GEOTEXTILES

Transit New Zealand Research Report No. 6

GEOTEXTILES

K. Hudson
and
G. R. W. East

Transit New Zealand Research Report No. 6

ISBN 0-478-04139-X
ISSN 1170-9405

P O Box 5084, Lambton Quay, Wellington, New Zealand
Telephone (04) 499-6600; Facsimile (04) 496-6666

Hudson, K. C., East, G. R. W. 1991. Geotextiles.
Transit New Zealand Research Report No. 6.

Keywords: geotextile, pavements, roads, separation, filtration, reinforcement, drainage, fibres, construction, reflective cracking, chip seals

DISCLAIMER

Transit New Zealand, its members, employees and agents undertake no duty of care in contract, tort, or otherwise, to users (whether direct or indirect) of this document and make no warranties or representations of any kind whatsoever in relation to any of its contents.

The document is only made available on the basis that all users of it, whether direct or indirect, must take appropriate legal or other expert advice in relation to their own circumstances and must rely solely on their own judgement and such legal or other expert advice.

Under no circumstances will Transit New Zealand, its members, employees and agents be liable in any way whatsoever, whether in contract, tort (including negligence), for breach of any statutory or regulatory duty (to the fullest extent permissible by law), or otherwise, to any user (whether direct or indirect) of this document for any loss or damage whatsoever arising directly or indirectly as a result of the use in any way of this document.

Each exclusion in the clauses of this disclaimer and each protection given by it is to be construed as a separate exclusion applying and surviving even if for any reason any of the exclusions or protections are held inapplicable in any circumstances.

CONTENTS

ACKNOWLEDGMENTS

Transit New Zealand, on behalf of the authors, thanks the New Zealand suppliers of geotextiles for their assistance, and for the provision of technical literature and product data. Messrs D. R. Lovatt and L. D. Houghton are also thanked for their comments and advice.

ABSTRACT

Geotextiles are permeable textile fabrics used in engineering applications.

This report is a review of geotextiles and their uses in pavements of both sealed and unsealed roads. Also reviewed is the use of geotextiles for filtration and sub-surface drainage.

Most of the readily available geotextiles are of either woven or non-woven construction using synthetic fibres. The types of synthetic geotextiles and their generalised properties are described. Specific properties are given for the geotextiles currently available in New Zealand. The test methods used to measure a geotextile's general, mechanical, hydraulic and durability properties are commented on.

The four main functions a geotextile can perform are separation, filtration, reinforcement and drainage. The function or functions a geotextile performs in any specific application depends both on the situation and on the geotextile. Understanding of the function(s) performed in each of the many situations covered is essential for proper geotextile selection and usage.

The uses of geotextiles in pavements of both sealed and unsealed permanent roads are discussed, as is the use of geotextiles in pavement construction, and in unsealed temporary roads.

Filtration is covered in some detail. Included is commentary on particle transportation and filtration in fine grained soils. Also included is filtration design to ensure selection of the appropriate geotextile properties in both cohesionless and cohesive soils.

Reflective cracking, its causes, and its reduction by the use of geotextiles under overlays on existing pavements are considered. Case histories of overseas pavement trials and the use of geotextiles in chip seals are also considered. Other uses for geotextiles include their use in embankments on weak foundations, under concrete block overlays, for MESL (membrane encapsulated soil layers), and in geocomposites for subsurface drainage.

CHAPTER 1

INTRODUCTION

1.1 GENERAL COMMENTS

Natural materials have been used for many centuries as an aid to road building. The Romans used woven reed mats when building roads over soft ground, and split-log "corduroy" roads over peat bogs date back to 3000 BC (Dewar 1962).

Over the last 20 years synthetic fabrics have been increasingly used in road pavements. The rate of increase in use of geotextiles for all purposes has been extremely rapid over the last 10 years. Estimates of the annual worldwide consumption of geotextiles indicate less than 5 million square metres were used in 1972, 260 million square metres in 1982 (Lawson 1982), between 300 and 400 million square metres in 1985, and it has been rapidly rising since.

Studies (Geotechnical Fabrics Report 1983, Hausmann 1984) conducted in 1982 give the following uses of geotextiles:

USA		EUROPE	
End Use	**1982 %**	**End Use**	**1982 %**
Sealed Roads	28	Roads, Road Embankments, Access Roads	60
Unsealed Roads	24	Drainage	15
Filtration & Drainage	13	Railroads	5
Reinforcement (stabilisation of embankments)	24	Protection with Geomembrane	10
Railroads	5	Erosion Control	5
Erosion Control	10	Other	5
Other (silt fences, flexible forms)	6		

Note that the predominant use of geotextiles on both continents is for roads.

Much of the early work and the impetus to use geotextiles came from the manufacturers themselves, such as ICI Fibres in England, Chemie Linz in Austria, and Du Pont and Mirafi in the United States (Koerner 1986), thus introducing geotextiles on a worldwide basis. Manufacture and use of geotextiles continues on an international basis with considerable quantities currently being used outside the country of manufacture.

2To some extent use has preceded detailed understanding by the users, but this is changing as more information becomes available.

A number of conferences have been held on the subject of geotextiles: the major ones being Paris in 1977, Las Vegas in 1982 and Vienna in 1986. Today many hundreds of papers and reports dealing with geotextiles are available. There are also numerous books and two regular journals (e.g. "Geotextiles and Geomembranes" published by Elsevier Applied Science Publishers).

The intensive research, investigations and trials of geotextiles over the last decade has advanced considerably the understanding of geotextiles and how they function in use.

To date the **reasons for using geotextiles on a project have usually been related to cost savings.** These cost savings may have been due to faster and/or cheaper construction, to reduced maintenance, or to a longer life structure. The basic motivation of cost saving is likely to continue in the future for road pavements.

Used wisely, geotextiles can now be used with confidence in many road pavement applications, but there are some applications in road pavements where detailed understanding of the actual geotextile performance mechanisms are currently not fully understood. Examples include control of reflective cracking, and thinner pavements from geotextile reinforcement. Research and trials are continuing so that understanding continues to increase. As understanding and confidence increases, the use of geotextiles is expected to continue to grow rapidly.

1.2 SCOPE

This Report provides information on the use of synthetic geotextiles in pavements of both sealed and unsealed permanent roads. Temporary unsealed roads are also covered.

The objectives of this Report are to give geotextile users an understanding of geotextiles, and to explain how they work in the various current pavement applications. This understanding revolves around consideration of the functions a geotextile will be required to serve. Improved understanding is expected to encourage the wise and thoughtful use of geotextiles. Some numerical examples are given, but the emphasis is on understanding rather than design calculations. Other texts are available to give detailed guidance on design calculations.

Of course, detailed consideration of the mechanisms by which a geotextile functions in a pavement quickly leads into considerations of soil mechanics and pavement engineering, and this report includes comment on these.

1.3 REFERENCES

Dewar, S. 1962. The Oldest Roads in Britain. *The Countryman* 59(3).

Geotechnical Fabrics Report 1983. Renewed Growth Expected for Geotextiles Throughout 1983. *Geotechnical Fabrics Report* 1(1).

Hausmann, M. R. 1984. Session 4 — General report: Soil improvement. Pp 170-176 in *4th Australia-New Zealand Conference on Geomechanics*, Pert.

Koerner, R. M. 1984. *Designing with geosynthetics*. Prentice-Hall, Englewood Cliffs, NJ.

Lawson, C. R. 1982. *Geotextiles - a short course*. Short course on soil and rock improvement techniques, including geotextiles, reinforced earth and modern piling methods. Asian Institute of Technology, Bangkok.

CHAPTER 2

TERMINOLOGY

2.1 GENERAL COMMENTS

"Geotextiles" is the term now commonly used to describe permeable textile fabrics which are employed in geotechnical engineering. They have also been termed geotechnical fabrics, filter fabrics, paving felts, and civil engineering fabrics. Geotextiles are the subject of this Report although they are but one of the various specific families that make up "geosynthetics" (Koerner 1986) which also includes geogrids, geomembranes, and geocomposites.

Geotextiles are porous to water flow across their manufactured plane, and in some cases also within their plane. While most of the commonly available geotextiles are constructed of human-made "plastic" fibres, this does not have to be so; geotextiles can be constructed of natural fibres, e.g. flax.

Geogrids represent a small but rapidly growing kind of geosynthetics. Geogrids are plastics formed into a very open netlike configuration (Koerner 1986).

Geomembranes are "impervious" thin sheets of rubber or plastic material. Geomembranes consist of any combination of geotextiles, geogrids and/or geomembranes themselves or with another material such as soil, polystyrene foam, steel cables, steel anchors, etc. (Koerner 1986).

Geocomposites include geocell (geoweb) cells/mattresses.

Terms relating to road pavements and pavement engineering used in this Report are explained in the "Road Maintenance Glossary" published by Transit New Zealand (Armitage undated).

2.2 DEFINITIONS

FABRIC, FILTER, CLOTH	Not a favoured term for geotextile.
GEOCELL	A three-dimensional structure filled with soil, thereby forming a mattress for increased bearing capacity and vehicle manoeuvrability on loose or compressible subsoils (Koerner 1986).

GEOCOMPOSITE	A manufactured material using geotextiles, geogrids and/or geomembranes in laminated or composite form (Koerner 1986).
GEOGRID	A deformed or non-deformed netlike material used with foundation, soil, rock, earth or any other geotechnical engineering-related material as an integral part of the human-made project, structure, or system (Koerner 1986).
GEOMEMBRANE	An essentially impermeable membrane used with foundation, soil, rock, earth or any other geotechnical engineering-related material as an integral part of the human-made structure or system (Koerner 1986).
GEOSYNTHETIC	The generic classification of all synthetic materials used in geotechnical engineering applications; it includes geotextiles, geocells, geogrids, geomembranes and geocomposites (Koerner 1986).
GEOTEXTILE	Any permeable textile used with foundation, soil, rock, earth or any other geotechnical material, as an integral part of human-made product, structure or system (ASTM D4439-85).
INDEX TEST	A test procedure which may contain a known bias, but which may be used to establish an order for a set of specimens with respect to the property of interest (ASTM D4885-88).
PERMEABILITY	The rate of flow of a liquid under a differential pressure through a material (ASTM D4439-85).
PERMITTIVITY	Of geotextiles. The volumetric flow rate of water per unit cross sectional area per unit head under laminar flow conditions, in the normal direction through a geotextile (ASTM D4439-85).
TRANSMISSIVITY	Of geotextiles. The volumetric flow rate per unit width of specimen per unit gradient in a direction parallel to the plane of the geotextile (ASTM D4716-87).

2.3 REFERENCES

Armitage, T. G. B. (undated). *Road maintenance glossary.* Road Research Unit Occasional Paper, Transit New Zealand.

ASTM 1985. Standard terminology for geotextiles. *ASTM D 4439-85.*

ASTM 1987. Constant head hydraulic transmissivity (in-plane flow) of geotextile related products." *ASTM D 4716-87.*

ASTM 1988. Determining performance strength of geomembranes by the wide strip tensile method. *ASTM D 4885-88.*

Koerner, R. M. 1986. *Designing with Geosynthetics.* Prentice-Hall, Englewood Cliffs, NJ.

CHAPTER 3

FIBRE AND GEOTEXTILE TYPES

3.1 GENERAL COMMENTS

The manufacture of geotextiles follows three general phases with the first being the choice of thermoplastic polymer raw material and additives to suit the environmental conditions and physical requirements. The second and third phases are the processing of polymers to form linear elements such as filaments, fibres, tapes and yarns which are then combined by an appropriate textile fabric manufacturing technology into a geotextile suitable for particular or engineering functions. Figure 3.1 shows the basic steps of the total manufacturing process. Geogrids are included under the general term "geotextile". These have similar polymer requirements but the filament phase is not necessarily required and the manufacturing technology is not that of the textile industry.

It has been impractical for manufactures to satisfy every applicant with a single geotextile type. In general, an individual manufacture has a technology that is utilised for wide range of products outside the geotextile industry but can provide a range of geotextiles with specific engineering function. Users should be wary of salespeople that claim their particular serves all purposes. Some manufacturers have a special interest in geotextiles and have made notable advances in product development. Combined products, loosely called "Geocomposites", are being manufactured to suit specific applications.

3.2 RAW MATERIALS

Fibres used in geotextile engineering are generally synthetic thermoplastics which can be manufactured with the chemical, physical and mechanical properties to suit particular applications in a ground-contact environment. Natural fibres have a tendency to biograde and are seldom utilised apart from where this property is required (e.g. temporary erosion control before vegetation growth).

In order of decreasing usage in geotextiles, Table 3.1 lists the thermoplastics materials that are dominant in the manufacturing of geotextiles and associated products.

All these materials are thermoplastic polymers and are composed of very large molecules built up in chain form from a large number of small, similar shaped units. The chemical composition of the smaller units identify the group name.

Common Name	Group Chemical Name	Symbol
Polypropylene	Polyolefin	PP
Polyester (Terylene)	Polyethylene Terephthalate	PETP
Nylon 6, 66	Polyamide	PA
Polyethylene	Polyolefin	PE
Polyvinyl Chloride	Chlorofibres	PVC

TABLE 3.1
Common Fibres Used in the Manufacture of Geotextiles

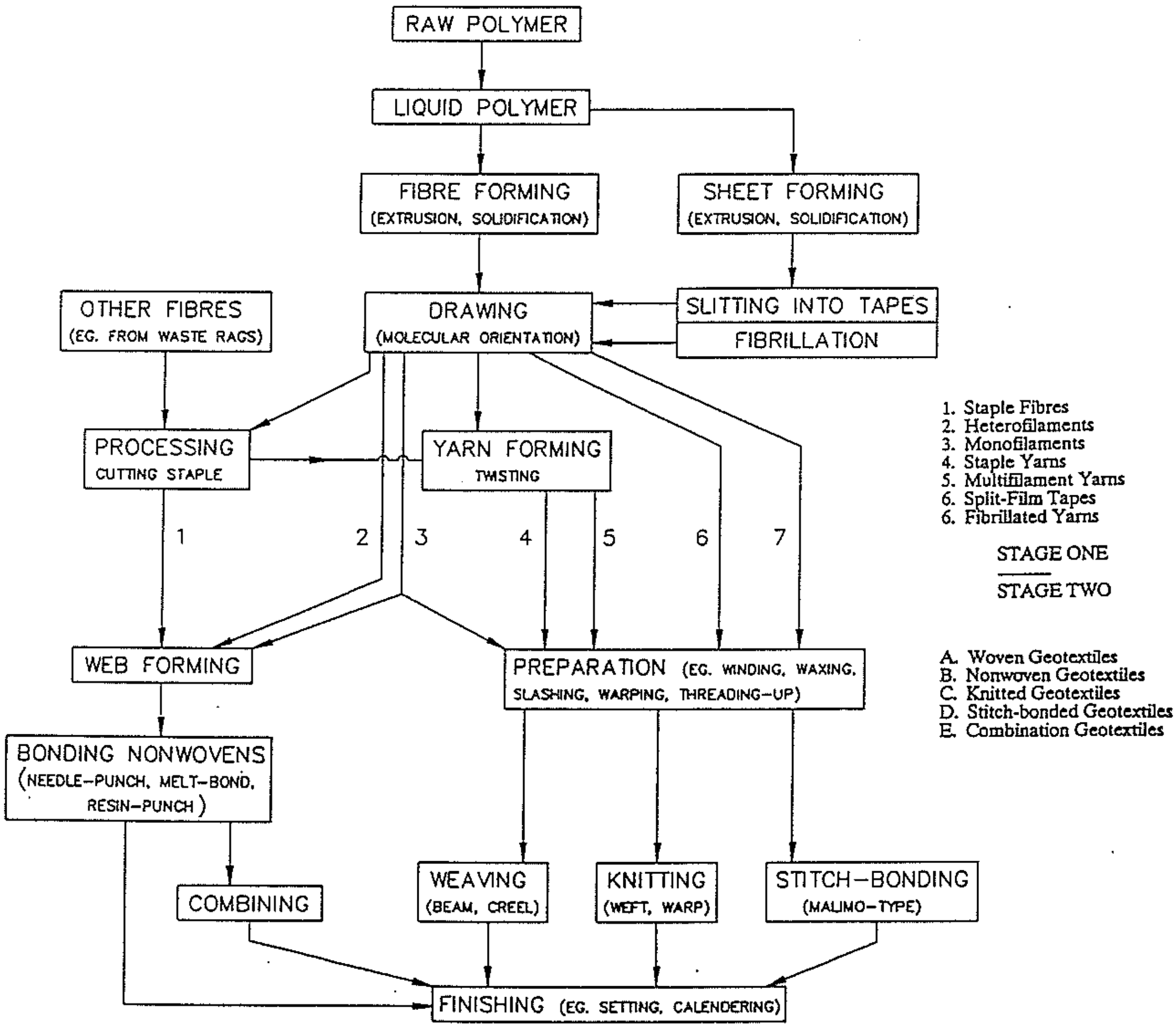


FIGURE 3.1
Simplified Outline of Steps in the Production of Geotextiles (Lawson 1986)

Fibres or filaments produced from each of the polymers have a unique range of properties, some of which are listed in Table 3.2. Manufacturers of geotextiles choose a polymer type by balancing the cost of supply and processing with the technical requirements of the engineering application and, in a number of cases, more than one polymer is used in the geotextile construction to optimise the geotextile properties and/or to facilitate the actual manufacture.

Fibre	**Ultimate Tensile Strength Mpa**	**Extension at Break %**	**Specific Gravity**	**Melting Point °C**	**Maximum Working Temp °C**
POLYOLEFINS					
Polypropylene	400-700	17-20	0.90	165	100
Polyethylene					
— low density	80-120	25-50	0.92	110	55
— high density	350-500	20-30	0.95	135	65
— ultra high density	2400-2700	3-4	0.90	165	100
POLYESTERS	500-1400	6-30	1.38	260	170
POLYAMIDE					
Nylon 6	450-470	20-40	1.13	215	70
Nylon 66	450-1000	12-30	1.14	260	130
CHLOROFIBRES					
Polyvinyl Chloride	340	20	1.38-1.40	160	80
Polyvinylidene Chloride	300-400	20-40	1.70	160	80
ARAMIDS (Kevlar)	500-2900	2-12	1.38-1.45	Decomp	450
GLASS	1750-3500	2-4	2.50	1300	1000
PRESTRESSING STEEL	1500-2000	2	7.80		

TABLE 3.2
Some Properties of Synthetic Fibres (Lawson 1986)

3.3 RESISTANCE OF GEOTEXTILE FIBRES

Although the synthetic thermoplastic fibres used for geotextile construction are chosen to have properties that generally complement engineering applications, the different molecular construction of the various polymers results in a corresponding difference in the resistance to molecular breakdown when the fibres are subjected to relatively harsh chemical or physical environments. Table 3.3 summarises the resistance of various polymers.

To a greater or lesser extent, all polymers are subject to both thermo-oxidation and photo-oxidation breakdown (brittlement) during processing and in use, but anti-oxidant additives are added to the thermoplastic in the processing stage to minimise this adverse property. UV-stabilisers are also added to the polymers where the geotextile may be exposed to ultraviolet light (UV) over long periods because the anti-oxidant alone does not give sufficient protection. In most cases, the geotextiles are not exposed to direct sunlight when in use but some polymer fibres, notably polypropylene, are less resistant than others against UV and stabiliser protection is desirable for storage and site handling.

TABLE 3.3
The Resistance of Various Geotextiles (Van Zanten 1986)

Polymer	**PP**		**PETP**		**PA (6 and 66)**		**PE**		**Soft PVC**[3]	
Loading Duration	**Short**[1]	**Long**[2]	**Short**	**Long**	**Short**	**Long**	**Short**	**Long**	**Short**	**Long**
Resistant Against:										
Dilute Acids	+ +	+ +	+ +	+	+	o	+ +	+ +	+	o
Concentrated Acids[4]	+ +	+	o	-	o	-	+ +	+	o	-
Dilute Alkali	+ +	+ +	+ +	o	+ +	+	+ +	+ +	+ +	+
Concentrated Alkali (pH >10)	+ +	+ +	o	-	o	-	+ +	+ +	+	o
Salt (Brine)	+ +	+ +	+ +	+ +	+ +	+ +	+ +	+ +	+ +	+ +
Oil (Mineral)	+	o	+ +	+ +	+ +	+ +	+	o	+	o
Glycol	+ +	+ +	+ +	o	+	o	+ +	+ +	+ +	+ +
Micro Organisms	+ +	+ +	+ +	+ +	+ +	+	+ +	+ +	+	o
UV Light	o	-	+	o	+	o	o	-	+	-
UV Light (Stabilised)	+ +	+	+ +	+	+ +	+	+ +	+	+ +	+
Heat, Dry (Up to 100°C)	+ +	+	+ +	+ +	+ +	+	+ +	o	+	o
Steam (Up to 100°C)	o	-	o	-	+ +	+	o	-	o	-
Moisture Absorption	+ +	+ +	+ +	+ +	+ +	+ +	+ +	+ +	+	+
Detergents	+ +	+ +	+ +	+ +	+ +	+ +	+ +	+ +	+ +	+ +
Tendency to Creep	+	o	+ +	+ +	+ +	+	+	o	+	o

Degree of Resistance: - Not Resistant o Moderate + Passable + + Good

This assessment of resistance is valid under normal conditions and temperatures.

1 During execution
2 During usage
3 Depends on type of softener at high relative humidities
4 Depends on type of acid and temperature

Resistance to burial deterioration has been an unknown quantity because of the relatively short history of geotextile utilisation. Results are forthcoming now that indicate that little deterioration occurs when the products are placed within soils with a typical range of in situ water chemistry and humidity (Van Zanten 1986, Colin *et al.* 1986, Halse *et al.* 1987a, b). Long-term tests of up to 12 years have been carried out on polypropylene, polyester and nylon and the losses in both the mechanical and hydraulic properties were nominal or undetectable. Geotextile filter drains in use are being studied over a period of time and no evidence of chemical or biological clogging has been apparent to date (Farrar and Samuel 1989). It is known that soft PVC can be attractive to micro-organisms but biostabiliser additives are generally added to the polymer for geotextile usage.

3.4 MANUFACTURE OF GEOTEXTILE FIBRES

The basic polymers are formed into filaments or fibres by melting and extruding them through a spinneret, a device with many fine holes. In the case of most geotextile filaments, the melt process is used where hardening is by cooling and simultaneously or subsequently they are drawn or stretched. Drawing increases the orientation of the molecular-structure within the filament structure and results in an increase in tensile strength and stress strain modulus. Draw ratios of 5:1 to 10:1 (final length : original length) are typical for geotextile type fibres. To facilitate the construction of some geotextiles, two polymers are combined in the process. Such fibres are called heterofilaments, the most common being polypropylene - polyethylene. These fibres are generalised by being called Continuous Monofilament because of their near-circular cross-section, and they form the basis for most geotextiles. Three further processes can be used in the manufacture of geotextiles:

(i) The monofilament fibres can be twisted to form a multifilament yarn.

(ii) The monofilament fibre is crimped and cut into short lengths called staples. These can be twisted and spun into a yarn for subsequent geotextile manufacture.

(iii) Instead of the spinneret, the extruded polymer is roll-formed into a thin sheet which is slit and drawn to form tape-like filaments, generally called slit-film or split-film tape fibres. These can be further split into a fibrous filament called a fibrillated yarn.

The above processing steps result in a number of different types of fibres or yarns for use in geotextile construction. These can be summarised (Lawson 1986) as follows:

Monofilament Fibres
These consist of single fibres of continuous length, generally with circular or elliptical cross-sectional shapes and produced from a single polymer component; also called monofils, homofils, homofilaments or continuous filaments. In practice, the term monofilament/monofil is usually reserved for continuous filaments with larger cross-sectional dimensions.

Heterofilament Fibres
These consist of single bicomponent fibres (i.e. two polymers) of continuous length with, in general, a skin-core or side-by-side arrangement of the two polymers in cross-section. Also called heterofils or bicomponent filaments.

Multifilament Yarns
These consist of a group or groups of continuous filaments twisted into a yarn structure; in general, a low level of twist is inserted.

Staple Fibres
These consist of short lengths, typically 50-200mm, of monofilament or heterofilament fibres; they are sometimes crimped.

Staple Yarns
These consist of staple fibres twisted into a yarn structure; a higher twist is required as compared to monofilament yarns. However, few geotextiles are made from staple yarns (also called spun yarns) because relatively low tensile strengths are realised due to the discontinuous nature of the constituent fibres.

Split-film Tapes
These consist of continuous lengths of single tapes, typically 1 to 4mm in width, and produced by splitting an extruded plastic sheet.

Fibrillated Yarns
These consist of plastic film strips which have been broken up into fibrous strands but these may still be interconnected to each other at various points along their lengths.

3.5 MANUFACTURE OF GEOGRIDS

Geogrids and meshes are constructed from a selection of similar polymers to that used for filaments and can be formed by two basic methods into Geosheets and Geowebs. The first is achieved by punching a series of holes in a thick extruded sheet of polypropylene or high density polyethylene followed by unidirectional or bidirectional drawing to form an octagonal grid (Geosheet). The second is by crosslaying thick webs of multiple filaments (Geostrips) and bonding at the crossover points (Geowebs). Any of the polymers or fibre types can be used in the construction of a geoweb. An example of the construction of geogrids is shown in Figure 3.4.

3.6 GEOTEXTILE TYPES

The resulting fibres are formed into three basic types of geotextiles by a number of manufacturing methods. The three types are (i) Woven, (ii) Knit and (iii) Non-woven, although knit fabrics are seldom used as geotextiles.

Other techniques are used to manufacture specialised products by combining the above (Combination Geotextiles) or sewing high strength fibres through the basic geotextile (Stitch-Bonding). Geotextiles, along with geogrids, are identified by their method of construction.

The methods can be summarised as follows (Hoare 1986) and comparative properties are given in Table 3.4 along with an indication of price efficiency:

3.6.1 Woven Geotextiles

Woven geotextiles are constructed by weaving two sets of filaments perpendicular to each other as demonstrated in Figure 3.2. The filaments parallel to the roll length are called the warp while those across the width are termed the weft.

Monofilament
The weft and warp fibres of the fabric are made up of single filaments, generally of circular cross-section. The filaments are relatively large in diameter and the weaving process produces a geotextile that has a medium strength, large pore sizes, and a high percentage open area that results in a high water permeability. The product is commonly used in hydraulic erosion applications where piping sands require retention.

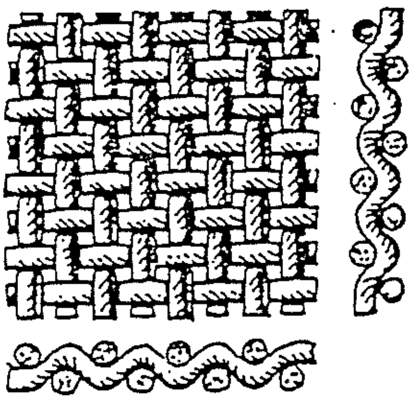
Multifilament
The weft and warp fibres of the fabric are made up of a yarn of many continuous filaments. The weaving process produces a geotextile which has a dense, high strength filament packing with a relatively low water permeability. The product is commonly used where reinforcement is required and polyester or polyamide polymer filaments are used in the stronger products. These provide high tensile strength and a high modulus, combined with low creep.

Tape
The weft and warp fibres of the fabric consist of continuous split-film tape fibres. Monofilament tape filaments are generally some 1 to 4mm in width and the weaving process produces a geotextile of medium strength with a low pore open area and a corresponding low water permeability. The product is commonly utilised in separation applications where a high permeability is not required. The products are not recommended for hydraulic applications. If the tapes are fibrillated the product can have greater strength and permeability relative to conventional tape.

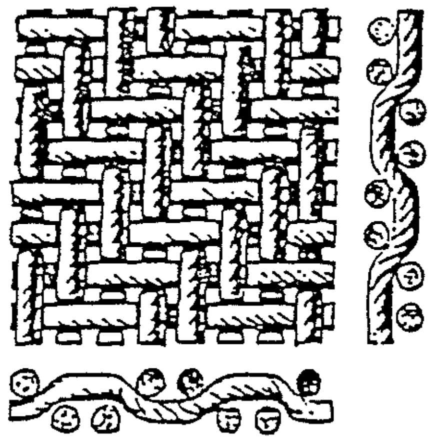
Combination
These are woven fabrics whose weft and warp fibres are made up of a combination of either tape and monofilament or monofilament and multifilament.

3.6.2 Knitted Geotextiles

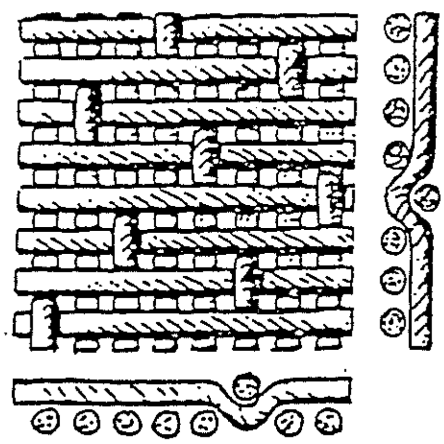
In general, knitted fabrics have structural strength in the direction across the roll (weft knit). These fabrics are generally very light and their structure is such that their physical and hydraulic properties alter when the fabric is subjected to tensile stresses in the warp direction. They also suffer from low resistance to tear propagation under stress and thus have found little use in geotechnical engineering. They have, however, been used on a small scale as a permeable wrapping around slotted subsoil drainage pipes to stop the ingress of the granular filter material surrounding the pipe.



(a) Plain Weave

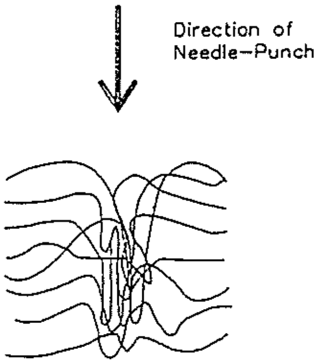


(b) A Twill Weave



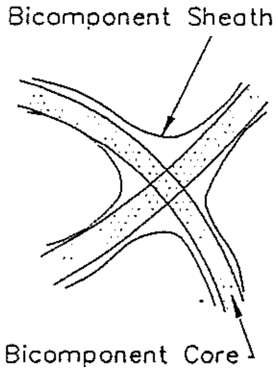
(c) A Satin Weave

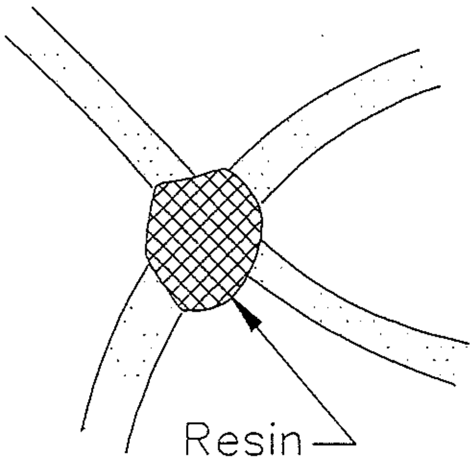
FIGURE 3.2
Schematic Diagrams of Some Woven Fabric Constructions (Lawson 1986)



(a) Needle-punched Non-woven: Mechanical Bond Formed by Entanglement of Fibres in a "Peg" or "Plug" of Fibres

(b) Melt-Bonded Non-woven: Example of Heterofilament Bond Point (Meld Point)





(c) Resin-Bonded Non-woven: Example of Resin Bond at Cross-Over Point Between Two Fibres

FIGURE 3.3
Schematic Diagrams of Bond Points in Non-woven Constructions (Lawson 1986)

A more specialised warp-knit fabric can be produced with structural strength in the direction of the roll. Heavy weight warp-knit fabrics have potential use as reinforcing layers in geotechnical structures (such as reinforced earth) because it is possible to knit high strength, high modulus, parallel fibres into the fabric.

3.6.3 Non-Woven Geotextiles

Non-woven geotextiles are constructed by bonding a loose, random (or oriented) mat or web or small diameter (50μm) continuous or staple filaments. The form of bonding differentiates the type of non-woven geotextile and these types are illustrated in Figure 3.3. Depending on the type of bonding a relatively wide range of weight grades and fabric thickness can be obtained from the process.

All non-woven geotextiles have a high porosity and a corresponding medium to high water permeability. Relative to weight/area the tensile strengths are low to medium and the elongation to failure are high compared to a woven geotextile, although the higher weight areas can approach the tensile strengths of the woven products. The pore sizes are generally low and the range is dependent on the type of bonding. The products are ideally suited for hydraulic and separation applications especially if filtration is required.

Needle-punched
This technique uses barbed needles to physically entangle either staple or continuous filament fibres into a flexible felt-like fabric. Relatively large amounts of filament are required to form a geotextile of adequate strength using this technique compared to other non-woven processes, although there is little restriction on the maximum weight or thickness of the geotextile. The weight grades can range from 130g/m^2 to in excess of 500g/m^2. A range of medium to low pore sizes is produced from this process. Higher weight grades result in a denser product of higher strength and lower pore size with a corresponding reduction in water permeability.

Melt-Bonded
Melt-bonded fabrics can be formed by either homofilament or heterofilament bonding. Homofilament melt-bonded non-wovens are produced by the preferential heat bonding of fibres of the same polymer type. Heterofilament melt-bonded non-wovens are manufactured by the preferential bonding of fibres made up of two different polymer types. The process restricts the thickness of the geotextile and weight grades are limited to a maximum of some 350g/m^2. A range of medium to low pore sizes is produced from this process. Higher weight grades result in a denser product of higher strength and lower pore size with a corresponding reduction in water permeability.

Resin-Bonded
These non-wovens are formed by impregnating a mass of filaments with a resin or glue. The resin or glue bonds the fibres together and gives the fabric dimensional stability. Doubts over the long-term durability of the bonding in a soil environment has limited the use of these products as geotextiles.

Combination
Where manufacturers have found it necessary to produce a lightweight needle-punched geotextile the fabric has required additional treatment to maintain its dimensional stability. The treatments adopted have been to combine either needle punching with melt bonding or needle punching with resin bonding.

3.6.4 Combination Geotextiles

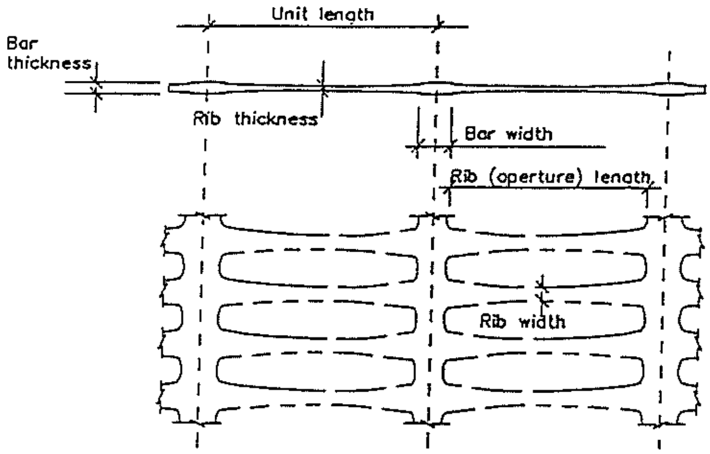
Woven fabrics with a needle-punched filling; woven fabrics with melt-bonded backing; melt-bonded and needle-punched fabrics with high tenacity fibres sewn in. Care should be exercised when choosing these products as the combination can be to the detriment of either the strength or the permeability of the separate components if standing alone. The applications are generally specific to a precise usage.

3.6.5 Geogrids, Nets and Meshes

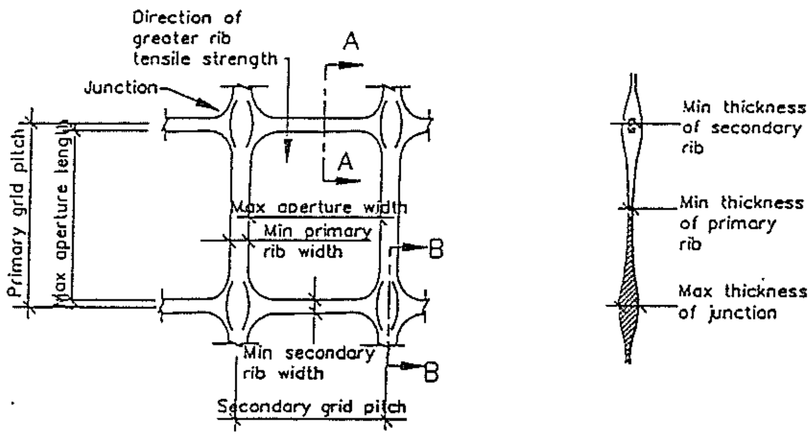
Very open mesh structures, constructed by two basic processes. Used primarily for earth reinforcement and soil stabilisation. Some examples of typical geogrids are shown in Figure 3.4.

3.6.6 Composite Geotextiles

Interwoven Webbings/Mats:	Also available with facing of melt-bonded, non-woven fabric.
Meshes:	Very open mesh structures, also incorporated as core in prefabricated drains.
Prefabricated Sheet Drains:	Combinations of woven or non-woven fabrics supported by an open mesh core.
Prefabricated Strip Drains:	Combinations of woven or non-woven fabrics supported on synthetic core or a thick fabric without a specific water-conducting core.

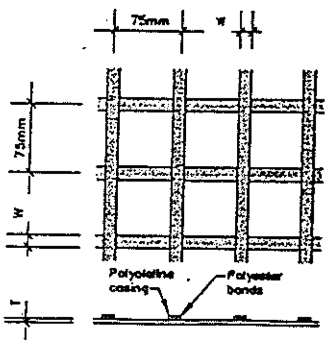


(a) Uniaxially Drawn



(b) Biaxially Drawn

Examples of Uniaxially and Biaxially Drawn Grids



Geogrid Formed by the Cross-Laying of Geostrips

FIGURE 3.4

Geogrid Construction (Koerner 1986)

Property	Woven Tapes	Woven Monofil	Woven Multifil	Needle-punched Staple	Needle-punched Cont Fil	Heat-Bonded Cont Fil
Tensile Strength	medium	medium	high	very low	low	low
Maximum Extension	low	low	very low	very high	high	medium-high
Tear Resistance	low	low	medium	very low	high	high
Impact Resistance	high	high	high	high	high	medium
Burst Resistance	high	high	high	low	medium	medium
Pore Size	medium	medium-high	medium-low	medium	medium	medium-low
Permeability	very low	high	medium-low	high-medium	high-medium	medium
Price Efficiency	high	low	high	low	medium-high	medium-high

cont = continuous
fil = filament

TABLE 3.4
General Properties of Various Geotextile Types
(Compared on the basis of mass per unit area) (Lawson 1986)

3.7 REFERENCES

Colin, G. 1986. The effect of soil burial exposure on some geotextile fabrics. *Geotextiles and Geomembranes*. 4(1):1-8.

Farrar, D. M., Samuel, H. R. 1989. Field study of filter drains at Swanley, Kent. *Transport and Road Laboratory (TRRL) Research Report* 179. 9p.

Halse, Y., Koerner, R. M., Lord Jr., A. E. 1987a. Effects of high level of alkalinity on geotextiles. Part 1: $Ca(OH)_2$ solutions. *Geotextiles and Geomembranes*. 5(4): 261-282.

Halse, Y., Koerner, R. M., Lord Jr., A. E. 1987b. Effect of high level of alkalinity on geotextiles. Part 2: NaOH Solutions. *Geotextiles and Geomembranes*. 6(4): 295-305.

Hoare, D. J. 1986. Geotextiles — compatibility and use. *Civil Engineering 1986.*

Koerner, R. M. 1986. *Designing with geosynthetics*. Prentice-Hall, Englewood Cliffs, NJ.

Lawson, C. 1986. Geotextiles or geosynthetics? *Proceedings Technical Sessions, Third Asian ICI Fibres Geotextiles Conference*, Thailand.

Van Zanten, R. V. (Editor). 1986. *Geotextiles and geomembranes in civil engineering*. J. Wiley, New York. 658p.

CHAPTER 4

TEST METHODS

4.1 GENERAL COMMENTS

To establish the suitability of a geotextile for a particular application, first define the functions that the geotextile will be required to perform in that application, second define and design the geotextile requirements, and third relate these to the relevant specific fabric properties. Test methods are needed to quantify these properties. Test methods are also required for quality control purposes by both the manufacturer and the purchaser.

4.1.1 Standard Geotextile Tests

At present no internationally agreed ISO standard geotextile tests are available. Initially many countries used tests based heavily, if not exclusively, on textile tests. Recently some purpose-written geotextile tests have been published by national organisations. Examples of these are:

USA:

- ASTM D 4354-84 "Sampling of Geotextiles for Testing".
- ASTM D 4355-84 "Deterioration of Geotextiles from Exposure to Ultraviolet Light and Water (Xenon-arc Type Apparatus) ".
- ASTM D 4491-85 "Water Permeability of Geotextiles by the Permittivity Method".
- ASTM D 4533-85 "Trapezoidal Tearing Strength of Geotextiles".
- ASTM D 4595-86 "Tensile Properties of Geotextiles by the Wide Width Strip Method".
- ASTM D 4632-86 "Breaking Load and Elongation of Geotextiles (Grab Method)".
- ASTM D 4594-87 "Effects of Temperature on Stability of Geotextiles".
- ASTM D 4716-87 "Constant Head Hydraulic Transmissivity (In-Plane Flow) of Geotextile Related Products".
- ASTM D 4751-87 "Apparent Opening Size of a Geotextile, Determining".

- ASTM D 4833-88 "Index Puncture Resistance of Geotextiles, Geomembranes and Related Products".

- ASTM D 4886-88 "Abrasion Resistance of Geotextiles (Sand Paper/Sliding Block Method)".

BRITAIN:

- BS 6906 Part 1 : 1987 "Determination of Tensile Strength of Geotextiles by Wide Width Test".

- BS 6906 Part 2 : 1989 "Determination of the Apparent Pore Size Distribution by Dry Sieving".

In the absence of internationally agreed testing criteria, engineers need to understand how geotextile properties have been evaluated. They can then make a balanced comparison between products and sets of published data, and objectively select the appropriate geotextile(s) for the intended purpose.

Geotextile tests may be one of two categories: index tests and design tests. Index test results allow for the relative qualitative comparison of geotextiles. Index tests are widely used for specification purposes but do not provide values appropriate for design. Design tests provide numerical values of properties relevant for design, and are typically performed using boundary conditions which simulate field conditions (Williams and Luna 1987).

Some of the recently published test methods are concerned with index tests and are a first step in the rationalisation of selection and design of geotextiles. Index tests are carried out on the geotextile alone, usually at relatively high speed. Index properties are not really directly applicable for design as the influence of the soil/geotextile interaction is not measured. Index properties are useful for quality control and acceptance testing.

In-soil geotextile design testing can be carried out but it tends to require complex equipment and be slow and expensive. Hence only a limited number of tests are performed.

For a given geotextile, the values obtained from index tests carried out on the geotextile in isolation do not necessarily correspond with the design-orientated results from in-soil testing. Mechanical properties tend to show a minor improvement with the latter method. In the case of hydraulic properties, however, a significant deterioration is indicated by in-soil testing as compared with in-isolation testing.

As a consequence of the lack of internationally accepted, geotextile test methods, a variety of methods are in use. Test results supplied by the manufacturers or importers may well be derived from any one of the many test methods, frequently quoted in differing units, and are difficult or impossible to relate. This complicates the comparison or selection of geotextiles.

Similar geotextiles can appear to have greatly different properties simply because different tests were used to characterise them. When considering a particular test value for a geotextile, the test method used to obtain the value must also be considered.

Laboratory test methods by which the properties of geotextiles are measured may be classified into the following groups:

- General
- Mechanical
- Hydraulic
- Survival
- Soil/Geotextile

Some of the test methods currently defined by national standards organisations are mentioned or outlined in Sections 4.2 to 4.5 below.

4.1.2 Sampling

Because of their inherent variability, sampling of geotextiles for testing should follow appropriate techniques if representative values are to be obtained. Similarly the size of a geotextile test specimen will influence the variability in the property measured. The larger the size of the test specimen, the smaller will be the variability of results between test specimens.

ICI Fibres (1988) propose that the piece of geotextile cut for testing should be taken at random anywhere along its length, and be at least 1m long by the full width of the roll wide. Test samples are obtained by folding this strip two or three times and stamping out sections, with a cutting die and press, to the required dimensions. No sample should be taken within 50mm of the edge.

To obtain a statistically significant result for a particular test method, a minimum number of tests have to be performed. The minimum number of tests required may be determined from

$$n \geq 0.154\tau^2$$

where n is the minimum of tests to give a precision of ± 5% at the 95% confidence level, and τ is the coefficient of variation of the results expressed as a percentage (ICI Fibres 1988).

ASTM D 4354-84: Instructions are given for dividing consignments of geotextiles into lots and for the determination of the number of production units in a lot.

4.2 GENERAL PROPERTIES

General properties include those which identify a geotextile by weight, geometry, material type, production method, and/or appearance.

4.2.1 Mass per Unit Area

This is a useful test for quality control, and to indicate site handling problems. For a given type of geotextile, general mechanical properties and cost tend to be directly related to mass per unit area.

One commonly used test is ASTM D 3776-85.

4.2.2 Thickness

The thickness is mainly of concern in determining hydraulic characteristics.

