizes calculated by different methods vary. Designers frequently rely on the conservatism of the static design to provide seismic stability of GRS structures. However, the conservatism of the static design procedures has decreased significantly in recent years. It is, therefore, not clear whether a GRS structure, designed to resist static loads only, will have an adequate level of seismic resistance.
9. Factors of safety used by different GRS design methods are different. Because of the empirical nature of the existing design methods, factors of safety for geosynthetic strength and for different modes of instability cannot be considered separately from the design method they are used with.
10. GRS structures constructed to date (1998) have been designed by different design methods and therefore have different levels of static and seismic resistance. There is clearly a need for design and construction guidelines for GRS structures in New Zealand. These design guidelines for static conditions could be based on the existing guidelines and standards developed by the UK, US, Japan and Germany. The design guidelines for seismic conditions could be based on the existing guidelines and standards developed by the US and Japan.
11. This research review summarises the existing design guidelines and forms a basis for further research work on the preparation of design guidelines for constructing GRS structures in New Zealand.

7. BIBLIOGRAPHY

- AASHTO. 1992. *Standard specification for highway bridges*. 15th Edition, American Association of State Highway and Transport Officials. Amended 1993, 1994, 1995.
- Andrawes, K.Z., Loke, K.H., Yeo, K.C., Murray, R.T. 1990. Application of boundary yielding concept to full scale reinforced and unreinforced soil walls. *Performance of Reinforced Soil Structures. British Geotechnical Society*: 79-83.
- Baker, R., Garber, M. 1977. Variational approach to slope stability. *Proceedings of the 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo 2*: 9-12.
- Balzer, E., Delmas, P., Matichard, Y., Sere, A., Thamm, B.R. 1990. Geotextile reinforced abutment: full scale test and theory. *Performance of Reinforced Soil Structures. British Geotechnical Society*: 47-52.
- Bathurst, R.J., Jarrett, P.M. 1990. Grid-reinforced retaining wall model tests. *Geotextiles, Geomembranes and Related Products*, Den Hoedt (Ed.): 119.
- Bathurst, R.J., Benjamin, D.J., Jarrett, P.M. 1988. Laboratory study of geogrid reinforced soil walls. *Geogrid Reinforced Soil Walls*. Royal Military College of Canada, Kingston, Ontario. *Proceedings of the Symposium on Geosynthetics for Soil Improvement, American Society of Civil Engineers (ASCE) National Convention, Nashville, Tennessee, R.D. Holtz (Ed.)*. *ASCE Geotechnical Special Publication No. 18*: 78-192.
- Bathurst, R.J., Koerner, R.M. 1988. Results of Class A predictions for the RMC reinforced soil wall trials. *The Application of Polymeric Reinforcement in Soil Retaining Structures*, P.M. Jarrett & A. McGown (Eds): 127-171.
- Bathurst, R.J., Wawrychuk, W.F., Jarrett, P.M. 1988. Laboratory investigation of two large-scale geogrid reinforced soil walls. *The Application of Polymeric Reinforcement in Soil Retaining Structures*, P.M. Jarrett & A. McGown (Eds): 1-125.
- Berg, R.R. 1993. Geosynthetic mechanically stabilized earth slopes on firm foundations. *Federal Highway Administration*: 1-43.
- Berg, R.R., Bonaparte, R., Anderson, R.P., Chouery, V.E. 1986. Design, construction and performance of two geogrid reinforced soil retaining walls. *Slope Protection and Retaining Walls. Third International Conference on Geotextiles, Vienna, Austria*: 401-406.
- Bonaparte, R., Holtz, R.D., Giroud, J.P. 1985. Geotextile testing and the design engineer. *Symposium, ASTM Special Technical Publication 952*: 69-115.

7. Bibliography

- Bonaparte, R., Schmertmann, G.R. 1988. Reinforcement extensibility in reinforced soil wall design. *The Application of Polymeric Reinforcement in Soil Retaining Structures*, P.M. Jarrett & A. McGown (Eds): 409-457.
- Bonaparte, R., Schmertmann, G.R., Williams, N.D. 1986. Seismic design of slopes reinforced with geogrids and geotextiles. *Foundations and Reinforced Embankments. Third International Conference on Geotextiles, Vienna, Austria*: 273-278.
- Broms, B.B. 1978. Design of fabric reinforced retaining structures. *Proceedings of Symposium on Earth Reinforcement, Pittsburg, Pennsylvania*: 282-304.
- British Standards Institute. 1995. Reinforced soils. Code of Practice: Strengthened/Reinforced Soils and Other Fills. *BS 8006: 1995*: 160.
- Burwash, W.J., Frost, J.D. 1991. Case history of a 9 m high geogrid reinforced retaining wall backfilled with cohesive soil. *Geosynthetics Conference, Atlanta, USA*: 485-493.
- Carroll, R.G., Richardson, G.N. 1986. Geosynthetic reinforced retaining walls. *Slope Protection and Retaining Walls. Third International Conference on Geotextiles, Vienna, Austria*: 389-394.
- Chandler, D., Kirkland, T. 1991. Design and construction of a geotextile wall. *Geosynthetics Conference, Atlanta, USA*: 755-761.
- Chew, S.H., Schmertmann, G.R., Mitchell, J.K. 1990. Reinforced soil wall deformations by finite element method. *Performance of Reinforced Soil Structures. British Geotechnical Society*: 35-40.
- Christopher, B.R., Gill, S.A., Giroud, J.P., Juran, I., Mitchell, J.K., Schtossler, F., Dunicliff, J. 1989. Reinforced soil structures. Summary of Research and Systems Information. *US Department of Commerce National Technical Information Service 2*: 1-158.
- Christopher, B.R., Holtz, R.D., Allen, T.M. 1990a. Instrumentation for a 12.6 m high geotextile-reinforced wall. *Performance of Reinforced Soil Structures. British Geotechnical Society*: 73-78.
- Christopher, B.R., Safdar, A.G., Giroud, J., Juran, I., Mitchell, J.K., Schlosser, F., Dunicliff, J. 1990b. Reinforced soil structures. *Design and Construction Guidelines 1*: 1-301. Federal Highway Administration, McLean, Virginia.
- Claybourn, A.F. 1990. A comparison of design methods for geosynthetic reinforced earthwalls. Thesis for University of Colorado, USA. 153pp.
- Claybourn, A.F., Wu, J.T.H. 1991. Case history comparison of geosynthetic-reinforced soil walls. *Geosynthetics Conference, Atlanta, USA*: 549-559.

Claybourn, A.F., Wu, J.T.H. 1993. Geosynthetic-reinforced soil wall design. *Geotextiles and Geomembranes 12*: 707-724.

Collin, J.G. 1986. Earth wall design. Doctoral Thesis, University of California, Berkeley.

Collin, J.G., Chourey-Curtis, V.E., Berg, R.R. 1992. *Earth reinforcement practice*, Ochiai, Hayashi & Otani (Eds), Balkema, Rotterdam: 223-228.

De Groot, M.B., Den Hoedt, G., Termaat, R.J. 1990. Geosynthetics: applications, design and construction. *Proceedings of the First European Geosynthetics Conference Eurogeo 1, Maastricht, Netherlands*: 1-24.

Department of Transport. 1987. Reinforced and anchored earth retaining walls and bridge abutments for embankments. *Technical Memorandum BE/78*. HMSO, UK. Published 1978, updated 1987. 49pp.

Department of Transport. 1994. Design methods for the reinforcement of highway slopes by reinforced soil and soil nailing techniques. *Advice Note HA 68/94*. HMSO, UK.

Deutsches Institut für Bautechnik. 1990. *Certificate No. Z20.1-102*.

Doi, Y., Mizushima, S., Tateyama, M., Murata, O. 1994. Geotextile-reinforced soil retaining wall: reconstruction of a railway embankment at Rokujizo, Kyoto. *Recent Case Histories of Permanent Geosynthetic Reinforced Soil Retaining Walls*, Tatsuoka & Leshchinsky (Eds): 197-204.

Dondi, G. 1990. Load test on a retaining wall reinforced with geosynthetics. *Geotextiles, Geomembranes and Related Products*, Den Hoedt (Ed.): 101-106.

Elias, V. 1990. *Durability/corrosion of soil reinforced structures*. Federal Highway Administration, McLean, Virginia. 173pp.

Elias, V. 1997. Corrosion/degradation of soil reinforcements for mechanically stabilized earth walls and reinforced soil slopes. *Demonstration Project 82 Reinforced Soil*: 1-105. US Department of Transportation Federal Highway Administration.

Elias, V., Christopher, B.R. 1997. *Mechanically stabilized earth walls and reinforced soil slopes design and construction guidelines*. US Department of Transportation Federal Highway Administration. 371pp.

Fannin, R.J. 1994. Field observations on the load-strain-time behaviour of geogrid reinforcement. *Canadian Geotechnical Journal 31*: 564-569.