One commonly used test is ASTM D 1777-64(1975) which, being a textile test, gives a selection of pressures on the textile at which to measure the thickness. A normal pressure of 2kPa has been recommended for this standard to measure the nominal thickness of geotextiles.

4.2.3 Specific Gravity

The specific gravity of a geotextile can be expressed as the specific gravity of the polymer fibres or the density of the fabric.

4.3 MECHANICAL PROPERTIES

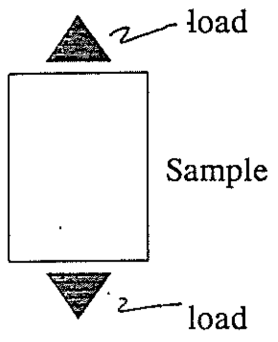
Mechanical tests describe the strength and deformation behaviour of geotextiles. Mechanical tests can be divided into two categories: those dealing with stress/strain characteristics, and those dealing with integrity or survivability characteristics. Stress/strain characteristics are obtained from tensile tests and creep tests. Integrity tests are obtained from tear propagation tests, and burst or impact tests.

4.3.1 Tensile Strength

Tensile strength is of interest in virtually all geotextile applications, whether as an indication of strength where reinforcement is one of the functions, or as one of the indicators of ability to resist installation stresses. Virtually all tensile tests assess the uni-directional stress/strain characteristics of the geotextile.

There are three types of tensile test :

- Strip
- Grab
- Plane Strain



Slowly applied load
e.g. Strip tensile
Grab tensile
Plane Strain tensile

FIGURE 4.1
Tensile Tests (ICI Fibres 1988)

4.3.1.1 Strip Tensile Test

In this type of test the rectangular sample is gripped for the full width of two opposite sides. A number of strip test methods exist with each tending to have different sample sizes and rates of loading, and the results are not comparable. The narrow sample width tests are useful for index values or quality control, while the wide sample width tests may indicate design values also. Some of the tests are described here and some of their specific limitations noted.

BS 2576 (Britain): A 50mm-wide sample is clamped to give a gauge length of 200mm, then strained at 200mm/min. Because of substantial fabric distortion it is not valid for comparing different geotextile types (non-wovens suffer Poisson's ratio effect, i.e. "Necking"). The results should not be extrapolated to kN/m.

ASTM D 1682-64 (USA): A 25 or 50mm-wide sample is clamped to give a gauge length of 75mm. Comments under BS 2576 apply. As D 1682 allows several different procedures (e.g. strip width, rate of testing, wet or dry, sample preparation), the detailed procedure actually used needs to be defined with the test results. See Section 4.3.1.2.

DIN 53857 (West Germany), AFNOR G 07-001 (France): Both use a 50mm-wide sample. The comments for BS 2576 apply as well.

ASTM D 4595-86: A 200mm-wide sample is used. The length between clamps is 100mm. Strain rate is 10%/min. While usually performed on dry geotextile samples, wet samples may also be tested. The test method defines the calculation of initial tensile modulus, offset tensile modulus, secant tensile modulus, and breaking toughness. Tensile strength and elongation are also given by this test.

The wide sample minimises the Poisson's ratio effect and provides a closer representation of expected geotextile behaviour in the field, i.e. it provides design parameters. The side effects caused by necking, while still present, have only a minor affect on the test results. This test may not be suitable for very high strength fabrics (>100kN/m) because of clamping difficulties and equipment limitations.

BS 6906 Pt 1: Uses a 200mm-wide sample clamped across its entire width. The length between clamps (gauge length) is 100mm. It is tested at a constant rate of strain of 7 to 13%/min. The test may be performed with the geotextile either wet or dry.

This British Standard quotes an accuracy of ±5% for tensile strength and ±10% of strain result. The comments made above for ASTM D4595 also apply.

AFNOR 38-014 (France): A 500mm-wide sample is clamped to give a gauge length of 100mm. It is strained at 100mm/min. Width reduction of the sample is considered in calculating the elongation at break.

4.3.1.2 Grab Tensile Test

In this type of test the rectangular sample is subjected to tensile stress through clamps that are narrower in width than the sample. The test gives an indication of the geotextile's ability to distribute concentrated loads, but results should not be used for design purposes. The test is also useful to give index values or for quality control purposes. A number of test methods exist, each tending to have different sample sizes, clamp dimensions and rates of loading and the results are not strictly comparable.

The test results are dependant not only on the strength of the tested fabric but also on its "stiffness" and type of manufacture. "Stiff" materials and non-wovens distribute the load exerted through the clamps over an area wider than the clamps giving an apparent increase in strength. Because of the small size of the sample actually tensioned, a large coefficient of variation has been found for some fabrics. For example some thin non-wovens are not consistent across the specimen, producing magnified strength variations created by the small test width.

ASTM D 1682-64: A 100mm-wide sample is clamped by 25mm-wide jaws and strained at a rate such that the specimen breaks in 20 ± 3 seconds. As this method allows some 18 different procedures, the detailed procedure actually used needs to be defined with the test results. It is primarily a textile test.

DIN 53858, AFNOR 07-120: These are similar to ASTM D 1682-64 except the strain rate for DIN 53858 is 100mm/min while for AFNOR 07-120 the time to rupture should be 20 seconds.

ASTM D 4632-86: A 100mm x 200mm sample is clamped by 25mm-wide jaws and strained at 300mm/min. The sample may be either wet or dry; though testing is normally done dry. This is an index test.

4.3.1.3 Plane Strain Test

The sample is restrained from width reduction in this test by wooden laths held across the fabric with steel pins penetrating the fabric. Because the fabric is prevented from distortion laterally the test can be used to compare the strength of different types of fabrics if test conditions are the same. The test results are not comparable with results from strip and grab tensile tests.

4.3.2 Tear Strength

There are two aspects of geotextile integrity: that dealing with tear propagation, and that dealing with tear initiation (puncture). The tear propagation resistance is a measure of the force required to propagate a tear from an existing puncture. The puncture resistance is a measure of the force, or energy, required to create that puncture. Tear propagation tests are covered here (Section 4.3.2). Puncture resistance tests are covered in Sections 4.3.3 and 4.3.4.

If the geotextile is damaged it is important that the fabric does not continue to tear. Tear propagation test methods are of two classes: non-ballistic and ballistic. Trapezoidal tear, wing tear, and tongue tear are non-ballistic, while the Elmendorf tear test, being ballistic, uses a falling pendulum. In these tests, woven and non-woven fabrics behave differently so the results of different fabric types cannot be compared. For wovens, if the yarns can move freely they tend to bunch adjacent to the tear point so increasing the tear strength. With non-wovens this tends not to happen. Also tear strength can be governed by bond failure as compared to fibre failure, i.e. fibres pulling out rather than breaking.

The tests can be used as index tests and to indicate likely tear resistance in the field. The results from the different test methods are not comparable.

4.3.2.1 Trapezoidal Tear Test

The sample is loaded so that the stress is concentrated at a cut to cause tear propagation. With the larger sample sizes it is claimed that a clean tear is obtained and the fabric deformation inherent in the smaller sample sizes is avoided. Some modification to the clamping technique may be needed for strong or glass fibre fabrics to prevent slippage. See Figure 4.2 and Table 4.1. The test parameters for test ASTM D1117-80 are similar to those shown in Table 4.1 for test ASTM D4533-85.

TABLE 4.1 Trapezoidal Tear Test Parameters

Parameter	**DIN 53363**	**ASTM D4533-85**	**AFNOR 38015**
Sample Size (length/width) (mm)	120/50	200/76	850/455
Incision length (mm)	25	15	70
Clamped length on cut side (mm)	50	25	225
Clamped length on opposite side (mm)	77	100	670
Test rate (mm/min)	100	300	50

ICI Fibres (1988) consider the Trapezoidal Tear test to be the most appropriate tear propagation test available for geotextiles because the load is applied in the plane of the geotextile, which approximates in-service stress conditions.

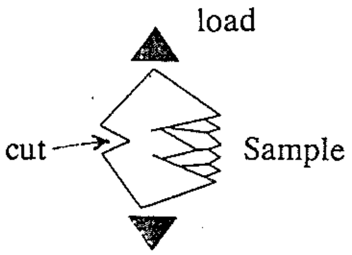


FIGURE 4.2
Trapezoidal Tear Test (ICI Fibres 1988) : Load Applied in Plane of Geotextile

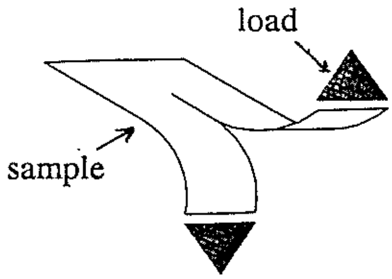
4.3.2.2 Wing Tear and Tongue Tear Tests

BS 4303 defines the Wing Tear test, and ASTM D 2261 defines the Tongue Tear test. Both are tear propagation tests with the load applied normal to the plane of the geotextile. Wing and Tongue Tear tests results tend to be higher than results from Trapezoidal Tear tests because fibres bunch up at the tear.

4.3.2.3 Ballistic Tear

ASTM D 1424 defines the Elmendorf Tear test which determines the average force required to propagate a tear starting from a pre-made cut in the woven fabric. The tear is propagated by means of a falling pendulum apparatus (Figure 4.3). The test has questionable validity for non-woven fabrics.

BS 4253 is similar to ASTM D 1424 but the results are not comparable because pendulum details, etc. differ.



Load applied normal to the plane of geotextile
Slowly applied load
e.g. Wing Tear
Tongue Tear

Dynamic applied load
e.g. Elmendorf Tear

FIGURE 4.3
Tear Propagation Tests. Load Applied Normal to the Plane of the Geotextile (ICI Fibres 1988)

4.3.3 Burst Strength

Burst strength tests are those where the geotextile is confined by a circular clamp and a load is applied slowly and normal to the plane of a geotextile. The major disadvantage of this type of test is that the test specimen is very small so a highly localised area is stressed. Hence the results may not be representative of the entire geotextile or conditions that would occur in the field. The Mullen burst and Ball burst tests are essentially index tests which are widely used for quality control. The Californian Bearing Ratio (CBR) Plunger test appears to simulate the stresses imposed on a geotextile, laid on a soft subgrade, during spreading and compaction of coarse granular material.

4.3.3.1 Mullen Burst Test

In this, a sample is held against a 30mm-diameter rubber membrane by a circular clamp. The membrane is hydraulically distended until the fabric sample fails. A disadvantage of this test is that only a highly localised area is stressed. Also the method cannot test high strength fabrics.

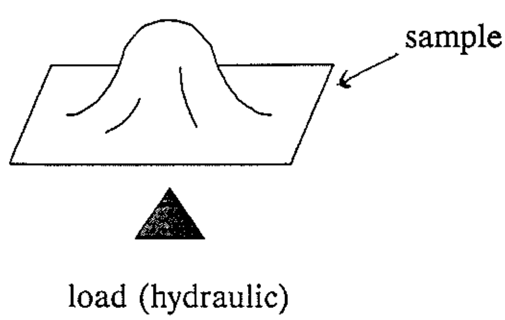


FIGURE 4.4
Mullen Burst Test (ICI Fibres 1988)

ASTM D 3786: A 100mm-diameter sample is held by an circular clamp of 30mm-inner diameter.

BS 4768, DIN 53861: Similar to ASTM D 3786.

4.3.3.2 Ball Burst Test

ASTM D 3787: Similar to the Mullen Burst test except a 25mm-diameter steel ball is forced through the geotextile. A disadvantage of this test is that only a highly localised area is stressed. It is not recommended for woven fabrics.

4.3.3.3 CBR Plunger Test

This test can be considered as an axisymmetric strength test and can be used to indicate tensile strength (Murphy and Koerner 1988, Myles 1987). It is considered here, however, as a puncture test.

DIN 54307 E: Using CBR Plunger test apparatus, the geotextile is clamped between two steel rings of 150mm inside diameter and the 50mm-diameter plunger is pushed against the fabric at a constant strain rate of 50 mm/min (Figure 4.5).

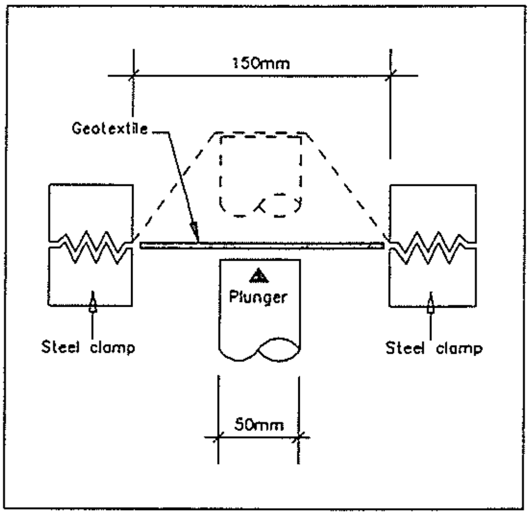


FIGURE 4.5
CBR Plunger Test According to DIN 54307E
(ICI Fibres 1988)

4.3.3.4 ASTM Puncture Test

ASTM D 4833-88: The test specimen is clamped without tension between circular plates. An 8mm- diameter steel rod is forced (at a rate of 300mm/min) against the unsupported centre part of the geotextile until rupture occurs. The maximum force is the puncture resistance. This is an index test and it may be inappropriate for some woven geotextiles with large pore sizes.

4.3.4 Impact Tests

These tests have been developed to assess the impact puncture resistance of a geotextile to sharp edged or pointed rocks that may fall on it during installation.

4.3.4.1 Drop Cone Test

The fabric is clamped between two steel rings and a cone is dropped point first. Typically the cone weighs 1 kilogram, has a 45° point, a maximum diameter of 50mm, and is dropped 500mm. The diameter of the resulting hole is measured.

It is an extreme test as it provides no soil support under the geotextile. The test only gives significant results for non-woven geotextiles with unit weights in the range 140-300g/m^2. For non-woven geotextiles less than 140g/m^2 the energy developed by the falling cone enables it to pass straight through the geotextile. For non-woven geotextiles heavier than 300g/m^2 and for all woven geotextiles, the energy developed by the falling cone is insufficient to provide measurable variations (ICI Fibres 1988).

4.3.5 Creep

In many applications the geotextile will be subjected to either sustained loads or repeating loads. Since the polymers used in geotextiles tend to be creep-prone materials, it is an important property to evaluate. The test specimen should be of the 'wide width' variety. In principle, the wide-strip tensile test procedure can be adopted for the following three time-dependent tests:

i. Creep — in which a constant load, usually by means of hanging weights, is applied to the geotextile and elongation is measured as a function of time.

ii. Stress relocation — in which a constant elongation is applied to the geotextile and the load is monitored as a function of time.

iii. Cyclic loading — in which the geotextile is subjected to a repetitive load cycle with both strain and load continuously recorded.

4.4 HYDRAULIC PROPERTIES

Hydraulic testing is performed to determine the effectiveness of the geotextile as a filter or as a drain. The geotextile factors of primary interest are permeability and filtration.

Permeability is obtained by permittivity and transmissivity tests: Permittivity tests measure the flow of water through a geotextile normal to its plane; Transmissivity tests measure the flow of water within the plane of the geotextile.

Filtration tests measure the compatibility of the soil and geotextile to ensure water can flow through the geotextile without soil particles passing through it as well, and that the soil particles do not excessively restrict the flow of the water through the geotextile.

The hydraulic tests usually applied to geotextiles fall into three groups: apparent opening size tests, normal permeability tests, and planar permeability tests. Because traditional textile tests did not measure hydraulic properties, new and original test methods are being developed. See also Sections 4.4.2 and 4.4.3.

4.4.1 Apparent Opening Size

The apparent opening size (AOS) of a geotextile is a measurement of its effective pore channel diameters. Once the pore size distribution of the geotextile is obtained the apparent opening size can be determined. The AOS is usually expressed as a percentage of the distribution of pore size, e.g. O_{95} refers to the pore size below which 95% of the geotextile's pore sizes lie.

A number of test methods have been developed to determine the pore-size distribution, and thence AOS, of geotextiles. The methods developed include sieving, impregnation, suction, and visual means. Because of the limitations of the various techniques, one single method cannot be used to measure the pore-size distribution of all geotextiles. Moreover, each of the different test methods will produce different pore-size distribution results for the same geotextile sample. The range of pore sizes for which each measurement technique is applicable (ICI Fibres 1988) is shown in Figure 4.6.

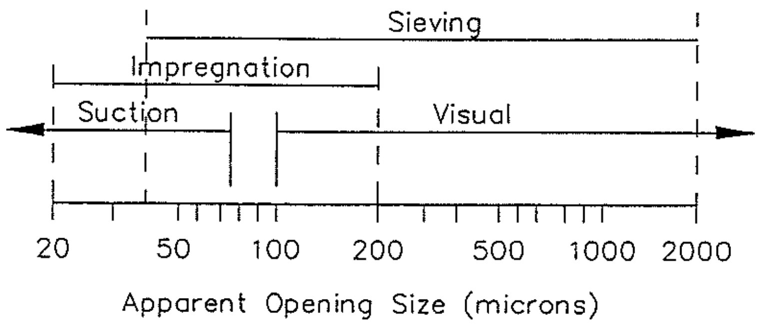


FIGURE 4.6
Approximate Ranges for Geotextile Pore Size Determination Tests (ICI Fibres 1988)

The most commonly used pore-size measurement technique is sieving. This involves sieving particles of known size through the geotextile. Both dry sieving and wet sieving methods have been used.

Dry sieving utilises sand fractions or glass beads. These are sieved through the geotextile in order of descending particle size fraction. The amounts of each particle size fraction retained on the geotextile give the pore-size distribution.

Sieving using glass beads can be strongly influenced by electrostatic forces and dampness. It is impractical to use beads smaller than about 30μm.

Sieving with sand fractions overcomes some of these disadvantages but requires careful attention to ensure the test method is followed; the smallest particle that can be sieved is about 40μm. For the draft Australian Standard it has been proposed that two sand sieving methods be adopted — dry sieving and wet sieving. Dry sieving is more applicable to woven, knitted, stitch-bonded and thin non-woven geotextiles, while the wet sieving method is more suited to thick needle-punched geotextiles.

Visual methods require measurements of pore spaces either directly visually, or from enlarged photographs. While this may be valid for monofilament wovens and thin non-wovens, it is not suitable for thicker non-wovens.

The different pore-size measurement methods described here can give markedly varying results on the same geotextile.

AFNOR 38-017: The grain size distribution of soil flushed through the geotextile is determined. The test is conducted in a pan fitted with a geotextile bottom, that is filled with the soil sample. The pan is immersed in a water-filled trough and pulled out at regular intervals. At the end of the test the particle size distribution of the soil collected in the water trough is established by sieving and sedimentation analysis.

While this is a time-consuming test requiring special apparatus, it has some resemblance to field conditions.

TNZ(NRB) T/6 (New Zealand): Single sized sand particles are placed on the geotextile sample which is then agitated. This is repeated for a range of sand sizes. The pore-size distribution is given by the sand sizes passing through the geotextile.

ASTM D 4751-87: Single-sized glass beads are placed on the geotextile sample which is then agitated. This is repeated for a range of bead sizes. The pore-size distribution is given by the weights of the glass bead sizes passing through the geotextile. As the results can be significantly influenced by damp beads or static electricity the test method goes to some length to avoid these conditions.

BS 6906 Part 2: Dry spherical lead glass beads are placed on the geotextile sample which is then agitated. This is repeated for a range of bead sizes. The pore-size distribution is given by the glass bead sizes passing through the geotextile. The test method goes to some length to ensure that the beads are dry.

4.4.2 Permeability Normal to the Geotextile

Permittivity tests measure the water flow rate normal to the plane of the geotextile.

The most common equation explaining flow through a porous media is Darcy's Law. Darcy's Law assumes laminar flow through the medium, and this approximates the case in thick non-woven geotextiles.

Thin non-woven, knitted, stitch-bonded, and most woven, geotextiles however can experience semi-turbulent flow conditions. The applicability of Darcy's Law to geotextiles is discussed in Van Der Sluys and Dierickx (1987). However, it is becoming common to assume that flow through geotextiles obeys Darcy's Law as follows:

Flow rate $q = k\,i\,A$

and express the relationship between flow rate and hydraulic gradient in terms of permittivity:

$$q = k\,(\Delta h/t)\,A$$

or

$$\frac{k}{t} = \frac{q}{(\Delta h)\,(A)}$$

where:

q	=	flow rate
Δh	=	head lost
A	=	area of fabric under test
k	=	permeability coefficient
t	=	thickness of the geotextile
$\frac{k}{t}$	=	permittivity
i	=	hydraulic gradient

Permittivity is defined by ASTM D 4491-85 as — the volumetric flow rate of water per unit cross sectional area per unit head, under laminar flow conditions in the normal direction through a geotextile.

The permeability of geotextiles, particularly thick non-wovens, is affected to some degree by external stresses, such as overburden pressures from soil above the geotextile.

Since the flow through the geotextile for the ASTM D4491-85 and TNZ(NRB) T/7 tests should be laminar, the results from the test methods should be comparable using Darcy's Law if the thickness of the geotextile is known.

If laminar flow is not occurring, describing the permeability in terms of volume flow rate at a specific constant water head will give results that can easily be used to compare different geotextiles tested using the same test (ICI Fibres 1988).

ASTM D 4491-85: This includes two test methods: the Constant Head Method and the Falling Head Method. They are performed on the geotextile in isolation in its uncompressed state using de-aired water.

Constant Head Test — a head of 50mm of water is maintained on the geotextile throughout the test and the quantity of flow is measured versus time. The test should only be performed under laminar flow conditions through the geotextile. The test method includes a check that this is the case. If not, the constant head should be adjusted between 10 and 75mm if this will give laminar flow. The Constant Head Test is used when the flow rate of water through the geotextile is so large that it is difficult to obtain readings of head change versus time in the Falling Head Test.

Falling Head Test — a column of water is allowed to flow through the geotextile and the head change versus time is measured. The flow rate of water through the geotextile must be slow enough to obtain accurate readings.

ASTM D 4491-85 states that data have shown agreement between the constant head and falling methods.

SN 640550 (Switzerland): The Swiss tests are conducted with de-aired de-mineralised water with one or more layers of geotextile with normal loads of 20 and 200 kN/m^2. At 20 kN/m^2 the hydraulic gradient should be 10, at 200 kN/m^2 the hydraulic gradient should not be more than 20.

TNZ(NRB) T/7: Permeability normal to the plane of the geotextile is measured by this test. The head loss is measured at a water flow rate of 10mm/second through one layer of geotextile. Permeability measured by this test is expressed as millimetres head loss at a flow velocity of 10mm/sec.

4.4.3 In-Plane Permeability

For geotextiles used for drainage internally, within the plane of the geotextile, the measurement of water permeability is important. Two common expressions are used for planar permeability: transmissivity and volume water flow.

Transmissivity is determined similarly to permittivity and is expressed as:

$$\Theta = k_p t$$

where:

Θ = transmissivity
k_p = Darcy's permeability coefficient in the plane of the fabric
t = thickness of the geotextile

Strictly speaking, transmissivity should only be applied to laminar flow situations, where Darcy's coefficient of permeability applies. This is true for thick needle-punched geotextiles but may not be true for the composite drainage geotextiles.

Transmissivity of geotextiles is measured by two basic methods: radial flow and parallel (rectangular) flow. In the radial flow method the water is inserted in the centre of a circular geotextile sample and collected from the circumference. One major disadvantage of radial flow tests is that they cannot determine transmissivity directional variability in the geotextile sample. In parallel flow tests, water is introduced at one end and collected at the other.

Single geotextile layers and multiple geotextile layers have been used in both test methods. However multiple layers of geotextiles may not give the same value per geotextile as one layer; one reason would be that water passes between the individual geotextile layers.

One unknown factor is the adequacy of laboratory tests to represent the in-situ effects of soil migration into the geotextile on its ability to drain within itself. Heerten (1982) found that thick needle-punched geotextiles, which exhibit relatively high laboratory transmissivities, after being buried in soil, contained up to eight times their original mass per unit area in soil particles trapped within their structure, resulting in drastic reductions in transmissivity. When the geotextile is placed in the soil it will be subject to earth pressures. These pressures can dramatically reduce geotextile planar permeability as well.

Planar water permeability can also be expressed in terms of volume water flow. These volume water flow tests are more commonly carried out on composite drainage materials (geotextile filters separated by a plastic spacer) because the flows along these may be turbulent or semi-turbulent. Then Darcy's Law does not apply. Volume flow rates are usually expressed in terms of volume of water passing through, per unit width of geotextile, per unit time, per unit water head.

ASTM D 4716-87: This is a parallel flow test. The hydraulic gradient and compressive stress are selected to model a given set of field parameters. It is intended as either an index test or a design aid.

4.5 SURVIVAL PROPERTIES

Survival testing of geotextiles aims to indicate their long-term response to chemical or environmental exposure, i.e. their durability. There are very few national standard tests for durability at present. Tests used or under consideration tend to compare the strength loss of a sample exposed to accelerated ageing, etc. to a control sample. Thus the results are comparative and indicative rather than proven by long-term performance.

The durability of a geotextile is largely a function of the durability of its components, i.e. its fibres and resins.

4.5.1 Ultraviolet Light

Exposure to ultraviolet light (UV) is a major cause of loss of strength of a geotextile. UV stabilisers in the fibre structure only delay the degradation, they do not stop it.

A range of apparatus for simulating natural ultraviolet degradation is available. Sources of radiation are mainly xenon burners, carbon arcs and special fluorescent tubes. The mechanical property which is affected and measured as a function of the duration of exposure is the tensile strength.

ASTM D 4355-84: This test exposes the geotextile to ultraviolet light in a xenon-arc apparatus. For part of the time the sample is also sprayed with water. Three different exposure time tests enable a degradation curve to be drawn.

This test does not simulate all practical variables so the results are unlikely to relate directly to a specific site. It is not an index test, but is intended to indicate to users the tendency of a geotextile to deterioration under UV light.

4.5.2 Chemical and Microbiological

Organic and inorganic chemicals in the soil may affect the chemical composition of the geotextile by oxidation and/or hydrolysis. Limited tests have shown that geotextiles have generally good resistance to attack by acids, alkalis, oxidising agents and microbiological organisms. At the present time there are no standard geotextile test methods for the evaluation of chemical or microbiological resistance.

It is usual to rank geotextiles for this on chemical and microbiological resistance data for the constituent fibre polymers.

4.5.3 Temperature

When a geotextile is used in a chip seal or asphaltic concrete overlay, exposure to the hot bitumen or hot mix can also cause loss of strength if the polymers melt. Geotextiles may be exposed to abnormally high temperatures during shipping or storage.

ASTM D 4594-87: This test determines the effects of climatic temperature on tensile strength and elongation properties of geotextiles. The test temperatures are selected to be typical of those which the geotextile will be exposed to in the field but within the range -127°C to +100°C.

ASTM D 794: This test applies continuous heat in an oven until the geotextile fails. An alternative method in this standard uses cyclic heat. Failure is defined as a change in appearance, weight, dimension or other property.

4.5.4 Abrasion

Heavy repetitive loads, such as under railway ballast or under sub-base on a road, induce a rubbing motion between the aggregate and the geotextile. Abrasion testing may subject a geotextile to cyclic rubbing.

ASTM D 4886-88: In this test the stationary geotextile is rubbed with sand paper under controlled conditions of pressure and abrasive action. The weight loss indicates the abrasion resistance.

4.6 SOIL — GEOTEXTILE PROPERTIES

All specific tests discussed up to this point have been for the geotextile in isolation. In situ the geotextile forms a composite system with the soil.

The effects of confining pressures, the effects of soil at the interface and the effects of soil particles penetrating into the geotextile may all contribute to the soil-geotextile composite properties being different to the geotextile in isolation properties.

To a large extent soil-geotextile interaction tests are models of actual or proposed field applications, particularly if care is taken to duplicate hydraulic gradients, ground water chemistry, etc. Although many possible soil-geotextile interaction tests may be performed, most fall into one of three major areas: in-soil mechanical behaviour of the geotextile; friction characteristics at the soil-geotextile interface; and the filtering characteristics of the soil-geotextile composite.

While many different soil-geotextile test procedures have been used, there are no national standard methods in existence.

4.6.1 In-Soil Mechanical Properties

4.6.1.1 Stress-Strain Characteristics

When embedded in a soil the geotextile experiences confining pressures and friction which inhibit fibre mobility and any consequent fibre realignment. This can give significant differences between in-soil and in-isolation values. Work by McGown *et al.* (1982) showed needle-punched non-wovens to display the greatest difference between in-soil and in-isolation stress strain properties. Melt-bonded non-wovens were also significantly affected. In general the differences are more pronounced for non-woven geotextiles than for woven geotextiles.

McGown *et al.* (1981) reported a device to enable the load-extension behaviour of geotextiles to be measured while the geotextile was subjected to a normal compressive stress through a soil medium. This showed that the load extension behaviour of the two wovens tested was significantly affected by the orientation of the test samples but was not influenced by the level of compressive stress. In contrast the two non-wovens tested proved to be essentially isotropic but the effect of confining pressure on the load-extension properties was quite marked.

4.6.1.2 Creep

McGown *et al.* (1982) used the same stress-strain device for creep testing. The test compared the in-soil creep results with unconfined in-isolation creep results, and found for the geotextiles tested that in-isolation testing grossly over-estimated long-term strains.

4.6.1.3 Soil-Geotextile Friction

Two types of soil-geotextile friction tests can be performed: shear box and pull-out tests. Pull-out tests — the geotextile is pulled out of the confining soil at a constant rate of strain; Shear box test — the confining soil is moved over the geotextile sample. Ingold (1983a, b) found that the results of these two tests are not comparable, with the pull-out method tending to give higher values.

The soil-geotextile friction is controlled primarily by the geotextile's physical form together with the soil type and confining pressure. Thin fabrics with relatively large openings, such as some of the wovens, can develop very high friction. Indeed there may be actual physical interlock. With heavier and thicker fabrics the soil particles do not penetrate the geotextile to contact the soil on the other side. If these fabrics have a relatively smooth surface then the friction is mostly due to the soil particles sliding on the fabric surface. If the fabric surface is matted then some interlocking between the surface fibres and the soil can be expected.

Research has indicated that in loose sand the geotextile-soil friction angle is approximately equal to the friction angle of the soil. This applies for non-woven fabrics without slick finishes and for woven fabrics where the soil grains are approximately the same size as the fabric pore sizes (Haliburton *et al.* 1978, Holtz 1977).

For materials with larger openings than the soil grains, studies by Ingold (1983a, b) indicate that the pull-out resistance is a function of cumulative strand area normal to the direction of the pullout. Haliburton *et al.* *(1978)* indicated that the geotextile-soil friction angle in dense sand will be somewhat less than the angle of internal friction. For rounded soil grains that are larger than the pore openings in the fabric, a friction angle somewhat less than the soil angle has been found (Collios *et al.* 1980). If the aggregate particles are sharp and angular they may generate high frictional resistance. However since well defined relationships do not appear to exist, testing should be performed when designers require values for specific conditions.

4.6.2 Geotextile Soil Filtration

In real geotextile-soil filtration situations, geotextiles can clog or blind. Clogging is the term usually applied to the penetration and entrapment of soil particles within the geotextile structure. Blinding usually refers to soil particles blocking the entrances or openings at the geotextile surface. Both clogging and blinding reduce the geotextile permeability.

To indicate the long-term permeability of a geotextile-soil system a permeameter may be used. In this the soil is placed on top of the geotextile. The soil sample is usually remoulded into the permeameter but with care "undisturbed" soil could be used. A hydraulic gradient is then applied. The head and flow rate of the water is recorded, any soil particles passing through the geotextile are dried and weighed. Reasonable assessments of long-term filter performance can be obtained from this test, but selecting the water to be used in the test needs to be appropriate (e.g. tap water, de-aired, water with deflocculating agent, or actual ground water from proposed drain location). Inappropriate choice of water can give erroneous results, e.g. a deflocculating agent will break up soil particles which in reality will clump together and so be easier to filter.

A variation on the permeameter test is to omit the soil and use a water/fine soil particle slurry. This has the advantage of speeding up the test and will usually give conservative test results (i.e. overestimate clogging and blinding). However this variation has not been fully evaluated or reported at the time of writing so the soil-geotextile test is preferred.

To fully evaluate the in-soil filtration requirements requires consideration not only of the actual soil particle sizes and the geotextile, but also of the likely tendency of the soil particles themselves to pipe i.e. be transported under the in situ hydraulic gradients. This is covered in detail in East and Hudson (1987).

4.7 REFERENCES

This list includes additional reading.

Ball, J. 1982. The development of test methods by ASTM. *Proceedings of Second International Conference on Geotextiles,* Las Vegas, USA.

Chemie Linz 1986. *Polyfelt TS. Design and Practice.* Chemie Linz AG, Austria.

Christopher, B.R., Holtz, R.D. 1985. *Geotextile engineering manual - course text.* National Highway Institute, Federal Highway Administration, Washington, DC.

Collios, A., Delmas, P., Gourc, J.P., Giroud, J.P. 1980. *Experiments on soil reinforcement with geotextiles. The use of geotextiles for soil improvement.* ASCE National Convention, Portland, Oregon.

East, G.R.W., Hudson, K.C. 1987. Geotextile subsoil drains: development towards their use in fine grained soils. *New Zealand Roading Symposium* 4: 135-148.

Haliburton, T.A., Anglin, C.C., Lawmaster, J.D. 1978. Testing of geotechnical fabric for use as reinforcement. *ASTM Geotechnical Testing Journal* 1(4): 203-212.

Heerten, G. 1982. Dimensioning the filtration properties of geotextiles considering long-term conditions. *Proceedings of Second International Conference on Geotextiles,* Las Vegas, USA.

Hoare, D.J. 1978. Permeable synthetic fabric membranes 2 : factors affecting their choice and control in geotechnics. *Ground Engineering* 11(8): 25-31.

Hoare, D.J. 1983. Geotextiles in the UK. *Ground Engineering* 16(5): 30-38.

Holtz, R.D. 1977. Laboratory studies of reinforced earth using a woven polyester fabric. *Proceedings of International Conference on the Use of Fabrics in Geotechnics,* Paris.

ICI Fibres 1981. *A guide to test procedures used in the evaluation of civil engineering fabrics.* ICI Geotextiles, Pontypool, Wales.

ICI Fibres 1988. *Designing for subsurface drainage.* ICI Geotextiles, Pontypool, Wales.

Ingold, T.S. 1983a. *Some observations on measurements in the laboratory of soil-geotextile bond.* Bordas - Dunod.

Ingold, T.S. 1983b. Laboratory pull-out testing of grid reinforcements in sand. *ASTM Geotechnical Testing Journal* 6(3): 101-111.

Koerner, R.M. 1986. *Designing with geosynthetics.* Prentice-Hall, Englewood Cliffs, NJ.

Koerner, R.M., Bove, J.A. 1983. In-plane hydraulic properties of geotextiles. *ASTM Geotechnical Testing Journal* 6(4): 190-105.

Lawson, C.R., Curiskis, J.I. 1985. *Geotextiles : including manufacture, properties, and use in pavements, embankments, subsurface drainage and erosion control.* Lawson Consulting, Sydney, Australia.

McGown, A., Andrawes, K.Z., Wilson-Fahmy, R.F., Brady, K.C. 1981. A new method of determining the load-extension properties of geotechnical fibres. *Transport and Road Research Laboratory, Supplementary Report 704.* 14p.

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, USA.

Murphy, V.P., Koerner, R.M. 1988. CBR strength (puncture) of geosynthetics. *ASTM Geotechnical Testing Journal* 11(3): 167-172.

Myles, B. 1987. A review of existing geotextile tension testing methods. Pp. 57-68 in *Geotextile testing and the design engineer,* J.E. Fluet Jr (Ed.). *American Society for Testing and Materials STP* 952. 183p.

Suits, L.D., Carroll Jr, R.G., Christopher, B.R. 1985. ASTM Geotextile Committee testing update. *ASTM Geotechnical Testing Journal* 8(4): 191-198.

Van der Sluys, L., Dierickx, W. 1987. The applicability of Darcy's Law in determining the water permeability of geotextiles. *Geotextiles and Geomembranes* 5(4): 282-299.

Wates, J.A., Wittstock, G.H. 1986. Selection of geofabrics for filtration applications with particular reference to gold tailings. *The Civil Engineer in South Africa,* March 1986.

Williams, N.D., Luna, J. 1987. Selection of geotextiles for use with synthetic drainage products. *Geotextiles and Geomembranes* 5(1): 45-61.

Wittstock, G.H., Emery, S.J., Savage, J. 1984. Towards the standardisation of geotextile test methods and parameters in South Africa. *The Civil Engineering Contractor,* October 1984.

CHAPTER 5

TYPICAL PROPERTIES OF GEOTEXTILES

5.1 GENERAL COMMENTS

The use of geotextiles in civil engineering requires a knowledge of the mechanical, hydraulic and durability properties of the various products to make a suitable choice for the intended application. A range of typical general properties is shown in Table 5.1 although it should be emphasised that some properties are dependent on the polymer used and the precise method of manufacture (Chapter 3). The properties quoted by the manufacturer are from laboratory tests to a prescribed standard and do not necessarily reflect end use values. Chapter 4 relates the appropriate test methods. Some judgement is required to assess the end use performance and this will be addressed in later chapters.

New Zealand has a history of using light to medium weight woven-tape geotextiles in preference to non-woven and heavy monofilament woven products or geogrids primarily due to cost and availability but, in recent times, the latter geotextiles have become readily available and cost competitive. Compromises with the required properties need not be made with the range of geotextiles available today.

This chapter reviews the properties of the geotextiles readily available in New Zealand. It does not cover the less common and more specialised products such as resin-bonded and knitted geotextiles. Some indication of the range in properties is illustrated in Table 5.1 but any user of such products should consult the supplier for detailed properties.

5.2 PHYSICAL PROPERTIES

The physical properties of a geotextile refer to the manufactured textile condition as distinct from the filament or polymer properties. The two principal physical properties are weight and thickness. Both are used as a comparative index for geotextile products.

5.2.1 Weight

The unit "weight" is the mass per unit area of a geotextile in the manufactured condition in units of g/m^2. The range of typical values for most geotextiles is from 100 to 700 g/m^2.

Note: Some care is required when comparing the unit weight of a geotextile as the specific gravity of the construction polymer may differ, e.g. a polypropylene geotextile

(SG = 0.9) will have 1.5 times the thickness of an equivalent unit weight polyester (SG = 1.38) product. This does not markedly affect the comparison of the mechanical properties of polypropylene and polyester geotextiles with unit weight as the polymer filament of the latter has a proportionally higher strength. This condition is also applicable to polyethylene (SG = 0.95) and nylon (SG = 1.13) products but some care is required when comparing PVC geotextiles (SG = 1.4) which have a relatively low filament strength.

5.2.2 Thickness

The thickness of a geotextile is the distance between the upper and lower surfaces of the product, measured at a specified pressure. The pressure for this measurement is generally 2.0kPa but some manufacturers also give values for other pressures. Thickness of commonly used geotextiles range from 1mm to 3mm.

5.3 MECHANICAL PROPERTIES

The mechanical properties indicate the geotextile's resistance to mechanical stresses mobilised from applied loads and/or installation conditions. The mechanical tests can be divided into short term and long term and it is perhaps unfortunate that only the former has formal test standards from which results can be quoted.

5.4 SHORT-TERM MECHANICAL PROPERTIES

The short-term mechanical tests applied to geotextiles may be classified in three categories, depending on the direction of load application relative to the plane of the geotextile and the nature of the test: tension tests, tear propagation tests, and burst tests (Chapter 4). However, although there are three categories of mechanical testing, only two geotextile mechanical response characteristics are measured: stress-strain characteristics (tensile tests), and integrity characteristics (tear propagation, burst, and impact procedure tests). Some geotextiles, notably geogrids, are unsuitable for the standard integrity tests.

Depending on the construction, the maximum peak strength of a woven geotextile may be in the warp direction (along length of geotextile) whereas non-woven geotextiles can be considered isotropic.

TABLE 5.1
Range of Values for Some Representative Properties of Available Geotextiles
(Lawson 1985)

Geotextile Construction	Unit Weight (g/m^2)	Tensile Strength (kN/m)	Maximum Extension (%)	Apparent Opening Size (mm)	Volume Water Permeability ([1]litres/m^2/s)
Wovens					
Monofilament	150-300	20-80	5-35	0.07-2.5	25-2000
Multifilament[2]	250-1300	40-600	5-30	0.2-0.9	20-80
Tape	100-250	8-90	15-20	0.05-1.25	5-20
Non-wovens					
Melt-Bonded	70-350	3-25	20-60	0.03-0.35	25-150
Needle-Punched	130-700	7-35	50-80	0.05-0.25	25-400
Resin-Bonded	130-800	4-30	30-50	0.01-0.35	20-100
Knitteds					
Weft		2-5	300-600	0.1-1.2	60-800
Warp		20-120	12-15		
Stitch-Bonded	250-1200	15-800	15-30	0.04-0.4	30-80
Geogrids	150-900	10-200	3-25	25-75	very high

1 Normal to the plane of the geotextile with 10cm constant head
2 Fibrillated tapes are included in this category

5.4.1 Stress-Strain Characteristics

For tensile tests, the load is applied in the direction of the plane of the geotextile. The three types of tensile tests used have been illustrated in Chapter 4: strip, grab and plane strain tensile tests.

The peak strength of geotextiles for a given construction is broadly proportional to the unit weight (mass per unit area). A general relationship is shown in Figure 5.1 and it can be seen that, for a given weight, woven geotextiles have at least twice the peak strength of a melded or needle-punched non-woven. The construction of non-woven geotextiles limit the maximum peak tensile strength to the order of 35kN/m for the heavy weight grades (550g/m^2) whereas some of the woven multifilament polyester reinforcement geotextiles can achieve a peak of 600kN/m in the warp direction.

In general, woven geotextiles exhibit peak strengths at a low elongation of 10% to 30% in comparison to 30% to 90% for non-woven products. For this reason, the extension modulus is significantly higher for woven geotextiles. A typical range of stress-strain results for various geotextiles normalised to 300g/m^2 weight is shown in Figure 5.2 (McGown 1976).

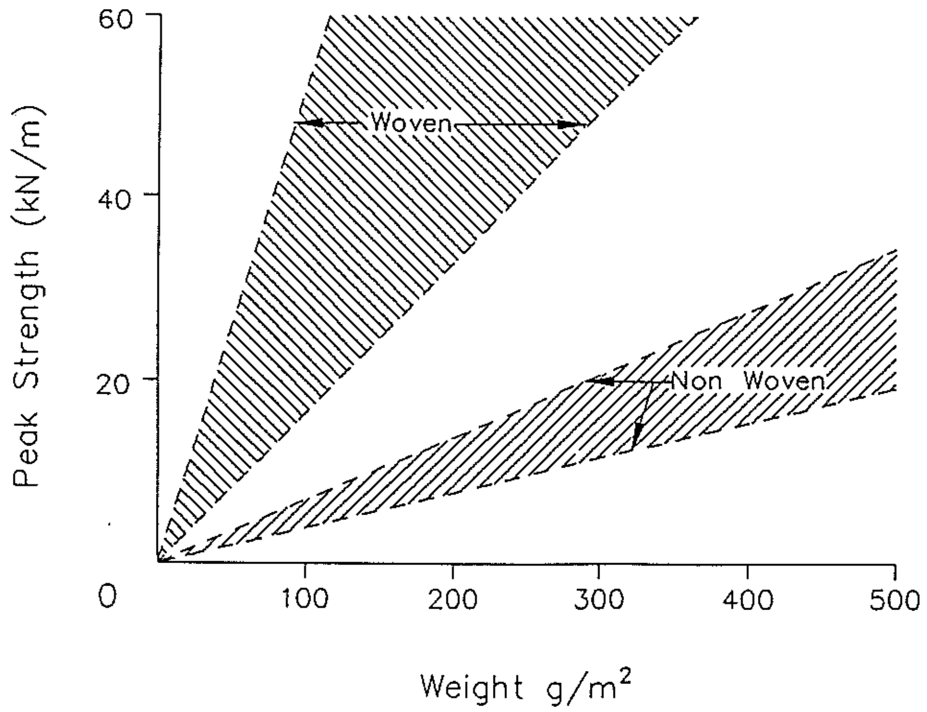


FIGURE 5.1
Relationship Between Tensile Strength and Weight

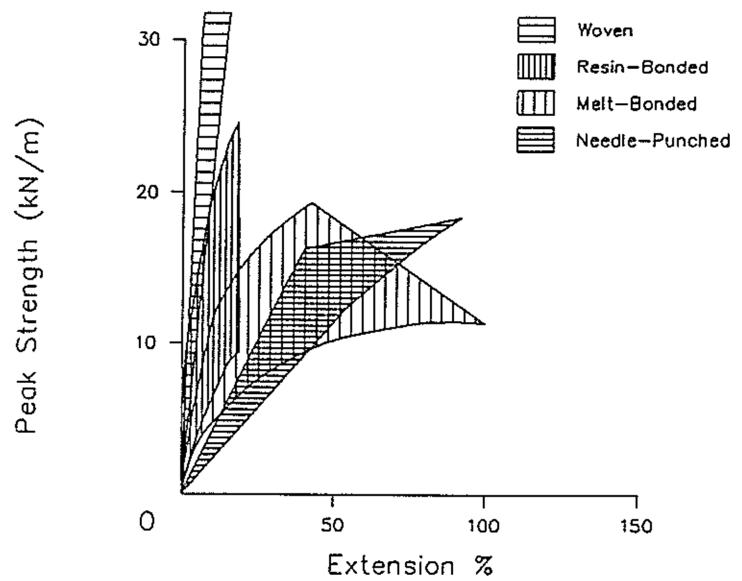


FIGURE 5.2
Typical Plane Strain Test Data for Various Generic Geotextile Types, Normalised to $300g/m^2$ Fabric Mass (McGown 1976)

The drawn, polymer-aligned, geogrid products have similar peak strength and stress-strain modulus properties to those of the high strength woven geotextiles.

5.4.2 Integrity Response Characteristics

Integrity tests are basically concerned with the characteristics of a geotextile under conditions of installation. The three basic conditions and associated tests are tear, burst and impact puncture. The test methods have been described in Chapter 4.

5.4.3 Tear Test Characteristics

Although the mechanism of tearing differs between woven and non-woven geotextiles, the results of both types are broadly apportioned to the unit weight (g/m^2) irrespective of the peak strength and stress-strain relationships. With few exceptions, the ASTM D1117 test results for products ranging in weight between $100g/m^2$ and $500g/m^2$ will have a corresponding tear resistance of 200N to 1000N. Within this range, there are products that have a tear resistance of between two and three times the numerical value of the weight (Lawson 1985). Geotextiles with relatively low tear resistance (<2 x weight) are non-woven products manufactured with short staple fibres and the higher strength tape wovens.

5.4.4 Burst Test Characteristics

Burst tests are carried out by Mullen Burst (MPa) or CBR Plunger (kN) procedures. Using the units quoted, both tests give almost identical numerical values.

In general, the burst strength of non-woven geotextiles are proportional to the peak tensile strength. For tensile strengths of 10kN/m to 35kN/m the corresponding burst strength range is 1.2 to 5.6 (MPa or kN). A ratio factor of 15% to 20% is apparent, the difference being due to the different geotextile construction (Lawson 1985).

Woven geotextiles have a similar relationship with tensile strength although the higher strength products cannot be tested by Mullen Burst test. With these products, the burst strength is approximately 10% of the tensile strength.

It can be noted that, because of the proportional relationship between weight and tensile strength, there is a relationship between the burst strengths of woven and non-woven geotextiles and the corresponding unit weight (Main Roads Department 1983).

5.4.5 Impact Puncture Characteristics

In the standard test, the diameter of the hole formed by the dropped cone is measured. With this test significant results are only obtained with non-woven geotextiles and woven geotextiles of unit weight less than $300g/m^2$. If the general results are correlated with peak tensile strength (or unit weight) the puncture resistance is greatest for woven products (5-10mm) followed by needle-punched non-woven products (7-30mm). The melded heat-bonded products have least resistance (30-45m), most likely caused by the slight stiffness that results from the geotextile construction (Lawson 1985, Main Roads Dept 1981 1983).

5.5 LONG-TERM MECHANICAL PROPERTIES

In many applications, the loads imposed on geotextiles are short term but in some applications the geotextile will be subject to sustained or repeated loads. In this latter condition, the time component of the geotextile's mechanical behaviour must be considered. The three properties generally considered are:

(i) Creep under sustained loading.
(ii) Stress relaxation under a constant strain.
(iii) Cyclic loading where the loading is repetitive.

All are associated with the visco-elastic creep characteristics of the geotextile. In general, the long-term creep characteristics are primarily dependent on the component polymer filaments although some non-woven geotextiles are more complex because of the variable nature of the bonding.

At a given stress level, polymer materials have a linear strain or extension against the logarithm of time but the gradient of the creep curve increases as the stress level increases. Depending on the polymer, stress level and time of loading, the geotextile may eventually strain to failure (Tertiary Creep). The above can be summarised by the relationship:

$$e = e_o + b \text{ Log } t$$

where e = the total strain or extension over time period t,
e_o = the instantaneous strain or extension from the short-term tests,
b = creep coefficient.

The creep coefficient is dependent on the level of stress or load, the polymer type, material construction, soil confinement and temperature. Figures 5.3(a) and 5.3(b) (Lawson 1985) illustrate the ranges of the creep coefficient for various polymer filaments and geotextile construction against the load level as a percentage of the ultimate load. Of the more common polymers, it can be seen that polyester has significantly superior creep characteristics in comparison to polypropylene. For a long-term design life, the maximum allowable load level to avoid tertiary creep failure is 20% and 40% of the ultimate load for polypropylene and polyester fibres respectively (Lawson 1985). There is some evidence to suggest that these values may be conservative in a confined environment (Fock and McGown 1987).

Similar characteristics apply to constant strain conditions.

Under repetitive loading, a residual strain will occur and the stress-strain modulus will increase to a level compatible to the rate of loading.

Little information is available on the long-term characteristics of geotextiles in general and what is available concentrates on the creep properties of heavy weight woven geotextiles and geogrids. This is not surprising as these products are used primarily as a reinforcement material. Ministry of Works, Central Laboratories (now Works Technical Services Laboratories) (Millar 1985, 1986) have undertaken two series of tests on some geotextiles available in New Zealand. The products tested included light to medium weight polypropylene wovens, a melt-bonded non-woven, and a staple filament, polyester, needle-punched geotextile. In general, the results coincide with those shown on Figures 5.3a and 5.3b.

5.6 HYDRAULIC PROPERTIES

The hydraulic tests usually applied to geotextiles can be divided into three groups: apparent opening size tests, normal permeability tests and in-plane permeability tests (drainage). Chapter 4 reviews the test methods. The hydraulic properties of geotextiles are influenced by the size and shape of the constituent fibres and the method of construction.

5.6.1 Apparent Opening Size (Pore Size)

Geotextiles cannot be manufactured with a uniform pore size and the apparent opening size (AOS) is expressed as a percentage of the distribution of pore size. The most common pore sizes quoted are O_{50} and O_{90}. These are the pore sizes of the geotextile that have respectively 50% and 90% of the pore sizes smaller than the quoted size. It is common practice to express the apparent opening size in the form of a measurement (i.e. in mm or microns). However, in North American countries, the apparent opening size is commonly expressed in terms of a US Standard Sieve number. The relationship between some US sieve numbers and apparent opening size is shown in Table 5.2.

Woven geotextiles are generally characterised by a more constant pore size than the three-dimension non-woven products although, in comparison to soils, all products can be considered as "poorly graded" or "well sorted". The most common method of determining the pore-size distribution involves the sieving of particles of known diameter through the geotextile (Chapter 4).

The geotextile construction controls the range of pore size that can be obtained. Woven products generally have an O_{90} value restricted to the order of 250μm because of the physical requirements of needing a moderately large diameter (denier) filament and permeability restrictions. Some products can have O_{90} values as low as 75μm at the expense of a low permeability.

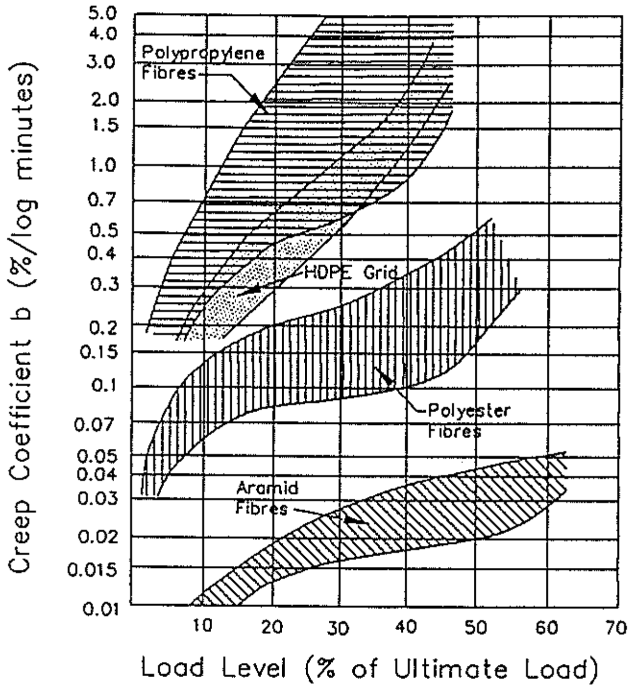


FIGURE 5.3a
Creep Coefficient Versus Load Level for Aramid, Polyester and Polypropylene Fibres and HDPE Grid

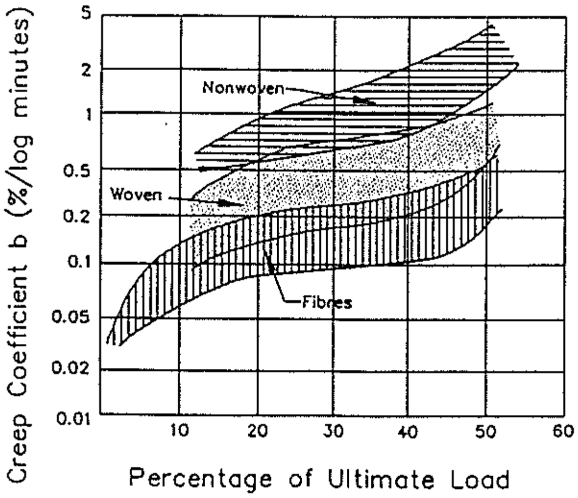


FIGURE 5.3b
Creep Coefficient Versus Load Level for Polyester Fibres, Polyester Woven Geotextiles, and Polyester Non-woven Geotextiles (Lawson 1985)

TABLE 5.2
Relationship between US Sieve Sizes and Geotextile Opening Sizes

US Sieve No	Opening Size (mm)
400	0.037
300	0.063
200	0.075
140	0.106
100	0.150
70	0.212
50	0.300
40	0.425
16	1.18
8	3.35
4	4.75

Non-woven needle-punched products generally have a range of O_{90} values from 100μm to 200μm — the value decreasing with the unit weight of the geotextile. Melt-bonded (melded) geotextiles have a similar range but the heavier grade products (>200g/m^2) can have an O_{90} pore size as low as 30μm without significant restriction to permeability.

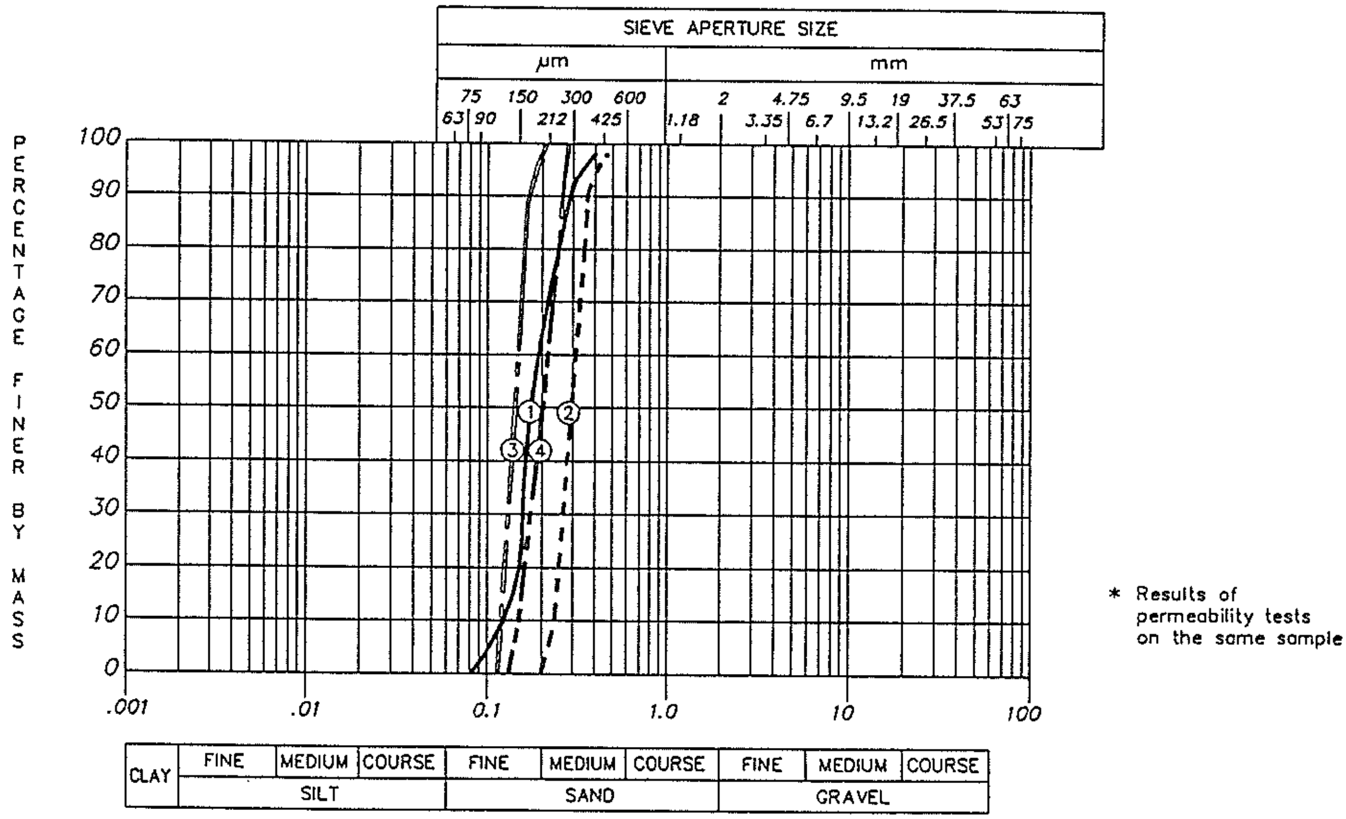
5.6.2 Permeability (Normal)

The permeability of a geotextile is generally expressed in terms of permittivity with units of litre/m^2/sec at a 100mm head loss. More recently the head loss (mm) at a controlled seepage velocity (e.g. 1cm/sec) has been used to express the permeability. The latter test method overcomes the non-laminar flow conditions of the former. The current TNZ(NRB) F/6 (Transit New Zealand 1985-1986) specification uses the head loss expression as the permeability requirement. Both values are determined by similar apparatus. Some test results are shown on Figure 5.4 for a selection of products available in New Zealand. Neither value expresses the permeability in true units (m/sec) but simple formulae can be used to convert the test values to permeability units. Two such expressions are reviewed in Chapter 8.

The range of permeability values for woven geotextiles is highly dependent on the Percentage Open Area (porosity). Monofilaments having a range POA of 10% to 30% and slit tape films are generally below 10% depending on the denier of the yarn. The values for light to medium weight (<300g/m^2) monofilament generally range from 100 to 300 litres/m^2/sec and a head loss in the order of 5mm at 1cm/sec velocity. Tape-woven values range from 3 to 20 litres/m^2/sec and a head loss at 1cm/sec in the order of 100 to 500mm. These values for tape-wovens indicate a low permeability and such products are not recommended for filtration use. Multifilament and heavy weight monofilament woven products have values between the above.

The permeability of non-woven geotextiles depends to some extent on the method of construction. Needle-punched products are the most permeable with values generally ranging from 100 to 200 litres/m^2/sec and a head loss of less than 5mm. The values for the melt-bonded melded products generally range from 40 to 80 litres/m2/sec and a head loss of 5 to 15mm. In both construction types, permeability depends on the unit weight and pore size of the products. The lower permeability values within the range apply to the higher weight grades and/or lower pore size products.

Figure 5.4 illustrates the pore-size distribution of a selection of products available in New Zealand (Millar 1986, East and High 1989).



INDEX	CONSTRUCTION PRODUCT	O 50 μm	O 90 μm	* Q 100 litre/m^2/sec	* a mm
1	Tape woven (Geotex 16/16)	180	270	9	130
2	Monofilament Woven (Polyweave F)	300	370	120	5
3	Melt Bonded Non Woven (Terram 1000)	150	170	65	6
4	Needle Punched Non Woven (Bidim U24)	190	230	300	2

FIGURE 5.4
Pore- Size Distribution

It should be noted that the normal stress on the geotextile can reduce the permeability. Neither the woven or melt-bonded melded products are markedly affected but the permeability of the more compressible needle-punched geotextiles can be reduced by some 50% with a normal stress of 100kPa. Little reduction occurs with higher stresses. Under such conditions, the needle-punched and melt-bonded geotextiles have a similar permeability range.

5.6.3 In-plane Permeability (Drainage)

Some typical values for in-plane permeability at 40kPa normal stress are given on Table 5.3 (Koerner 1986).

TABLE 5.3
Typical Values of In-Plane Permeability

Geotextile Construction	Transmissivity T (m^2/sec)	Permeability k_p (m/sec)
Woven tape	1.3×10^{-8}	2×10^{-5}
Woven monofilament	3.0×10^{-8}	4×10^{-5}
Non-woven heat-bonded	3.0×10^{-9}	6×10^{-6}
Non-woven needle-punched	2.0×10^{-6}	4×10^{-4}

Note: Transmissivity $T = k_p b$ where b is geotextile thickness at normal pressure.

At higher than normal pressures, the values decrease. The more compressible needle-punched product will be most affected with, perhaps, a 10-fold reduction in permeability for an additional 100kPa normal stress ((Van Zanten 1986; Lawson and Curiskis 1985).

Apart from perhaps the thicker needle-punched products, the in-plane permeability properties have limited application in general road construction. The low in-plane transmissivity of geotextiles has resulted in the introduction of drainage geocomposites. These products are constructed by wrapping a geotextile around a high flow core. Further details of these products are given in Chapter 8.

5.7 DURABILITY

Durability includes two time-related aspects of geotextile integrity: insurance that the geotextile does not degrade critically either after it is placed in the soil or before it is placed in the soil. The chemical composition of the geotextile determines the extent to which both occur.

The potential for degradation once the geotextile is placed in the soil is governed by the resistance to degradation of the polymer components in the geotextile. As reviewed in

Chapter 3 degradation does not appear to be a problem with the commonly used polymers.

The prime consideration concerning stability of the geotextile before it is placed in the soil is the effect of prolonged exposure to ultraviolet (UV) light. All synthetic fibres are susceptible to some extent to degradation by UV light. The degradation process results in a gradual decrease in the mechanical properties generally caused by filament brittlement. Needless to say, once the geotextile is covered adequately by soil or rock no further UV degradation occurs. It is common practice for the more highly susceptible fibres (polypropylene, polyethylene, and polyamide) to have a UV stabiliser incorporated in the fibre during manufacture. Two types of UV stabilisers are used in geotextiles: one type consists of black carbon particles ("Carbon Black") which renders the geotextile black in colour; the other type is a complex chemical which does not change the appearance of the virgin material at all. Although some synthetic fibres exhibit good natural UV resistance and in others UV stabilisers may be incorporated in other fibre types, geotextiles should not be left exposed for prolonged periods as this natural or synthetic UV stability only delays the degradation but does not prevent it.

5.8 SUMMARY AND COMPARISON OF GEOTEXTILE PROPERTIES

(a) The peak tensile strength is proportional to the unit weight of the product although for a given weight, woven geotextiles have a higher strength than non-woven.

(b) The elongation at peak strength of non-woven products is some 3 to 10 times that of woven products in unconfined tests. Consequently, the stress-strain modulus for woven geotextiles is higher than for non-woven geotextiles. There are indications that the differences are less in confined conditions.

(c) The tear resistance is proportional to the unit weight of the product irrespective of whether the geotextile is woven or non-woven.

(d) The burst strength is proportional to the peak tensile strength of the product. For a given unit weight, woven geotextiles have a higher burst strength than non-woven.

(e) The impact puncture resistance is dependent on the method of geotextile construction. For a given unit weight, the puncture resistance is greatest for woven products followed by needle-punched non-woven and melt-bonded non-wovens. For a given method of construction, the resistance increases with the unit weight.

(f) In conditions that require sustained loading or constant strain, woven or geogrid products with polyester polymer filaments are indicated to have superior creep properties over woven products with polypropylene filaments and over all non-woven geotextiles. A low proportion of the peak tension strength is required to avoid creep strain to failure or stress relaxation.

Note: Care should be exercised when using relatively low strength, high modulus, polypropylene woven products under conditions of constant strain, such as at the base of an embankment over compressible soils.

(g) The pore sizes of woven products vary considerably with the method of geotextile construction but, in general, the minimum apparent opening size O_{90} is in the region of 250μm. Non-woven products can have significantly lower pore sizes. For a given method of construction (needle-punched or melt-bonded), the apparent opening size, O_{90} generally, decreases with an increase in the unit weight of a non-woven geotextile.

(h) The permeability of a geotextile is dependent on the method of construction. Monofilament woven products have a high permeability but tape wovens have an extremely low permeability. All non-woven products have a reasonably high permeability with the needle-punched geotextiles having the higher values. For a given method of construction (needle-punched or melt-bonded), the permeability is proportional to the pore size of non-woven geotextiles. The permeability of the more compressible needle-punched non-woven products will decrease with high normal loads but not to an extent that would be significant to their use.

(i) The in-plane permeability is highly dependent on the method of geotextile construction. Only the needle-punched non-woven products can be considered to have in-plane permeability for limited practical applications and, as with the normal flow direction, the permeability will decrease with normal load. High flow capacity drainage geocomposites can be used if this property is required for particular applications.

5.9 REFERENCES

East, G.R.W., High, R. 1989. *Geotextile subsoil drain investigation.* Transit New Zealand /Road Research Unit, Research Project PD/14.

Fock, G., McGown, A. 1987. The creep behaviour of a needle-punched polypropylene geotextile. *Proceedings of Ninth South-east Asian Geotechnical Conference,* Bangkok, Thailand.

Koerner, R.M. 1986. *Designing with geosynthetics.* Prentice-Hall, Englewood Cliffs, NJ.

Lawson, C. 1986. Geotextiles or geosynthetics? *Proceedings of Technical Sessions, Third Asian ICI Fibres Geotextiles Conference,* Thailand.

Lawson, C.R., Curiskis, J.I. 1985. *Geotextiles : including manufacture, properties, and use in pavements, embankments, subsurface drainage and erosion control.* Lawson Consulting, Sydney, Australia.

McGown, A. 1976. *The properties and uses of permeable fabric membranes.* Research workshop on materials and methods for low cost road, rail and reclamation works, Leura, Australia.

Main Roads Department 1981. Evaluation of engineering fabrics for use by MRD. *Main Roads Department, Materials Branch Ref RP705, TT70.*

Main Roads Department 1983. Evaluation of geotextiles. *Main Roads Department, Materials Branch, Report R1.*

Millar, P.J. 1985. Properties of geotextiles manufactured in New Zealand. *Ministry of Works & Development, Central Laboratories Report M2 85/6.*

Millar, P.J. 1986. Assessment of geotextiles, physical properties and performance tests. *Ministry of Works & Development, Central Laboratories Report 2 86/11.*

Transit New Zealand (NRB) 1985-1986. Specification for fabric-wrapped aggregate subsoil drain construction. *TNZ(NRB) F/6, 1985 and TNZ(NRB) F/6 Notes, 1986.*

Van Zanten, R.V. (Editor) 1986. *Geotextiles and geomembranes in civil engineering.* J.Wiley, New York. 658p.

CHAPTER 6

PROPERTIES OF GEOTEXTILES AVAILABLE IN NEW ZEALAND

6.1 GENERAL COMMENTS

This chapter lists the geotextiles that are currently available in New Zealand, and their properties. Only those considered useful to roading or associated applications are included. The properties have been obtained from the manufacturers' data sheets and other sources authorised by the National Roads Board (now Transit NZ) but cannot be guaranteed. When in doubt appropriate testing should be carried out.

New brands and grades are constantly being introduced and hence the product list (Section 6.3) cannot be considered comprehensive. In addition, more specialised types and grades that have been utilised in New Zealand and can generally be indented by the appropriate agent, are listed in Section 6.4. The agent should be contacted for the properties of these geotextiles.

No one product type can be used for all the applications where geotextiles are required. Different applications generally require properties that cannot all be attained by one method of manufacture. With some exceptions, manufacturers generally concentrate on one type of geotextile construction, depending on their background in the textile industry. Historically, New Zealand manufacturers have been the supplier through an associated retailer but, with the relatively recent importation of a number of overseas products, suppliers have broadened their stocks to offer the civil engineering industry a wide range and choice of geotextiles and geocomposites. The product lists are categorised by geotextile construction, not by the suitable application. The geotextile construction types covered are:

(a) Woven (Tape and Monofilament)
(b) Non-woven (Heat-Bonded and Needle-Punched)
(c) Geogrids
(d) Geocomposites

6.2 PROPERTIES

The various geotextile manufacturing techniques and raw thermoplastic result in a wide range of properties. Five basic properties need to be considered to evaluate the potential use or application:

(a) Physical (e.g. mass per unit area)
(b) Mechanical (e.g. strength, extension)
(c) Hydraulic (e.g. permeability, pore size)

(d) Durability (e.g. chemical resistance, UV light resistance)
(e) Commercial (e.g. roll width, length)

The bitumen absorption properties and melting temperatures are also required for geotextiles with an application for use in the upper layers of pavements.

Many test methods are available to evaluate these properties, and these have been discussed in Chapter 4. It must be emphasised that some test methods are more relevant to a particular use than others, e.g. an adequate filter is generally not required to have a high strength, and pore size is not applicable in a reinforcement application.

The index of products list the properties, as published by the manufacturer, against test procedures generally accepted by the industry and users in Tables 6.1 to 6.5. A blank form (Table 6.6) is also provided so that the properties of a new product that becomes available can be entered. Comparisons between products can be made from these tables but some care is required as in some instances the tests have not been carried out to the same standard. It should be noted that for a given product, there is a moderate range in any property value that reflects the variations that can occur in both the manufacturing and testing techniques.

For some products audit tests have been carried out, to verify the published property values, by the MWD Central Laboratory (now WORKS Technical Services Laboratory) in addition to a special NRB (Transit NZ) permeability test (Test T/6). Audit tests are noted by asterisks in the tables.

Some property listings are not complete. Generally this is because the test is not appropriate for the product (e.g. permeability of a geogrid). Durability properties are not included as they are generally not published. Such properties are dependent on the fibre composition and have been discussed in Chapter 3.

6.3 PRODUCTS AVAILABLE IN NEW ZEALAND

6.3.1 Woven Geotextiles

(a) Tape Woven (Split Film)

Product (Manufacturer)	Grade	Principal Supplier/Retailer	Data Reference
GEOTEX (Donaghys)	110, 170	Ground Engineering P O Box 18-294 AUCKLAND Ph (09) 598-215 P O Box 2543 CHRISTCHURCH Ph (03) 62-849	Table 6.1.1
POLYWEAVE (Sarlon)	R, HR	Cruickshank BSL Ltd P O Box 2842 AUCKLAND Ph (09) 390-533	Table 6.1.2
PROPEX MUDSTOP (Amoco)	2000, 2002 3220, 3227	Permathene Plastics Ltd P O Box 71-015 Avondale AUCKLAND Ph (09) 885-179	Table 6.1.3

(b) Monofilament Woven

Product (Manufacturer)	Grade	Principal Supplier/Retailer	Data Reference
GEOTEX (Donaghys)	300	Ground Engineering P O Box 18-294 AUCKLAND Ph (09) 598-215 P O Box 2543 CHRISTCHURCH Ph (03) 62-849	Table 6.1.1
POLYWEAVE (Sarlon)	F	Cruickshank BSL Ltd P O Box 2842 AUCKLAND Ph (09) 390-533	Table 6.1.2

6.3.2 Non-Woven Geotextiles

(a) Heat-Bonded

Product (Manufacturer)	Grade	Principal Supplier/Retailer	Data Reference
TERRAM (ICI)	500, 700, 1000, 1500, 2000, 3000, 4000	Ground Engineering P O Box 18-294 AUCKLAND Ph (09) 598-215 P O Box 2543 CHRISTCHURCH Ph (09) 62-849	Table 6.2.1
TYPAR (Du Pont)	3337, 3407, 3607, BM41 (Pav)*	Fletcher CSP P O Box 22-745 Otahuhu AUCKLAND Ph (09) 641-239 P O Box 1955 CHRISTCHURCH Ph (03) 487-050 Also Hamilton, Dunedin, Palmerston North	Table 6.2.2

(b) Needle-Punched

Product (Manufacturer)	Grade	Principal Supplier/Retailer	Data Reference
BIDIM (Geofabrics Australasia)	U12, U14, U24, U34, U44, U64 PF1 (Pav) PF2(Pav)	Maccaferri Gabions NZ Ltd P O Box 12-536 Penrose AUCKLAND Ph (09) 646-495 CHRISTCHURCH Ph (03) 495-600	Table 6.3.1
POLYFELT (Chemie Linz)	Ecofelt, 22T TS500, TS600, TS700, TS750 PGM14(Pav)	Nylex NZ Ltd P O Box 58-001 East Tamaki AUCKLAND Ph (09) 274-5149	Table 6.3.2

*Grades designated (Pav) are pavement geotextiles.

6.3.3 Geogrids

Product (Manufacturer)	Grade	Principal Supplier/Retailer	Data Reference
PARAGRID (ICI)	50S/50S 100S/100S 100S/25S	Ground Engineering P O Box 18-294 AUCKLAND Ph (09) 598-215 P O Box 2543 CHRISTCHURCH Ph (03) 62-849	Table 6.4.1

6.3.4 Drainage Geocomposites

Product (Manufacturer)	Grade	Principal Supplier/Retailer	Data Reference
GEODRAIN (Ground Engineering)		Ground Engineering P O Box 18-294 AUCKLAND Ph (09) 598-215 P O Box 2543 CHRISTCHURCH Ph (03) 62-849	Table 6.5.1
STRIPDRAIN (Nylex)	Depths 300-900 mm	Nylex NZ Ltd P O Box 58-001 East Tamaki AUCKLAND Ph (09) 274-5149	Table 6.5.1

6.4 PRODUCTS THAT CAN BE INDENTED TO NEW ZEALAND

Note: In some cases the supplier may hold a limited stock of these products.

6.4.1 Woven Geotextiles

Product (Manufacturer)	Grade	Principal Agent/Retailer
TERRAM (ICI)	Heavy Weight Polypropylene W/5-5 to W/20-4	Ground Engineering P O Box 18-294 AUCKLAND Ph (09) 598-215
	Heavy Weight Polyester WB/20-5 to WB/60-5 plus other geo-composite products	P O Box 2543 CHRISTCHURCH Ph (03) 62-849

6.4.2 Non-Woven Geotextiles

Product (Manufacturer)	Grade	Principle Agent/Retailer
TYPAR (Du Pont)	Full range of Typar heat-bonded products, not included in 6.3.2	Fletcher CSP P O Box 22-745 Otahuhu AUCKLAND Ph (09) 641-239
		P O Box 1955 CHRISTCHURCH Ph (03) 487-050
FIBRETEX (A/S Fibretex)	A range of needle-punched staple fibres products — some partially heat-bonded G-100, F-2B, F-25, F-4M, S-300	Hammerking Products Ltd P O Box 38-177 Howick AUCKLAND Ph (09) 274-8568

6.4.3 Geogrids

Product (Manufacturer)	Grade	Principal Agent/Retailer
TENSAR (Netlon)	A range of polymer (Netlon)-aligned high density polyethylene products SR2, SR55, SR80, SR110, ARI (Pav)*	Ground Engineering P O Box 18-294 AUCKLAND Ph (09) 598-215 P O Box 2543 CHRISTCHURCH Ph (03) 62-849

*(pav) pavement geotextiles

TABLE 6.1 Properties of Woven Geotextiles

PRODUCT NAME: Geotex   MANUFACTURER: Donaghys   DATE: November 1989

Product Designation			GEOTEX	
Construction			Woven Tape	Woven Monofilament
Composition			Polypropylene	Polypropylene
Grades		110	170	300
PHYSICAL PROPERTIES				
1. Mass Per Unit Area - Nominal	g/m^2	110	170	310
2. Thickness - Under Load 2kPa	mm	.4	.5	-
MECHANICAL PROPERTIES				
1. Strip Tensile - BS2576				
Breaking Load - Length	kN/m	18	30	57
Extension at Failure - Length	%	22	30	32
Breaking Load - Across	kN/m	15	32	49
Extension at Failure - Across	%	20	23	30
2. Grab Tensile Strength				
100mm Wide Sample - ISO 5082				
Breaking Load	N	-	1300/1300	2300/1800
Extension at Failure	%	-	-	-
3. Tear Strength				
Trapezoidal - ASTM D4533	N	350	310	430
4. Burst Strength				
Cone Drop - EMPA	mm	-	-	-
CBR - DIN 54307	kN	2.1	3.2	4.5
Mullen - BS4768	MPa	2.2	3.5	5.3
HYDRAULIC PROPERTIES				
1. Permeability				
Volume Water Flow(10cm head)	$Litres/m^2/sec$	13	28	130
2. Head Loss - NRB T/7 *	mm	133	-	-
3. Pore Size				
Apparent Opening Size				
- O_{90}	μm	290	540	385
- O_{50}	μm	280	-	-
BITUMEN ABSORPTION				
1. Bitumen retention -TS25	$Litres/m^2$			
2. Melting Temperature	^oC			
COMMERCIAL DATA				
1. Roll Width	m	3.65	3.8	3.5
2. Roll Length	m	150	150	100
3. Roll Diameter	m	-	-	-
4. Roll Weight	kg	-	-	-

*Audit test

TABLE 6.1.1
Properties of Geotex Geotextiles

TABLE 6.1 (continued)

PRODUCT NAME: Polyweave   MANUFACTURER: Sarlon   DATE: November 1989

Product Designation		POLYWEAVE		
Construction		Woven Tape		Woven Monofilament
Composition		Polypropylene		Polypropylene
Grades		R	HR[1]	F
PHYSICAL PROPERTIES				
1. Mass Per Unit Area - Nominal	g/m^2	102	150	206
2. Thickness - Under Load 2kPa	mm	.4	.5	.6
MECHANICAL PROPERTIES				
1. Strip Tensile - ASTM D1682 : *				
Breaking Load - Length	kN/m	18		46
Extension at Failure - Length	%	40		30
Breaking Load - Across	kN/m	13		30
Extension at Failure - Across	%	25		20
2. Grab Tensile Strength 100mm Wide Sample - ASTM D1682				
Breaking Load	N	580	≈940	1360
Extension at Failure	%	25/18	-	25/18
3. Tear Strength Trapezoidal - ASTM D1117	N			
4. Burst Strength				
Cone Drop - EMPA	mm	-	-	-
CBR - DIN 54307	kN	1.1	-	2.7
Mullen - ASTM D3786	MPa	2.0	-	3.5
HYDRAULIC PROPERTIES				
1. Permeability Volume Water Flow(10cm head)	$Litres/m^2/sec$	12	-	135
2. Head Loss - NRB T/7 *	mm	170	-	5
3. Pore Size Apparent Opening Size				
- O_{90}	µm	-	-	250
- O_{50}	µm	120	-	-
BITUMEN ABSORPTION				
1. Bitumen retention	$Litres/m^2$			
2. Melting Temperature	°C			
COMMERCIAL DATA				
1. Roll Width	m	3.86/5.0	3.85/5	1.83/3.66
2. Roll Length	m	250	100	250
3. Roll Diameter	m			
4. Roll Weight	kg			

Note 1. New product at time of publication, test not complete

*Audit test

TABLE 6.1.2
Properties of Polyweave Geotextiles

TABLE 6.1 (continued)

PRODUCT NAME: Propex Mudstop   MANUFACTURER: Amoco Fabrics   DATE: November 1989

Product Designation		PROPEX MUDSTOP			
Construction		Woven Tape		Woven Tape/Needle punched	
Composition		Polypropylene		Polypropylene/Polyester	
Grades		2000	2002	3220	3227
PHYSICAL PROPERTIES					
1. Mass Per Unit Area - Nominal	g/m^2	105	160	345	415
2. Thickness - Under Load 2kPa	mm	.3	.5	-	-
MECHANICAL PROPERTIES					
1. Strip Tensile - ASTM D1682					
Breaking Load - Length	kN/m	19*	21*	-	-
Extension at Failure - Length	%	90	38	-	-
Breaking Load - Across	kN/m	15	28	-	-
Extension at Failure - Across	%	41	35	-	-
2. Grab Tensile Strength					
100mm Wide Sample - ASTM D1682					
Breaking Load	N	790*	960*	500	550
Extension at Failure	%	48	31	10/50	10/50
3. Tear Strength					
Trapezoidal - ASTM D1117	N				
4. Burst Strength					
Cone Drop	mm	-	-	-	-
CBR - DIN 54307	kN	1.3	2.7	2.2	2.4
Mullen - BS4768	MPa	1.9	2.7	2.8	3.0
HYDRAULIC PROPERTIES					
1. Permeability					
Volume Water Flow(10cm head)	$Litres/m^2/sec$	2*	30*	10	8
2. Head Loss - NRB T/7 *	mm	680	11	-	-
3. Pore Size					
Apparent Opening Size					
- O_{90}	μm	*	*	105	105
- O_{50}	μm	0/50	375	-	-
BITUMEN ABSORPTION					
1. Bitumen retention	$Litres/m^2$				
2. Melting Temperature	^oC				
COMMERCIAL DATA					
1. Roll Width	m	3.83	3.83	3.83	3.83
2. Roll Length	m		1000, 200, 100, 10		
3. Roll Diameter	m				
4. Roll Weight	kg				

*Audit test

TABLE 6.1.3
Properties of Propex Geotextiles

TABLE 6.2 Properties of Non-Woven Heat-Bonded Geotextiles

PRODUCT NAME: Terram - Heat Bonded MANUFACTURER: ICI Fibres, UK DATE: November 1989

Product Designation / Construction / Composition / Grades		TERRAM Heat Bonded Nonwoven Polypropylene/Polyethylene						
Grades		500	700	1000	1500	2000	3000	4000
PHYSICAL PROPERTIES								
1. Mass Per Unit Area - Nominal	g/m^2	70	100	135	190	230	270	350
2. Thickness - Under Load 2kPa	mm	0.4	0.5	0.6	0.8	1.0	1.1	1.4
MECHANICAL PROPERTIES								
1. Strip Tensile - ASTM D1682								
Breaking Load - Length	kN/m	3.5	5.0	8.0	12.0	15.0	18.0	23.0
Extension at Failure - Length	%	20	20	30	30	30	30	30
Breaking Load - Across	kN/m	3.5	5.0	8.0	12.0	15.0	18.0	23.0
Extension at Failure - Across	%	20	20	30	30	30	30	30
2. Grab Tensile Strength 100mm Wide Sample - ASTM D1682								
Breaking Load	N	320	480	630	930	1100	1300	1700
Extension at Failure	%	50	50	55	55	60	60	60
3. Tear Strength Trapezoidal - ASTM D1117	N	150	250	300	450	600	700	850
4. Burst Strength								
Cone Drop - EMPA	mm	45	45	40	35	30	25	20
CBR - DIN 54307	kN		1.2	1.6	2.1	2.6	3.1	4.0
Mullen - ASTM D3786	MPa	0.5	1.0	1.4	2.2	2.8	3.2	4.2
HYDRAULIC PROPERTIES								
1. Permeability Volume Water Flow(10cm head)	Litres/m^2/sec	150	80	50	35	33	30	25
2. Head Loss - NRB T/7 *	mm	-	3	6	26	13	-	-
3. Pore Size Apparent Opening Size								
- O_{90}	µm	350	180	100	60	50	40	30
- O_{50}	µm	200	120	70	40	30	20	10
BITUMEN ABSORPTION								
1. Bitumen retention -TS25	Litres/m^2							
2. Melting Temperature	°C							
COMMERCIAL DATA								
1. Roll Width	m	4.5	4.5	4.5	4.5	4.5	4.5	4.5
2. Roll Length	m	200	150	100	100	100	100	50
3. Roll Diameter	m	0.32	0.33	0.31	0.35	0.37	0.41	0.35
4. Roll Weight	kg	70	75	70	95	110	130	85

*Audit test

TABLE 6.2.1
Properties of Terram Heat-Bonded Geotextiles

TABLE 6.2 (continued)

PRODUCT NAME: Typar MANUFACTURER: Du Pont DATE: November 1989

Product Designation		TYPAR				
Construction		Heat Bonded Nonwoven				
Composition		100% Polypropylene				
Grades		3337	3407	3607	3857	BM41
PHYSICAL PROPERTIES						
1. Mass Per Unit Area - Nominal	g/m^2	110	136	190	290	140
2. Thickness - Under Load 2kPa	mm	0.45	0.46	0.58	0.78	0.70
MECHANICAL PROPERTIES						
1. Strip Tensile DIN 53857						
Breaking Load - Length	kN/m	4.0	6.3	7.5	16.0	4.9
Extension at Failure - Length	%	25	25	32	33	45
Breaking Load - Across	kN/m	4.0	6.3	7.5	16.0	4.9
Extension at Failure - Across	%	25	26	32	33	45
2. Grab Tensile Strength						
200mm Wide Sample - DIN 53858						
Breaking Load	N	470	680	1000	1440	-
Extension at Failure	%	>60	>60	>60	>60	-
3. Tear Strength						
Trapezoidal - ASTM D1117	N	270	370	480	680	-
4. Burst Strength						
Cone Drop - SORLIE	mm	36	29	23	17	-
CBR - DIN 54307	kN	0.8	1.3	1.9	3.0	-
Mullen - ASTM D3786	MPa	1.0	1.4	1.7	2.5	-
HYDRAULIC PROPERTIES						
1. Permeability						
Volume Water Flow(10cm head)	Litres/m^2/sec	160	100	40	20	-
2. Head Loss - NRB T/7	mm					
3. Pore Size						
Apparent Opening Size						
- O_{95}	µm	210	170	130	80	-
- O_{50}	µm	145	110	75	40	-
BITUMEN ABSORPTION						
1. Bitumen retention -TS25	Litres/m^2	-	-	-	-	0.53
2. Melting Temperature	^{o}C	-	-	-	-	165
COMMERCIAL DATA						
1. Roll Width	m	5.2	5.2	5.2	5.2	3.8/1.9
2. Roll Length	m	150	150	100	100	100/200
3. Roll Diameter	m	0.30	0.31	0.28	0.31	-
4. Roll Weight	kg	99	119	111	157	60/60

TABLE 6.2.2
Properties of Typar Geotextiles

TABLE 6.3 Properties of Non-Woven Needle-Punched Geotextiles

PRODUCT NAME: Bidim MANUFACTURER: Geofabrics Australasia Pty Ltd DATE: November 1989

Product Designation		BIDIM							
Construction		Needlepunched Continuous Filament Nonwoven							
Composition		Polyester							
Grades		U12	U14	U24	U34	U44	U64	PF1	PF2
PHYSICAL PROPERTIES									
1. Mass Per Unit Area - Nominal	g/m^2	130	150	210	270	340	550	140	180
2. Thickness - Under Load 2kPa	mm	1.5	1.6	2.0	2.4	2.8	4.2	0.6	0.8
MECHANICAL PROPERTIES									
1. Strip Tensile - DIN 53857									
Breaking Load - Length	kN/m	6	7	10.4	14.6	19.0	37.0	10	16
Extension at Failure - Length	%	70	70	70	70	70	70	30	30
Breaking Load - Across	kN/m	6	7	10.4	14.6	19.0	37.0	10	16
Extension at Failure - Across	%	70	70	70	70	70	70	30	30
2. Grab Tensile Strength 100mm Wide Sample - DIN 53858									
Breaking Load	N	430	500	850	1050	1450	2500	-	-
Extension at Failure	%	85	85	80	75	65	60	-	-
3. Tear Strength Trapezoidal - DIN 53363	N	220	280	420	570	770	1130	-	-
4. Burst Strength									
Cone Drop - EMPA	mm	30	30	23	20	15	7	-	-
CBR - DIN 54307	kN	1.3	1.4	2.3	3.1	3.4	5.7	-	-
Mullen - DIN 53861	MPa	-	1.7	2.2	2.5	3.6	5.5	-	-
HYDRAULIC PROPERTIES									
1. Permeability Volume Water Flow(10cm head)	Litres/m^2/sec	180	150	120	120	110	90		
2. Head Loss - NRB T/7 *	mm	-	3	2	-	-	-		
3. Pore Size Apparent Opening Size									
- O_{90}	µm	220	210	210	200	150	130		
- O_{50}	µm	-	-	-	-	-	-		
BITUMEN ABSORPTION									
1. Bitumen retention -TS25	Litres/m^2	-	-	-	-	-	-	1.8	2.4
2. Melting Temperature	^oC	-	-	-	-	-	-	260	260
COMMERCIAL DATA									
1. Roll Width	m	2/4	2/4	2/4	2/4	2/4	2/4	4.0	4.0
2. Roll Length	m	150	150	150	150	150	100	150/300	150/300
3. Roll Diameter	m	-	-	-	-	-	-	-	-
4. Roll Weight	kg	-	-	-	-	-	-	-	-

*Audit test

Note: PF product thickness measured at 20kPa

TABLE 6.3.1
Properties of Bidim Geotextiles

TABLE 6.3 (continued)

PRODUCT NAME: Ecofelt and Polyfelt MANUFACTURER: Chemie Linz, Austria DATE: November 1989