Fannin, R.J., Hermann, S. 1990. Performance data for a sloped reinforced soil wall. *Canadian Geotechnical Journal 27*: 676-686.

7. Bibliography

- Fishman, K.L., Desai, C.S. 1991. Response of a geogrid earth reinforced retaining wall with full height precast concrete facing. *Geosynthetics Conference, Atlanta, USA*: 691-700.
- Goehring, R.L. 1991. Raleigh Street embankment - Orlando, Florida. *Geosynthetics Conference, Atlanta, USA*: 727-738.
- Gourc, J.P., Gotteland, Ph., Delmas, Ph. 1988. Design of geosynthetic retaining walls: 'displacement method' and 'two blocks method' comparison and charts. *International Geotechnical Symposium on Theory and Practice of Earth Reinforcement, Fukuoka, Japan*: 517-522.
- Gourc, J.P., Ratel, A., Delmas, P.H. 1986. Design of fabric retaining walls: the 'displacement method'. *Slope Protection and Retaining Walls. Third International Conference on Geotextiles, Vienna, Austria*: 1067-1072.
- Gourc, J.P., Ratel, A., Gotteland, Ph. 1988. Design of reinforced soil retaining walls: analysis and comparison of existing design methods and proposal for a new approach. *The Application of Polymeric Reinforcement in Soil Retaining Structures*, P.M. Jarrett & A. McGown (Eds): 459-506.
- Greenwood, J.H. 1990. The creep of geotextiles. *Geotextiles, Geomembranes and Related Products*, Den Hoedt (Ed.): 645-650.
- Hanna, T.H., Touahmia, M. 1991. Comparative behaviour of metal and Tensar geogrid strips under static and repeated loading. *Geosynthetics Conference, Atlanta, USA*: 575-585.
- Holtz, R.D., Christopher, B.R., Berg, R.R. 1995. *Geosynthetic design and construction guidelines*. National Highway Institute, Federal Highway Administration, Washington, DC. 395pp.
- Horii, K., Kishida, H., Tateyama, M., Tatsuoka, F. 1994. Computerized design method for geosynthetic-reinforced soil retaining walls for railway embankments. *Recent Case Histories of Permanent Geosynthetic Reinforced Soil Retaining Walls*, Tatsuoka & Leshchinsky (Eds): 205-218.
- Ingold, T.S. 1982. An analytical study of geotextile reinforced embankments. *Proceedings of Second International Conference on Geotextiles, Las Vegas*: 683.
- Ingold, T.S. 1988. Long term performance requirements for polymeric soil reinforcement in the United Kingdom. *International Geotechnical Symposium on Theory and Practice of Earth Reinforcement, Fukuoka, Japan*: 523-528.
- Ingold, T.S., Miller, K.S. 1988. *Geotextiles Handbook*. Thomas Telford, London. 36pp.

Jarrett, P.M., McGown, A. 1988. Analytical techniques and design methods. *The Application of Polymeric Reinforcement in Soil Retaining Structures*, P.M. Jarrett & A. McGown (Eds): 555.

Jenner, C.G. 1990. A study of the influence of soil on the reinforcement load in polymer grid reinforced soil structures. *Performance of Reinforced Soil Structures. British Geotechnical Society*: 21-24.

Jewell, R.A. 1998. Reinforced soil wall analysis and behaviour. *The Application of Polymeric Reinforcement in Soil Retaining Structures*, P.M. Jarrett & A. McGown (Eds): 365-408.

Jewell, R.A., Paine, N., Woods, R.I. 1984. Design methods for steep reinforced embankments. *Proceedings of Conference on Polymer Grid Reinforcement, London*: 70.

Jones, C.J.F.P. 1994. Economic construction of reinforced soil structures. *Recent Case Histories of Permanent Geosynthetic-Reinforced Soil Retaining Walls*, Tatsuoka & Leshchinsky (Eds): 103-115.

Jones, C.J.F.P. 1988. Predicting the behaviour of reinforced soil structures. *International Geotechnical Symposium on Theory and Practice of Earth Reinforcement, Fukuoka Japan*: 535-540.

Jones, C.J.F.P. 1996. *Earth reinforcement and soil structures*. Thomas Telford Publishing, London. 379pp.

Jones, C.J.F.P., McGown, A., Varney, D.J. 1988. Construction methods, economics and specifications. *The Application of Polymeric Reinforcement in Soil Retaining Structures*, P.M. Jarrett & A. McGown (Eds). 555pp.