Product Designation Construction Composition Grades		ECOFELT 22T	 Needlepunched Continuous Filament Nonwoven TS500	 TS600	 Polypropylene TS700	 TS750	POLYFELT TS800	 PGM14
PHYSICAL PROPERTIES								
1. Mass Per Unit Area - Nominal	g/m^2	110	140	200	280	350	400	140
2. Thickness - Under Load 2kPa	mm	1.1	1.6	2.1	2.6	3.0	3.3	1.5
MECHANICAL PROPERTIES								
1. Strip Tensile - ASTM - D1682								
Breaking Load - Length	kN/m	6	8	11.5	16	19	22	9
Extension at Failure - Length	%	50/80	50/80	50/80	50/80	50/80	50/80	50
Breaking Load - Across	kN/m	6	8	11.5	16	19	22	9
Extension at Failure - Across	%	50/80	50/80	50/80	50/80	50/80	50/80	50
2. Grab Tensile Strength								
100mm Wide Sample - ASTM D1682								
Breaking Load	N	450	600	870	1150	1270	1590	-
Extension at Failure	%	70	70	70	70	70	70	-
3. Tear Strength ASTM *								
Trapezoidal — ASTM D1117	N	250	300	430	650	800	900	-
4. Burst Strength								
Cone Drop — TRCF	mm	-	15	12	10	8	7	-
CBR — DIN 54307E	kN	1.2	1.5	2.0	2.6	2.8	3.3	-
Mullen - AS 2001.2.4	MPa	1.3	1.5	2.0	2.5	3.2	3.7	-
HYDRAULIC PROPERTIES								
1. Permeability								
Volume Water Flow(10cm head)	Litres/m^2/sec	380	310	240	190	130	120	-
2. Pore Size								
Apparent Opening Size								
- O_{95}	μm	120	120	110	90	80	70	-
- O_{50}	μm							
BITUMEN ABSORPTION								
1. Bitumen retention -TF25	Litres/m^2	-	-	-	-	-	-	1.35
2. Melting Temperature	^oC	-	-	-	-	-	-	-
COMMERCIAL DATA								
1. Roll Width	m	2.0/4.0	2.0/4.0	4.0	3.8/4.0	4.0	3.8	1.9/3.8
2. Roll Length	m	300	200	175	125	100	100	150
3. Roll Diameter	m	-	-	-	-	-	-	0.5
4. Roll Weight	kg	70/140	70/140	140	140	140	150	40/80

*Audit test

TABLE 6.3.2
Properties of Ecofelt and Polyfelt Geotextiles

TABLE 6.4 Properties of Geogrids

PRODUCT NAME: Paraproducts MANUFACTURER: ICI Fibres Ltd, UK DATE: November 1989

Product Designation		PARAGRID		
Construction		Welded Strips		
Composition		Polyester Reinforcement/Polyethylene Sheath		
Grades		50S/50S	100S/100S	100S/25S
PHYSICAL PROPERTIES				
1. Mass Per Unit Area - Nominal	g/m^2	530	770	520
2. Thickness - Under Load 2kPa	mm	3.0	3.0	3.0
MECHANICAL PROPERTIES				
1. Strip Tensile ASTM D4595				
Breaking Load - Length	kN/m	50	100	100
Extension at Failure - Length	%	11	11	11
Breaking Load - Across	kN/m	50	100	25
Extension at Failure - Across	%	11	11	11
2. Grab Tensile Strength				
100mm Wide Sample				
Breaking Load	N			
Extension at Failure	%			
3. Tear Strength				
Trapezoidal	N			
4. Burst Strength				
Cone Drop	mm			
CBR	kN			
Mullen	MPa			
HYDRAULIC PROPERTIES				
1. Permeability				
Volume Water Flow(10cm head)	$Litres/m^2/sec$			
2. Head Loss	mm			
3. Pore Size				
Apparent Opening Size				
- o_{90}	µm			
- o_{50}	µm			
BITUMEN ABSORPTION				
1. Bitumen retention	$Litres/m^2$			
2. Melting Temperature	°C			
COMMERCIAL DATA				
1. Roll Width	m	4.5	4.5	4.5
2. Roll Length	m	50	50	50
3. Roll Diameter	m	0.4	0.4	0.4
4. Roll Weight	kg	128	180	125

TABLE 6.4.1
Properties of Paraproducts

TABLE 6.5 Properties of Drainage Geocomposites

PRODUCT NAME:	Geodrain, Stripdrain	MANUFACTURER:	Ground Engineering, Nylex	DATE: November 1989

Product Designation		Geodrain	Stripdrain
Construction		Composite	Composite
Composition		Terram 1000	Terram 1000
Grade		Netlon Core	HDPS Waffle Core
PHYSICAL PROPERTIES			
1. Mass Per Unit Area - Nominal	g/m^2	1000	
2. Thickness - Under Load 2kPa	mm	6	40
MECHANICAL PROPERTIES			
1. Strip Tensile			
Breaking Load - Length	kN/m		
Extension at Failure - Length	%		
Breaking Load - Across	kN/m		
Extension at Failure - Across	%		
2. Grab Tensile Strength			
100mm Wide Sample			
Breaking Load	N		
Extension at Failure	%		
3. Tear Strength			
Trapezoidal -	N		
4. Burst Strength			
Cone Drop	mm		
CBR	kN		
Mullen	MPa		
HYDRAULIC PROPERTIES			
1. Permeability (Normal)			
Volume Water Flow(10cm head)	$Litres/m^2/sec$	36	40
2. Permeability (In-Plane)			
Volume Water Water (i = 1 and 40kPa normal stress)	Litre/m/sec	0.5	10
3. Pore Size			
Apparent Opening Size			
- o_{90}	µm	100	100
- o_{50}	µm	70	70
BITUMEN ABSORPTION			
1. Bitumen retention	$Litres/m^2$		
2. Melting Temperature	^oC		
COMMERCIAL DATA			
1. Roll Width	m	2	0.2,0.3,0.6,0.9
2. Roll Length	m	30	-
3. Roll Diameter	m	-	-
4. Roll Weight	kg	-	-

TABLE 6.5.1
Properties of Drainage Geocomposites

TABLE 6.6 Properties of New Product

PRODUCT NAME:		MANUAFCTURER:	DATE:
Product Designation Construction Composition Grades			
PHYSICAL PROPERTIES			
1.	Mass Per Unit Area - Nominal	g/m^2	
2.	Thickness - Under Load 2kPa	mm	
MECHANICAL PROPERTIES			
1.	Strip Tensile		
	Breaking Load - Length	kN/m	
	Extension at Failure - Length	%	
	Breaking Load - Across	kN/m	
	Extension at Failure - Across	%	
2.	Grab Tensile Strength		
	100mm Wide Sample		
	Breaking Load	N	
	Extension at Failure	%	
3.	Tear Strength		
	Trapezoidal	N	
4.	Burst Strength		
	Cone Drop	mm	
	CBR	kN	
	Mullen	MPa	
HYDRAULIC PROPERTIES			
1.	Permeability		
	Volume Water Flow (10cm head)	$Litres/m^2/sec$	
2.	Head Loss - NRB T/7	mm	
3.	Pore Size		
	Apparent Opening Size		
	O_{90}	μm	
	O_{50}	μm	
BITUMEN ABSORPTION			
1.	Bitumen retention	$Litres/m^2$	
2.	Melting Temperature	°C	
COMMERCIAL DATA			
1.	Roll Width	m	
2.	Roll Length	m	
3.	Roll Diameter	m	
4.	Roll Weight	kg	

CHAPTER 7

GEOTEXTILE FUNCTIONS

7.1 GENERAL COMMENTS

To establish the suitability of a geotextile for a particular application in engineering, it is first necessary to define the functions that the geotextile will be required to perform in that application.

It has become convention for manufacturers and users to specify a particular geotextile as a product which can fulfil at least one of the following functions when in contact with water, soil, and/or rock:

Separation,
Filtration,
Reinforcement,
Drainage.

An additional function called water/particle proofing is applied to bitumen-impregnated geotextiles and geomembranes. This function is available for moisture and water control applications. It must be emphasised that for any one application several functions may be required with perhaps weighted emphasis. Such weighting is up to the designer, but one such example has been demonstrated in chart form by Rankilor (1982) (Figure 7.1). The various manufacturing processes are such that each product may have a dominant function that dictates a primary application, although the corollary is more likely that the product is inadequate in a particular function for the application. In this context the geotextile properties must be relatively rigorously specified to ensure the required functions for the application are achieved. As the technology advances, geocomposites will be designed for specific applications. The strip/fin drain is an example of this kind.

The following sections define the functions of geotextiles and their principle applications to roading. A broad overview of the type of geotextile construction that is suitable and economic for each function are also listed.

7.2 SEPARATION

Geotextile separation can be defined as the introduction of a flexible barrier between dissimilar materials so that the integrity of both materials can remain intact or be improved.

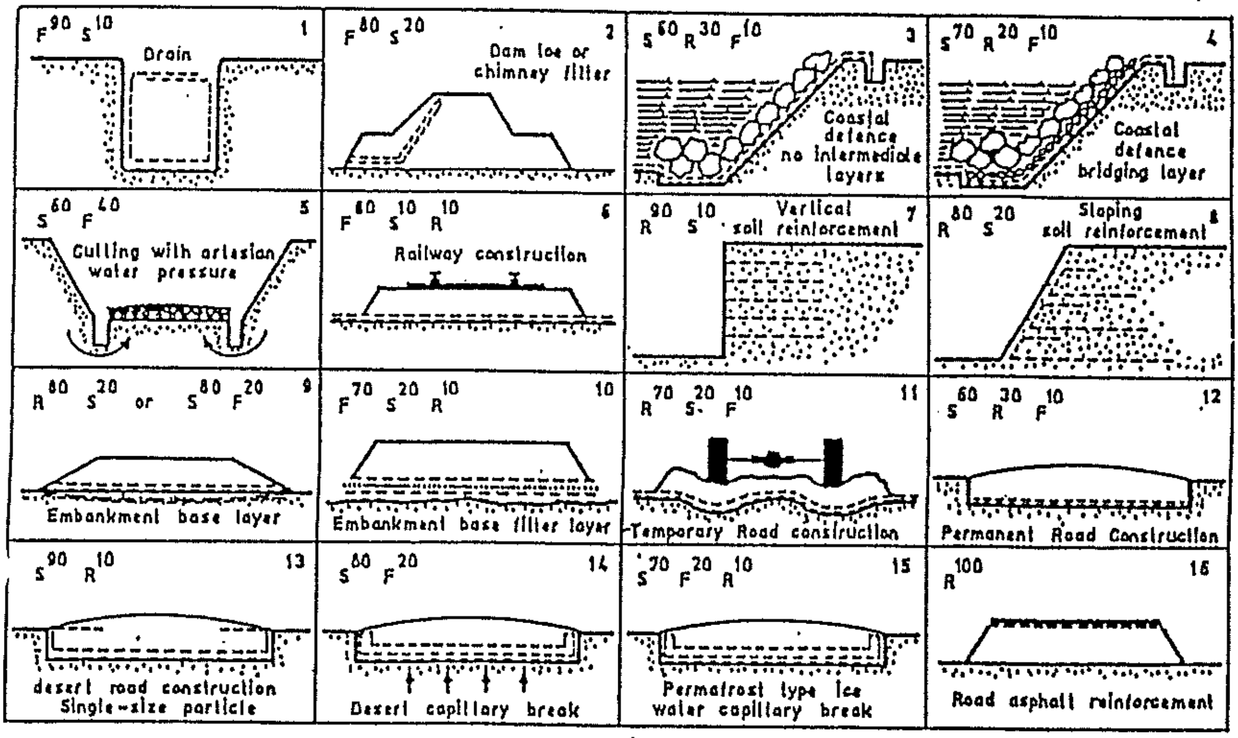


FIGURE 7.1
Design Variations Assessing the Importance of Filtration, Separation and Reinforcement (Rankilor 1982)

For roads, this generally means the isolation of two different soil/rock types to prevent intermixing. Two types of adverse mechanisms can be prevented by separation (Figure 7.2): first separation prevents soil fines from the subgrade entering the aggregate layer ("pumping"), and second separation prevents the penetration or intrusion of aggregate into the subgrade. Separation retains the original dimensions, strength and drainage capability of the aggregate layer.

Before the use of geotextiles the separation function was often undertaken by introducing a "sacrificial" layer of aggregate between the subgrade and the structured aggregate basecourse. To do this, road pavement design generally builds in an additional thickness of pavement (c.50mm). Using geotextiles can save this procedure. The savings are proportional to bearing strength (CBR soaked) of the subgrade. With high permanent strength, cohesive soil subgrades (perhaps lime stabilised), this separation application may not be required. The use of geotextiles in this application of separation was incorrectly deemed to be a reinforcement function at the beginning of geotextile technology.

Separation is the most underrated of all functions, largely because the presence of almost any geotextile type marketed will offer some intermixing protection. It is rarely designed on its own merit, but criteria are available to enable this to be carried out.

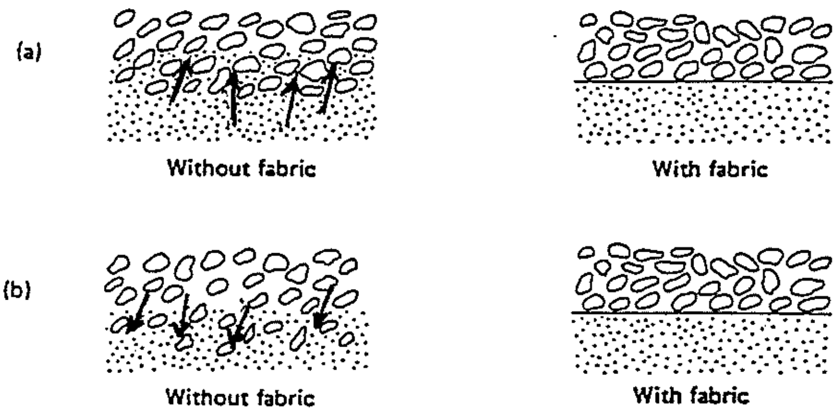


FIGURE 7.2

"Different mechanisms involved in the use of geotextiles involved in the separation function.
(a) Mechanism of soil fines pumping into stone voids and its prevention using geotextiles;
(b) Mechanism of stone intrusion into soil subgrade and its prevention using geotextiles" (Koerner 1986).

Where firm subgrades are present, the geotextile parameters that are relevant to this function for general roading are the Puncture/Burst Strength, Tensile Strength, and Tear Resistance, in conjunction with Permeability and Pore size (Filtration function). All woven and non-woven geotextiles have some degree of properties suitable for separation. The more robust products have the greater unit weight (Chapter 5). Tape-woven products should be avoided where significant water is required to pass through the geotextile (Chapter 9).

Where very soft compressible subgrades are present, road construction and traffic loading may mobilise high strains in the geotextile and only the low modulus, non-woven products may be suitable. The latter products can deform without developing high tensile stresses while still retaining the separation function. Alternatively a high strength, high modulus, woven product may be considered to restrain the deformations (Reinforcement function) in addition to separation (Chapter 11). Such high strength (>40kN/m) woven geotextiles are becoming more readily available in New Zealand although the most suitable products, with a tensile strength in excess of 200kN/m, may need to be imported.

Where the geotextile is required to resist sustained static loadings, in addition to separation, the geotextile's construction may be significant. An application of this type is the placement of a separation geotextile under an embankment founded on soft compressible soils. In this situation non-woven products will deform without developing high tensile stresses but woven products with the higher modulus may develop tensile stresses that approach the strength capacity of the product. Under these conditions the woven geotextile is attempting to utilise the reinforcement function and, in general, products with high strength and low creep properties would be required. Specific design is required for this application.

Geotextiles are now being used more frequently in another application of the separation function in pavement design. To alleviate the transfer of horizontal shear and cracks into an overlay from cracks in the original pavement (reflective cracking), non-woven geotextiles are being laid and bitumen-impregnated, before the overlay construction. These geotextiles need to be non-woven products that can absorb bitumen (Chapters 4, 13, 14). The geotextile industry is undertaking considerable research on this application (Lytton 1989).

7.3 FILTRATION

The geotextile function of filtration involves the movement of water through the fabric face (normal to the plane) while retaining the soil on the upstream face. The geotextile is required to have adequate permeability and soil retention at the same time. In addition, a long-term soil/fabric compatibility must exist so that the system will not completely clog during the required lifetime.

It can be seen that the requirements conflict by requiring a filter geotextile to be both open and tight. Such filtration concepts have been well established for granular filters, and similar conditions are being applied to the design of geotextile filters. Filtration offers an economic solution to many geotechnical and hydraulic problems but requires suitable design to be used in practice.

Two primary applications of geotextiles being used in the filtration function are available for the road situation. The first applies to the construction of subsoil or interceptor drains. In this application a filter geotextile surrounding an open graded aggregate can be used as an economic substitute for granular filters (Chapter 8). The second application is the placement of the geotextile between the subgrade and granular sub-base which, in conjunction with the separation function, will alleviate the contamination of sub-base from soil fines "pumped" from the subgrade during dynamic traffic loading (Chapter 9).

The filtration function requires design and a knowledge of the grading and permeability of the protected soil(s). Geotextile parameters which are important to filtration are the pore size, fabric permeability and the long-term soil/fabric permeability. While manufacturers offer filtration design charts for their products, the application to New Zealand conditions with a predominance of cohesive soils needs to be addressed (Chapter 8).

In general only non-woven geotextiles are suitable for the filtration function although monofilament woven products can be used to protect coarser sand/gravel soils. Tape-woven products should be avoided because of their low permeability.

7.4 REINFORCEMENT

Reinforcement is the increase in strength or stability of a soil or rock body. Geotextiles are a tensile strength material, which can complement soils and rocks which are weak in tension. For this reason they can be particularly suitable for reinforcement and load distribution because of the limited thickness and the ability to use the friction of the surrounding material to resist tensile forces.

With the reinforcement function, the three mechanisms are:

(i) membrane,
(ii) shear, and
(iii) anchorage,

but for these to act, the stiffness of the fabric must be matched to that of the surrounding soil (strain controlled). The failure or deformation mechanism of construction without the geotextile must be known to determine the most effective position to place the geotextile and the most suitable product to use. Similarly the failure and deformation mechanisms of construction with the geotextile must be known in order to know the required properties and installation procedures. In addition, subsidiary requirements such as transfer of stress across fabric overlaps and/or sewn seam joining needs to be addressed. Specific design is required for any project using the reinforcement function of geotextiles.

The reinforcement function is significant in civil engineering structures that can be associated with roads such as embankments and retaining walls, but in road pavements the applications are limited (Chapters 11 and 12).

The principle mechanism of a conventional woven (or non-woven) geotextile under a pavement system is the membrane type and relatively large deformations are required before any significant tensile stress is developed in the geotextile (Chan *et al.* 1989). Applications are generally limited to unsealed temporary roads over relatively soft subgrades where large local surface rutting can be initially tolerated (Chapter 11). High strength, low modulus, woven geotextiles are the most suitable products although there is some evidence to suggest that non-woven geotextiles may have similar properties when confined by granular or sand soils. The use of high strength, relatively stiff, geogrids is a promising development of this application. When using geogrids the granular sub-base partially penetrates the apertures of the grid and additional local tensile stresses can be developed at the base of the granular layer (Netlon Ltd 1989). In all cases the application is only applicable to low load-bearing soils (CBR <4).

Another possible road application of the reinforcement function is the alleviation of crack propagation in concrete road slabs into an asphaltic overlay. In this case the geotextile is placed at the base of the overlay. This application has been extensively researched by the Netlon manufacturers using moderately high strength, low modulus, Tensar geogrids. These products still allow the overlay to bond with the lower pavement layer and encapsulate the geotextile.

Geotextile parameters that are important to the reinforcement function are the tensile strength, modulus and creep characteristics, the latter being more significant to static loading than to the dynamic road pavement stresses. Soil properties that are significant are the strength (e.g. CBR), modulus and the geotextile/soil friction parameter.

In general only the high strength woven geotextiles or geogrids (≥ 40 kN/m) are suitable for reinforcement. For static loading very high strength products are generally required (≥200 kN/m) and the polymer type is significant when assessing the creep properties. The heavy weight woven polyester or polyamide polymer geotextiles or geogrids appear to be the most suitable for sustained static loading although the polymer- aligned, high density, polyethylene geogrids are very suitable when used at the appropriate stress level specified by the manufacturer (Chapter 5).

7.5 DRAINAGE

The drainage function is the ability of a geotextile to transmit water in the plane of the fabric. All geotextiles can provide this function to a very limited degree. Only the thicker needle-punched geotextiles have sufficiently high transmissivity for practical purposes although the flow rate is highly dependent on the applied normal pressure (Chapter 5). The relatively narrow section of conventional geotextiles results in a low discharge capacity which generally precludes utilising this function in road situations without addition of adjoining sand or aggregate drainage courses. The development of very thick needle-punched products may alter this requirement.

Notwithstanding this, geocomposites are now being manufactured that can discharge water at a significant rate. These are generally constructed by wrapping a filter geotextile around a drainage core. The products with higher transmissivity can replace the conventional subsoil and pavement drains (Koerner 1989, Dempsey 1989; Chapter 8).

7.6 REFERENCES

Chan, F., Barksdale, R.D., Brown, S.F. 1989. Aggregate base reinforcement of surface pavements. *Geotextiles and Geomembranes* 8(3): 165-189.

Dempsey, B.J. 1989. Hydraulic requirements of geocomposite fin-drain materials utilised in pavement sub-drainage. *Geotextiles and Geomembranes* 8(3): 191-215.

Koerner, R.M. 1986. *Designing with geosynthetics.* Prentice-Hall, Englewood Cliffs, NJ.

Lytton, B.L. 1989. Use of geotextiles for reinforcement and strain relief in asphalt concrete. *Geotextiles and Geomembranes* 8(3): 217-237.

Netlon Limited 1989. *"Tensar" Geogrids in Civil Engineering.* Netlon Ltd, UK.

Rankilor, P.R. 1982. Design procedures. Fabric Report. *Civil Engineering July 1982*: 38-39.

CHAPTER 8

SUBSURFACE DRAINAGE

8.1 GENERAL COMMENTS

Because of the New Zealand climate, topography and geology, water tables near the ground surface are the general rule. For this reason, subsurface drainage is of significant importance in roading. The ingress of free water or a water table in the pavement layer can generate severe reductions in strength, stiffness and the useful life of the composite system. This is especially significant if the pavement is allowed to become fully saturated.

To enable subsurface drainage to be relatively permanent and maintenance-free, a filter is required that alleviates the migration of subsoil particles into the drain while maintaining an adequate permeability. Geotextile filters have been found to make an excellent replacement for the traditional graded granular filters. They have a number of advantages over granular filters in that they have consistently well defined properties and are easy to install, thus offering cost effectiveness. In addition, greater flexibility with the type of drainage system is available, notably replacing the traditional perforated pipe with an open graded aggregate in a trench, lined with a geotextile filter ("french drain"), or with a completely prefabricated geocomposite drain (Section 8.12). Both these replacements can lead to further cost saving. Figure 8.1 shows a variety of subsurface drainage systems that can be used. A typical TNZ(NRB) F/6 Specification is that shown in Figure 8.1a.

As with aggregate filters, the use of geotextile filters requires design. Consideration must be given to the hydraulic requirements of the drain and the relevant properties of the soil to be retained in addition to the geotextile filter properties. The flow characteristics of open graded aggregates need to be assessed (Section 8.11). Secondary considerations are mechanical (Section 8.9) and durability (Section 8.10) requirements.

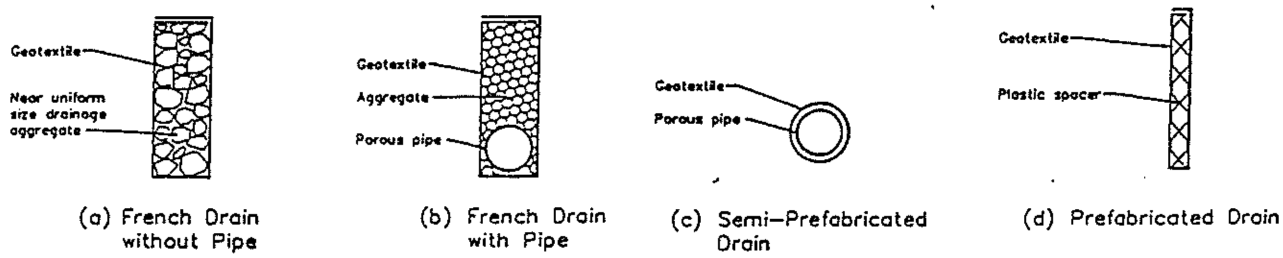


FIGURE 8.1
Types of Subsurface Drainage Structures Using Geotextiles

8.2 HYDRAULIC CONDITIONS OF SUBSOIL DRAINS, AND SOIL TRANSPORTATION

The basic requirement of any subsoil drain is to intercept and control the infiltrated subsurface water so that a road pavement, building platform or similar structures are not inundated with ground water. Subsoil drains are not generally designed to intercept surface run off water, and are overlain with compacted clay or a similar low permeability material.

The use of a granular or geotextile filter system is primarily required to alleviate siltation such that the drain can function over a long period of time without a detrimental increase in the hydraulic gradient and subsequent failure of the drain by blockage and/or erosion of the upstream soil.

Geotextile or granular filters are not highly permeable systems (k_f = 10^{-3} to 10^{-4}m/sec) and can not be expected to carry the water flow from very high capacity aquifers. In such cases a pipe(s) open to the aquifer should be used in conjunction with the filter. The primary purpose of road subsoil drains is to permanently control the free water surface to a minimum of 150mm below the lowest part of the granular sub-base and/or basecourse layers of the pavement, and at least 400mm below permanent seal level. These minimum depths should be increased for roads carrying heavier traffic.

The most critical situations to control include pavements between deep cut excavations and/or steep longitudinal road grades over water-bearing subgrade soils. These situations can have hydraulic gradients which may result in an artesian head of water above a pavement level. The former situation may require deeper intercepter subsoil drains and the latter a supplementary transverse drainage system.

The development and action of the free water surface as influenced by a subsurface drain is significant to the filter. The hydraulic conditions are transient. If a soil having a high water table is trench excavated the initial exit hydraulic gradient is high ($\approx$>5). This decreases at a decreasing rate to a steady state condition with a low exit hydraulic gradient ($\approx$0.5). The period of time to reach this steady state may be several days for very permeable soils to years for low permeability soils. Another transient hydraulic condition imposed on the system is the periodic influx of infiltration water from rainfall which controls the final steady state condition and influences the rate of the development of the free water surface towards this state. The steady state condition is thus partially transient with a cyclic hydraulic gradient depending on the infiltration. With an adequately designed and performing drainage system, however, this will always be significantly lower than the condition which would have prevailed without the subsoil system.

The susceptibility of soils to erosion and piping is dependent on the soil grading and consistency. A general range of these soil properties is shown in Figure 8.2 along with a specific soil grading that is susceptible to piping erosion.

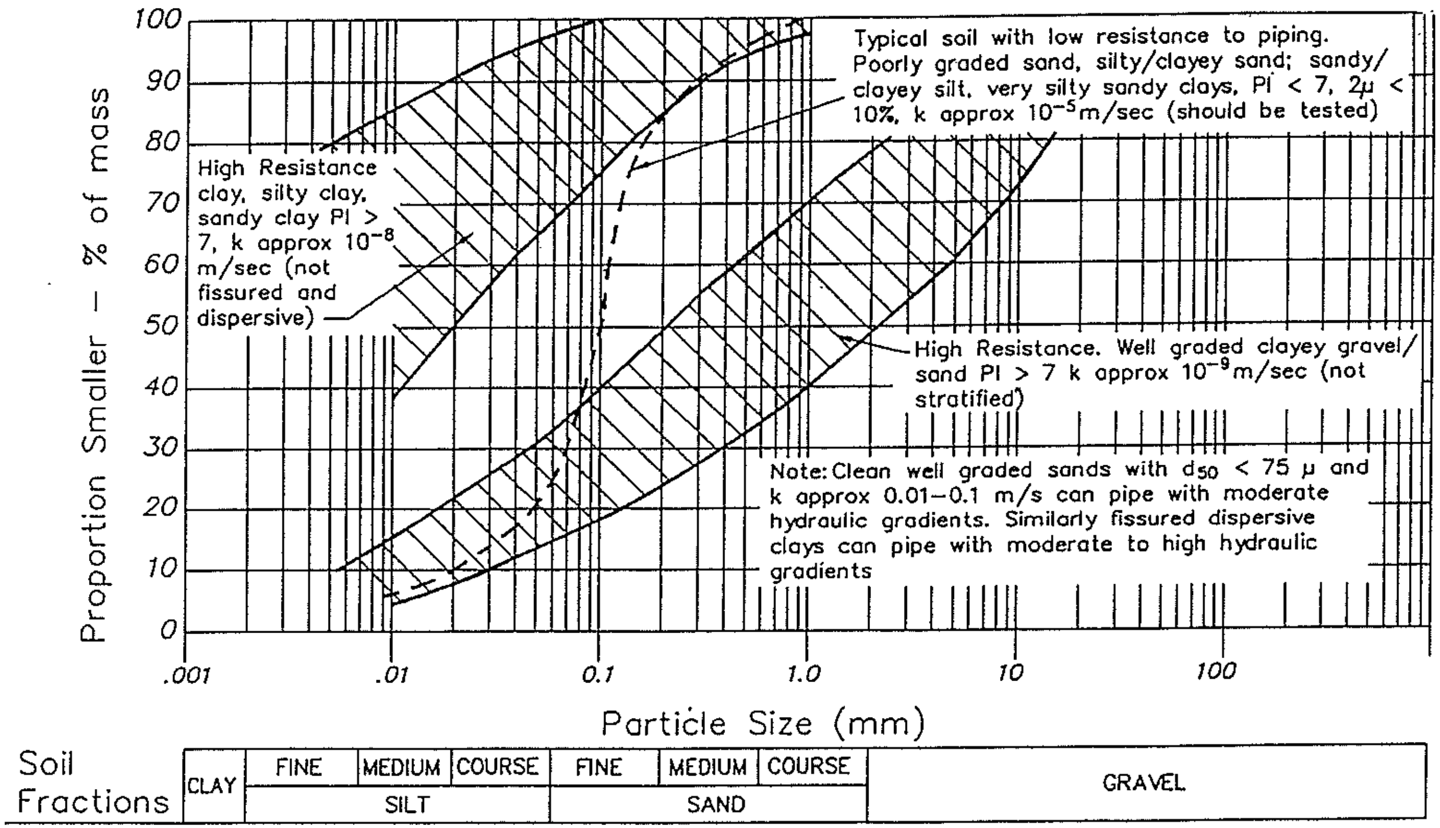
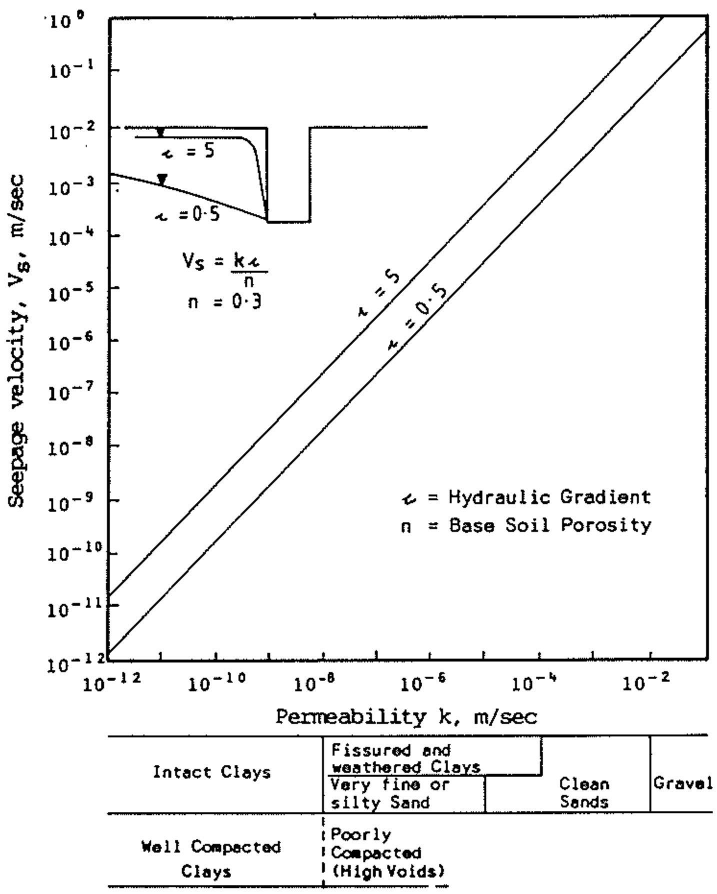
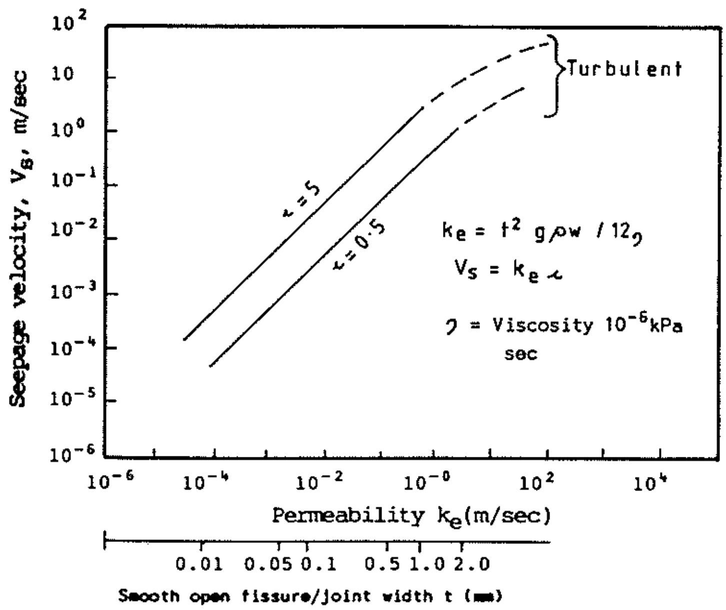


FIGURE 8.2
Susceptibility of Soils to Erosion and Piping



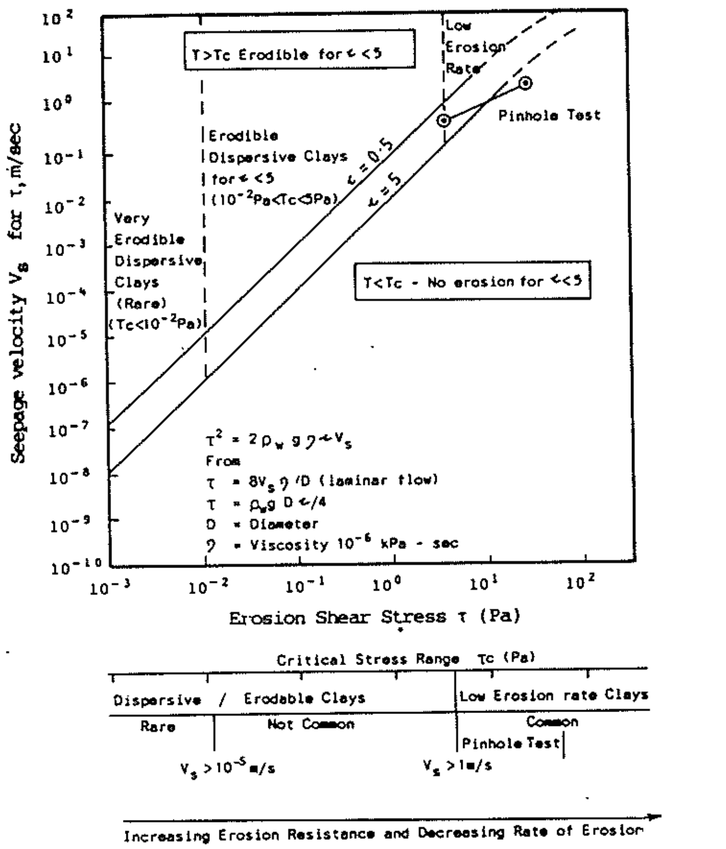
(a) Relationship between seepage velocity and permeability for intact soils.



(b) Relationship between seepage velocity and permeability for parallel, non-filled fissures

FIGURE 8.3
Erosion Properties of Soils Under the Hydraulic Conditions of Subsoil Drains and Geotextile Filters (East and Hudson 1987)

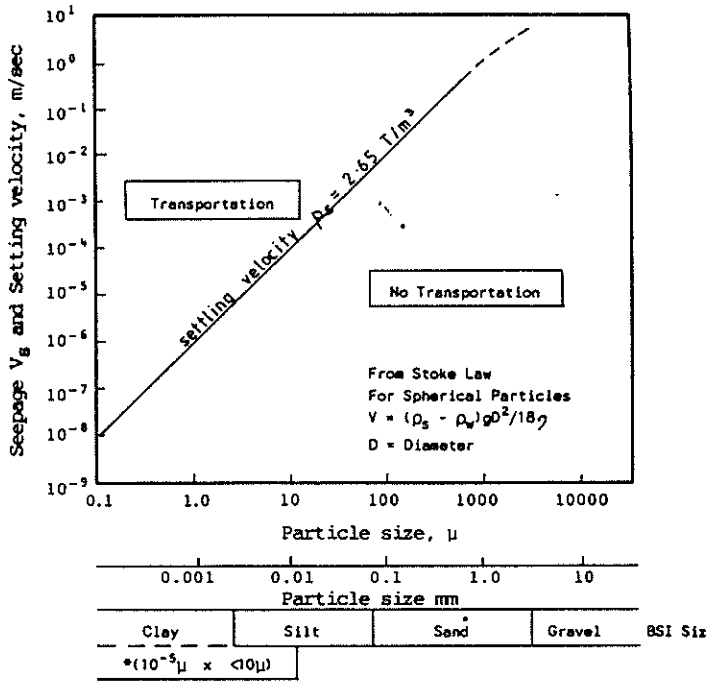
(c)



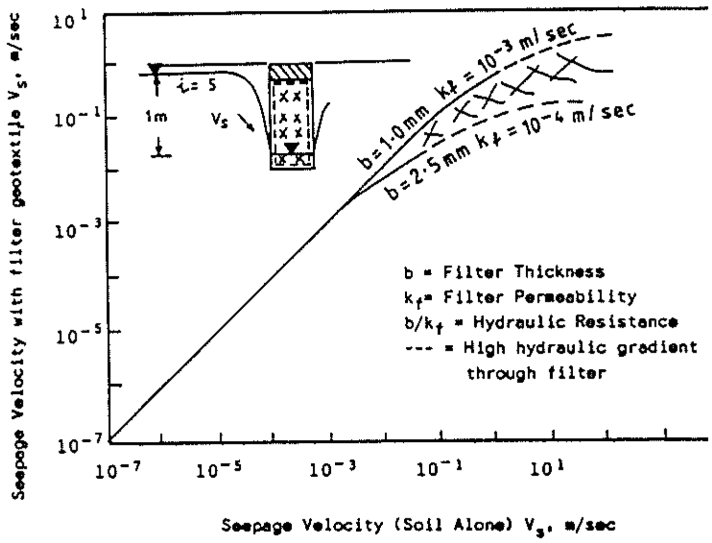
Note 1 : Above relationship is for compacted soil and distilled water as the erosion fluid. Water with soluble salts and undisturbed soils will have a higher erosion resistance (τc) and a lower rate of erosion.

Note 2 : For parallel fissures $V_s = V_s$ Holes x 1.33.

(d)



*True (Thickness x length and/or width dimension range of common clay minerals (13)



(c) Relationship between erosion shear stress (T), critical shear stress (Tc), for clays ($2\mu > 12\%$), and seepage velocity (V_s) in small diameter cylindrical holes.

(d) Relationship between settling velocity and particle size.

(e) Seepage velocity of a soil abutting a geotextile filter for an overall gradient of 5 and a 1.0m hydraulic head.

(e)

FIGURE 8.3 continued

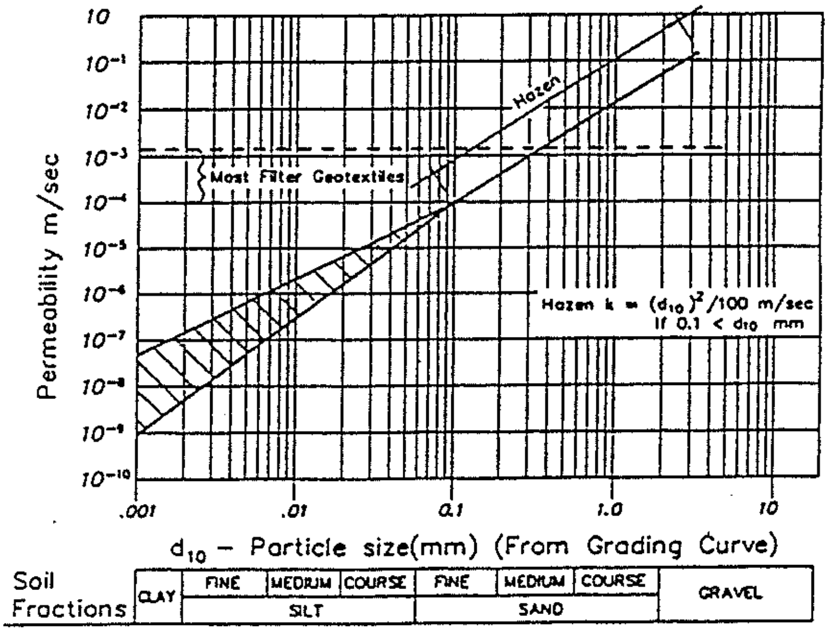


FIGURE 8.4
Approximate Permeability Relative to D_{10} for Intact Soils

The seepage velocity (V_s) through the soil governs the erodibility of the soil and the size of the particle that can be transported by the pore water without settling. Seepage velocity is governed by the hydraulic gradient (i) for a given intact soil of permeability (k) by the relationship $v_s = ki/n$ (where n is the porosity of a soil ~0.3) (Figure 8.3a). A similar relationship exists for fissured/structured soils (Figure 8.3b).

The piping potential can be evaluated by applying the realistic hydraulic gradient (i = 5 to 0.5) to the relationship between seepage velocity, soil macrostructure, permeability, erodibility and particle transportation. For a piping condition to exist at the hydraulic exit of a trench drain, the seepage velocity must first be capable of eroding the soil and second transport the eroded soil particles. These conditions have been reviewed in detail by East and Hudson (1987) (Figures 8.3 a to d).

Cohesionless soils are considered erodible under even very low seepage velocities although the degree of erosion will depend on the hydraulic gradient which is unlikely to exceed unity critical value in practical cases. Under this condition the soil particles can be transported although the maximum size is controlled by the seepage velocity (Figure 8.3d). The permeability of cohesionless soils can be assumed from the "effective size" d_{10} obtained from a grading test. This relationship is shown in Figure 8.4 where it has been extended to include intact cohesive soils.

Most intact cohesive soils require seepage velocities well in excess of any value to be encountered in practice and hence piping does not occur in these soils (Figure 8.3 a, c). Piping is possible in the water-bearing discontinuities of fissured cohesive soils and especially in the relatively uncommon "dispersive" soils which are highly erodible in any state. Some South Island loess soils are of this latter type.

The significant difference between the piping potential of cohesionless and cohesive soils is such that the two types should be treated separately for filter design and application. As real soils can have a particle distribution encompassing a range of soil sizes some soils are difficult to specify. For filter design the properties of cohesionless soils would, in general, be such that the PI<7 and/or clay size 2μm <10%. Some intermediate cases may require further evaluation by erosion testing but, in general, all soils with a PI>15 can be considered cohesive. The permeability of the soils is also significant and can be assessed from Figure 8.4 for cohesionless and intact cohesive soils. For fissured soils Figure 8.3b can be used for an assessment.

From Figures 8.3a-d, it can be seen that at the initial high hydraulic gradient stage of the subsoil drain, larger eroded particles can be transported and as the steady state is approached only progressively smaller particles can be transported. For cohesive soils the erosion potential is similarly reduced. It will be shown that this is significant to the development of the filter drain and it can be postulated that in future the design geotextile pore size (or d_{15} granular filter size) will be related to the seepage velocity of the protected soil for filter piping criteria.

The presence of a filter at the hydraulic exit can govern the overall seepage velocity and effectiveness of the drain but, providing the permeability (k_f) of the geotextile is of the same order or greater than that of the base soil (k), the hydraulic resistance (C_f = thickness b/k_f) is not significant and little reduction in seepage velocity will result across the filter at the commencement of flow. This condition is a criterion of geotextile filter design ($k_f > k$). This condition was reviewed by East and Hudson (1987) (Figure 8.3e) and for the general range of filter geotextiles (k_f = 10^{-3} to 10^{-4}m/sec) C_f = 0.5 to 30m/m/sec, the resulting seepage velocity cannot transport particle sizes coarser than medium sand size (<0.5mm). Note that monofilament woven geotextiles with a higher permeability than 10^{-3} m/sec are available but the higher pore size and corresponding reduced retention properties should be carefully assessed before using them.

If the soil is eroded and transported towards the subsoil drain in a unidirectional flow the final permeability of the filter is that of the retained soil "filter cake" built up on the geotextile interface. The permeability of the soil/filter cake/geotextile is generally lower than that of the initial soil/geotextile but, with an adequate filter design, this reduction is insufficient to significantly effect the overall performance of the drain. A reduction in seepage velocity may accompany this action with a corresponding reduction in erodibility and transported particle size. Similar conditions occur with granular filters.

Note : *The above applies to subsoil drains constructed as cut offs to protect a road pavement or property against inundation from conventional infiltration. Cases of protection from instability where deep cut off drains or downslope counterfort trenches may be required to control high flow aquifers and lower the ground water level of the unstable area require special consideration. In these cases the steep hydraulic gradient may not be transient and the conditions could be similar to those of a hydraulic structure. Special care is required when using geotextile (or granular) filters in these cases to ensure that the long-term "filter cake" permeability matches or exceeds the dewatering requirements. The currently available high permeability non-woven geotextiles should be adequate in such situations but the addition of a non-perforated pipe open to the water source(s) outside the geotextile is considered to be good insurance. Each site would need to be evaluated because the hydraulic conditions govern the requirements.*

8.3 ADVANTAGES AND DISADVANTAGES OF GEOTEXTILE FILTERS

Traditionally, granular filters have been used in unidirectional flow subsurface drains. One disadvantage of granular filters is the difficulty and uncertainty of obtaining a consistent uniform supply of the correct grading. Prevention of segregation is another problem. Although at present there is no shortage of hard rock quarries in New Zealand, a true granular filter is generally a high cost, low volume product of blended and/or crushed aggregate which is not always readily available. These factors, and the transport costs from the quarry source, can lead to the use of less costly alternative grades of aggregate to the detriment of the long-term performance of the subsoil drain, especially in sands and silts and other soils with a high susceptibility to piping erosion (Figure 8.2).

Use of a geotextile filter avoids these disadvantages and in most situations offers a lower installed cost. A geotextile filter surrounding a relatively economic and more readily available open graded aggregate results in a subsoil drain with consistent properties with little difference in construction techniques (Figure 8.1a). For example, a less costly, near single-size aggregate can be used, perhaps from an alternative source.

The recently marketed geocomposite or geotextile-wrapped, waffle-core strip drains (Figure 8.1d) may offer further economic advantages due to a significant decrease in trench width and corresponding reduction in volume of excavation and backfill.

However, geotextile filters have their own potential problems. These problems include degradation under ultraviolet light, possible puncturing and tearing during installation, and adequate specification and testing of the geotextile. UV degradation can be prevented by non-exposure to sunlight, puncturing by appropriate geotextile selection and careful handling, and progress on testing methods is being made in many countries.

8.4 RELEVANT GEOTEXTILE PROPERTIES FOR FILTRATION

One of the principle problems when comparing design criteria is the lack of standardisation of test methods and definitions of the geotextiles. This necessitates some conservatism when applying the criteria.

The two most relevant properties for filtration are pore size and permeability. The various methods for measuring these properties have been reviewed in Chapter 4.

8.4.1 Geotextile Pore Size

The manufacturers use a wide range of test methods to determine pore size which makes comparison difficult.

In general, the total pore size distribution is tested and any pore size can be selected. Manufacturers' data generally quote two sizes, O_{50} and O_{90}. The former is the opening size where 50% of the pores are smaller and the latter where 90% of the pores are smaller.

The pore size parameter is essential to the design of the filter but some decision is required to assess which particular pore size effectively controls the filtration behaviour of the geotextile. Two equivalent terms can be used for this pore size: O_e the "Effective pore size", or AOS the "Apparent opening size". The latter is used in this publication. For non-woven filter geotextiles, O_{90} realistically controls the geotextile behaviour. For woven geotextiles, with discrete openings, O_{50} (the true "Equivalent opening size" or EOS) controls the behaviour but there is little difference in this size and 0_{90} in a well made geotextile.

Unfortunately the measured pore size determination is highly dependent on the test method and, at this stage of the technology, some conservatism is required in the design criteria to allow for test methods that may not be applicable to actual conditions.

8.4.2 Geotextile Permeability

Geotextile filter permeability is generally tested by a quantity of water passing per unit area per unit time (litre/m^2/sec) under a specified small head of water (generally 50mm or 100mm). This water volume flow rate measurement has been called Q (Q_{50} or Q_{100} in this report. Although the heads are small, the hydraulic gradient and seepage velocities are unrealistically high when compared with field conditions. As steady laminar flow conditions do not occur with this testing, Darcy's Law no longer applies. More recently, testing has been undertaken to control the seepage velocity or flow rate so that approximate laminar flow does exist (e.g. 1cm/sec). The head of water (mm) for this condition is measured and used as a criterion for filter geotextiles (Transit New Zealand/NRB F/6 1985, 1986).

Most geotextile criteria requires the filter permeability (k_f) in true permeability units (m/sec), as measured by Darcy's Law, to compare with the permeability of the soil. To obtain the k_f value from the measured parameters the following relationship must be used:

(i) k_f (m/sec) = (10^{-3} x Q x b)/h where:

Q	=	water volume flow rate (litres/m^2/sec)
h	=	head applied (mm)
b	=	thickness of geotextile (mm)

The relationship is strictly only applicable at very low heads where laminar flow exists. Nevertheless, the relationship is conservative in the case of non-laminar flow. A typical thin (t<1mm) filter geotextile has a Q value of 100 litre/m^2/sec at 100mm head which is equivalent to a permeability (k_f) of at least 10^{-3}m/sec.

or

(ii) k_f (m/sec) = (v x b)/a where:

a = measured head (mm)
v = seepage velocity (m/sec)
b = thickness of geotextile (mm)

If the seepage velocity is in the order of 10^{-2} m/sec the conditions should be laminar (Tan *et al.* 1982). Current TNZ(NRB) filter criteria restrict the head to 10mm at a velocity of 10^{-2} m/sec (1cm/sec), which is equivalent to a minimum k_f value in the order of 10^{-3} m/sec for the general range of thin geotextiles (b < 1mm).

It would be perhaps better to use the water volume flow rate (Q) directly into filtration criteria to avoid calculations which rely on both laminar flow and a good assessment of geotextile thickness. One such relation has been given by Lawson (1986b) and is shown on Figure 8.5 which also includes allowance for minimum long-term filter clogging.

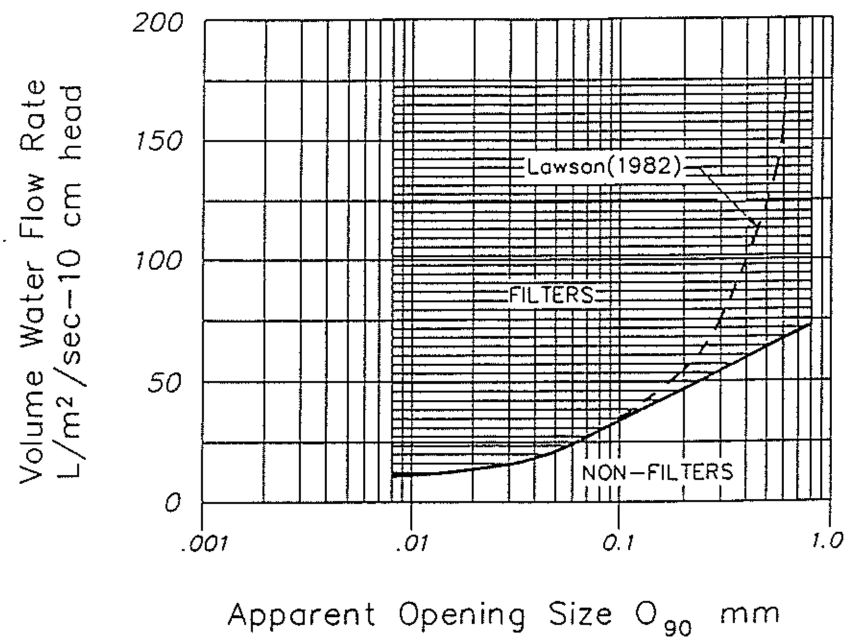


FIGURE 8.5
Minimum Volume Water Flow Requirements for Geotextile Filters (Lawson 1986b)

8.5 GEOTEXTILE FILTRATION PRODUCTS

Rapid advancements have been made in the understanding and performance of thermoplastic geotextiles and the industry now manufactures a wide range of products that fulfill specialised functions in place of the "do anything" fabrics that were available in the past. Geotextile filters are now available in a wide range of pore sizes (O_{50} = 30 to 250μm) and thicknesses (0.5 to 3mm) with good permeability (k_f = 10^{-3} to 10^{-4}m/sec). Sufficient long-term experience has been gained to indicate that little significant degradation occurs with time if the fabric is buried. Geocomposites are becoming available which include the geotextile-wrapped waffle core which is replacing granular backfill and/or perforated pipe (Figure 8.1d). These filtration geotextiles are generally the non-woven type (e.g. needle-punched or melt-bonded thermoplastic felts) but for coarser soils the monofilament woven fabrics can be utilised. One advantage of the thicker non-woven types is that a pore size grading approaching that of a graded granular filter can be built into the geotextile although the hydraulic resistance is generally greater than the thinner fabrics. The low cost tape (slit film)-woven fabrics are not recommended for filtration use as they have a very low pore open area (POA) with a corresponding low permeability.

There is no unique relationship between permeability and pore size although there is some correlation when the method of manufacture is considered. In general, for a given pore size, non-woven needle-punched fabrics are the most permeable (Q_{100} L/m^2/sec = 1.14 $O_{90}\mu$m*) followed by non-woven melded and spunbond (Q_{100} L/m^2/sec = 0.64 $O_{90}\mu$m*) with woven fabrics having an extreme range depending whether they are monofilament (Q_{100} L/m^2/sec = 0.5 $O_{90}\mu$m*) to tape (Q_{100} L/m^2/sec = 0.05 $0_{90}\mu$m*) (Waters 1984).* The range in permeability for woven fabrics is highly dependent on the Percentage Open Area (POA) with monofilament having a range of 10% to 30% and tape generally below 10%, depending on the denier of the yarn. It is for this reason that tape-woven fabrics are not generally recommended for filtration, in addition to their higher susceptibility to clogging.

The Apparent Open Pore Size (O_{90}) also depends to some extent on the method of manufacture. Melded non-woven products can have O_{90} values as low as 30μm yet have good permeability. Needle-punched products have O_{90} values generally in excess of 100μm with excellent permeability. Woven fabrics can have O_{90} values as low as 75μm but 250μm is the usual order because of the permeability restrictions and the economics of producing a fine denier weave (Chapters 3 and 5).

A number of combination geotextiles have been developed that can alter the properties of the basic geotextiles and give a lower pore size distribution (Chapter 3).

8.6 UNIDIRECTIONAL FILTRATION

In developing the design criteria for filters for subsoil drains, the assumptions and laboratory testing have been based on the concept of unidirectional flow, i.e. water flows only from the soil into the subsoil drain and it never reverses. This assumption is realistic given intelligent subsoil drain design in practice and, in particular, that surface water should not be discharged into subsoil drains, and that the outlets of subsoil drains will not become submerged.

*Note: use relationship with caution.

8.6.1 General Conditions And Design Criteria

When a geotextile is placed against a soil and water is allowed to flow from the soil, a complex interaction occurs between the soil particles and the pores of the geotextile. Idealistically a bridging network or "filter cake" of soil develops after a period of time (Figure 8.6). In the long term this "filter cake" acts as the filter, and the geotextile is the medium which enables the "cake" to form. With matching of the geotextile pore size and the soil grading, the initial formation of the filter cake partially blocks the geotextile with particles larger than the pores, and then a slower "blocking" as finer soil particles progressively build up behind larger particles. The time required to form the final "cake" varies from several days to several months, largely depending on the grain sizes of the soil and the hydraulic gradient/water velocity (Carroll 1983, Koerner and Ko 1982, Lawson 1982, 1986b, Millar *et al.* 1980). The thickness of "cake" can be in the order of 10 to 20mm (Miller *et al.* 1980). As the filter zone forms, the particles finer than the pores of the geotextile are washed through until "blocking" occurs. The same action occurs in granular filters within a thin zone of the aggregate adjacent to the soil. In practice the conditions are likely to be more complex (Miller *et al.* 1980) especially with non-woven geotextiles. In non-woven geotextile filters, the initial stages of the filter zone can be formed by two combined actions. The first is by external "blocking" where the particles are deposited on the interface of the filter. Such particles generally have sizes greater than the pore sizes exposed at the interface. The second is by internal "clogging" where the particles become interlocked in the internal fibre structure of the filter but are of sufficient size or shape to be retained. In a single layer, monofilament, woven geotextile, the filter zone can only form by the "blocking" action while the "clogging" action is more likely to occur in the thicker non-woven products.

The maximum size of the particles transported from the soil to the drain is limited by the seepage velocity of the soil and/or geotextile. This size and smaller are transported to the geotextile with, initially, many particles finer than the geotextile pore size passing through the geotextile. The coarser particles gradually "block" the geotextile pores and commence the development of the "filter cake".

The filter requirements are that the "cake" is able to build up without a significant loss of fines from the soil into the protected drain and that the final permeability of the filter zone/geotextile is sufficient to allow flow without an excessive build up of hydraulic gradient (Figure 8.7). A low final permeability (detrimental clogging) is generally considered more of a problem than excessive wash through of fines (Hoare 1982, 1984). The availability of high permeability non-woven geotextiles with the low pore size ($<200\mu m$) and numerous flow paths (high POA) has effectively eliminated a low final permeability as the high initial water velocity can more successfully transport larger particles to commence the "blocking" and control the final filter zone permeability and piping. Tape-woven geotextiles are not recommended as filter forming geotextiles as the low permeability and low POA of these products make them susceptible to permeability clogging (Koerner 1986).

Woven monofilament geotextiles are suitable, providing that the POA is in excess of 40% for cohesionless soils and 10% for cohesive soils but it should be noted that such products cannot be manufactured with low pore sizes ($\geq 200\mu m$).

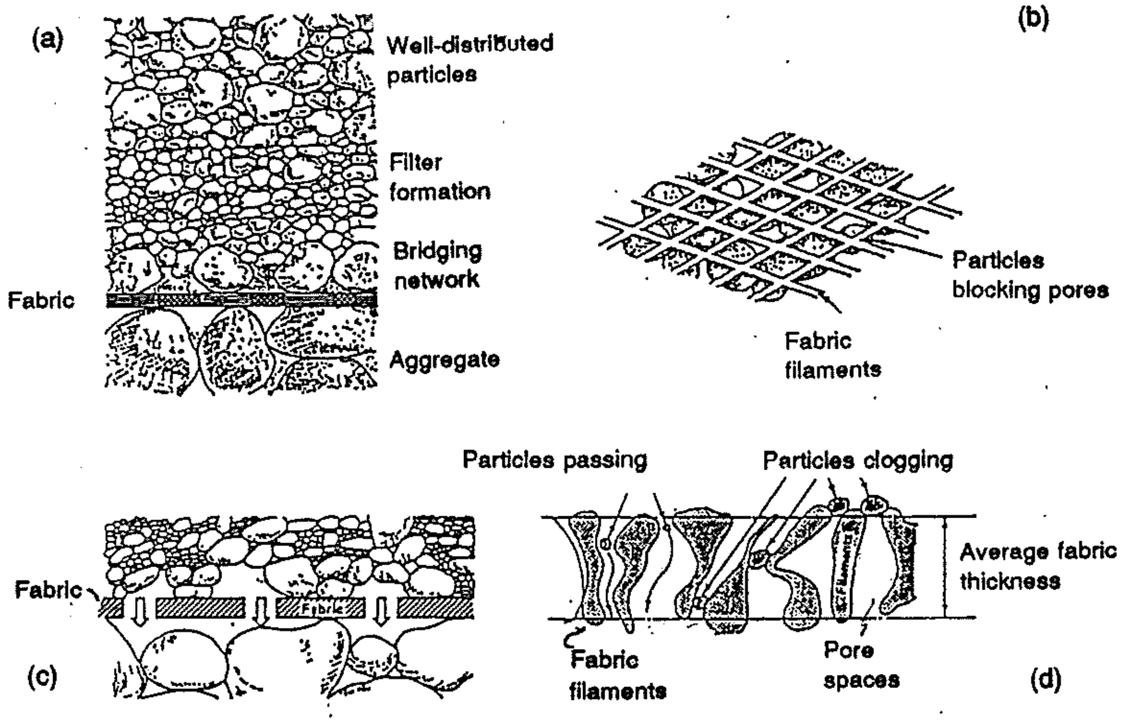


FIGURE 8.6
Various Hypothetical Mechanisms Involved in Long-Term Soil-to-fabric Flow Compatibility: (a) Formation of an upstream soil filter; (b) Upstream particles blocking geotextile openings; (c) Upstream particles arching over geotextile openings; (d) Soil particles clogged within geotextile structure. (Koerner 1986)

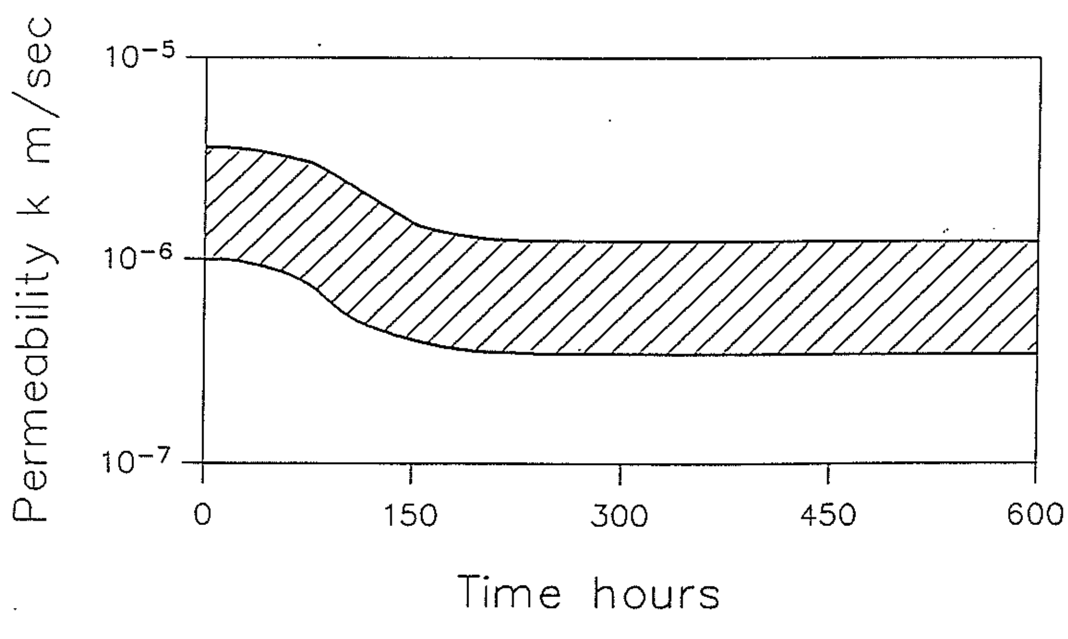


FIGURE 8.7
Reduction in Permeability with the Formation of a Filter Cake in a Range of Sandy Silt Soils.

8.7 DESIGN METHOD FOR COHESIONLESS SOILS

The flow of water through a cohesionless soil can initiate movement of soil particles which may be considered in three degrees of intensity:

(a) Local motion of a few fine particles which may give the first indication that more vigorous movement is imminent.

(b) Migration of fine particles through the voids between larger particles in the soil either to be carried away with the water or to percolate into the voids in an adjacent mass of soil. This process is known as "suffusion".

(c) Internal erosion and removal of particles, known as "piping", which occurs when the hydraulic gradient exceeds the critical value.

Two mechanisms involving suffusion are:

(i) A fine grained soil (A) overlying a coarse grained soil (B) where water flowing downwards is likely to carry fine particles from soil A into the relatively large voids in soil B.

(ii) A soil consisting of fine particles contained in a matrix of coarser particles where water flowing from this soil may carry fine particles away with it if the velocity of flow is great enough.

Suffusion of fine particles can occur not only from or into materials which have been placed or compacted as fill but also from one naturally deposited soil in to another if they have never been subjected to a significant hydraulic gradient and a movement of water is artificially imposed.

In the practical roading situation, suffusion of types (a) and (b) are the most likely conditions. It would be a relatively unique situation if a subsoil drain were placed solely in a cohesionless soil with a water table above the drain invert and the hydraulic gradient exceeds the critical value (c). Nevertheless, true piping situations may arise where thin sand layers are encountered in the trench. For the purpose of filter design, all three degrees of intensity will be called piping as, in this context, all conditions result in erosion and transported soil.

8.7.1 Filter Requirements And Filtration Criteria For Cohesionless Soils

Subsoil drain criteria for cohesionless soils have been based on the well proven granular filter design and the relationship that the average pore size of a soil (0_{50}) is related to the 10% soil grading (D_{10}) by $0_{50} = 0.2\ D_{10}$ (Atterberg relationship for spherical particles) and hence the average pore size of a granular filter is $0_{50} = 0.2\ D_{10}$. The Equivalent

Opening Size (EOS) of a geotextile with discrete openings (woven) is considered to be equivalent to the average pore size of a granular filter (Hoare 1982).

Based on the above criteria, the following rules apply for unidirectional flow geotextile filters:

Rule 1 : To alleviate piping

$$\frac{\text{EOS } (O_{50})}{85\% \text{ size of protected soil } (D_{85})} < 1$$

Rule 2 : To maintain permeability

$$\frac{\text{EOS } (O_{50})}{15\% \text{ size of protected soil } (D_{15})} < 1$$

Note: This is the basis for the current TNZ(NRB) F/6 Specification (1985, 1986).

These design criteria for geotextile filters are relatively conservative for very erodible cohesionless soils. Recent European developments in geotextile filtration have indicated possible relaxations in these design criteria for some conditions. Using the facts that the thicker non-woven geotextiles have a coefficient of uniformity ($U_f = 0_{60}/0_{10}$) (Schober and Teindl 1979) and protected soils with a high coefficient of uniformity ($U_B = D_{60}/D_{10}$) are less susceptible to piping (Rycroft and Jones 1982), the piping rule ($0_{50}/D_{85} < 1$) can be substantially relaxed. Laboratory tests on fine grained cohesionless soils (D_{85} = 10μm to 300μm) have backed up these conditions. The conventional piping rule approximates to a U_f and U_B value of unity. American testing authorities have verified a criteria of EOS/D_{85} of 2 to 3 for a wide range of cohesionless soils and geotextiles (Carroll 1983, Tan *et al.* 1982).

In a number of the more recent developments the 50% size of the base soil (D_{50}) has been utilised, but a review (Lawson 1986b) has verified that D_{85} is the more appropriate parameter for simple filter design. Of significance is the pore size of the geotextile to be used in the filter criteria. Both Hoare (1982) and Lawson (1982, 1986b) advocate the use of the Apparent Opening Size (AOS) of the geotextile filter which is equivalent to the 90% pore size O_{90}. Based on these modifications to the "granular equivalent" method the design criteria advocated by Lawson are recommended for cohesionless soils, ie

Rule 1 : To alleviate piping

$$\frac{\text{AOS } (O_{50})}{85\% \text{ size of protected soil } (D_{85})} < 1$$

Rule 2 : To maintain permeability

$$\frac{\text{AOS } (O_{50})}{15\% \text{ size of protected soil } (D_{15})} < 1$$

Lawson also qualified this latter criterion by relating the AOS to the permeability of the geotextile (Figure 8.5).

East and Hudson (1987), indicated that one further modification may be required depending on the grading of the soil. The protected soil grading to be used for filter design should contain only particle sizes that can be transported by the seepage velocity through the soil and geotextile. In general this limits the design D_{85} size to a maximum value in the order of $\leq$0.5mm (medium/coarse sand), i.e. $D_{85} \leq 0.5$mm should be used in the above piping criteria.

8.7.2 Clogging And Filter Permeability in Cohesionless Soils

Laboratory soil/geotextile permeability tests are more commonly carried out on cohesionless soils for engineering projects and various researchers (East and Hudson 1987, TNZ(NRB) F/6 1985, 1986, Lawson 1986a, b, Millar *et al.* 1980, Rycroft and Jones 1982, Wade *et al.* 1983) have indicated that the conventional piping design criterion (AOS $< D_{85}$) is applicable and conservative. The final permeability is not significantly different to that of the protected soil providing that reasonably high POA geotextiles are used. The final soil/geotextile interface permeability is the result of external blocking and internal clogging. To alleviate internal clogging at the expense of some initial piping it is generally accepted that the geotextile permeability (k_f) should have a higher permeability than the protected soil ($k_f/k > 5$ to 10) and that the AOS should be as large as the piping criteria will allow. East and Hudson (1987) reported testing indicated that the final interface permeability can reduce to below that of the protected soil but as this also occurs with granular filters there should be little concern in practice. In more critical situations (hydraulic structures) explicit measurements can be made in the region of the interface (Gradient Ratio Test — East and Hudson 1987, Koerner and Ko 1982, Haliburton and Wood 1982).

A low final permeability is considered more of a problem than excessive wash through of fines (Lawson 1986b) and hence it is desirable to maintain the AOS (O_{90}) as close as possible to the D_{85} size of the soil while still maintaining the piping criteria.

8.7.3 Design Method For Cohesionless Soils

(a) Carry out a grading test (sieve/hydrometer) on the soil (NZS4402 or ASTM 4421).

(b) If the soil is considered marginally cohesionless carry out a consistency test (PI) on the soil (NZS4402) and confirm PI < 7 and 2μm <10%.

(c) Check the D_{85} size of the soil grading and obtain the maximum AOS O_{90} pore size from the piping criteria. If the grading curve indicates $D_{85} > 0.5$mm (500=μm), then fix $D_{85} = 0.5$mm.

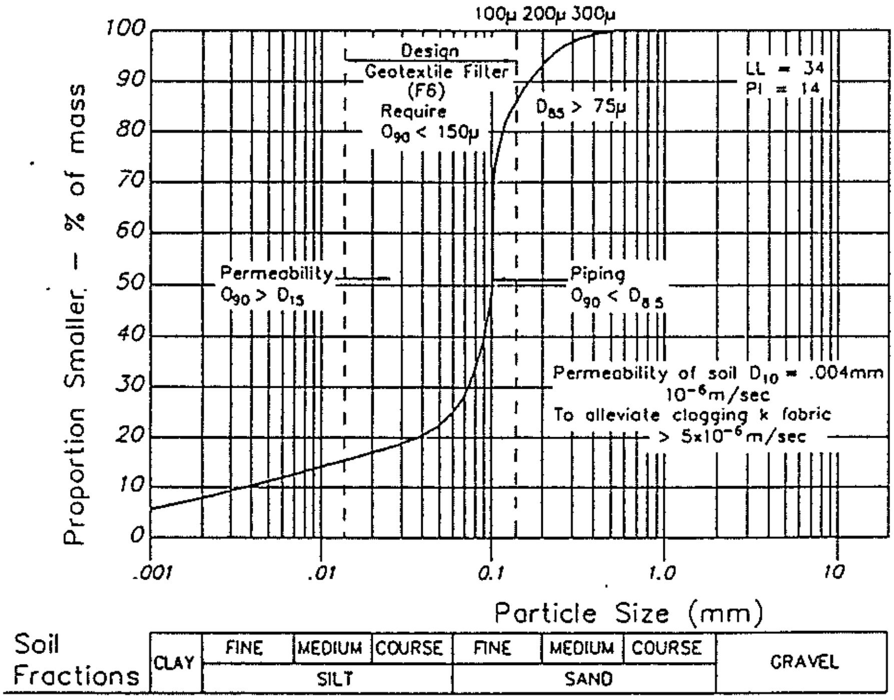
(d) Check the D_{15} size of the grading and obtain the minimum AOS 0_{90} pore size from the permeability criteria.

(e) Confirm that the geotextile permeability (k_f) is at least five times that of the soil (k). k_f can be obtained from the manufacturer's data (Section 8.4) or by testing. k can be obtained from the D_{10}/Permeability graph (Figure 8.4). In addition a woven geotextile must have an open pore area greater than 40% (ie tape-woven fabric cannot be used).

(f) Choose the appropriate geotextile for

$D_{15} < O_{90} < D_{85}$
$k_f > 5 \times k$
POA > 40% (i.e. non-woven or monofilament woven geotextile only can be used).

Example: Silty Sand Cohesionless Soil



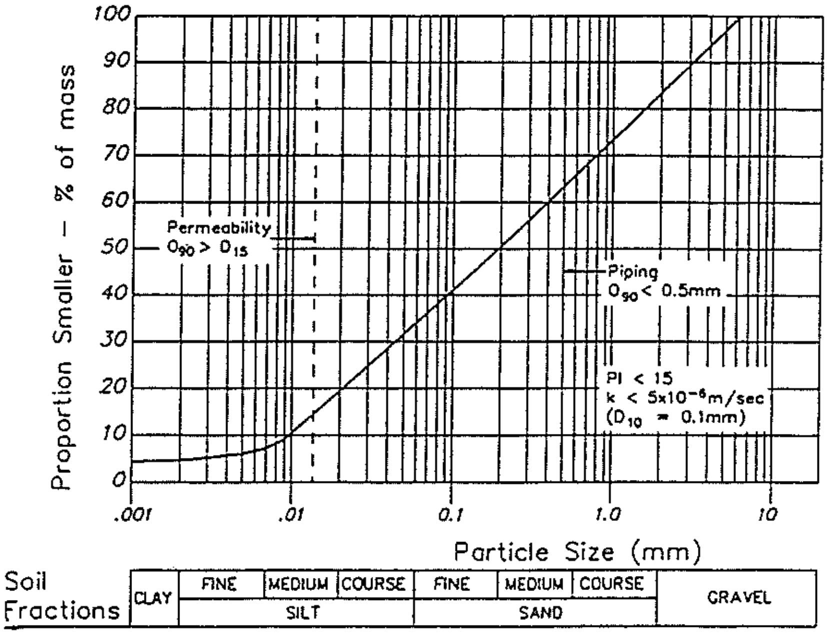
(a) Grading test (as above)
(b) PI = 14 but 2μm < 10%, therefore Cohesionless Soil
(c) D_{85} = 150μm (.15mm)
(d) D_{15} = 15μm (.015mm)
(e) D_{10} = 3μm (.003mm), therefore k ≈ 10^{-6}m/sec (Figure 8.4)

Decision:
(i) Required geotextile pore size 15μm < O_{90} < 150μm
(Upper limit should be used to result in maximum possible permeability)

(ii) Required geotextile permeability $k_f > 5 \times 10^{-6}$m/sec
(This should not be a problem as all non-woven and monofilament woven geotextiles have $k_f = 10^{-2}$ to 10^{-3}m/sec)

i.e. A non-woven geotextile with an AOS O_{90} less than 150μm should be suitable.

Example: Very Coarse Cohesionless Soil



(a) Grading test (as above)
(b) PI < 7 and $2\mu m$ < 10%, therefore Cohesionless Soil
(c) $D_{85} > 500\mu m$ (0.5mm), therefore design for piping based on $D_{85} = 500\mu m$ (0.5mm)
(d) $D_{15} = 15\mu m$ (.015mm)
(e) $D_{10} = 10\mu m$ (.01mm), therefore $k \approx 5 \times 10^{-5}$m/sec (Figure 8.4)

Decision:
(i) Required geotextile pore size $15\mu m < O_{90} < 500\mu m$

(ii) Required geotextile permeability $k_f > 2.5 \times 10^{-4}$m/sec

i.e. Any lightweight non-woven or monofilament woven geotextile within the typical AOS O_{90} range of $150\mu m$ to $250\mu m$ should be suitable.

8.8 DESIGN METHOD FOR COHESIVE SOILS

In the roading situation cohesive soils can be present in one of three states that govern the permeability of the soil:

(i) In situ, undisturbed, intact soils.

(ii) In situ, undisturbed and structured soils with fissures including remnant joints, bedding, layering, or abutting jointed rock at various intervals between the intact clay. Note that structured soils are likely to be more prevalent in natural deposits than generally accepted. These will become more pronounced in the trench wall as the initial hydraulic gradient will tend to open the fissures with the reduction in confining stress from trench excavation.

(iii) Disturbed and compacted soils. These soils can be either well compacted or poorly compacted with high voids.

With intact or well compacted cohesive soils the seepage velocities are insufficient to erode and/or transport soil particles under the considered practical hydraulic gradients. Piping conditions can occur in fissured structured cohesive soils with water-bearing discontinuities but only the uncommon, easily erosive, dispersive clays are likely to cause significant problems. Although the soils are considered to break down completely in the fissure, the maximum transportable particle size will likely be governed by the geotextile permeability and the seepage velocity, and is unlikely to exceed 0.5mm. Both erosion and transportation will decrease as the transient hydraulic gradient decreases to the steady state. High void, poorly compacted clays may have similar properties (East and Hudson 1987).

Sherard *et al.* (1984), demonstrated the non-fissured situation by subjecting a large number of well compacted clay and silt samples to extremely high hydraulic gradients ($i = 500$) in which no erosion took place until higher gradients resulted in hydraulic fractures. Sherard *et al.* also demonstrated, with a slot through a compacted soil, that under extremely high hydraulic gradients ($i>300$) and seepage velocities (15m/sec — turbulent) all soils are erodible.

The fissured situation has been demonstrated by Rohan *et al.* (1986) using a pinhole apparatus with two non-dispersive soils. Significant erosion took place only at exceedingly high seepage velocities but the results also indicated that slight erosion can take place in the range of velocities that could be present in subsoil drains.

The pinhole erosion apparatus is used for dispersive soils and there is no doubt that these relatively uncommon soils will completely erode under the hydraulic conditions of subsoil drains.

Some testing of cohesive New Zealand soils using a modified pinhole apparatus on both non-dispersive and dispersive undisturbed soils verified the above erosion properties.

Note on Dispersibility : *If the soil is non-dispersive, the clay particles aggregate ("floc") in the "normal" ground water suspension to a grading of predominantly silt-size particles. A dispersive soil will not flocculate and the grading is that of the "true" clay size particles. Figure 8.3d shows that dispersive clay particles ($<2\mu m$) can be transported at velocities in the order of 10^{-6}m/sec whereas the non-dispersive clay/silt floc require some 100 times this velocity to be transported. All dispersive clays have high sodium ions ($Na+$) in the pore water, relative to other cations (Mg^{2+}, K^+, Ca^{2+}). In general, the clay type will be smectite (montmorillonite) and to a lesser extent illite. The dispersive condition of weak particle bonding is due to ionic repulsion of the clay particles. Real soils are a mixture of clay types and clay/silt/sand size particles, and hence have a range of dispersibilities.*

The soil grading, determined by a conventional mechanical hydrometer soil analysis using dispersing agents in the suspension water (NZS4402 Test 9 — Sodium hexametaphosphate), can be grossly misleading as the test will always result in the "true" dispersed clay size and not the aggregate floc grading of a non-dispersive or partial dispersive soil. More meaningful is a similar test carried out without the dispersing agent (ASTM 4221 — distilled water) or, as a further refinement, using the natural ground water. True dispersive clays will have similar gradings to the conventional test.

A number of methods are available to determine if a soil is dispersive, ranging from the simple "Crumb Test", and "Pinhole Test" (usually considered the most reliable indicator), to the "Concentration of Na^+ ions". However the most suitable test for filter design is the Double Hydrometer (SCS) Test (ASTMD 4221) where the conventional hydrometer test is performed with and without the dispersing agent. The "percentage dispersion" can be obtained by the ratio of 5µm size of the two tests and a value in excess of 50% is considered dispersive (Sherard et al. 1976a,b). The grading obtained without the dispersing agent should be evaluated for filter design. An example of this test on a non-dispersive clay is shown in Figure 8.8.

A change in water chemistry from that within the natural or compacted soil can also alter the dispersive nature of a clay. This is not considered significant in the roading situation where the soils are in place or compacted from a local borrow area and are unlikely to experience marked change in water chemistry after construction.

It should be noted that the addition of quicklime or hydrated lime (3%) to a compacted soil will alter a dispersive clay (high Na^+) to a non-dispersive clay (Ca^{2+}).

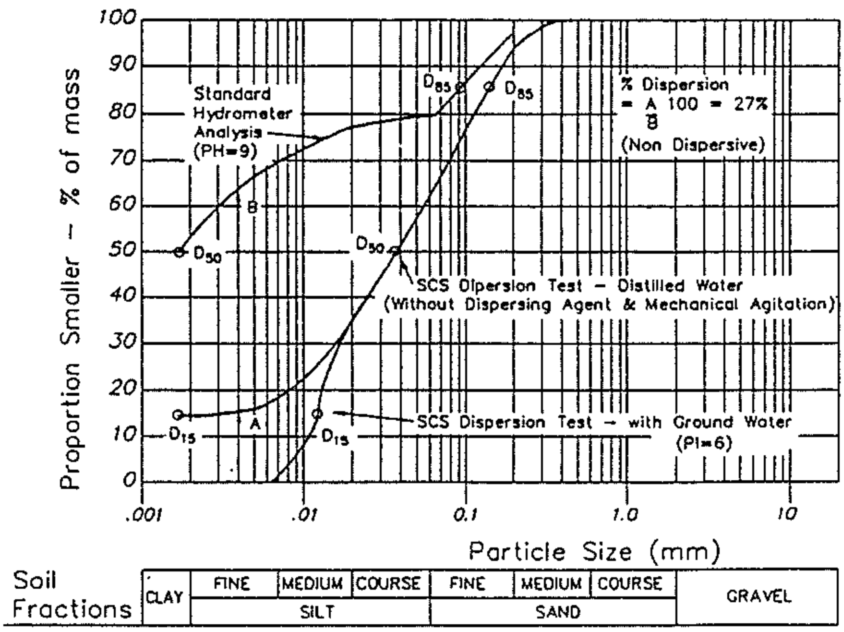


FIGURE 8.8
Double Hydrometer Test on a Non-Dispersive Clay
(ASTMD 4221)

8.8.1 Filter Requirements And Filtration Criteria For Cohesive Soils

As has been demonstrated, only the cohesive soils with a fissured or open structure may be subject to piping under the hydraulic conditions of a conventional trench drain. Erodible dispersive soils are of more concern than the slowly eroding non-dispersive soils. Cohesive soils with preferential flows in fissure structures resulting from a number of geological conditions are likely to be more prevalent in practice than anticipated, although in most cases the fissures are widely spaced within the intact soil. The extent to which New Zealand's roads are founded on dispersive soils is not known at this time but the absence of problems would seem to indicate that this soil condition is not prevalent. The Port Hills, near Christchurch, is one known area where such a soil (from loess) exists.

The use of geotextile filters (or granular filters) in cohesive soils is not well understood. Low dispersive clay particles tend to move through water as a "floc" or "aggregate" and not as the "true" particle grading as determined by hydrometer analyses using a dispersing agent.

The geotextile filter design for cohesive soils should be based on the hydrometer grading without a dispersing agent (Figure 8.8). Real dispersive cohesive soils will include the clay mineral size particles with some non-dispersive silt-sized flocs in addition to any true silt present. Non-dispersive soils should contain few clay size particles in suspension.

Under the initial hydraulic gradient created by the construction of the drain, it is considered that the soil lining a fissure in a cohesive soil will break down to release the eroded clay fraction and the transportation water will carry all size fractions up to approximately 0.5mm diameter (medium/coarse sand) to the filter. Under a realistic hydraulic gradient, larger size particles cannot be transported if the seepage velocity is controlled by the permeability of the filter ($k_f < 10^{-3}$m/sec). The rate of erosion will depend on dispersivity of the clay.

Cohesive soils do not have the same filtration criteria as cohesionless soils with respect to the AOS pore opening size (or D_{15} in granular filters). Geotextiles having a considerably higher, coarser, AOS (O_{90}) can be used for piping protection ($O_{90}/D_{85} > 1$).

Note: *Possible Reasons for $O_{90}/D_{85} > 1$ in cohesive soils:*

(i) *The Stokes Law hydrometer test (for spherical particles) does not represent the grading of the greatest dimension of the clay particles and flocs (Figure 8.3d).*

(ii) *Elongated non-spherical particles can be trapped within the tortuous flow path (clogging) creating a finer pore size filter.*

(iii) *Adhesion bonding by the clay particles to one another and/or the filter at the filter interface.*

(iv) *The seepage velocity is reduced with a combination of conventional blocking, internal clogging and adhesion to that of the intact soil such that transportation is alleviated.*

8.8.2 Piping Criteria for Cohesive Base Soils

A review by Hoare (1982) has shown that the filter criteria for cohesive soils have generally fixed a limiting pore size for any soil and have been based on the soil/filter tests:

$O_{50} < 80\mu m$ (from granular filter design), and
$O_{50} \approx 150\mu m$ (various researches for melt-bonded), presumably using non-dispersive soils.

Waters (1984) undertook some limited testing on a non-dispersive soil with a PI of 56 and noted that the tested D_{85} size of the soil increased from $10\mu m$ with dispersing agent to $100\mu m$ without the dispersing agent. He further commented that for minimal piping on non-dispersive cohesive soils, using a non-woven geotextile under steady state conditions, the following criterion can be used:

$$AOS < 85\mu m$$

This criterion does not appear particularly satisfactory for real cohesive soils having a wide range of grading, clay minerals and dispersivity.

East and Hudson (1987) reviewed the results of slurry testing by Sherard *et al.* (1976a, b) and Vaughan and Soares (1982) for granular filters which tentatively can be used for non-woven geotextiles where the flow paths are similarly tortuous. The equivalent filtration criteria for geotextiles can be based on EOS $O_{50} = 0.2\ D_{15}$ granular filter and $D_{15} = (k_f/3.5 \times 10^3)^{\frac{1}{2}}$ (Sherard *et al.* 1984). In the granular filter testing by Sherard *et al* a wide range of D_{15F}/D_{85B} resulted for successful filters but all ratios were above a value of 9. Vaughan using the filter permeability (k_f) as a criterion obtained a similar relationship. All soil sizes were obtained without a dispersing agent. The equivalent limit for successful geotextile filters can be given as EOS $O_{50}(\mu m) \leq 8 \times (D_{85}\mu m)^{0.75}$.

This piping retention criterion for successful filters can be represented in the more traditional form by the relationship:

$$\frac{\text{EOS } (O_{90})}{85\% \text{ size of protected cohesive soil } (D_{85})} < C$$

where C is a constant depending on the D_{85} soil size: e.g. for D_{85} values of $50\mu m$, $100\mu m$ and $200\mu m$ the values for C are 3.0, 2.5 and 2.0 respectively.

This criterion is less demanding than it is for cohensionless soils where C = 1.

To assess the validity of this relationship East and High (1989) carried out a series of falling head slurry tests on a limited range of New Zealand soils using four commonly used lightweight needle-punched and melt-bonded non-woven geotextiles. The geotextiles pore-size distribution was checked using TNZ(NRB) F/6 (1985, 1986) and the D_{85} size determined by a hydrometer sedimentation test without a dispersing agent (ASTMD 4221). The results (Table 8.1) indicate that the above piping retention criterion is reasonable although a little optimistic. Based on the testing, a conservative simplification for the range of non-woven geotextiles typically used has been determined (East and High 1989). This also uses the O_{90} (AOS) size as for the recommended cohensionless soil criterion:

$$\frac{\text{AOS } (O_{90})}{\text{85\% size of protected cohesive soil } (D_{85})} < 2.3$$

This relationship is shown in Figure 8.9, with the test results. Where the soils and geotextiles properties are in the "effective" zone, all filters retained at least 90% of the soil to be protected. Subsequent permeability tests with water indicated little further loss of soil and hence the initial charge of soil was sufficient to maintain the filter zone. The "non-effective" zone was tested using the dispersive loess soil (D_{85} = 60μm).