Juran, I., Farrag, K.H., Richmond, L. 1991. Short and long term performance of polymeric geogrids. *Proceedings of Geosynthetics Conference, Atlanta, USA*: 587-599.

Kanazawa, Y., Ikeda, K., Murata, O., Tateyama, M., Tatsuoka, F. 1994. Geosynthetic-reinforced soil retaining walls for reconstructing railway embankment at Amagasaki. *Recent Case Histories of Permanent Geosynthetic Reinforced Soil Retaining Walls*, Tatsuoka & Leshchinsky (Eds): 233-242.

Koerner, R.M. 1994. *Designing with geosynthetics*. Prentice Hall, Englewood Cliffs. 395pp.

Koga, Y., Washida S. 1992. Earthquake resistant design method of geotextile reinforced embankments. *Earth Reinforcement Practice*, Ochiai, Hayashi & Otani (Eds): 255-259.

7. Bibliography

- Koga, Y., Ito, Y., Washida, S., Shimazu, T. 1988. Seismic resistance of reinforced embankment by model shaking table tests. *International Geotechnical Symposium on Theory and Practice of Earth Reinforcement, Fukuoka, Japan*: 413-418.
- Kutara, K., Aoyama, N., Yasunaga, H., Kato, T. 1988. Long term pull-out tests of polymergrids in sand. *International Geotechnical Symposium on Theory and Practice of Earth Reinforcement, Fukuoka, Japan*: 117-122.
- Leclercq, B., Schaeffner, M., Delmas, Ph., Blivet, J.C., Matichard, Y. 1990. Durability of geotextiles: pragmatic approach used in France. *Geotextiles, Geomembranes and Related Products*, Den Hoedt (Ed.): 679.
- Leschinsky, D., Volk, J.C. 1986. Predictive equation for the stability of geotextile reinforced earth structure. *Slope Protection and Retaining Walls. Third International Conference on Geotextiles, Vienna, Austria*: 383-388.
- Leshchinsky, D. 1993. Keynote lecture: Issues in geosynthetic-reinforced soil. *Earth Reinforcement Practice*, Ochiai, Hayashi & Otani (Eds): 871-897.
- Leshchinsky, D. 1994. Active versus reactive reinforcement force: design implications. *Recent Case Histories of Permanent Geosynthetic Reinforced Soil Retaining Walls*, Tatsuoka & Leshchinsky (Eds): 247-252.
- Leshchinsky, D. 1995. Design procedure for geosynthetic reinforced steep slopes. *Technical Report REMR-GT-120* (temporary number), US Army Engineer Waterways Experiment Station, Vicksburg, MS.
- Leshchinsky, D., Perry, E.B. 1987. A design procedure for geotextile-reinforced walls. *Session 1B: Slopes and Walls, Geosynthetic Conference, New Orleans, USA*: 95-107.
- Leshchinsky, D., Reinschmidt, A.J. 1985. Stability of membrane reinforced slopes. *ASCE Journal of Geotechnical Engineering 111 (11)*: 1285.
- Ling, H.I., Leshchinsky, D., Perry, E.B. 1997. Seismic design and performance of geosynthetic- reinforced soil structures. *Géotechnique 47 (5)*: 933-952.
- Marr, A., Werden, S. 1997. *Reinforced slope stability, a microcomputer program*. US Department of Transportation Federal Highway Administration. 92pp.
- McCaul, C., Snowdon, R.A. 1990. Early performance of a reinforced earth bridge abutment. *Performance of Reinforced Soil Structures. British Geotechnical Society*: 87-88.