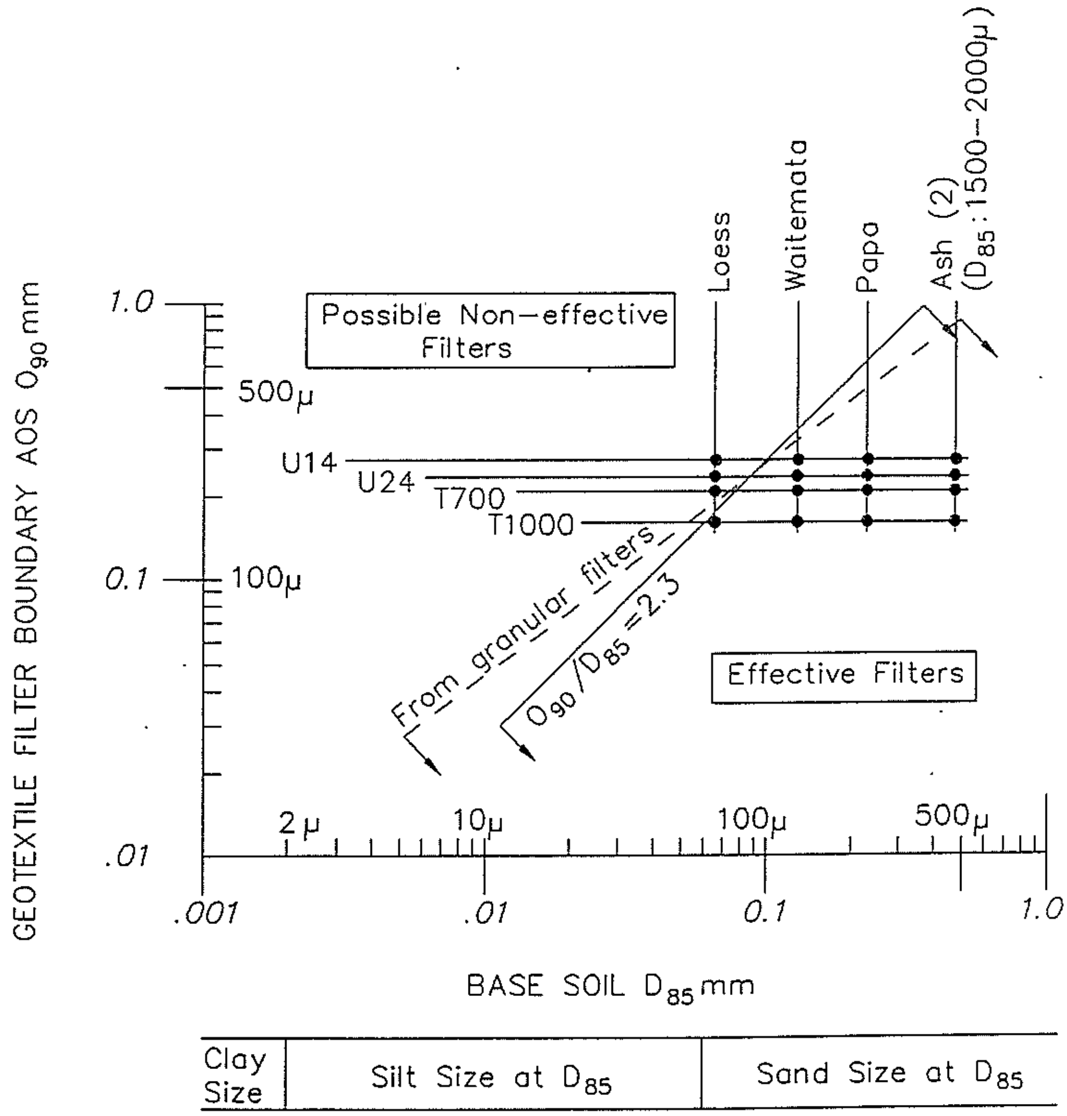
To allow for some factor of safety the recommended piping retention criterion for cohesive soils is:

$$\frac{\text{AOS } (O_{90})}{\text{85\% size of protected cohesive soil } (D_{85})} < 2.0$$

Some variation in pore size between the manufacturers' data and tests on isolated samples has been apparent. If the manufacturers' data is utilised without quality assurance testing, the piping retention criterion should be modified to:

$$\frac{\text{AOS } (O_{90})}{\text{85\% size of protected cohesive soil } (D_{85})} < 1.75$$

It is suggested that these criteria be used for all cohesive soils with the proviso that the protected soil D_{85} be determined in a hydrometer test without the dispersing agent (ASTMD 4421). This does not negate the single pore size criteria for non-dispersive clay because, for the general range of real cohesive soils (D_{85} = 75-150μm), the above criterion is compatible.



Note: D_{85} is determined without the dispersing agent (ASTMD 4221)

FIGURE 8.9
Geotextile Filter Piping Criteria for Cohesive Soils and Non-Woven Geotextiles
(East and High 1989)

TABLE 8.1 % Retained on Geotextile by Slurry Test (25g/litre Concentration)

Soil (Grading)[1]	Geotextile	Pore Size $O_{50}, O_{90}\mu m$	% Retained	% Passed	Passed Particle Size D_{50}	D_{85}	$D_{90}\mu m$
Wanganui Papa	U14	210,260	95	5	30	55	60
D_{85} = 220μm	U24	190,230	94	6	30	60	65
D_{50} = 90μm	T700	180,220	95	5	30	65	75
<75μ = 46%	T1000	150,170	92	8	25	50	66
<2μ = <5%							
Auckland Waitemata Gp	U14	210,260	97	3	25	55	60
D_{85} = 120μm	U24	190,230	83	17	30	70	80
D_{50} = 40μm	T700	180,220	93	7	35	75	85
<75μ = 68%	T1000	150,170	95	5	30	70	80
<2μ = 10%							
Hamilton Ash	U14	210,260	98	2	40	85	100
D_{85} = 1500μm	U24	190,230	99	1	35	55	60
D_{50} = 200μm	T700	180,220	99	1	45	90	—
<75μ = 42%	T1000	150,170	100	0	—	—	—
<2μ = 0%							
Taranaki Ash	U14	210,260	91	9	35	65	75
D_{85} = 2000μm	U24	190,230	90	10	35	65	70
D_{50} = 500μm	T700	180,220	92	8	35	65	75
<75μ = 21%	T1000	150,170	92	8	35	65	70
<2μ = <1%							
Christchurch Loess[2]	U14	210,260	2	98	45	70	80
D_{85} = 70μm	U24	190,230	10	90	30	85	65
D_{50} = 25μm	T700	180,220	48	52	35	60	70
<75μ = 87%	T1000	150,170	33	67	35	70	85
<2μ = 18%							

1 Note: Grading without dispersing agent (ASTMD 4421)
2 Note: A dispersive soil

TABLE 8.2 Permeability of Retained Soils by Slurry Test (25g/litre Concentration)

Soil (Grading)[1]	Geotextile	Initial Geotextile Q100=1/m²/sec	Permeability k_fm/sec	Retained Thickness mm	Retained[2] Permeability k-m/sec	Retained[3] Void Ratio e
Wanganui Papa	U14	260	7×10^{-3}	12	5×10^{-7}	1.92
D_{10} = 6μ	U24	410	1×10^{-2}	13	3×10^{-7}	2.30
<75μ = 40%	T700	330	2×10^{-3}	10	2×10^{-7}	1.45
<2μ = 5%	T1000	270	3×10^{-3}	11	1×10^{-7}	1.75
Auckland Waitemata Gp	U14	260	7×10^{-3}		3×10^{-7}	2.44
D_{10} = 2μ	U24	410	1×10^{-2}	14	4×10^{-7}	2.68
<75μ = 60%	T700	330	2×10^{-3}	14	1×10^{-7}	2.39
<2μ = 10%	T1000	270	3×10^{-3}	22	6×10^{-7}	4.2
Hamilton Ash	U14	260	7×10^{-3}	26	1×10^{-5}	5.04
D_{10} = 15μ	U24	410	1×10^{-2}	25	2×10^{-5}	4.79
<75μ = 42%	T700	330	2×10^{-3}	24	1×10^{-5}	4.55
<2μ = 0%	T1000	270	3×10^{-3}	28	2×10^{-5}	5.36
Taranaki Ash	U14	260	7×10^{-3}	15	4×10^{-6}	2.93
D_{10} = 25μ	U24	410	1×10^{-2}	15	3×10^{-6}	3.04
<75μ = 17%	T700	330	2×10^{-3}	16	5×10^{-6}	3.16
<2μ = <1%	T1000	270	3×10^{-3}	16	6×10^{-6}	3.16

1 Note: Grading without dispersing agent (ASTMD 4421)
2 Note: Permeability of retained soil and the geotextiles are determined from the steady state condition of a falling head test with distilled water after the initial retention test with the slurry.
3 Note: Void Ratio obtained from e = $\underline{P}_D/\underline{P}_{S-1}$ where $\underline{P}_D$= retained dry density, $\underline{P}_S$ = tested solid density.

8.8.3 Clogging And Filter Permeability With Cohesive Soils

Few details are known about the final permeability of the filter interface against cohesive soils that can be eroded and transported. The eroded soil must originate from water-bearing fissures which may have a significantly higher flow capacity than the geotextile and intact soil. To retain the eroded soil it is necessary to reduce the fissure flow. The slurry tests carried out by Sherard *et al.* (1984) and Vaughan and Soares (1982) did not report the final permeability. Other authors (e.g. Wade *et al.* 1983) indicated a 20% lower permeability from that of the intact soil using a realistic hydraulic gradient through a dispersive soil in a slurry condition.

Further comment can be made on the conditions of the retained soil. The cohesive soil retained on the geotextile will be in a flocculated, loose aggregate, non-dispersed state and the "filter cake" grading will be, in general, coarser than the natural soil (unless that is dispersive). The approximate permeability relative to the effective size (D_{10}) (Figure 8.4) is considered sufficiently accurate for filter design. It is suggested that the effective size (D_{10}) of the hydrometer testing without the dispersing agent is more appropriate to assess the permeability of the retained soil, whereas the natural soil permeability can be assessed from D_{10}, obtained from the conventional test using the dispersing agent. Using these criteria, a typical non-dispersive soil with D_{10} values of 2μm under dispersed testing and of 50μm non-dispersed (double hydrometer test ASTMD 4221) would have an intact permeability in the order of 10^{-7}m/sec and a retained soil permeability in the order of 10^{-5}m/sec, i.e. the retained soil has a permeability 100 times that of the intact soil. A dispersive soil would have a similar permeability for both cases. Similar conditions can be assessed by comparing the void ratio (e) or porosity (n) of the retained soil and the natural soil ($\mathrm{Log}k \propto e$). The void ratio of a non-dispersive retained soil can be significantly greater than the intact soil. This assumes that the geotextile filter is not clogged to an extent that the permeability (or hydraulic resistance) is lower than the retained soil. This is a realistic assumption if the high POA non-woven geotextiles are employed.

The permeability results obtained from slurry tests previously mentioned are shown on Table 8.2. There is reasonable correlation between the permeability assessed from the D_{10} size of the protected soil (tested without dispersing agent) and the test results. In addition, the retained void ratio is greater than that of the natural soil. The intact soil permeability is significantly higher than the tested retained soil values (East and High 1989). The porosity of the effective filter geotextiles reduced by less than 15% from the original value indicating that little internal clogging took place (East and High 1989).

It should be emphasised that the eroded soil must originate from water-bearing fissures in the cohesive soil and these fissures could have a significantly higher permeability (Figure 8.3b) than the retained soil. The fissure will have a significant reduction in flow by using a geotextile that has also retained the eroded soil, although this reduced flow should be greater than that in the intact soil adjacent to the fissure.

Notwithstanding this, in general roading conditions even a complete seal over the local fissures is not likely to be detrimental to the overall performance of a well designed subsoil drain because the flow will progressively transfer to the adjacent intact soil.

8.8.4 Design Method For Cohesive Soils

(a) Assess whether the soil is likely to be significantly fissured or contain water bearing discontinuities and/or is dispersive. If intact or well compacted and non-dispersive, a design is not required.

(b) Carry out a consistency test (PI) on the soil (NZS 4402).

(c) Carry out a double hydrometer test on the soil (ASTMD 4421).

(d) Confirm that the soil is cohesive ($PI>7$ and $2\mu m>10\%$) using the standard hydrometer test with the dispersing agent.

(e) Check the D_{85} size of the non-dispersed grading and obtain the minimum pore size O_{90} for piping retention from the design criteria.

$O_{90} < 2.0\ D_{85}$ for values of O_{90} obtained from tested representative geotextile samples

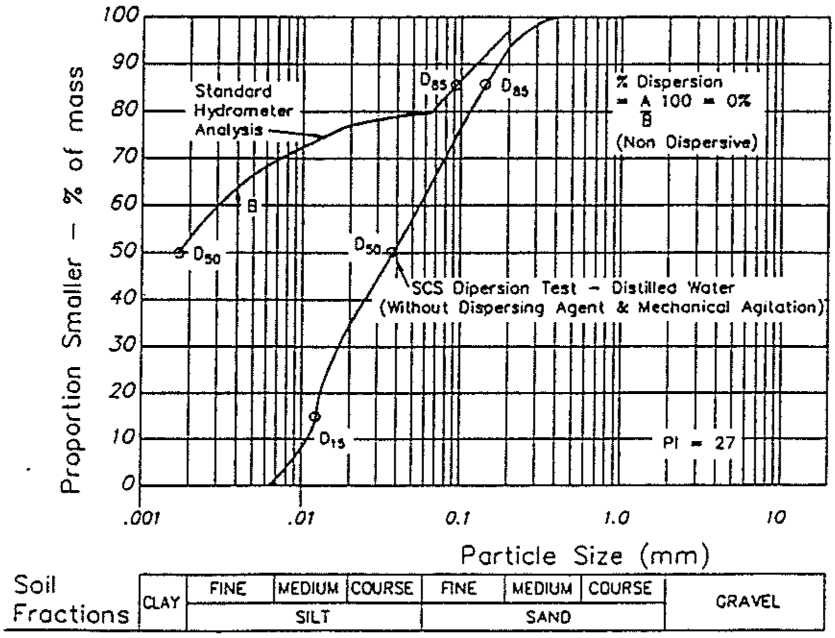
$O_{90} < 1.75\ D_{85}$ for values of O_{90} from manufacturers' data

(f) Check the D_{10} size of non-dispersed grading and obtain the approximate permeability of the retained soil from the D_{10}/Permeability graph (Figure 8.4).

(g) Compare the permeability of the intact soil from the D_{10} size of the dispersed grading (Figure 8.4).

(h) Assess if the retained soil permeability is not detrimental to the overall performance of the subsoil drain in the prevailing hydraulic situation. Unless the drain is designed to de-water very high flow, fissured, aquifers (Figure 8.3b), the hydraulic condition should be adequate and superior to conditions where the fissures are completely blocked.

Example: Non-dispersive Cohesive Soil



(a) Fissures are present but the soil is non-dispersive
% Dispersion = $\frac{A}{B} \times 100 = 0\%$
Design required but erosion will be minimal

(b) PI = 27

(c) Inspect the graph of the double hydrometer gradings

(d) PI > 7, 2μm > 10%, therefore Cohesive Soil

(e) D_{85} non-dispersed = 130μm
O_{90} < 260μm If representative geotextile samples have been tested for O_{90}.

O_{90} < 225μm If the manufacturers' O_{90} size is used

(f) D_{10} non-dispersed = 10μm
Retained soil k ≈5 x 10^{-5}m/sec (Figure 8.4)

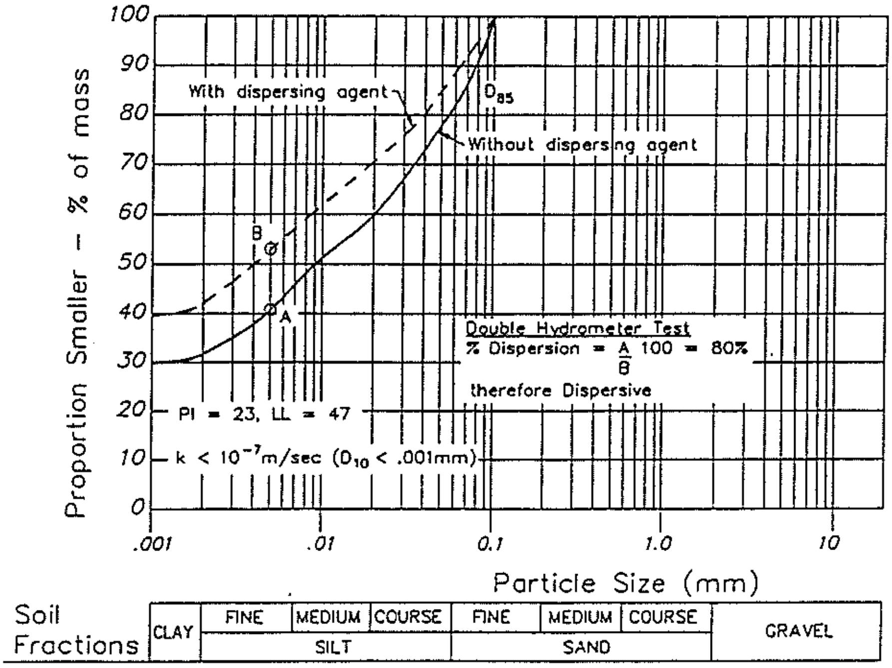
(g) D_{10} dispersed = <2μm
Intact soil k < 10^{-7}m/sec (Figure 8.4)

(h) k retained > k intact, therefore the drain should perform adequately.

Decision: A geotextile with pore size O_{90} < 225μm will be suitable if the water-bearing fissures are not critical to the subsoil drain.

i.e. A typical commercial light-weight needle-punched or melded non-woven geotextile should be suitable.

Example: Dispersive Cohesive Soil



(a) Soil is dispersive
% Dispersion = $\frac{A}{B}$ x 100 = 80%

Design required and erosion should be expected

(b) PI = 23

(c) Inspect the graph of the double hydrometer gradings

(d) PI > 7, 2μ > 10%, therefore Cohesive Soil

(e) D_{85} non-dispersed = 60μm
O_{90} < 120μm If representative geotextile samples have been tested for O_{90}

O_{90} < 105μm If the manufacturers' O_{90} size is used

(f) D_{10} non-dispersed < 1μm
Retained soil k < 10^{-7}m/sec (Figure 8.4)

(g) D_{10} dispersed < 1μm
Intact soil k < 10^{-7}m/sec (Figure 8.4)

(h) k retained ≈ k intact. Any fissures present will be effectively blocked and the drain will act as if it is draining intact soil.

Decision: A geotextile with a pore size $O_{90} < 105\mu_m$ will be suitable providing water-bearing fissures are not critical to the subsoil drain performance (the more common condition).

This is a relatively low pore size for the common range of geotextiles. A low pore size melt-bonded product may be necessary. As this soil will definitely erode, the pore size should be chosen to be well below 105μm and/or retention testing may be considered necessary.

8.9 MECHANICAL REQUIREMENTS

While the primary function of the geotextile is to filter the soil (hence the dominant requirement is hydraulic) the mechanical requirements of the geotextile are also quite important to ensure the mechanical integrity of the geotextile is maintained during its installation. It does not matter how much effort has gone into establishing appropriate hydraulic requirements for a geotextile filter if it is going to be punctured and torn during installation.

Where geotextiles are used as filters, integrity is the sole mechanical constraint because the only mechanical stresses are exerted during installation of the geotextile. Lawson (1986a) has developed empirical relationships between observed, in situ, mechanical performance and geotextile puncture and tear resistances for both subsurface drainage and erosion control structures. The two geotextile integrity tests which have particular relevance to subsurface drainage filters are Mass Per Unit Area (for dynamic puncture resistance) and Trapezoidal Tear (for tear propagation resistance). It has been observed (in the field) that the two likely modes of mechanical damage (tear and puncture) are best described in terms of these two tests. Standard test methods have been established to measure the relevant properties — ASTM D1117 for Trapezoidal Tear resistance and ASTM D1910 for Mass Per Unit Area (Chapter 4).

The relationships developed by Lawson (1986a) for the mechanical integrity requirements for subsurface drainage filters are as follows:

(a) Dynamic Puncture Criterion: $\mu \geq 1000\ H^{0.5}\ D_{85}$

(b) Tear Propagation Criterion:

- For Firm Smooth Soil Surface, FT $\geq 750\ D_{85}^{0.45}$
- For Rough Soil Surface, $F_T \geq 1500\ D_{85}^{0.75}$ for $D_{85} > 0.1$m dia.

where μ is the Mass Per Unit Area of the geotextile in g/m^2, H is the height of aggregate placement in metres, D_{85} is the characteristic diameter in mm of the drainage aggregate in the drainage trench, and F_T is the Trapezoidal Tear resistance of the geotextile in Newtons. The relationships are shown graphically in Figure 8.10.

The relationships indicate that the mechanical requirements are not particularly arduous for the conventional one-metre deep geotextile-subsoil drain backfilled with 50mm aggregate. All commonly available filter geotextiles in excess of 100g/m^2 unit weight should be suitable. Mechanical requirements are significant when constructing deep interceptor drains (french drains) where the drop height may be in excess of 3 metres and the aggregate size is some 100mm in diameter. In this application the unit weight of the geotextile may need to be the order of 200g/m^2 and the Tear Resistance value in excess of 300N. Care is required if low tear resistance short staple-fibre geotextiles are to be used (Chapter 5).

8.10 DURABILITY REQUIREMENTS

Comprehensive durability criteria should cover two time-related aspects of geotextile integrity: ensurance that the geotextile does not degrade critically <u>after</u> it is placed in the soil and ensurance that the geotextile does not degrade critically <u>before</u> it is placed in the soil. The chemical composition of the geotextile determines the extent to which both occur.

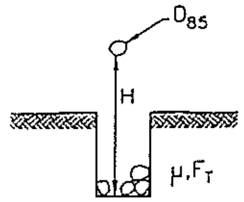
The potential for degradation once the geotextile is placed in the soil is governed by the resistance to degradation of the components in the geotextile. Polypropylene, polyethylene and polyester are the most commonly used geotextile polymers primarily because of their excellent durability characteristics in soil environments (Chapter 3).

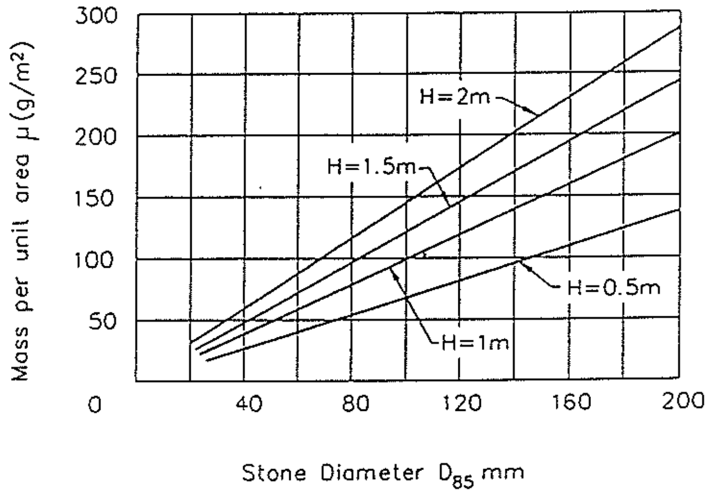
The prime consideration concerning stability of the geotextile before it is placed in the soil is the effect of prolonged exposure to ultraviolet (UV) light. All synthetic fibres are susceptible to some extent to degradation by UV light (Chapter 3) although once the geotextile is covered adequately by soil or rock no further UV degradation occurs. It is considered essential practice to have a UV stabiliser incorporated in the more highly susceptible fibres (polypropylene, polyethylene and polyamide) during manufacture.

It should be noted that while some thermoplastic synthetic fibres exhibit good natural UV resistance and UV stabilisers may be incorporated in other fibre types, geotextiles should not be left exposed for prolonged periods as this UV stability (either natural or synthetic) only delays the degradation, it does not prevent it.

8.11 BACKFILL AGGREGATE AND OTHER PRACTICAL CONSIDERATIONS

The geotextile should be placed in the drain trench so that it conforms loosely to the shape of the trench. The transverse joints should be overlapped by at least 150mm (Lawson 1986a) or to the TNZ(NRB) F/6 (1985, 1986) specification. No longitudinal joints or overlaps should be permitted apart from the top closure of the wrapped aggregate.





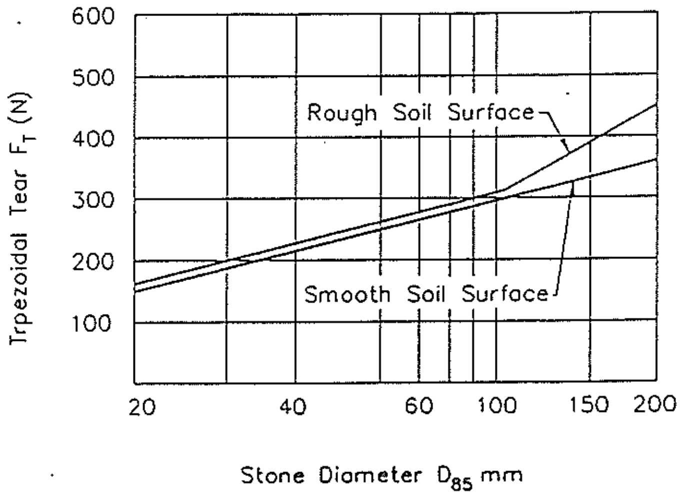


FIGURE 8.10

Mechanical Integrity Requirements in Terms of Mass Per Unit Area and Trapezoidal Tear For Geotextile Filters Used in Subsurface Drainage (Lawson 1986a)

The backfill drainage aggregate must be durable and clean, without clay/silt fines. The flow capacity of the drain (or permeability) is dependent on the stone size and the uniformity of the aggregate grade. The permeability (k) can be assessed from Figure 8.11 (Dept of the Navy (USA) 1982) and the approximate capacity (Q) from Darcy's Law assuming laminar flow.

$Q\ (m^3/s) = (k \times A \times i)$ where:

Q = quantity of water discharged per unit time
k = permeability coefficient (m/sec)
A = area of the drain (m^2)
i = gradient of drain

A typical subsoil drain complying with TNZ(NRB) specification F/6 would have an aggregate grading of 10 to 50mm, an area of $0.6m^2$ (1m x 0.6m), and perhaps installed at a gradient of 1:100. From Figure 8.11 the permeability of a clean aggregate will be the order of 10^{-1} m/sec and the resultant capacity is $6 \times 10^{-4} m^3$/sec or 0.6 litres/sec. Figure 8.11 also shows that a near-single size aggregate would increase the flow capacity and the introduction of fines would dramatically lower the flow capacity.

A subsoil or interceptor drain outside the pavement system must be capped by at least 300mm of low permeability soil or equivalent, to reduce the ingress of surface run-off water. Under the pavement system a subsoil drain abuts the sub-base aggregate.

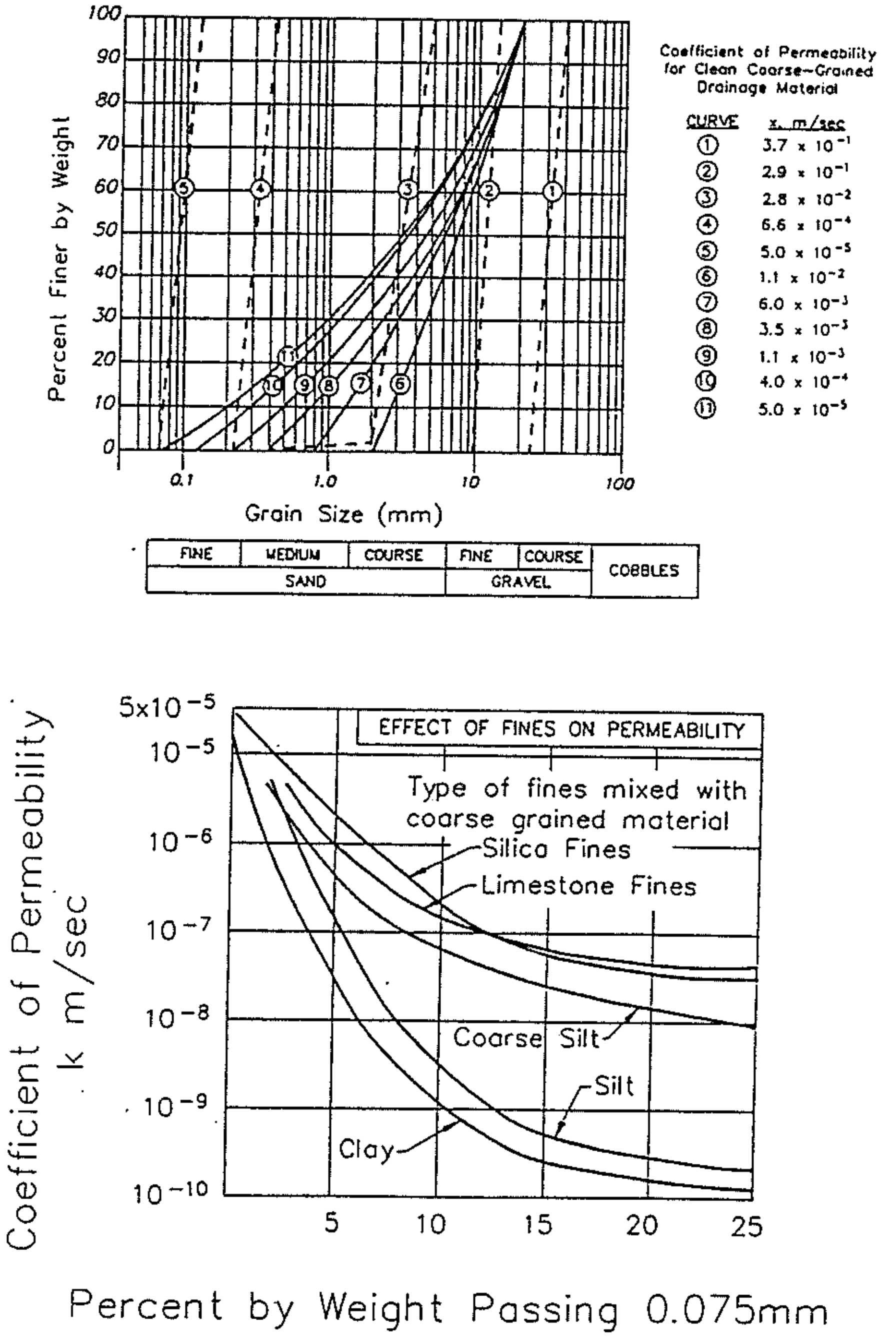


FIGURE 8.11
Permeability of Drainage Materials (Dept of the Navy (USA) 1982)

8.12 GEOCOMPOSITES FOR SUBSURFACE DRAINAGE

Synthetic drainage products, commonly called strip drains or fin drains, are now available as a replacement for the conventional subsoil drain and pavement drain. These high drainage products have an overall application in situations where high hydraulic gradients result in relatively large water discharges.

These drainage geocomposites generally comprise a core protected by a geotextile filter. The requirements of the geotextile are exactly those of conventional geotextile- protected drains including the piping retention and permeability criteria.

The cores of the geocomposites vary widely between products. Their core widths range from 10mm to 50mm, and are made up of ribs, waffles, grids, webs and corrugations. They are generally constructed to be suitable for at least a 900mm deep drain. A list of the products available in North America (Koerner 1986) and Australasia is given in Table 8.3. Currently in New Zealand, only the Stripdrain and Geodrain products are available. Both are protected by the non-woven heat-bonded geotextile Terram 1000.

Product	**Manufacturer**	**Core Characteristics**		
		Thickness (mm)	**Shape**	**Material**
Amerdrain II	American Wick	80	ribs corrugated	polyethylene
Cordrain	Burcan	25 and 50	waffle	polystyrene
Enkadrain	Enka	10 and 20	wire web	nylon
Filtram	ICI Fibres	5	grid	polypropylene
Geodrain	Ground Eng	5	grid	polypropylene
Geotech	Geotech Systems	50	sphere	expanded polystyrene
Hydraway	Monsanto	28	column	polyethylene
Miradrain 4000	Mirafi	25	waffle	polystyrene
Permadrain	NW Fabrics	20	waffle	polyethylene
Stripdrain (Hitec)	Nylex	40	waffle	polystyrene

TABLE 8.3
Range of Geocomposite Drainage Products

All these geocomposites respond differently under normal soil pressures, and short-term strains vary from product to product. The flow characteristics of a number of products against normal pressure are shown in Figure 8.12 along with the flow demands of various drainage applications. These flow characteristic values are those under full conduit conditions, and greater flows per unit width will result under normal use. Of the products currently available in New Zealand only the Nylex Stripdrain can replace the conventional subsoil or pavement drain. In the roading application the Geodrain product may be suitable for interface drainage between the subgrade and sub-base as well as its indirect applications for retaining wall drainage. Some different core structures are shown in Figure 8.13.

As little is known about the long-term creep properties of these products, appropriate testing should be carried out if the geocomposite is to endure high permanent loading. This is not considered significant with the subsoil- and pavement-drain applications in which the permanent normal pressures are relatively low.

Similar products, with smaller dimensions, are being manufactured to replace conventional vertical sand drains although the core is generally constructed of stiffer material.

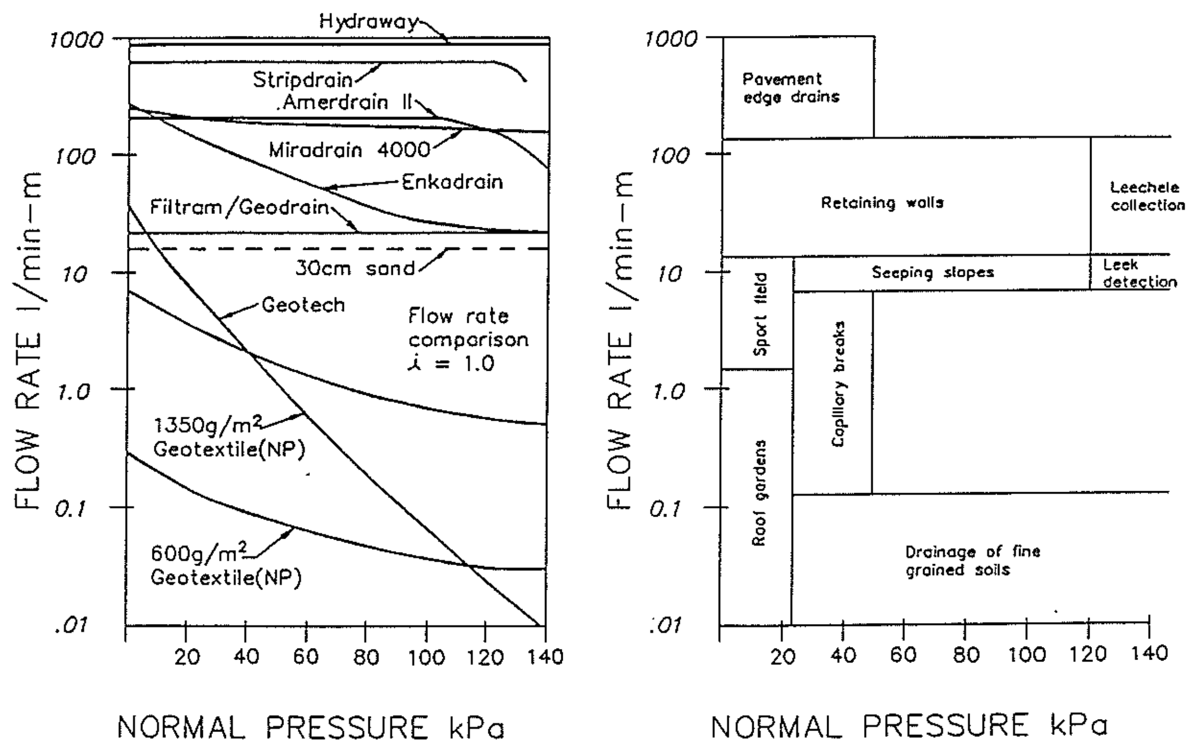


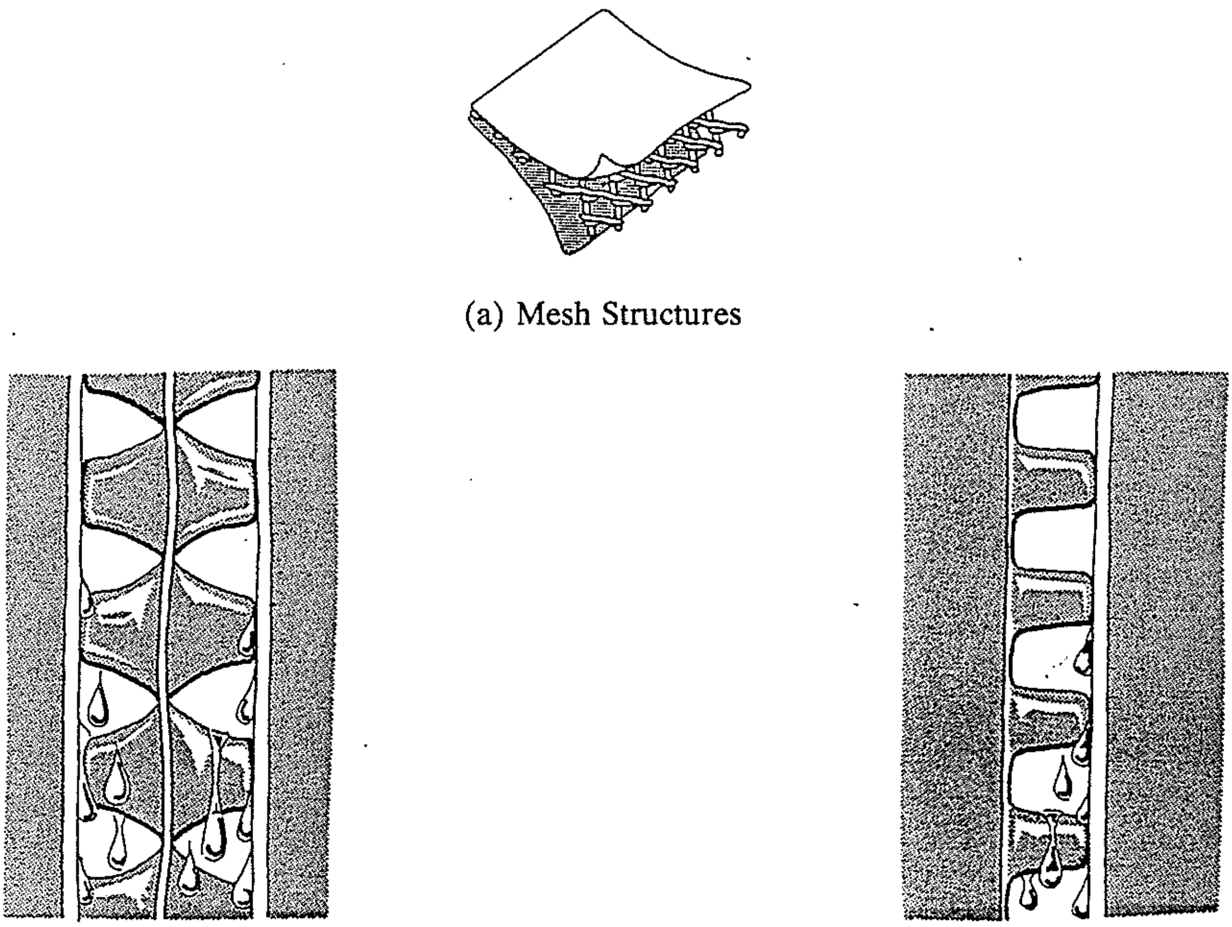
FIGURE 8.12
Flow Characteristics of Drainage Geocomposites and under Different Applications (Koerner 1986, Keith 1985)

8.12.1 Some Practical Considerations

The economic advantage of drainage geocomposites is that a relatively narrow trench and hence minimal backfill is required. The product should directly abut the trench face side with the prevailing water source while the opposite side of the trench is backfilled. In New Zealand a well graded sand (as for concrete) is generally recommended for backfill in this application. This can be compacted by progressive washing (jetting) and tamping where the trench is very narrow. If high flows or steep longitudinal gradients are anticipated, short lengths of recompacted excavated material should be placed at regular intervals to alleviate the possibility of the sand being transported. In the subsoil drain application, the trench and geocomposite must be capped by at least 300mm of low permeability soil. In the pavement drain application the geocomposite abuts the sub-base and the excavated soil should be recompacted against the product.

8.12.2 Geotextile-Wrapped Pipes

In some applications geotextile-wrapped perforated pipes (Figure 8.1c) may be used but careful consideration should be given to the drainage requirements. If used in subsoil drains the backfill aggregate properties should comply with granular filter criteria which eliminates the economics of using geotextile filters. In addition, such products can not adjoin the total face of the trench with high flow capacity properties. It is considered that geotextile-wrapped perforated pipes are suitable for pavement drains where the drain is surrounded by the sub-base aggregate, or in situations where a designed granular backfill is finer than the perforations of the pipe (Diam Perf. > D_{85} Backfill). Other uses also include the filter protection of bored horizontal perforated PVC pipe drains used for stabilising slopes.



(a) Mesh Structures

(b) Waffle Structure

(c) Alveolite Structure

FIGURE 8.13
Some Different Core Structures Used in Prefabricated Drainage Composites
(Lawson 1986a)

8.13 REFERENCES

Carroll, R.G. 1983. Geotextile filter criteria. *Transportation Research Record* 916: 46-53. Transportation Research Board, Washington DC.

Department of the Navy (USA) 1982. Soil Mechanics Design Manual 7.1. *Navfac DM-7.1.*

East, G.R.W., High, R. 1989. Geotextile subsoil drain investigation stage 2. *Transit New Zealand /RRU Project PD/14 Report* GS 89/20.

East, G.R.W., Hudson, K. 1987. Geotextile subsoil drains : developments towards their use in fine grained soils. *New Zealand Roading Symposium* 4: 135-148.

Haliburton, T.A., Wood, P.D. 1982. Evaluation of the US Army Corp of Engineering gradient ratio test for geotextile performance. *Proceedings of Second International Conference on Geotextiles,* Las Vegas, USA.

Hoare, D.H. 1982. Synthetic fabrics as soil filters : a review. *ASCE Journal of Geotechnical Engineering* 108 (GT 10): 1230-1245.

Hoare, D.H. 1984. Geotextiles as filters. *Ground Engineering* 17(2): 29-44.

Keith, D. 1985. *Stripdrain flow characteristics and design methods.* Hitex Ltd, March 1985.

Koerner, R.M. 1986. *Designing with geosynthetics.* Prentice-Hall, Englewood Cliffs, NJ.

Koerner, R.M., Ko, F.K. 1982. Laboratory studies on long term drainage capability of geotextiles. *Proceedings of Second International Conference on Geotextiles,* Las Vegas, USA.

Lawson, C.R. 1982. Filter criteria for geotextiles. Relevance and use. *ASCE Journal of Geotechnical Engineering* 108 (GT 10): 1300-1317.

Lawson, C.R. 1986a. Geotextiles or geosynthetics? *Proceedings of Technical Sessions, Third Asian ICI Fibres Geotextiles Conference,* Thailand.

Lawson, C.R. 1986b. Geotextile filter criteria for tropical residual soils. *Proceedings of Third International Conference on Geotextiles,* Vienna, Austria.

Millar, P.J., Ho, K.W., Turnbull, H.R. 1980. A study of filter fabrics for geotechnical applications in New Zealand. *Ministry of Works and Development Central Laboratories Report* 2-80/5.

Rohan, K., Lefebvre, G., Douville, S., Milette, J. 1986. A new technique to evaluate erosivity of cohesive material. *ASTM Geotechnical Testing Journal* 9(2): 87-92.

Rycroft, D., Jones, D.P. 1982. Geotextile filtration performance and current filter criteria. *Proceedings of Second International Conference on Geotextiles,* Las Vegas, USA.

Schober, W., Teindl, H. 1979. *Filter criteria for geotextiles. Design Parameters in Geotechnical Engineering. Proceedings of Seventh European Conference SMFE, 2.*

Sherard, J.L., Dunnigan, L.P., Decker, R.S., Steele, E.F. 1976a. Pinhole test for identifying dispersive soils. *ASCE Journal of Geotechnical Engineering* 102 (GT 1): 69-86.

Sherard, J.L., Dunnigan, L.P., Decker, R.S. 1976b. Identification and nature of dispersive soils. *ASCE Journal of Geotechincal Engineering* 102 (GT 4): 287-302.

Sherard, J.L., Dunnigan, L.P., Talbot, J.R. 1984a. Filters for silts and clays. *ASCE Journal of Geotechnical Engineering* 110(6): 701-718.

Sherard, J.L., Dunnigan, L.P., Talbot, J.R. 1984b. Basic properties of sand and gravel filters. *ASCE Journal of Geotechnical Engineering* 110(6): 684-700.

Tan, H.H., Weimer, R.D., Chen, Y.H., Demery, D.M., Simons, D.B. 1982. Hydraulic function and performance of various geotextiles in drainage and related applications. *Proceedings of Second International Conference on Geotextiles,* Las Vegas, USA.

Transit New Zealand (NRB) 1985-1986. Specification for fabric-wrapped aggregate subsoil drain construction. *TNZ(NRB) F/6, 1985 and TNZ(NRB) F/6 Notes, 1986.*

Vaughan, M., Soares, H.F. 1982. Design of filters for clay core of dams. *ASCE Journal of Geotechnical Engineering* 108(GT 1): 17-32.

Wade, G.T., Klaiber, Lohres 1983. *Permeability tests of selected filter fabrics for use with a loess-derived alluvium. Engineering Fabrics in Transportation Construction.* Transportation Research Board, Washington DC.

Waters, T.J. 1984. Evaluation of geotextiles. *Proceedings of Australian Road Research Board Conference* 12(2).

CHAPTER 9

SUB-BASE PROTECTION

9.1 GENERAL COMMENTS

Intermixing of the sub-base aggregate with the subgrade soil can occur in roads founded on soft wet subgrades. This intermixing is variously described as subgrade intrusion, fines migration, or pumping. It is a combination of the upwards movement of fines, and punching down of coarse aggregate particles down into the soft subgrade.

A geotextile at the subgrade/sub-base interface can prevent this punching, and reduce or prevent this intrusion. This is the geotextile **separation** function (described in Chapter 7). Christopher and Holtz (1985) point out that, given adequate pavement design, any geotextile should be capable of providing **separation**, provided it is strong enough to survive construction activities. If the holes in the geotextile, however, are too fine or too few its permeability to water may be insufficient to allow dissipation of subgrade pore pressures leading to subgrade weakening. Subgrade/sub-base separation is covered in more detail in Chapter 10.