- McGown, A., Andrawes, K.Z., Kabir, M.H. 1982. Load-extension testing of geotextiles confined in soil. *Proceedings of Second International Conference on Geotextiles, Las Vegas, Nevada, IFAI*: 793-796.
- Miki, H., Kutara, K., Minami, T., Nishimura, J., Fukuda, N. 1988. Experimental studies on the performance of polymer grid reinforced embankment. *International Geotechnical Symposium on Theory and Practice of Earth Reinforcement, Fukuoka, Japan*: 431-436.
- Muir-Wood, Sir A. 1985. *Polymer grid reinforcement*. Thomas Telford Limited, London. 11pp.
- Murato, O., Tateyama, M., Tatsuoka, F. 1990. Steep slopes reinforced with a planar geotextile having a rigid facing. *Geotextiles, Geomembranes and Related Products*, Den Hoedt (Ed.): 122.
- Murray, R.T. 1984. Reinforcement techniques in repairing slope failures. *Proceedings of a Conference on Polymer Grid Reinforcement, London*: 47.
- Nakamura, K., Tamura, Y., Tatsuoka, F., Iwasaki, K., Yamauchi, H. 1988. Roles of facings in reinforcing steep clay slopes with a non-woven geotextile. *International Geotechnical Symposium on Theory and Practice of Earth Reinforcement, Fukuoka, Japan*: 553-558.
- Nakanishi, A., Sakagushi, M. 1990. Seismic behaviour of the reinforced embankment by geotextiles. *Geotextiles, Geomembranes and Related Products*, Den Hoedt (Ed.): 121.
- Ng, H.Y., Mak, C.H. 1988. A 14 metre high reinforced soil embankment as the abutment of a steel bridge in Tuen Mun, Hong Kong. *International Geotechnical Symposium on Theory and Practice of Earth Reinforcement, Fukuoka, Japan*: 449-454.
- Nimmegern, M., Bush, D. 1991. The effect of repeated traffic loading on geosynthetic reinforcement anchorage resistance. *Proceedings of Geosynthetics Conference, Atlanta, USA*: 665-672.
- Nishimura, J., Hirai, T., Iwasaki, K., Saitoh, Y., Morishima, M., Shintani, H., Yoshikawa, S., Yamamoto, H. 1996. Earthquake resistance of geogrid-reinforced soil walls based on a study conducted following the Southern Hyogo earthquake. *Proceedings of International Symposium on Earth Reinforcement, Kyushu, Japan*. Balkema, Rotterdam.
- Paul, J. 1996. Coded Language. *Reinforced Soil Magazine*: 25, 26.

7. Bibliography

- Public Works Research Institute. 1992. Design and construction manual of geotextile reinforced soil structures. *Public Works Research Institute Report No. 3117*: 404 (in Japanese).
- Rankilor, P.R. 1990. The weathering of fourteen different geotextiles in temperate, tropical, desert and permafrost conditions. *Geotextiles, Geomembranes and Related Products*, Den Hoedt (Ed.): 719-722.
- Resl, S., Schneider, H., Werner, G. 1988. Theory and practice of 'reinforcing' steep slopes with non-woven geotextiles. *International Geotechnical Symposium on Theory and Practice of Earth Reinforcement, Fukuoka, Japan*: 565-570.
- Richardson, G.N., Lee, K.L. Seismic design of reinforced earth walls. *Journal of the Geotechnical Engineering Division*: 167-188.
- Rigo, J.M. 1988. The researchers' point of view. *Durability of Geotextiles*. RILEM (International Union of Testing and Research Laboratories for Materials and Structures), London, UK: 15-19.
- Rowe, R.K., Ho, S.K. 1988. Application of finite element techniques to the analysis of reinforced soil walls. *The Application of Polymeric Reinforcement in Soil Retaining Structures*, P.M. Jarrett & A. McGown (Eds): 541-554.
- Schardin-Liedtke, H. 1990. Geotextiles for the support of steep slopes: approval procedure. *Geotextiles, Geomembranes and Related Products*, Den Hoedt (Ed.): 79-85.
- Schlosser, F., De Buhan, P. 1990. Theory and design related to the performance of reinforced soil structures. *Performance of Reinforced Soil Structures*. *British Geotechnical Society*: 1-14.
- Schmertmann, G.R., Chouery-Curtis, V.E., Johnson, R., Bonaparte, R. 1987. Design charts for geogrid reinforced soil slopes. *Session 1B: Slopes and Walls Geosynthetic Conference, New Orleans, USA*: 108-120.
- Schneider, H.R., Holtz, R.D. 1986. Design of slopes reinforced with geotextiles and geogrids. *Geotextiles and Geomembranes* 3: 29.
- Segrestin, P. 1994. GRS structures with short reinforcements and rigid facing - discussion of previous papers published by Prof. F. Tatsuoka, M. Tateyama and O. Murata. *Recent Case Histories of Permanent Geosynthetic Reinforced Soil Retaining Walls*, Tatsuoka & Leshchinsky (Eds): 317-322.
- Simac, M.R., Christopher, B.R., Bonczkiewicz, C. 1990. Instrumental field performance of a 6 m geogrid soil wall. *Geotextiles, Geomembranes and Related Products*, Den Hoedt (Ed.): 53-59.