For soft wet subgrades, pronounced subgrade intrusion can occur when repeated dynamic loads are applied. With subgrade intrusion, a mixture of water and fine soil particles tends to move up into the sub-base; the **filtration** function (also described in Chapter 7) of the geotextile attempts to restrain these soil particles and yet allow the free passage of water. Under these conditions and with repeated dynamic loadings from passing heavy vehicles, the flow through the geotextile may well alternate its direction, i.e. alternating flow.

Chapter 8 considered in detail geotextile **filtration** for drainage applications where the flow through the geotextile was in **one direction** only. This Chapter (Chapter 9) considers filtration for the **alternating** flow situation for soft wet subgrades under dynamic loads.

9.2 RISK SITUATIONS

A risk of subgrade intrusion into the sub-base occurs only if the subgrade is wet, or has a high moisture content. Thus any use of a geotextile between subgrade and sub-base should be preceded or accompanied by pavement subsurface drainage, if economic.

Very severe geotextile filtration requirements will occur if the subgrade "pumps". This occurs in saturated conditions and the application of repeated dynamic loads on fine cohesionless or low cohesion subgrade soils (e.g. silt or fine sand) in which the permeability is insufficient to dissipate the water pressure. Under these conditions the soil pore pressure is increased by the dynamic loading (e.g. from heavy vehicles) to the point that it is greater than the strength of the soil, and a soil slurry (liquefaction) is formed. This slurry then attempts to "pump" up into the sub-base. The geotextile's function is to retain the soil particles yet allow free passage of the water (filtration).

A similar phenomenon can occur in cohesive soils such as clays. In these, cohesion of the soil particles to the subgrade soil mass is broken by a combination of increased pore pressures and erosion. This requires free water and severe dynamic loads. This can occur in voids under the joints of concrete slab pavements over which frequent heavy vehicles pass. The susceptibility to this action depends on the critical erosion shear stress of the soil and the dynamic seepage velocity, i.e. the drag on the soil particles caused by the velocity of the water is enough to pull soil particles off the soil mass (East and Hudson 1987). Because these conditions are particular to a given situation, they are difficult to quantify; suffice to say that dispersive soils have the greatest risk.

If pumping of the subgrade is not likely, there may still be a requirement for the filtration function of the geotextile. This could arise from a number of causes including expulsion of water from a wet subgrade, removal of water that has accumulated in the subgrade from rain or construction practices, or water welling up through the subgrade from a high water table. The filtration requirement in these cases could be for either one- directional flow or for alternating flows. Which of these, and their severity, will depend on the traffic, subgrade soil type, sub-base grading, and the amount of water.

The actual magnitude and number of repetitions of the dynamic loads necessary to initiate liquefaction and pumping is not well defined. It is also difficult to characterise soils susceptible to liquefaction. Saturated non-cohesive and weakly cohesive, fine grained soils tend to be most susceptible (Christopher and Holtz 1985).

The problem can be very severe under rigid pavements over soils or sub-bases susceptible to pumping. Similarly it can be very severe under railway crossings due to the intensive rhythmic dynamic loads from trains. The problem can also occur during road construction on these soils which tend to be susceptible to disturbance under construction activities.

Alternatively if pumping of the subgrade is not likely, the subgrade and sub-base may still intermix if the subgrade is soft and wet, as in some wet clays. The geotextile in these cases fulfils a separation function.

9.3 FILTRATION OF ALTERNATING FLOW : THEORY

Filtration under conditions producing alternating hydraulic flows is more severe than for one-directional flow. Filter cakes and bridging networks do not build up.

Schober and Teindl (1979), as a result of laboratory tests, proposed for turbulent alternating flow:

"Dependent on the intensity of flow and on the allowable account of soil-loss, the grain diameter that must be retained is set equal to the (geotextile) opening size O_{90}."

That is, positive restraint of a given soil particle size demands an effective opening size smaller than the particle to be retained .

The coefficient of uniformity of the soil $U = d_{60}/d_{10}$, was also employed. The results are plotted in Figure 9.1 for the various soil sizes to be retained on the geotextile.

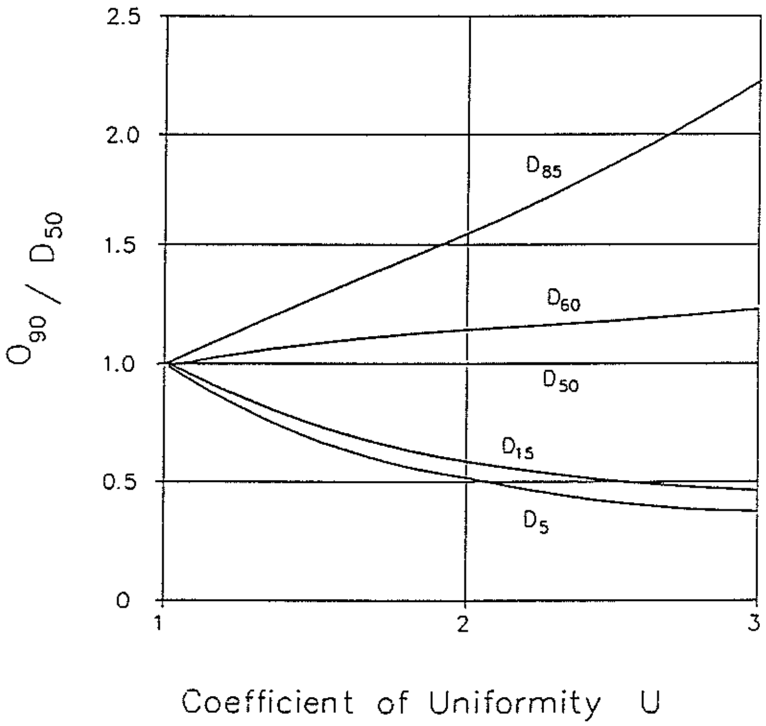


FIGURE 9.1
Filter Criteria for Turbulent Alternating Flow
(Ingold 1985)

Ogink (1975) suggested for alternating flows that : $d_{15} > O_{90}$. This is very similar to Schober and Teindl's (1979) conclusions. Ogink had also brought in consideration of the coefficient of uniformity of the soil, U.

The equations by Ogink and by Schober and Teindl are extreme. For example, suppose that a silt with $d_{50} = 10\mu m$ and $U = 3$ is to be protected under alternating flow, the requirement from the geotextile is $O_{90} = 4.6\mu m$. In practice, the smallest O_{90} of commercially available geotextiles is an order of magnitude higher than this value.

List undertook alternating flow filtration tests. On the basis of these tests List (Hoare 1984) was not prepared to predict performance of a fabric without actually testing the soil system.

Ingold (1985) proposed what he called his revised theory which allows larger geotextile pore sizes than those allowed by Schober and Teindl (1979), at least for a reversing hydraulic gradient of only ±1 (possibly not realistic under truck wheel loads). His theory, however, allowed 15% of the soil weight to pass through the geotextile. This is shown in Figure 9.2 together with Schober and Teindl's criteria for 85% retention for ease of comparison.

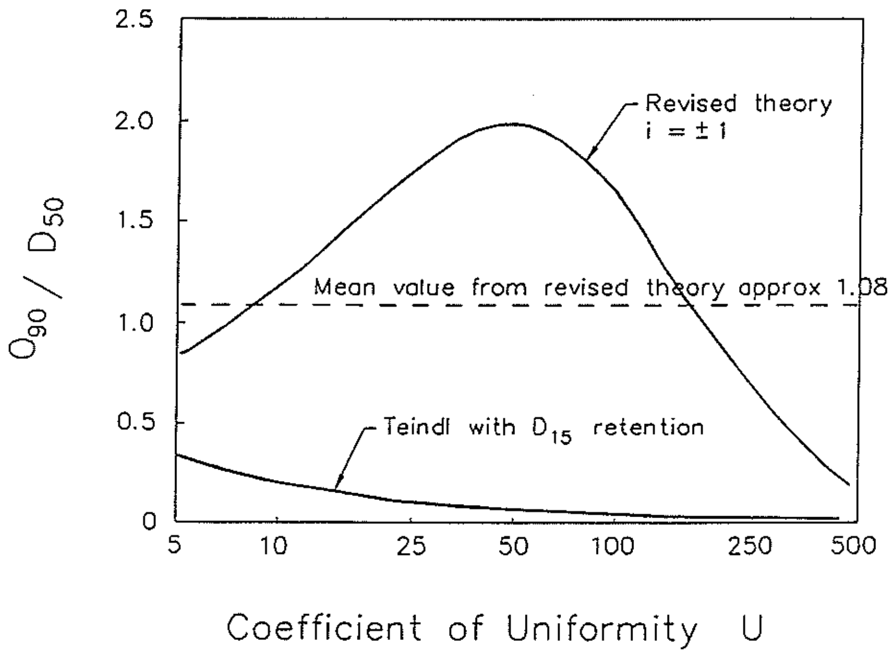


FIGURE 9.2
Variation of O_{90}/d_{50} with Coefficient of Uniformity U, for i = ±1 (Ingold 1985)

Consideration of all the above formulae for alternating flow show that complete protection of the sub-base aggregate from contamination with subgrade fines requires the filtering geotextile to have very fine pore sizes. This is not considered to be practical because geotextiles of this order are not generally available, and if they were they would be expensive. They are also likely to have low permeability and to be prone to blocking, either of which property may inhibit adequate subgrade drainage and allow excessive pore pressures to develop in the subgrade soil.

Thus in practice it is generally not realistic to rigorously follow the above rules for filtration for alternating flows. A more relaxed criterion is required, together with acceptance either that some fines will pass through the geotextile, or that additional measures will be taken to stop the passage of fines (e.g. add a sand layer, or granular filter layer, next to the geotextile, or provide effective pavement drainage).

To support this, a large number of laboratory tests have been carried out in connection with research for the International Union of Railways, and Hoare (1982b) reports that no commercially marketed fabrics can be found in Europe which can prevent the passage of fines from the clay soils tested. However, railway train loadings are more likely to cause pumping than road traffic.

9.4 FILTRATION OF ALTERNATING FLOW : FIELD AND LABORATORY EXPERIENCE

The use of geotextiles as long-term separators and filters in pavements has been the subject of considerable research, as is indicated by the references for this Chapter. The concern has generally been the protection of the pavement material from subgrade soil migrating upwards under the dynamic loading effect of traffic.

Subgrade softening due to the presence of water will be a prerequisite for the occurrence of this migration. Thus **pavement drainage should always be constructed before or in association with geotextile installation.** However it is not always possible to prevent this softening, perhaps because of the groundwater or soil characteristics at the site, or because of inadequate permeability of the sub-base layer.

A small increase in the quantity of contaminants can dramatically reduce sub-base or basecourse strength. For example, Bell *et al.* (1981) show that the static shear strength of three different sub-base materials was reduced by 20 to 40% when cohesive fines, in quantities ranging from approximately 2 to 4% of the dry weight of the sub-base aggregate, were added to the basecourse. Other basecourses may tolerate slightly greater quantities of fines, depending on their particle size distribution and the quality of these fines but, generally, relatively small additional quantities of fine particles can drastically reduce the strength of a basecourse.

While somewhat variable results have been reported, the main findings regarding sub-base protection using geotextiles for filtration of fine-grained wet subgrades subjected to repeated dynamic loads may be summarised as:

- The geotextile pore sizes or O_{95} are a major factor in controlling fine particle migration (presumably the ratio of **effective** soil particle size to geotextile pore size is important).

- The smaller the geotextile pore size the less migration of fines.

- The behaviour of woven and thin non-woven geotextiles (i.e. < 200 g/m^2 or 2mm in thickness) are similar.

- The smaller the geotextile pore size, the slower the rate of dissipation of subgrade pore-water pressure. This suggests that geotextiles with small pore sizes (in this situation) are restricting the flow of water as well as soil particles, i.e. inhibiting pore-pressure dissipation.

- Subgrade migration into a dense graded sub-base will be less than into an open graded sub-base.

- Migration of fines appears to occur at the high stress contact points where the sub-base particles contact the underlying geotextile and apply pressure to the subgrade.

- The higher the subgrade moisture content, the greater the migration of fines through the geotextile.

- The greater the number of load repetitions the greater the amount of migration of fines.

- There is some evidence that thick non-woven needle-punched or melt-bonded geotextiles perform better than other geotextiles.

- No commercially available geotextiles are able to completely filter fine particles under severe dynamic conditions (Ayres 1986).

- Properly designed thick granular filters perform better than any of the geotextiles; they prevent sub-base contamination, and also allow relatively rapid dissipation of excess pore-water pressures.

- The use of a layer of sand (minimum thickness 25mm, effective pore size 5 to 30μm) or a granular filter in conjunction with (even a thin) geotextile may be the surest means of preventing migration of fines.

- Sub-base contamination levels can be expected to be acceptable if the geotextile satisfies normal one-directional flow filter criteria (given in Chapter 8).

From the above findings it is evident that there are conflicting requirements of the geotextile. A balance between retention of soil particles, flow of water, and thickness (and therefore cost) of geotextile is required. This suggests some movement of soil particles through the geotextile should be accepted in practice, at least in severe conditions of moisture and dynamic loading. Bell *et al.* (1982) suggest that sub-base contamination can be kept within acceptable levels for most roads if a geotextile with pore sizes conforming to the one-directional flow criteria, as given in Chapter 8, is used.

9.5 GEOTEXTILE MECHANICAL PROPERTIES

A geotextile cannot perform the filtration functions discussed above, or indeed any useful function, unless it survives its initial placement and covering and remains intact under trafficking. Hence physical properties of the geotextile to ensure **survivability** (Haliburton and Barron 1983) or **integrity** (Lawson and Curiskis 1985) are of prime importance. These requirements are detailed in Section 10.4.1.

9.6 CONCLUSION

The migration of fines from a soft wet subgrade into a granular sub-base can have a detrimental effect on the overlying pavement. For severe conditions under dynamic loadings (such as could occur under a poorly drained, heavily trafficked pavement), none of the current geotextiles can completely prevent this migration, although virtually any geotextile will markedly reduce it. The smaller the geotextile pore size (relative to subgrade effective particle size), the greater the reduction in fines migration. Thick non-woven geotextiles tend to perform best. However it is not practical (for fine grained subgrades) to use the smallest pore-sized geotextiles available because of cost, and they may tend to restrict subgrade drainage and pore-pressure dissipation. To encourage this pore-pressure dissipation, a geotextile with a high percentage open area is preferred, which tends to favour non-wovens.

It is suggested (Bell *et al.* 1982) that sub-base contamination can be kept within acceptable levels for most roads if a **geotextile conforming to the one-directional flow filter criteria, outlined in Chapter 8, is used.**

Other methods of reducing or preventing migration of subgrade fines include improved drainage, lime or cement stabilisation of the subgrade, or using a granular filter layer separately or in conjunction with a geotextile.

9.7 REFERENCES

Ayres, D.J. 1986. Geotextiles or geomembranes in track? British Railways Experience. *Geotextiles and Geomembranes* 3(2 & 3): 129-142.

Bell, A.L., McCullough, L.M., Gregory, B.J. 1981. Clay contamination in crushed rock highway sub-bases. *Proceedings of Second Australian Conference on Engineering Materials.*

Bell, A.L., McCullough, L.M., Snaith, M.S. 1982. An experimental investigation of sub-base protection using geotextiles. *Proceedings of Second International Conference on Geotextiles,* Las Vegas, USA.

Christopher, B.R., Holtz, R.D. 1985. *Geotextile engineering manual - course text.* National Highway Insitute, Federal Highway Administration, Washington, DC.

East, G.R.W., Hudson, K.C. 1987. Geotextile subsoil drains : developments towards their use in fine grained soils. *New Zealand Roading Symposium* 4: 135-148.

Haliburton, T.A., Barron, J.V. 1983. Optimum depth method for design of fabric-reinforced unsurfaced roads. *Transportation Research Record* 916: 26-32. Transportation Research Board, Washington DC.

Hoare, D.J. 1982a. Synthetic fabrics as soil filters : a review. *ASCE Journal of Geotechnical Engineering* 108(GT10): 1230-1245.

Hoare, D.J. 1982b. A laboratory study into pumping clay through geotextiles under dynamic loading. *Proceedings of Second International Conference on Geotextiles,* Las Vegas, USA.

Hoare, D.J. 1984. Geotextiles as filters. *Ground Engineering* 17(2): 29-44.

Hudson, K.C. 1978. Geotextiles in road pavements. *Road Research Unit Newsletter.*

Ingold, T.S. 1985. A theoretical and laboratory investigation of alternating flow filtration criteria for woven structures. *Geotextiles and Geomembranes* 2(1): 31-45.

Ingold, T.S., Crowcroft, P.S. 1984. The notion of geotextiles as separators in roads. *Ground Engineering* 17(1): 27-34.

Lawson, C.R., Curiskis, J.I. 1985. *Geotextiles : including manufacture, properties, and use in pavements, embankments, subsurface drainage and erosion control.* Lawson Consulting, Sydney, Australia.

Ogink, H.J.M. 1975. Investigations of the hydraulic characteristics of synthetic fabrics. *Publication No. 146, Waterloopkundig Laboratorium,* Delft Hydraulics Laboratory.

Schober, W., Teindl, H. 1979. Filter criteria for geotextiles. *Proceedings of Seventh European Conference SMFE,* 2.

Waters, T.J. 1984. Evaluation of geotextiles for drainage and reinforcement applications. *Proceedings of Australian Road Research Board Conference* 12(2).

CHAPTER 10

ROAD CONSTRUCTION

10.1 GENERAL COMMENTS

Temporary or permanent roads often have to be constructed over weak subgrades. These weak subgrades can make access for construction vehicles and equipment difficult or impossible, aggregate becomes "lost" into the subgrade, and compaction of pavement layers is inhibited. A thick pavement is required to carry the traffic over such subgrades.

One generally successful technique which is frequently used for dealing with these situations is to place a geotextile directly over the poor subgrade, cover it with an aggregate layer, then use construction machinery. The initial economic benefit from this use of geotextile may well arise from the ability to physically get aggregate and machinery on to the subgrade quickly and without loss, i.e. as a construction expedient. While this initial benefit on its own may justify the inclusion of the geotextile, as long as the geotextile retains its integrity it may well continue to benefit the pavement throughout the entire life of the pavement, if the subgrade remains soft. Chapter 12 discusses the long-term benefits of a geotextile for sub-base protection.

In road construction applications the main function of the geotextile is almost always **separation**; it prevents the sub-base aggregate sinking into and mixing with the soft ground. **Filtration** is also usually desirable, allowing the excess water in the subgrade to dissipate through the geotextile into the sub-base while restraining the subgrade soil particles.

During pavement construction (and under final traffic for an unsealed temporary road) the geotextile may also provide **reinforcement**. This can allow heavy vehicles to cross the construction site with less aggregate cover on the subgrade than may otherwise be necessary.

Reinforcement for temporary roads will be discussed in Chapter 11. However, as covered in Chapter 12, little or no long-term reinforcement should be credited to the geotextile in sealed roads or, generally, in unsealed permanent roads.

10.2 SHORT-TERM BENEFITS

The main benefit of using a geotextile on top of the subgrade during construction is separation, to prevent subgrade intrusion or subgrade/sub-base mixing.

It should be noted that subgrade/sub-base mixing is only likely to be a problem over soft wet subgrades such as those with in situ CBRs of 5 or less, as noted in Chapter 9. Also there are a number of alternative approaches to solving the problem including lime and/or cement stabilisation, and improved subgrade drainage.

If a geotextile is used to aid road construction **then drainage should also be provided** if practicable, to aid the long-term strength gain of the subgrade. This should include provision for draining the sub-base. While thick non-woven geotextiles have some in-plane permeability it is good practice to provide a permeable sub-base and appropriate gradients and flow lengths to drains.

10.3 LONG-TERM BENEFITS

As detailed in Chapter 12, the long-term benefits of separation can also be very worthwhile, particularly on soft wet subgrades. In permanent roads separation to prevent subgrade/sub-base mixing (subgrade pumping or intrusion) can have a variety of benefits:

- It prevents or reduces contamination of the sub-base or basecourse.
- It avoids or reduces the need to have an additional sacrificial thickness of sub-base (i.e. the portion that will become contaminated) which may allow thinner granular pavement layers.
- It allows the use of a more open graded sub-base. This will be more permeable, allowing improved drainage of the pavement layers including the subgrade.
- Improved subgrade drainage via the geotextile and sub-base may allow the subgrade to strengthen.

Maintenance benefits accruing from the use of geotextiles are referred to by Lawson (1986) who mentions reports by Anderson and Freden (1977), and Lawson and Ingles (1982). This maintenance benefit is difficult to quantify as conditions vary from site to site and little data are currently available.

10.4 SELECTION OF GEOTEXTILES

10.4.1 Mechanical Properties

Any geotextile that will survive construction stresses could be used as a separator to aid road construction. Construction stresses imposed on the geotextile will vary from site to site:

- Use of a coarse angular sub-base aggregate will increase puncturing and tearing stresses.

- Use of a dense graded sub-base aggregate with rounded stones will reduce puncturing and tearing stresses.

- The softer the existing subgrade the greater the tensile stresses and/or the greater the elongation. Department of Main Roads, Queensland (1983) suggest that for a subgrade of CBR 2.5 to 5, a moderately robust geotextile is required, one of CBR 1 to 2.5 requires a robust geotextile, while subgrades of CBR < 1 require a very robust geotextile (see Section 11.7.1).

- Vegetation stumps left from clearing will increase puncture, tearing and tensile stresses.

- Site clearance effects, e.g. removal of root mat, may influence the effective subgrade CBR and hence geotextile stresses.

- The thinner the aggregate cover the higher the puncture, tearing and tensile stresses under traffic.

- The heavier the construction plant and the higher the amplitude of the vibrating rollers the greater the puncture, tearing and tensile stresses.

- The larger the magnitude of differential settlements of the subgrade with time the higher the tensile stress and/or elongation.

A geotextile cannot perform any of its functions unless it survives its initial placement and covering. Hence physical properties of the geotextile to ensure its **survivability** (Haliburton and Barron 1983) or **integrity** (Lawson and Curiskis 1985) are of prime importance.

Attempts have been made to relate survivability to standard fabric strength measurements such as that from Hausmann (1987) shown in Table 11.2. Lawson and Curiskis (1985) and Lawson (1986) have also commented on this. The properties they consider important for mechanical integrity of a geotextile for separation at the subgrade/sub-base interface are shown in Table 11.2 and Figure 12.1. Figure 12.1 enables the determination of integrity requirements in terms of geotextile trapezoidal tear and CBR puncture resistance, given a subgrade strength and a characteristic sub-base aggregate diameter (D_{85}).

In addition to the above mechanical properties, consideration needs to be given to a combination of geotextile strength and geotextile strain to break, especially when the subgrade is weak. A weak subgrade tends to rut under areas of wheel loading. A high strength geotextile will change the failure mode of the subgrade by its reinforcing effect. A low strength geotextile is not able to much influence the failure mode of the soil, so has to strain to follow the subgrade deformations. So, while geotextile strength alone is not of great interest for separation, a combination of mobilised stress and strain is. Van den Berg and Kenter (1984) refer to this as the stress-strain energy.

10.4.2 Hydraulic Properties

As noted in Chapter 9, the geotextile should also be a filter, retaining the subgrade soil particles while allowing water from the subgrade to pass into the permeable sub-base and drains away. This helps to prevent a build-up of subgrade pore pressures with corresponding loss of subgrade strength. Thus it is desirable for the separating geotextile to be designed also for filtration (i.e. permeability and pore size) in accordance with the criteria for one-directional flow given in Chapter 8. As explained in Chapter 9 these criteria are aimed for a balance between allowing pore pressure dissipation, and inhibiting intrusion of subgrade fines up into the sub-base. They may, however, allow some subgrade fines to pump into the sub-base through the geotextile.

10.5 PAVEMENT CONSTRUCTION DENSITY REQUIREMENTS

Compaction of the pavement layers to a reasonable density (e.g. to achieve a dry density of 100% of standard compaction) is desirable during construction. Dense pavement layers spread the wheel loads to the subgrade better, and reduce any tendency to show wheel track ruts from further compaction under traffic.

Over soft subgrades good compaction of the initial pavement layers is usually not possible as the subgrade does not provide a good "anvil" for the rollers to compact against. Also the subgrade may weave and/or weaken under repeated pressures from the rollers.

One of the problems facing the construction engineer when constructing pavements over soft ground is to determine how much aggregate must be placed before density requirements can be achieved in the upper portion (say 300mm) of fill. As will be discussed in Section 12.5, use of a geotextile does not allow increased densities to be obtained in the covering aggregate. Hence apart from a reduction of subgrade/sub-base mixing, use of a geotextile will not result in thinner cover requirements to obtain the required density.

Use of a lime-stabilised or cement-stabilised sub-base aggregate above the geotextile can provide a firmer base for subsequent pavement layers if the lime or cement is given time to set or cure.

10.6 COVER REQUIREMENTS

Given a suitable geotextile it would be possible to lay the geotextile on soft ground, peg the sides to put it under tension, then drive vehicles directly over it. This would require a very robust geotextile, and substantial wheel ruts would result after a few vehicle passes. All the geotextiles commonly used in road construction on soft subgrade require a covering of gravel or aggregate before any trafficking, to prevent damage to the geotextile and/or over-straining of the subgrade. The depth of this initial layer can be critical.

The minimum cover before any construction traffic is allowed on the geotextile will depend on the type and weight of the construction machinery, subgrade strength, geotextile strength and sub-base aggregate size, shape and grading. Normal practice over strong, competent subgrades is for approximately 200mm lifts. On very soft subgrades it is prudent to place at least 300mm of gravel or aggregate, but some situations and geotextiles will require more than this thickness. Chapter 11 (Section 11.3) provides guidance on cover which can be used to indicate thickness of aggregate cover that will carry construction traffic.

10.7 OVERLAP

Overlap of the geotextile provides frictional resistance and can be used to provide continuity between adjacent rolls. The amount of overlap needed depends primarily on the soil conditions and the likely rut depth. Since the rutting potential and CBR are related, CBR can be used to determine minimum overlap requirements (Table 10.1).

Subgrade CBR	**Minimum Overlap (mm)**
Greater than 2	300 - 450
1 - 2	600 - 900
0.5 - 1	900 or sewn
Less than 0.5	sewn

TABLE 10.1
Recommended Minimum Overlap Requirements
(Christopher and Holtz 1985)

Where high settlements are expected (e.g. over peat), the overlap should be increased up to 1.5m (ICI Fibres 1988).

To avoid displacing the geotextile during sub-base spreading, the transverse laps should have the exposed edge pointing away from the advancing sub-base.

Koerner (1986) notes that the sewing of geotextiles has advanced rapidly to the point that for all geotextile construction on soft ground, consideration should be given to sewn joints. However, with sewing there are potential problems, e.g. dampness can cause thread or needle break, bobbins need covering, etc. See Figure 11.9.

Geotextiles can be joined by stapling, using corrosion resistant staples inserted with an industrial stapler. Stapled seam strengths are likely to be considerably lower than those achieved by sewing (ICI Fibres 1988). (See Figure 11.9.)

The use of wider rolls of geotextile (e.g. 4m rather than 2m wide) can reduce the need for joints.

10.7.1 Anchorage

To obtain any reinforcing effect from the geotextile requires careful consideration of length needed for anchorage. Anchorage slippage can occur either by "pulling out" of the geotextile leaving the pavement aggregate behind, or by movement of the entire geotextile/basecourse segment.

Chemie Linz (1986) recommend a minimum geotextile length from the outer wheel track to the road edge of at least 1.25m + D (Figure 11.10). Other options to gain anchorage include turning under the geotextile into the aggregate or into an anchor trench.

10.8 CONSTRUCTION

Preparation of the subgrade will depend on the type of soil and how soft it is. A typical construction procedure is:

- Remove large boulders and sharp objects.
- Trees and shrubs should be removed and tree trunks cut as close to the ground as possible. On some very soft subgrades, where inclusion of an organic layer will not be detrimental, retention of the root mat may add to the ground's strength.
- Shape the subgrade.
- Roll out the geotextile and overlap, weld, staple, or sew the joints. Limiting the uncovered length of geotextile to 10 to 15m will reduce lifting by the wind.
- Weight the edges with aggregate.
- Protect the geotextile from prolonged exposure to UV light.
- Avoid traffic directly on the geotextile itself.
- Tip the gravel or sub-base aggregate on to existing previously spread aggregate then spread it over the geotextile. Do not tip directly onto the geotextile.
- Spread the sub-base aggregate over the geotextile with a tracked machine, as tracks spread the load better than wheels.
- Once a sufficient thickness of gravel or aggregate (Section 10.6) covers the geotextile, compact using a steel-wheeled roller. The first few passes should be with the vibrations off.

10.9 GEOGRIDS

Chaddock (1988) investigated the use of geogrids between the sub-base and subgrade to provide reinforcing. The trials were done on gravel surfaced roads in a test pit. Subgrades of CBR 0.4, 1.6 and 4.9 were trialled. Being a trial the load repetitions were applied over a short time, approximating the unsealed temporary road situation.

Inclusion of a geogrid over the subgrades of CBR 1.6 and 4.9 did provide some effective reinforcement and effective separation. Inclusion of the geogrid allowed about 3.5 times more traffic to be carried before wheel track rutting reached 40mm depth, compared to the same pavement without a geogrid. Inclusion of the geogrid over the subgrade of CBR 0.4 provided little benefit and the clay subgrade intruded through the grid into the sub-base aggregate.

This suggests that, for subgrades with CBRs 1.5 to 5, a geogrid may allow increased trafficking, or a thinner pavement, for unsealed temporary roads. This is in accord with the benefits expected from a geotextile as indicated in Chapter 11 and Section 10.1. In contrast to geotextiles however, geogrids appear to offer little benefit on very soft subgrades.

10.10 REFERENCES

Anderson, O., Freden, S. 1977. The influence of plastic fabric upon the pavement at frost break. *Frost Action in Soils, No. 18.* National Road and Traffic Research Institute, Sweden.

Chaddock, B.C.J. 1988. Deformation of road foundations with geogrid reinforcing. *Transport and Road Research Laboratory (TRRL), Research Report* 140. 8p.

Chemie Linz 1986. *Polyfelt TS. Design and practice.* Chemie Linz AG, Austria.

Christopher, B.R., Holtz, R.D. 1985. *Geotextile engineering manual — course text.* National Highway Institute, Federal Highway Administration, Washington, DC.

Department of Main Roads 1983. *Evaluation of geotextiles.* Main Roads Department, Queensland, Australia.

Haliburton, T.A., Barron, J.V. 1983. Optimum-depth method for design of fabric-reinforced unsurfaced roads. *Transportation Research Record* 916: 26-32. Transportation Research Board, Washington DC.

Hausmann, M.R. 1987. Geotextiles for unpaved roads - a review of design procedures. *Geotextiles and Geomembranes* 5(3): 201-233.

ICI Fibres 1988. *Designing for subsurface drainage.* ICI Geotextiles, Pontypool, Wales.

Koerner, R.M. 1986. *Designing with geosynthetics.* Prentice-Hall, Englewood Cliffs, NJ.

Lawson, C. 1986. Geotextiles or geosynthetics? *Proceedings of Technical Sessions, Third Asian ICI Fibres Geotextiles Conference,* Thailand.

Lawson, C.R., Ingles, O.G. 1982. Long term performance of MESL roads sections in Australia. *Proceedings of Second International Conference on Geotextiles,* Las Vegas, USA.

Van der Berg, C., Kenter, C. 1984. *Design methods and guidelines for geotextile applications in unpaved low volume roads.* Geotextile Technology, Imperial College, London, UK.

CHAPTER 11

TEMPORARY ROADS

11.1 GENERAL COMMENTS

Temporary roads can be arbitrarily defined as aggregate surfaced (unsealed) roads that carry less than 10,000 vehicle passes. Temporary roads may have a short life, e.g. less than a year, and may well develop deep wheel track ruts.

Most temporary road design methods which have been developed relate to roads that offer only minimal riding characteristics and assume that **substantial rutting is acceptable** (e.g. 150mm rut depths). This may be acceptable for trucks and tractors, but will not be acceptable for most on-highway vehicles (Section 11.3).

Temporary roads and/or temporary pavements may be applicable to access roads, haul roads, forest roads, detours, military roads, etc., that will carry relatively few vehicles and possibly have a short life. If these roads are on soft subgrades, conventional unreinforced pavement design procedures indicate that substantial thicknesses of aggregate will be needed to prevent excessive subgrade shear or rut formation. Ways to reduce these substantial thicknesses of aggregate include improved subgrade drainage, lime or cement stabilisation of the subgrade, or inclusion of a geotextile in the pavement.

In addition to the design of temporary roads the procedures outlined in this Chapter can also be used to indicate minimum aggregate depths required to cover a geotextile to carry construction traffic during construction of permanent pavements.

Inclusion of a geotextile in a temporary pavement is likely to be economically attractive generally only for subgrades with in situ CBR 5 or less.

The geotextile will usually be placed directly on the subgrade, usually soft wet soil, with stone aggregate placed directly above.

Another layer of geotextile in the base layer may be beneficial in some cases. This will increase resistance to "shallow shear" failures in poor quality basecourse by placing the fabric across the normal failure surface. However, use of a "slippery" geotextile could induce failures by providing a slippery failure surface.

The benefits of using a geotextile in temporary pavements arise from faster and therefore cheaper construction, and thinner cheaper pavements, across soft subgrades.

11.2 GEOTEXTILE FUNCTIONS

While a geotextile in a temporary road pavement will perform separation, reinforcement, filtration and drainage functions (if the geotextile construction is suitable), its main functions will usually be **separation** and **reinforcement**.

As a separator, the geotextile prevents or inhibits contamination of the coarse aggregate with subgrade fines, and prevents the punching of coarse aggregate particles down into the soft subgrade. On saturated silty soils, pavement aggregate contamination by fines is accelerated by hydraulic action. Upward water flow (pumping) from the subgrade under passing wheel loads may cause irreversible soil particle movement (e.g. pumping). In such a situation the geotextile is expected to act as a filter and to maintain the integrity of the soil layer system. This filtration under alternating flows is covered in Chapter 9.

As reinforcement the geotextile may allow a reduction in pavement aggregate thickness. It is emphasised that, to gain any advantages from its reinforcement ability, the **geotextile must be tensioned by stretching due to deformation, and remain tensioned.** This requires rut formation in the subgrade and hence on the pavement surface, from trafficking. While the geotextile can be pretensioned across the direction of the road by other means, this is not practical as it is expensive and the pretension would probably be lost in time due to creep.

11.3 LEVEL OF SERVICE

To generate geotextile tension requires a minimum plastic deformation of 25mm at the surface of the pavement. Greater reinforcement benefits are possible with greater rut depths; for instance, rut depths of 300mm and 150mm are an option in Giroud and Noiray's (1981) design method and the Chemie Linz (1986) design method respectively. Rut depths of this magnitude, while acceptable to trucks, construction equipment, military vehicles and some 4-wheel drive vehicles, will render the road **impassable to ordinary cars.**

These ruts can be filled by the addition of more aggregate. While this costs money, it has the advantage of increasing the pavement thickness where it is most needed, i.e. in the wheel tracks, and hence reducing the subsequent rate of rut depth increase.

As discussed the design methods given in this Chapter result in pavements offering minimal levels of service with substantial rut depths. For these pavements the geotextile can be used as both a separation and a reinforcement layer. At the other end of the spectrum is a requirement for a temporary road which offers good levels of service, is required for a longer period of time, and is suitable for cars as well as trucks. For these pavements the role of the geotextile is essentially as a separation layer only (Lawson 1988).

11.4 REINFORCEMENT ACTION

By providing reinforcement, geotextiles can improve the performance of unpaved roads. For a given thickness of aggregate layer the traffic can be increased, or for the same traffic the thickness of the aggregate layer can be reduced (in comparison with the required thickness when no geotextile is used). This improvement in performance has been recognised by users in the field, but design procedures are still somewhat lacking.

Hausmann (1987) reviewed current design procedures for geotextiles in unpaved roads and found them to be based only on strength and deformation characteristics of fabrics and subgrades. The theoretical considerations of the effects of fabrics on bearing capacity, enhanced ability of the aggregate to distribute surface loads, and membrane support are relatively crude and lack comprehensive validation from field usage. It would really be desirable to obtain more field performance data before accepting these concepts as good approximations of the complex structural interaction of soil and fabrics.

In the interim, however, design methods such as proposed by Sellmeijer *et al.* (1982), Giroud and Noiray (1981), and Barenberg (1980) appear to give reasonable guidance (Hausmann 1987). In spite of its limitations the Giroud and Noiray design procedure is a significant attempt to understand the mechanics of unpaved roads (Milligan *et al.* 1989).

Putting it another way, it appears that the reinforcement mechanism for geotextiles in temporary roads is **not yet fully understood**. None the less, benefits appear to be there in practice. While currently far from perfect, design methods do exist that can be used to give guidance to roading practitioners. The work at Oxford University reported in Section 11.6.4 (Milligan *et al.* 1989) suggests that improved design procedures are under development but are not yet available.

In view of the limitations of the current design methods, the mechanisms of geotextile reinforcing assumed by them, particularly the Giroud and Noiray (1981) method, are examined in Section 11.5.

11.5 GEOTEXTILE REINFORCEMENT : MECHANISMS

The current product-independent design methods assume that benefits arise from one or more of the following mechanisms associated with including a geotextile at the sub-base/subgrade interface.

11.5.1 Control of Bearing Failure Mode

The classic Terzaghi-bearing capacity formula applicable for strip footings:

$$q = cN_c + \gamma\ DNq + 0.5\ _{\gamma 2}\ BN_{\gamma}$$

includes three bearing capacity factors (Nc, Nq and Nγ) that are a function of the angle of internal friction only, and vary with the assumed failure geometry. For a purely cohesive soil this formula reduces to $q_u = cN_c$.

There are two failure geometries: local shear and general shear.

Local Shear Failure is a condition where failure wedges develop only in the soil immediately below the footing; shear and densification of the stressed soil cause excessive settlement without noticeable bulging at the ground surface. An extreme case of local bearing failure is also referred to as **punching shear.** This would be typical in very loose sands, and very soft clays, with CBR values less than 3 (Hausmann 1987).

General Shear Failure is characterised by recognisable failure planes extending from the edge of the loaded area to the ground surface, causing significant upward bulging of the soil. General shear failure is typical for dense sands and stiff clays.

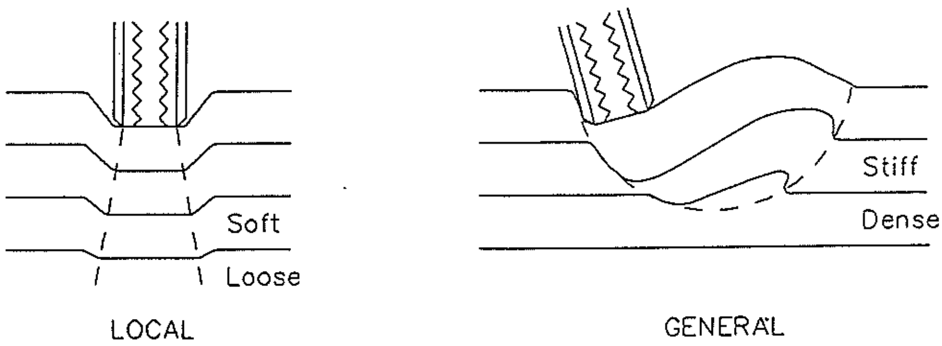


FIGURE 11.1
Local and General Bearing Failure in Cohesive Soils (Hausmann 1987)

Terzaghi suggested that for a purely cohesive soil the relationship of factor N in his equation for General Shear Failure to Local Shear failure is 5.4 to 3.4.

Thus changing the failure mode in the subgrade under a reinforcing geotextile from local shear to general shear can increase the bearing capacity by some 5.4/3.4, i.e. about 60%.

11.5.2 Membrane Effect

Settlement or rutting under the wheel tracks usually causes heave between and beyond the wheel tracks. Therefore a geotextile at or near the subgrade/sub-base interface develops a wavy shape and stretches after trafficking. When a stretched flexible material has a curved shape, pressure against its concave face is higher than against its convex face. This is known as the "membrane effect".

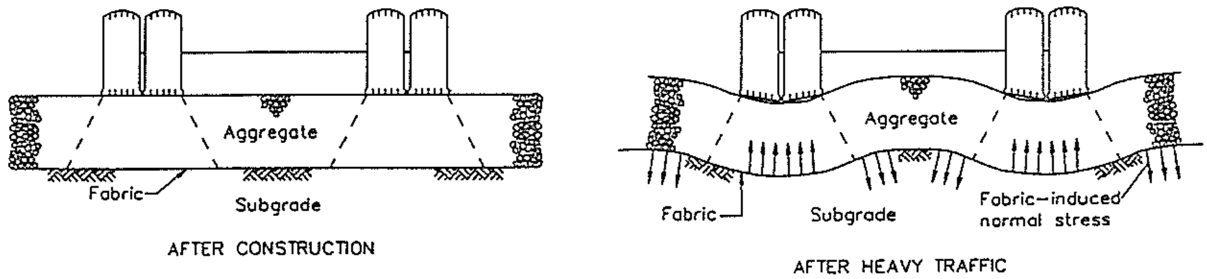


FIGURE 11.2
Membrane Effect for Unpaved Road with Geotextile (Robnett and Lai 1981)

This membrane effect has two benefits as shown in Figure 11.2:

(i) Confinement of the subgrade soil between and beyond the wheels, and

(ii) Reduction of the pressure applied by the wheels on the subgrade soil.

The same conclusion can be arrived at by assuming a tension in the geotextile and resolving the forces into their horizontal and vertical components under and between the loaded wheels.

11.5.3 Slab Action

Unbound granular materials are assumed to have no tensile strength characteristically. Consequently, when a layer of aggregate supported by a soft subgrade is subjected to an applied load, the load distribution effectiveness of the aggregate layer is limited by the shear stresses that develop at the interface between aggregate layer and subgrade (Bender and Barenberg 1978). A layer of fabric can increase the load distribution effectiveness of the aggregate layer by providing tensile strength. In effect, simplistically, the fabric in this system serves much the same function as does reinforcing steel in concrete.

11.6 DESIGN METHODS

As outlined in Section 11.4 above, Hausmann (1987) has reviewed a number of design procedures. The mechanisms of fabric performance upon which each of these procedures is theoretically based is summarised in Table 11.1 (Hausmann 1987):

Method (Reference)	Failure Mode			
	Control of Bearing	Slab Effect	Membrane Effect	Traffic Failure
Steward et al.(1977)	yes	no	no	no
Barenberg et al.(1975)	yes	yes	no	no
Haliburton Barron (1983)	yes	no	no	
Raumann (1982)	yes	some	yes	no
Barenberg (1980)	yes	yes	yes	no
Sellmeijer et al.(1982)	yes	no	yes	yes
Giroud Noiray (1981)	yes	no	yes	yes

TABLE 11.1
Failure Modes Considered in Design Methods
for Fabric-Reinforced Unpaved Roads

As previously mentioned, of these, Hausmann (1987) concluded that Sellmeijer *et al.* (1982), Giroud and Noiray (1981) and Barenberg (1980) appear to give reasonable guidance. Holtz and Sivakugan (1987) have also examined the product — independent design procedures available and concluded that Giroud and Noiray (1981) is the most elegant for quite a number of reasons. Milligan *et al.* (1989) notes that the Giroud and Noiray method is currently the most widely used. Accordingly Giroud and Noiray's method is the only one considered in detail in this report.

11.6.1 Giroud and Noiray Design Procedure

Giroud and Noiray (1981) developed design criteria for geotextile-reinforced unpaved roads. The results are presented in the form of charts, most of which have been produced by Holtz and Sivakugan (1987) using the Giroud and Noiray procedure.

The Giroud and Noiray design procedure is developed using a combination of:

(i) Formulae (based on an extensive test programme) relating to aggregate thickness and traffic for unpaved roads **without** geotextile, and

(ii) A quasi-static analysis comparing unpaved road behaviour with and without geotextile.

The charts are only applicable to purely cohesive subgrade soils and to roads subjected to light to medium traffic (1 - 10,000 truck passages over the lifetime of the road).

Assumptions made in the design method (with comment) appear to include:

- The rut depth increases only as a result of subgrade deformation, i.e. no significant basecourse shear occurs. This assumption may be questionable with heavy traffic, a soft subgrade, and a lively or poor quality basecourse, because low basecourse compaction levels and high pavement deflections tend to increase the likelihood of basecourse shear.

- In addition to reinforcement, the geotextile will fulfil both the separation and filtration functions as required to maintain base and sub-base integrity. Thus the design charts presumably show the benefits of both reinforcement and separation.

- Anchorage and longitudinal lapping of the geotextile prevents slippage.

- The geotextile is laid flat with no creases.