Sitar, N., Nova-Roessig, L., Ashford, S.A., Stewart, J.P. 1997. Seismic response of steep natural slopes, structural fills and reinforced soil slopes and walls. *Seismic Behaviour of Ground and Geotechnical Structures*: 341-348.

Smith, R.J.H., Worrall, P.K. 1990. Foundation options for reinforced earth on poor ground. *Performance of Reinforced Soil Structures. British Geotechnical Society*: 427-432.

Standards Australia. 1996. *Earth retaining structures (including reinforced soils)*. Draft Australian Standard for Comment. 96pp.

Steward, J.E., Williamson, R., Mohney, J. 1997. Guidelines for use of fabrics in construction and maintenance of low-volume roads. Chapter 5: *Earth Reinforcement*, USDA, Forest Service, Portland, Oregon.

Tateyama, M., Murata, O., Watanabe, K., Tatsuoka, F. 1994. Geosynthetic-reinforced retaining walls for Bullet Train yard in Nagoya City. *Recent Case Histories of Permanent Geosynthetic-Reinforced Soil Retaining Walls*, Tatsuoka & Leshchinsky (Eds): 141-150.

Tatsuoka, F., Koseki, J., Tateyama, M. 1996. Performance of reinforced soil structures during the 1995 Hyogo-kin Nanbu earthquake. *IS-Kyushu Special Report*. 36pp.

Tatsuoka, F., Murata, O., Tateyama, M., Nakamura, K., Tamura, Y., Ling, H.I., Iwasaki, K., Yamauchi, H. 1990. P2/4 reinforcing steep clay slopes with a non-woven geotextile. *Performance of Reinforced Soil Structures. British Geotechnical Society*: 141-146.

Tatsuoka, F., Tateyama, M., Koseki, J. 1996. Performance of soil retaining walls for railway embankments. *Special Issue of Soils and Foundations, Japanese Geotechnical Society*: 311-324.

Tensar Corporation. 1997. *The design of reinforced soil structures using tensar geogrids*. 45pp.

Thamm, B.R. 1990. Theory and design relate to the performance of reinforced soil structures. *Performance of Reinforced Soil Structures. British Geotechnical Society*: 15-20.

Thamm, B.R., Krieger, B., Krieger, J. 1990. Full-scale test on a geotextile reinforced retaining structure. *Geotextiles, Geomembranes and Related Products*, Den Hoedt (Ed.): 3-8.

Watts, G.R.A., Brady, K.C. 1990. Pull-out tests on geogrids. *Performance of Reinforced Soil Structures. British Geotechnical Society*: 65-71.

7. *Bibliography*

Wilson-Fahmy, R.F., Koerner, R.M. 1993. Unconfined and confined wide width testing of geosynthetics used in reinforcement applications. S.C.J. Cheng (Ed.), *ASTM STP 1190*: 49-63.

Wood, J.H., Elms, D.G. 1990. Seismic design of bridge abutments and retaining walls. *New Zealand National Roads Board, Road Research Unit Bulletin 84 (2)*: 1-90.

Wrigley, N.E. 1987. Durability and long-term performance of tensar polymer grids for soil reinforcement. *Materials Science and Technology 3*: 161-170.

Yako, M.A., Christopher, B.R. 1988. Polymerically reinforced retaining walls and slopes in North America. *The Application of Polymeric Reinforcement in Soil Retaining Structures*, P.M. Jarrett & A. McGown (Eds): 239-283.

Yoshioka, A., Delmas, Ph., Gourc, J.P., Gotteland, Ph. 1990. Validation of the 'Displacements Method' on an experimental reinforced wall at failure. *Geotextiles, Geomembranes and Related Products*, Den Hoedt (Ed.).

Zhongchun Ouyang. 1988. 26 m height reinforced soil quaywall: design and full-scale test. *International Geotechnical Symposium on Theory and Practice of Earth Reinforcement, Fukuoka, Japan*: 599-604.

APPENDIX A. APPROVAL PROCEDURES FOR GEOSYNTHETIC REINFORCEMENT

Certification Body	Country	Application / Design Method	Scope of Assessment	Significance of Approval
Deutsches Institut für Bautechnik	Germany	Steep embankments and retaining walls. Specific design method detailed in approval certificate.	Manufacturing quality control, QC test data, fitness for purpose, long-term creep data, grid-soil interaction, installation damage, ease of installation, durability.	Approval required prior to use of reinforcement in certain types of structure in Germany. Approval not required for private / commercial projects.
Geotechnical Engineering Office	Hong Kong	Steep embankments, retaining walls and bridge abutments. Design loads specified. Design method given in GEO Report No.34 for slopes (less than 70°) and GEOSPEC 2 for walls (70-90°).	Manufacturing quality control, QC test data, Fitness for purpose, long-term creep data, grid-soil interaction, installation damage, ease of installation, durability.	Approval required prior to use in reinforced soil applications in Hong Kong.
Roads and Traffic Authority (RTA)	New South Wales, Australia	Steep embankments and retaining walls and bridge abutments. Design loads specified. Design Method specified in R57 "Design of Reinforced Soil Walls".	Manufacturing quality control, QC test data, Fitness for purpose, long-term creep data, grid-soil interaction, installation damage, ease of installation, durability.	Approval required prior to use on RTA projects in New South Wales. Gives credibility in other states. Approval not required for private / commercial projects.

APPROVAL PROCEDURES FOR GEOSYNTHETIC REINFORCEMENT

Certification Body	Country	Application / Design Method	Scope of Assessment	Significance of Approval
British Board of Agreement (BBA)	UK	Retaining walls and bridge abutments designed to BE 3/78 (currently being changed to BD70/97). Design loads specified. Retaining walls to BD 70/97 & HA 68/94 .	Manufacturing quality control, quality control test data, fitness for purpose, long-term creep data, grid-soil interaction, installation damage, ease of installation, durability.	All soil proprietary reinforcing elements on major highways projects in UK require BBA certification to demonstrate fitness for purpose. BBA certificate is valued outside approved application and UK as confirmation of fitness for purpose - SR GEO certificate refers to BBA certificate. Not required for private / commercial projects.
British Standards Institution (BSI) Quality Assurance	UK	Manufacture only.	Audit of Quality Assurance procedure to requirements of ISO 9002.	Production quality control only.
Railtrack	UK	Steep Embankments, slope repairs and reinforced soil applications . Design method not specified. Structural reinforcement of railway ballast. Design method not specified.	Review of product data and existing relevant applications.	Allows easier approval of product on new projects with Railtrack.

APPENDIX B. LIST OF ORGANISATIONS CURRENTLY INVOLVED IN DEVELOPMENT OF DESIGN & TEST METHODS FOR GEOSYNTHETICS

1. International Geotextile Society (IGS) (international)
2. International Organisation for Standards (ISO) (international)
3. Permanent International Association of Road Congresses (PIARC) (international)
4. European Disposables and Nonwovens Associated (EDANA) (international)
5. Permanent International Association of Navigation Congresses (international)
6. Geosynthetic Research Institute (GRI) of Drexel University (international)
7. Ministry of Public Works (Belgium)
8. Canadian General Specification Board (CGSB) (Canada)
9. British Standards Institution (BSI) (United Kingdom)
10. The Institution of Civil Engineers (United Kingdom)
11. Transport and Road Research Laboratory (TRRL) (United Kingdom)
12. Technical Research Center of Finland (Finland)
13. Comité Français des Géotextiles et Géomembranes (France)
14. L'Association Française de Normalisation (AFNOR) (France)
15. Réunion Internationale des Laboratoires d'Essais et de Recherche sur les Matériaux et les Construction (RILEM) (France)
16. German Standards Committee for Geotextiles (DIN) (Germany)
17. Franzius Institut (für Wasserbau und Küsteningenieurwesen der Hannover) (Germany)
18. Bundesanstalt für Wasserbau (Germany)
19. Deutsche Gesellschaft für Erd- und Grundbau (Germany)
20. Nederlands Normalisatie Instituut (NNI) (The Netherlands)
21. Consiglio Nazionale Ricerche (CNR) (Italy)
22. ENEL/CRIS (Italy)
23. Unitex (Italy)
24. Korea Highway Corp. (South Korea)
25. Norwegian Road Research Laboratory (NRRL) (Norway)

26. João de Matos Rosa (Portugal)
27. The Swedish National Road Administration (Sweden)
28. Swedish Geotechnical Institute (Sweden)
29. Schweizerisches Verband der Geotextilfachleute (Switzerland)
30. South Africa Bureau of Standards (South Africa)
31. The American Society for Testing and Materials (ASTM) (United States)
32. US Army Corps of Engineers, Waterways Experiment Station (United States)
33. American Association of State Highway and Transportation Officials (AASHTO) (United States)
34. Geotechnical Engineering Office (Hong Kong)
35. Deutsches Institut für Bautechnik (Germany)

APPENDIX C. TEST STANDARDS RELEVANT TO GEOSYNTHETICS USED IN GRS STRUCTURES

AMERICAN SOCIETY FOR TESTING & MATERIALS STANDARDS

D543	Evaluating the Resistance of Plastics to Chemical Reagents
D746	Brittleness Temperature of Plastics and Elastomers by Impact
D751	Test Methods for Coated Fabrics (discontinued 1997, no replacement)
D792	Density and Specific Gravity of Plastics by Displacement
D794	Determining Permanent Effect of Heat on Plastics
D1388	Test Methods for Stiffness of Fabrics (discontinued 1995, no replacement)
D1424	Test Methods for Tearing Strength of Fabrics by Falling-Pendulum Type (Elmendorf) Apparatus (discontinued 1995, no replacement)
D1505	Density of Plastics by the Density-Gradient Technique
D1693	Environmental Stress-Cracking of Ethylene Plastics
D3786	Test Methods for Hydraulic Bursting Strength of Knitted Goods and Non-Woven Fabrics - Diaphragm Bursting Strength Tester Method (discontinued 1996, no replacement)
D4354	Sampling of Geosynthetics for Testing
D4355	Deterioration of Geotextiles from Exposure to Ultraviolet Light and Water (Xenon-Arc Type Apparatus)
D4439	Geosynthetics
D4533	Trapezoid Tearing Strength of Geotextiles
D4594	Effects of Temperature on Stability of Geotextiles
D4595	Tensile Properties of Geotextiles by the Wide Strip Method
D4632	Breaking Load and Elongation of Geotextiles (Grab Method)
D4759	Specification Conformance of Geosynthetics
D4833	Index Puncture Resistance of Geotextiles, Geomembranes, and Related Products
D4884	Seam Strength of Sewn Geotextiles
D5199	Test method for Measuring Nominal Thickness of Geotextiles and Geomembranes
D5208	Operating Fluorescent Ultraviolet (UV) and Condensation Apparatus for Exposure of Photodegradable Plastics
D5261	Test method for Measuring Mass Per Unit Area of Geotextiles
D5262	Test methods for Unconfined Tension Creep Behaviour of Geosynthetics
D5321	Test method for Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method
D5322	Immersion Procedures for Evaluating the Chemical Resistance of Geosynthetics to Liquids
D5397	Test methods for Stress Crack Resistance of Polyolefin Geomembranes Using Notched Constant Tensile Load Test
D5496	In Situ Immersion Testing of Geosynthetics
G21	Determining Resistance of Synthetic Polymeric Materials to Fungi
G22	Determining Resistance of Plastics to Bacteria
G53	Operating Light and Water Exposure Apparatus (Fluorescent UV - Condensation Type) for Exposure of Non-metallic Materials

GEOSYNTHETIC RESEARCH INSTITUTE TEST METHODS & STANDARDS

GEOTEXTILE (GT) RELATED

- GT3 Deterioration of Geotextiles from Outdoor Exposure
- GT5 Tension Creep Testing of Geotextiles (discontinued, see ASTM D5262)
- GT6 Geotextile Pullout
- GT7 Determination of Long-Term Design Strength of Geotextiles

GEOGRID (GG) RELATED

- GG1 Geogrid Rib Tensile Strength
- GG2 Geogrid Junction Strength
- GG3a Tension Creep Testing of Stiff Geogrids (discontinued, see ASTM D5262)
- GG3b Tension Creep Testing of Flexible Geogrids (discontinued, see ASTM D5262)
- GG4a Determination of the Long-Term Design Strength of Stiff Geogrids
- GG4b Determination of the Long-Term Design Strength of Flexible Geogrids
- GG5 Test Method for Geogrid Pullout

GEOSYNTHETIC (GS) RELATED (i.e. MULTIPURPOSE)

- GS1 CBR Puncture Strength
- GS2 Rupture Strength by Pendulum Impact for Geotextiles-Geomembranes-Geocomposites
- GS3 In-Situ Monitoring of the Mechanical Performance of Geosynthetics
- GS4 Time Dependent (Creep) Deformation Under Normal Pressure
- GS6 Interface Friction Determination by Direct Shear Testing (discontinued, see ASTM D5321)
- GS7 Determining the Index Friction Properties of Geosynthetics
- GS8 Determining the Connection Strength of Mechanically Anchored Geosynthetics
- GS9 Oxidative Induction Time of Polyethylene Geosynthetics by High Pressure Differential Scanning Calorimetry