- Creep and stress relaxation of the geotextile once tensioned are ignored. Results of laboratory tests on geotextiles in-air (Millar 1986) indicate creep could be a problem resulting in either stress relaxation or geotextile failure, for the majority of geotextiles available in New Zealand in late 1985. The geotextile strain will have two components, the permanent deformation (indicated by rut depth) and the elastic deformation under a passing wheel (as measured by the Benkleman Beam rebound). Fabric creep has been viewed as a highly complex interaction mechanism requiring considerable research (Shrestha and Bell 1980). In addition, in-soil creep tests indicate lower creep than in-air tests. (Kinney and Barenberg (1982) give a limited theoretical consideration of creep.) Creep and stress relaxation will be a consideration only at deeper rut depths. Using the Giroud and Noiray (1981) equations, at 75mm rut depth the induced strain in the geotextile is only about 1%, but at 300mm rut depth elongation can be up to 13%. Tolerable strains that will not result in significant creep will depend on the geotextile type (woven or non-woven) and fibre. Lawson (1988) suggests an arbitrary limit of 5% strain under repeated load conditions.

- Constant volume conditions pertain in the subgrade as it ruts under trafficking but Ingold and Crowcroft (1984) question this assumption. Also it may not apply to a compressible subgrade such as peat.

- The subgrade soil is purely cohesive. Under quick loading, such as traffic loading, it is assumed the material behaves in an undrained manner so the usual assumption is that its friction angle is nil.

- The geotextile is laid at the subgrade/sub-base interface.

- The fourth power law applies to convert axle loads to the standard axle load of 80kN. The fourth power law is well known for sealed roads.

- All vehicles will travel in the same wheel paths. If this does not occur some of the geotextile tension may be lost. This assumption is acceptable for narrower roads, but is doubtful for wide roads, yards, hardstands, etc.

- Wheel loads will be distributed uniformly over an area which increases with aggregate depth by an angle to the vertical = 64°.

- The reduction in aggregate thickness caused by using the geotextile does not relate to traffic volumes. Giroud and Noiray note that full scale tests should be performed to verify this assumption.

Giroud and Noiray (1981) claim that some (i.e. limited) full scale tests on unpaved roads with and without geotextiles carried out by the United States Army Corps of Engineers indicated good agreement with the curves when traffic is light, and the theoretical results appear conservative when traffic is heavy, for 300mm rut depth.

Practical design charts (Figures 11.3-11.7) are given, showing curves for the required aggregate thickness without geotextile h'_o for various numbers of vehicle passes (N = 10 to 10,000) and the reduction of aggregate thickness Δh due to the geotextile, all versus subgrade shear strength or CBR. The required aggregate thickness with geotextile h' is then determined from $h' = h'_o - \Delta h$. The design charts given relate to on-highway trucks. Reference to Giroud and Noiray (1981) will give design equations for off-highway trucks with very wide dual tyres.

The geotextile elongation can be checked from the design chart for rut depth of 300mm. This elongation needs to be checked against the elongation at failure of the geotextile, and needs to be considered relative to geotextile creep.

11.6.1.1 Design Charts

Examination of the following design charts (Figures 11.3 to 11.7), together with a review of the assumptions and results of Giroud and Noiray's method, indicate the following implications (Giroud and Noiray 1981; Holtz and Sivakugan 1981):

- Tyre inflation pressures ranging from 480 to 620 kPa have no influence on required aggregate thickness with geotextile, for subgrade CBR less than 1.8, and only minor influence above CBR 1.8.

- The required aggregate thickness is very sensitive to subgrade CBR, particularly for CBR less than 2.

- Because of small strains induced in the geotextile by smaller rut depths, the modulus of the geotextile is not important. Thus at small rut depths ($r \geq 75$ mm) the geotextile acts as a separation layer rather than a reinforcing layer.

- For 200 and 300mm rut depths and very low traffic volumes negative aggregate cover over the geotextile is needed for some subgrades. This is nonsensical, but suggests the geotextile is strong enough to carry the few trucks without needing an aggregate cover to spread the load. In practice, virtually all geotextiles will need a minimum aggregate cover (see Section 10.6) for protection and anchorage.

- The higher the geotextile modulus the thinner the required thickness of aggregate for rut depths greater than 100mm.

- For typical geotextiles, the reduction in aggregate thickness resulting from the use of a geotextile over a subgrade with CBR of one, and between 1,000 and 10,000 vehicle passes, generally ranges between 20% and 60%.

- The reduction in aggregate thickness arising from the use of a geotextile is independent of traffic. This was one of the assumptions made in deriving the method (Section 11.6.1).

Notation used on the charts (Figures 11.3 to 11.6) is generally the same as used by Giroud and Noiray and is:

Symbol			Unit
P	=	axle load	kN
r	=	rut depth	m
P_c	=	tyre inflation pressure	kPa
h'_o	=	aggregate thickness without geotextile	m
Δh	=	reduction in aggregate thickness resulting from the use of a geotextile	m
K	=	geotextile secant modulus in transverse direction to the road	kN/m
ϵ	=	elongation of geotextile, or strain	%
N	=	number of passages of axle load P	

For axle load P above if it is a dual tyred single axle of P=82kN then number of passages = Equivalent Design Axles, EDA. Similarly other dual-tyred single-axle loads can be approximately converted to EDA or equivalent 8.2 tonne axles by the fourth power law as described in the State Highway Pavement Design and Rehabilitation Manual (National Roads Board 1987). However, Giroud and Noiray (1981) use a different procedure together with unpublished design charts for axle loads other than 80kN; thus for axle loads much greater than 80kN reference should be made to Giroud and Noiray (1981), particularly for dual or triple axle groups.

11.6.1.2 Example from Giroud and Noiray (1981)

Consider 70 passages of trucks with rear tandem-axle load of 160kN and 340 passages of trucks with a rear single-axle load of 50kN. For a final rut depth of 300mm, what is the aggregate thickness and geotextile strain if subgrade CBR is 1 and the secant modulus of the geotextile is 90 kN/m?

Using the fourth power law for the axle loads and increasing the equivalent two single-axle loads for the tandem by 20% to allow for subgrade stresses from interaction between the two axles of the tandem (front axle loads should also be considered):

For tandem axle trucks

$$N \text{ at } P_{80} = \frac{70 \times 2 \ \ (160 \times 0.6)^4}{(80)} = 290$$

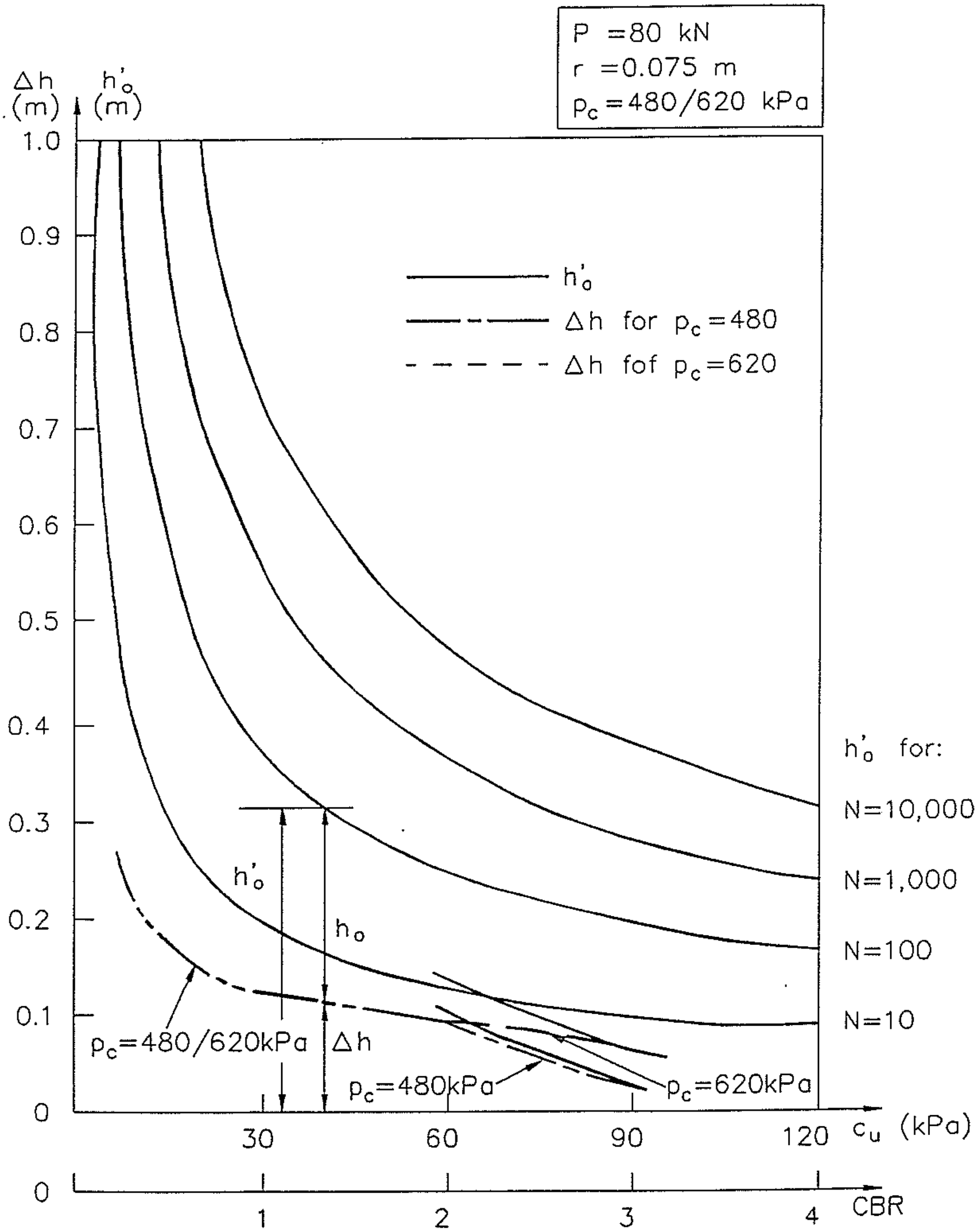


FIGURE 11.3

Aggregate Thickness h'_o Without a Geotextile and Possible Reduction in Aggregate Thickness Δh Using a Geotextile. On-highway truck. Rut depth 75mm (Holtz and Sivakugan 1987)

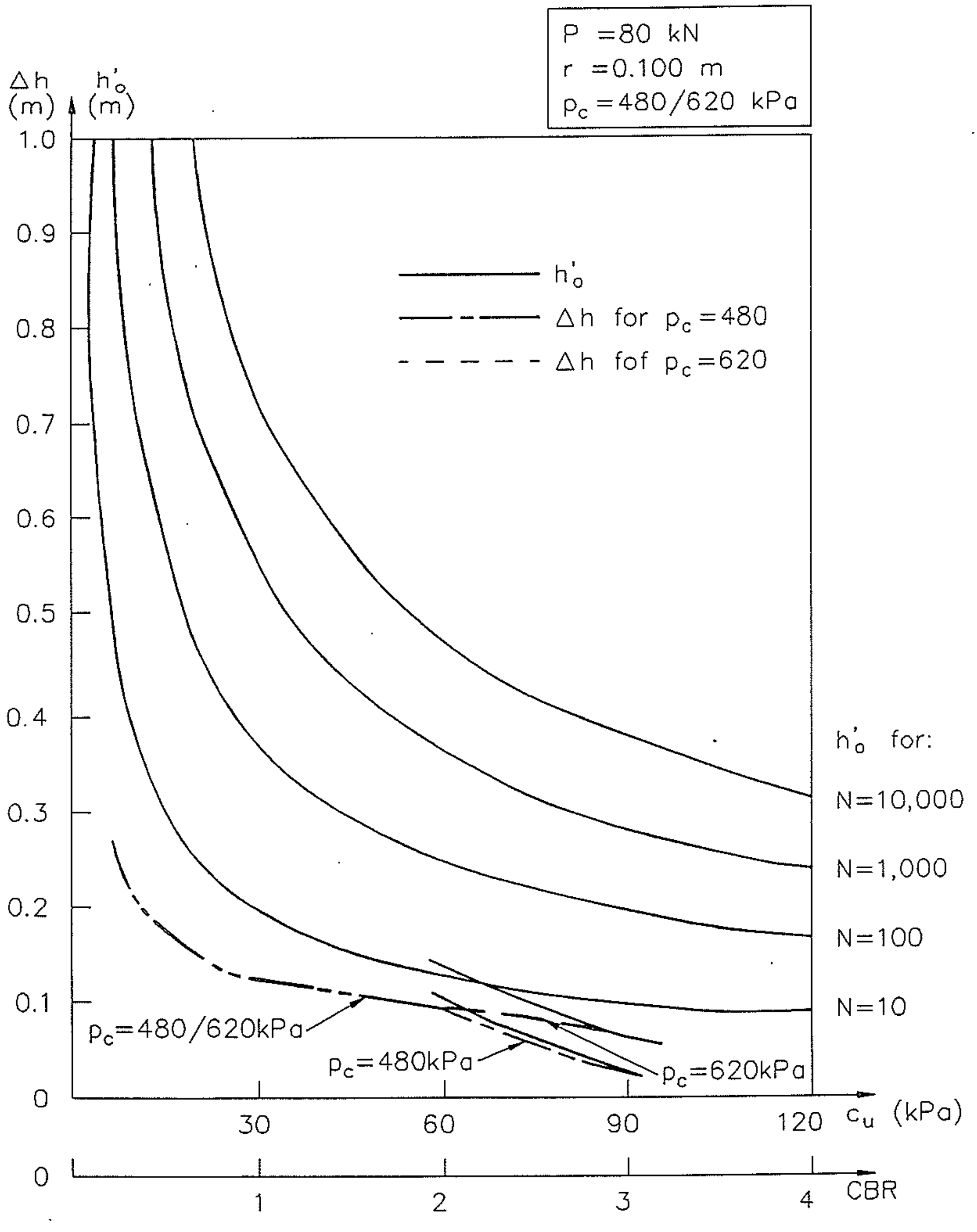


FIGURE 11.4
Aggregate Thickness h'$_o$ Without a Geotextile and Possible Reduction in Aggregate Thickness Δh Using a Geotextile. On-highway Truck. Rut depth 100mm
(Holtz and Sivakugan 1987)

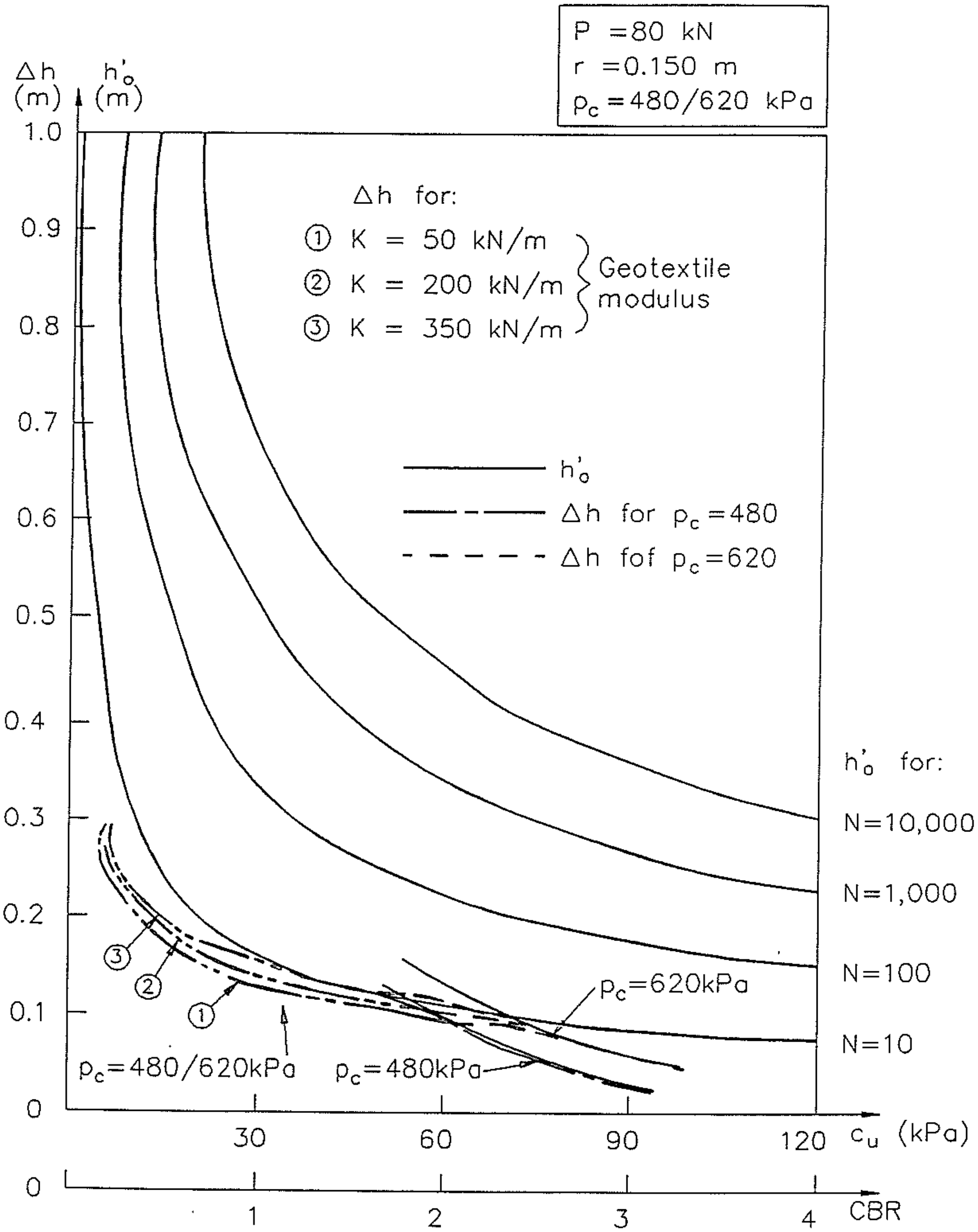


FIGURE 11.5
Aggregate Thickness h'_o Without a Geotextile and Possible Reduction in Aggregate Thickness Δh Using a Geotextile. On-highway Truck, Rut depth 150mm
(Holtz and Sivakugan 1987)

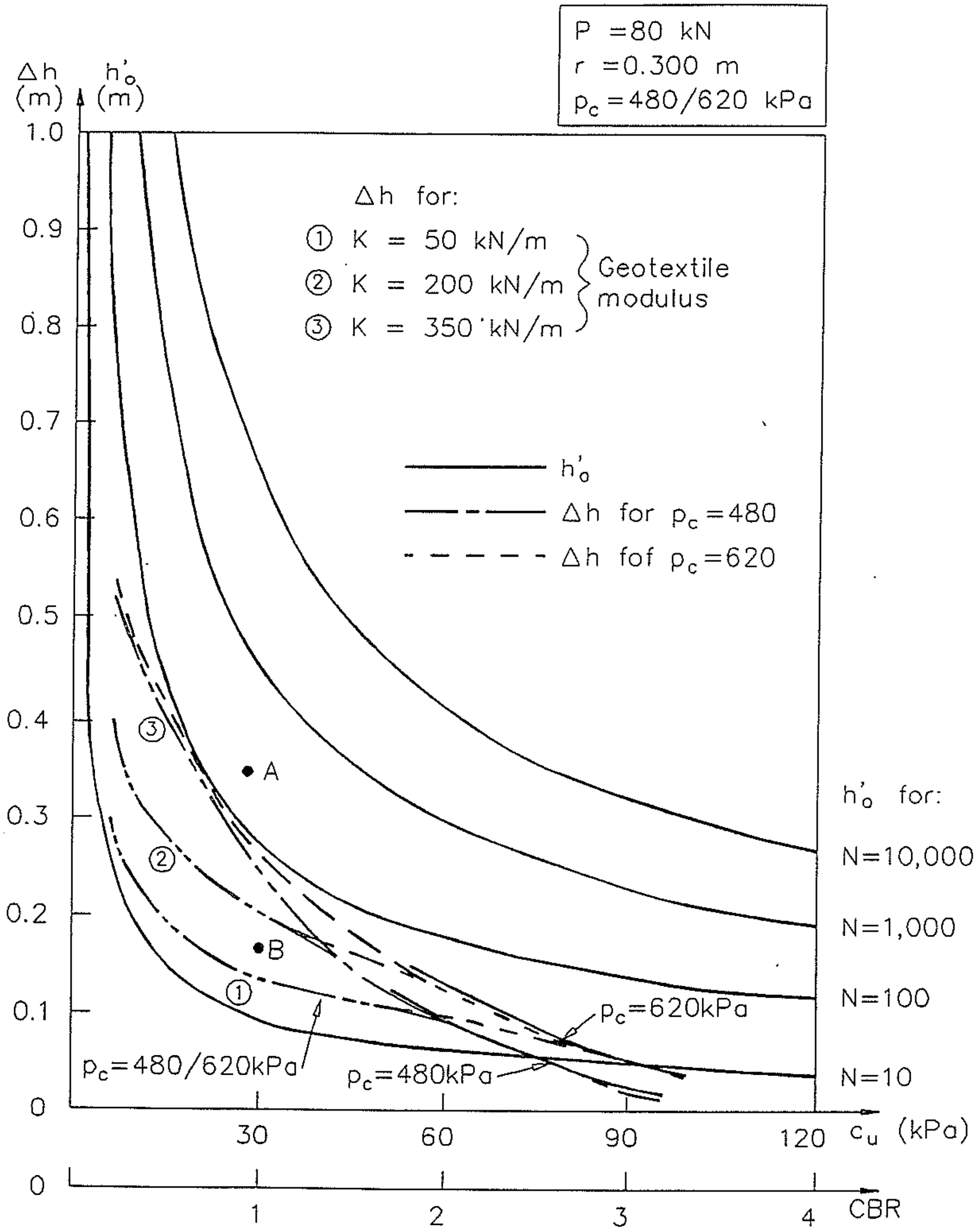


FIGURE 11.6
Aggregate Thickness h'_o Without a Geotextile and Possible Reduction in Aggregate Thickness Δh Using a Geotextile. On-highway Truck ;. Rut depth 300mm
(Giroud and Noiray 1981)

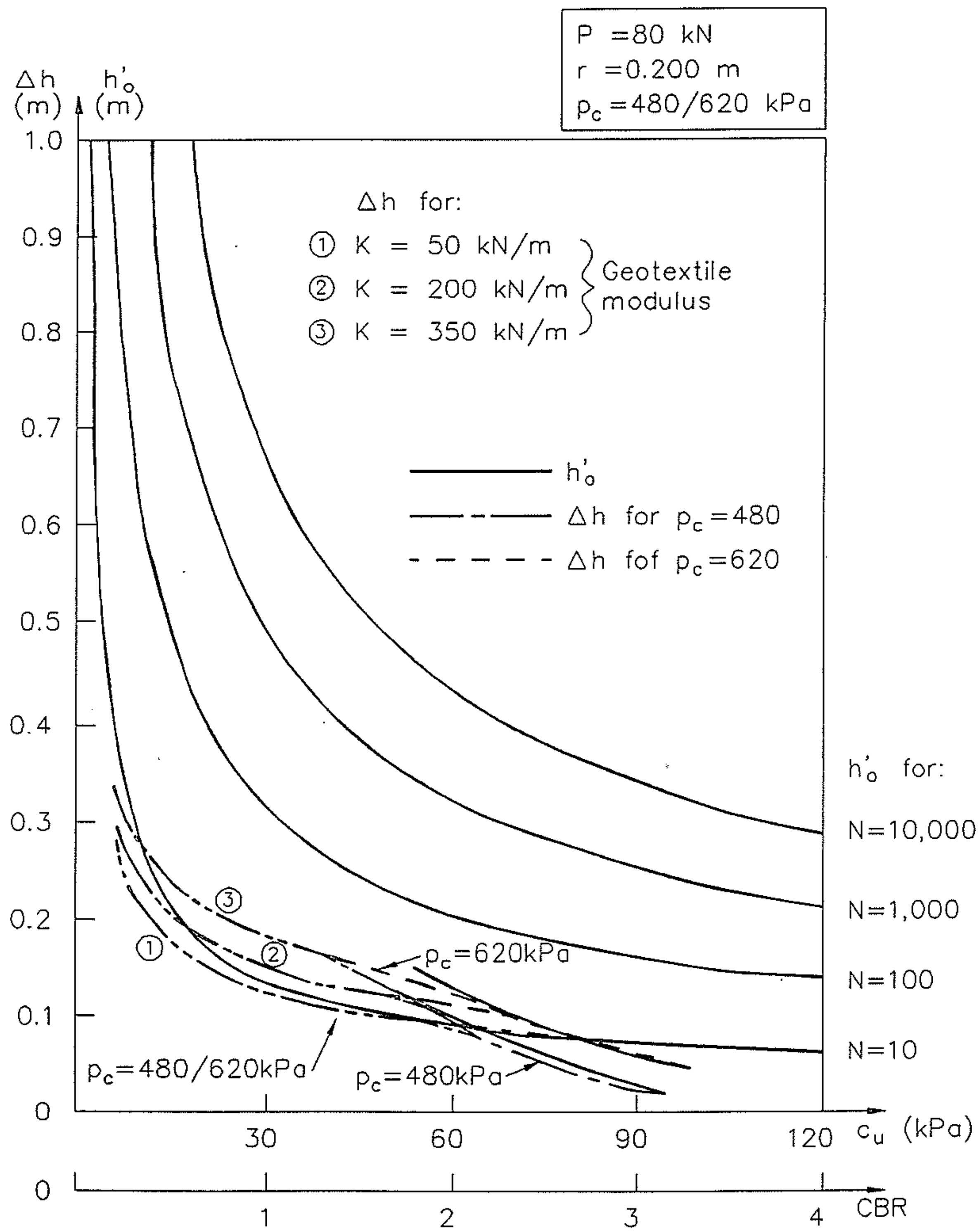


FIGURE 11.7
Aggregate Thickness h'$_o$ Without a Geotextile and Possible Reduction in Aggregate Thickness Δh Using a Geotextile. On-highway Truck. With Standard Axle Load. Rut Depth 200mm (Holtz and Sivakugen 1987)

For single axle trucks

$$N \text{ at } P_{80} = \frac{340 \times (50)^4}{(80)} \quad = \quad 52$$

$$\overline{342}$$

On Figure 11.6 for rut depth 300mm, P = 80 kN, design for number of passages N = 342.
Point A corresponding to CBR = 1 and N = 342 shows that the required thickness of aggregate layer when there is no geotextile would be 0.35m.
Point B corresponding to CBR = 1 and geotextile K = 90 kN/m shows that the reduction of thickness can be 0.15m.
Consequently the recommended thickness of aggregate is 0.20m.

Also Point B in Giroud and Noiray (1981) shows that the elongation (ϵ) of the geotextile is approximately 10%. The elongation factor of the chosen geotextile should be checked to ensure that it is much greater than this.

11.6.2 Bender and Barenberg Design Procedure

Bender and Barenberg (1978) developed a design procedure based on laboratory investigations and reported plate load tests. The design procedure was based on the use of Mirafi-140 geotextile. The results indicated that the geotextile inhibits subgrade strain so that a general shear failure rather than a local shear failure can occur. (Later research by Kinney and Barenberg (1982) led to a revision of this design procedure to include the membrane effect.)

Inspection of the curves on Figure 11.8 shows the savings when considering the geotextile only as a separator (shaded area) and as reinforcement.

11.6.3 Product-Specific Design Procedures

A number of the geotextile manufacturing companies have their own product-specific design procedures. These procedures are either primarily empirical or are based on unpublished tests and analyses (Christopher and Holtz 1985). Three of these procedures are commented on below.

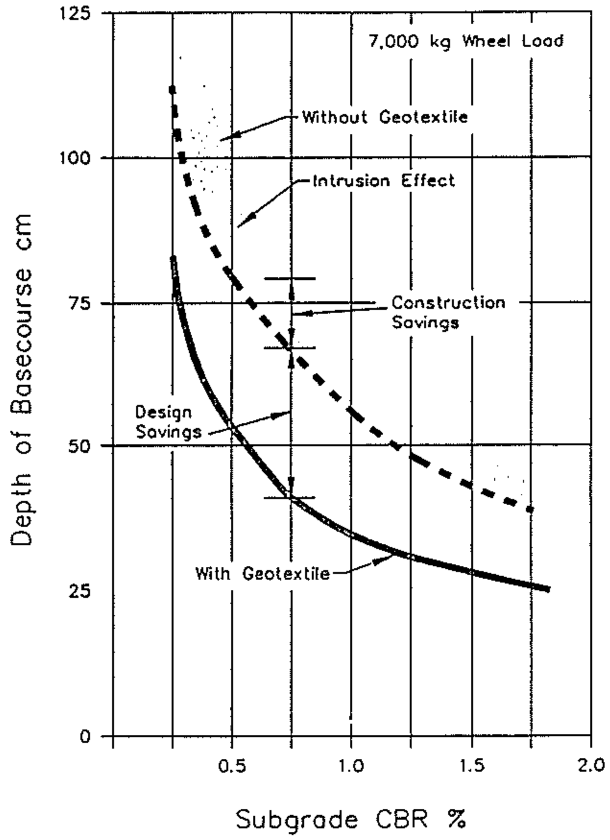


FIGURE 11.8
Example Thickness Design Curve for 67kN Wheel Load and 100 psi Tyre Contact Pressure (Bender and Barenberg 1978)

11.6.3.1 ICI Design Procedure
ICI of Great Britain has developed a procedure for the design of temporary access roads. The design procedure (ICI Fibres 1981) is based on laying a single layer of Terram 1000 on top of the subgrade. Required aggregate depths are given for various subgrade CBR's and axle loads. No account of number of vehicle passes is taken and limiting rut depths are not stated.

From the references given in their manual some of the technical back-up data seems to be from Barenberg's work. Christopher and Holtz (1985) indicate that the ICI method gives comparable results to the Bender and Barenberg (1978) results.

11.6.3.2 Du Pont Design Procedure
The Du Pont (1987) procedure for unpaved roads gives the option of two design methods:

(i) Their own design method, apparently based on the work of Robnett and Lai (1981), or

From the references given in their manual some of the technical back-up data seems to be from Barenberg's work. Christopher and Holtz (1985) indicate that the ICI method gives comparable results to the Bender and Barenberg (1978) results.

11.6.3.2 Du Pont Design Procedure

The Du Pont (1987) procedure for unpaved roads gives the option of two design methods:

(i) Their own design method, apparently based on the work of Robnett and Lai (1981), or

(ii) An approach using "local design" methods but empirically increasing the subgrade CBR by 3 points.

The Du Pont method (i) above makes allowances for subgrade CBR, axle loads, number of axle load applications and aggregate quality. The minimum recommended aggregate thickness is 150mm and Du Pont recommend filling the wheelpath ruts once they have formed.

The empirical increase of CBR (= actual CBR + 3) has received much criticism as it is empirically based on unrealistic laboratory models. This approach is especially suspect for higher strength soils (CBR > 3). In the lower CBR range the method will generally produce a design in the range of other methods (Christopher and Holtz 1985).

11.6.3.3 Chemie Linz Design Procedure

Chemie Linz (1986) of Austria has developed a design procedure for unpaved roads using the Polyfelt range of geotextiles. The basis of the design charts is not given apart from a minor reference to Comité Français des Géotextiles (1981). The method takes account of axle load, aggregate quality, subgrade CBR, number of axles and rut depth. A minimum aggregate cover depth of 400mm and shoulder width of 1.25mm + D are required for geotextile protection and anchorage respectively (Section 11.7.3). The method also gives guidance on geotextile selection considering construction stresses.

11.6.3.4 Milligan *et al.* Design Procedure

Recognising the limitations of the current design methods for reinforced unpaved roads, Milligan *et al.* (1989) have proposed a new approach as the outcome of an extensive research programme carried out at Oxford University. The research consisted of model studies of unreinforced and reinforced roads under steady and cyclic loadings, as well as a large strain finite-element analysis that considers static loading.

Milligan *et al.* (1989) propose moving away from the Giroud and Noiray concentration on the importance of the tensioned membrane effect (Section 11.5.2). The new ideas are briefly described as follows: when a vertical load is applied at the surface of the granular fill layer it causes high horizontal stresses, as well as vertical stresses, under the loaded area. The resulting horizontal thrust in the soil is partly resisted by horizontal stress in the fill outside the loaded area, but also results in outward shear stresses onto the surface of the clay below. The presence of such outward shear stresses reduces the appropriate bearing capacity factor for the clay, possibly to as little as one half of the value for purely vertical loading. If reinforcement is introduced these shear stresses are picked up by the reinforcement, which is put into tension, and purely vertical forces are transmitted to the clay below, allowing the full bearing capacity of the clay to be mobilised.

Such research helps to explain why reinforcement is able to improve road performance even at very small rut depths, as it does not depend on the concept of a curved tensioned membrane. For similar reasons the analysis is not restricted to the situation of channelled traffic. Since the tension in the reinforcement is seen as being primarily caused by outward shear stress, anchorage of the reinforcement is viewed as less important. Presumably however aggregate-geotextile friction becomes more critical to develop geotextile tension.

The axial stiffness, i.e. modulus, of the geotextile reinforcement also becomes an important consideration if large tensions are to develop at small rut depths.

The analysis demonstrates the benefits of reinforcement even at small surface deformations. This is argued as providing an explanation for observed improvements.

Currently Milligan and colleagues are seeking to verify and fine tune their approach against well documented case histories. They have not yet published design curves.

11.7 CONSTRUCTION ASPECTS

11.7.1 Survivability

The design methods described above do not cover all the requirements for successful use of geotextiles in temporary roads. A geotextile cannot perform any of its functions unless it survives its initial placement and covering. Resistance to damage during construction has been termed fabric **survivability** (Haliburton and Barron 1983) or **integrity** (Lawson and Curiskis 1985). It is related to the existing subgrade conditions, type of prior site preparation (if any), type, grading and angularity of cover aggregate, and type of equipment used for road construction.

Attempts have been made to relate survivability to standard fabric strength measurements such as those listed in Table 11.2 below (from Hausmann 1987):

Required Degree of Fabric Survivability	Minimum Grab Strength (N)	Puncture Strength (N)	Burst Strength (kPa)	Trapezoidal Tear Strength (N)
Very high	1200	490	2970	330
High	800	330	2000	220
Moderate	580	180	1450	180
Low	400	130	1000	130

TABLE 11.2
Minimum Fabric Properties Required for Fabric Survivability

Lawson and Curiskis (1985) have also commented on this. The properties they consider important for unpaved temporary roads are:

Property	Type	Proposed Test Method
Mechanical	Integrity – Tear Resistance – Puncture Resistance	 Trapezoidal Tear CBR Puncture
Hydraulic	Pore Size Permeability	Apparent Opening Size Permittivity

TABLE 11.3
Geotextile Properties of Importance for Unpaved Deformable Pavements

Lawson and Curiskis (1985) and ICI Fibres (1986) also present integrity requirements for geotextiles in terms of trapezoidal tear and CBR puncture resistance tests. Figure 12.1 enables the determination of these values, given a subgrade strength and a characteristic sub-base aggregate diameter (D85).

11.7.2 Overlap

Overlap can be used to provide continuity between adjacent fabric strips through frictional resistance between the overlaps. The amount of overlap depends primarily on the soil conditions and the likely rut depth. Since the rutting potential can be related to CBR it can be used as a guideline for minimum overlaps required, which are listed in Table 11.4.

Subgrade CBR	Minimum Overlap (mm)
Greater than 2	300 - 450
1 - 2	600 - 900
0.5 - 1	900 or sewn
Less than 0.5	sewn

TABLE 11.4
Recommended Minimum Overlap Requirements (Christopher and Holtz, 1985)

Where high settlements are expected (e.g. over peat), the overlap should be increased up to 1.5m (ICI Fibres 1986).

To avoid displacing the geotextile during sub-base spreading, the transverse laps should have the exposed free end pointing away from the advancing sub-base.

Koerner (1986) notes that the sewing of geotextiles has advanced rapidly to the point where it should be considered for all joints for fabric construction on soft ground (Figure 11.9). However, there are potential problems, e.g. dampness can cause thread or needle break, bobbins need covering, etc. (ICI Fibres 1988).

Geotextiles can be joined by stapling, using corrosion resistant staples which should be inserted with an industrial stapler (Figure 11.9). Stapled seam strengths are likely to be considerably lower than those achieved by sewing (ICI Fibres 1988).

The use of wider rolls of geotextile (e.g. 4m rather than 2m width) can reduce the need for joints, and so eliminate the risk of slippage at these joints.

11.7.3 Anchorage

To achieve design requirements for reinforcing effects, careful consideration of anchorage length is required. Anchorage slippage can be caused either by "pulling out" of the geotextile leaving the pavement aggregate behind, or by movement of the entire geotextile/basecourse segment.

Chemie Linz (1986) recommend a distance from the outside lane to the road edge of at least the aggregate depth + 1.25 metres (Figure 11.10). Other options to gain anchorage include turning the geotextile up into the aggregate, or down into an anchor trench.

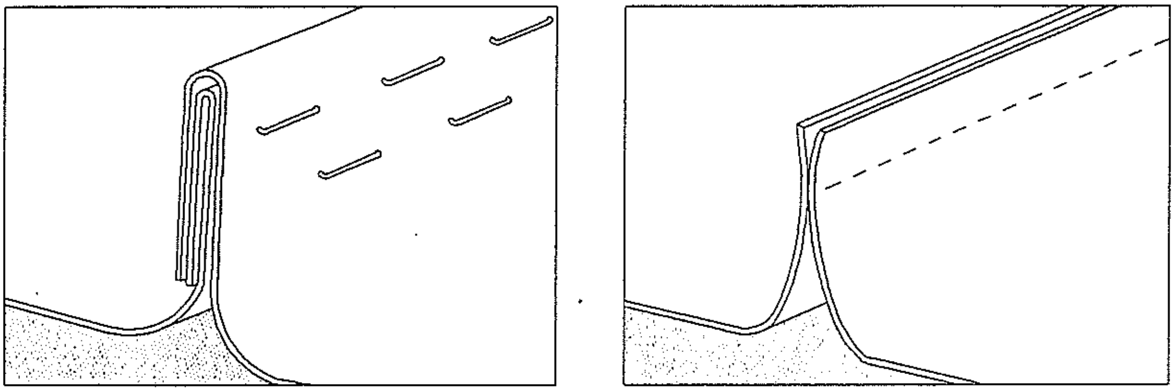


FIGURE 11.9
Stapled Seam and Sewn "Prayer Seam" (ICI Fibres 1988)

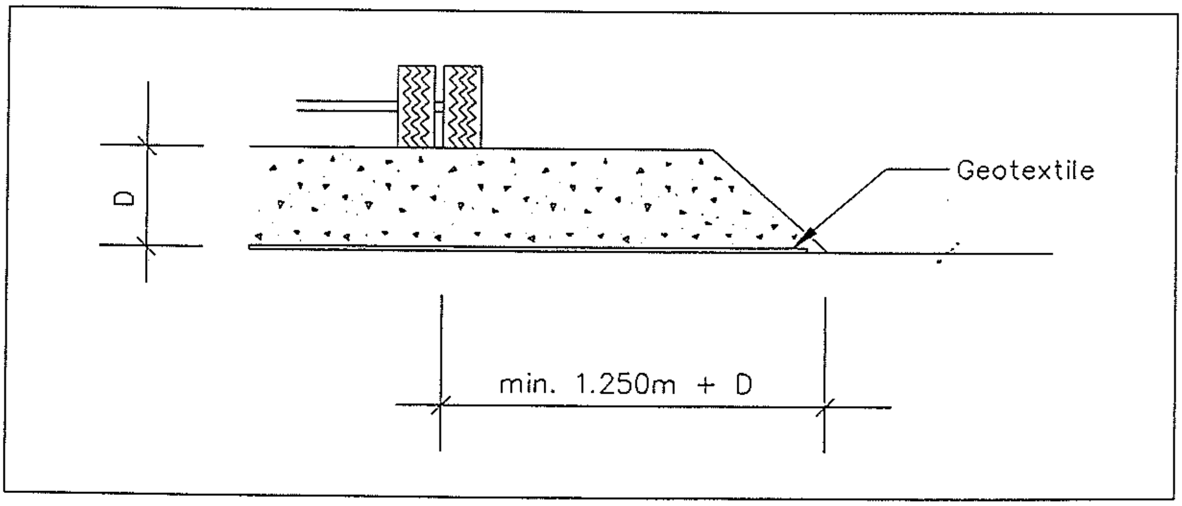


FIGURE 11.10
Anchorage Detail (Chemie Linz 1986)

11.8 REFERENCES

Barenberg, E.J. 1980. *Design procedures for soil-fabric-aggregate systems with Mirafi 500X fabric.* University of Illinois, Urbana-Champaign, USA.

Barenberg, E.J., Dowland Jr, J.H., Hales, J. 1975. *Evaluation of soil-aggregate systems with Mirafi fabric.* University of Illinois, Urbana-Champaign, USA.

Bender, D.A., Barenberg, E.J. 1978. Design and behaviour of soil-fabric-aggregate systems. *Transportation Research Record* 671: 64-75. Transportation Research Board, Washington DC.

Chemie Linz 1986. *Polyfelt TS. Design and practice.* A-4021, Chemie Linz AG, Austria.

Christopher, B.R., Holtz, R.D. 1985. *Geotextile engineering manual - course text.* National Highway Institute, Federal Highway Administration, Washington, DC.

Comité Français des Géotextiles 1981. Recommendations pour l'emploi des géotextiles dans les voies de circulation provisoire, les voies à faible traffic et les couches des forme. Document du Ministère des Transports, Direction des Routes et de la Circulation Routière.

Giroud, J.P., Noiray, L. 1981. Geotextile-reinforced unpaved road design. *ASCE Journal of Geotechnical Engineering* 107(GT9): 1233-1253.

Du Pont 1987. *"Typar". Designing aggregate bases with "Typar" spunbonded polypropylene.* Du Pont E81218.

Haliburton, T.A., Barron, J.V. 1983. Optimum depth method for design of fabric-reinforced unsurfaced roads. *Transportation Research Record* 916: 26-32. Transportation Research Board, Washington DC.

Hausmann, M.R. 1987. Geotextiles for unpaved roads - a review of design procedures. *Geotextiles and Geomembranes* 5(3): 201-233.

Holtz, R.D., Sivakugan, N. 1987. Design charts for roads with geotextiles. *Geotextiles and Geomembranes* 5(3): 191-199.

ICI Fibres 1981. *Designing with "Terram".* 2nd Edition. ICI Geotextiles, Pontypool, Wales.

ICI Fibres 1986. Geotextiles or geosynthetics? *Proceedings of Third Asian ICI Fibres Geotextiles Conference,* Thailand.

ICI Fibres 1988. *Designing for subsurface drainage.* ICI Geotextiles, Pontypool, Wales.

Ingold, T.S., Crowcroft, P.S. 1984. The notion of geotextiles as separators in roads. *Ground Engineering* 17(1): 27-34.

Karafiath, L.L. 1986. Trafficability of fabric reinforced haul roads. *Civil Engineering for Practising and Design Engineers* 5(5): 335-347.

Kinney, T.C., Barenberg, E.J. 1982. The strengthening effect of geotextiles on soil-geotextile-aggregate systems. *Proceedings of Second International Conference on Geotextiles,* Las Vegas, USA.

Koerner, R.M. 1986. *Designing with geosynthetics.* Prentice-Hall, Englewood Cliffs, NJ.

Lawson, C.R., Curiskis, J.I. 1985. *Geotextiles : including manufacture, properties, and use in pavements, embankments, subsurface drainage and erosion control.* Lawson Consulting, Sydney, Australia.

Lawson, C.R. 1988. Personal Communication. Exxon Chemical Geopolymers Limited, Pontypool, Wales.

Millar, P.J. 1986. Assessment of geotextiles, physical properties and performance tests. *Ministry of Works and Development, New Zealand. Central Laboratories Report* 2-86/11.

Milligan, G.W.E., Jewell, R.A., Houlsby, G.T., Burd, H.J. 1989. A new approach to the design of unpaved roads - Part 1. *Ground Engineering* 22(3): 25-29.

Raumann, G. 1982. Geotextiles in unpaved roads : design considerations. *Proceedings of Second International Conference on Geotextiles*, Las Vegas, USA.

Robnett, Q.L., Lai, J.S. 1982. Fabric reinforced aggregate roads - overview. *Transportation Research Record* 875: 42-50.

Sellmeijer, J.B., Kenter, C.J., Van der Berg, C. 1982. Calculation method for a fabric reinforced road. *Proceedings of Second International Conference on Geotextiles,* Las Vegas, USA.

Shrestha, S.C., Bell, J.R. 1980. Creep behaviour of geotextiles under sustained loads. *Proceedings of Second International Conference on Geotextiles*, Las Vegas, USA.

Steward, J., Williams, R., Mohney, J. 1977. *Guidelines for use of fabrics in construction and maintenance of low-volume roads.* USDA Forest Service, Oregon, USA.

Transit New Zealand (NRB) 1987. *State highway pavement design and rehabilitation manual.*

CHAPTER 12

PERMANENT ROADS

12.1 GENERAL COMMENTS

Unbound pavements of permanent roads are normally designed to limit compressive strains on the subgrade. Any bound layers have an additional requirement of limiting tensile strains. Permanent pavements are characterised by low or zero plastic deformations (e.g. wheel track rutting) and low to medium allowable elastic deformations (e.g. deflections). These limitations on plastic and elastic deformations are necessary to prevent over-straining of the subgrade and any bound layers, and/or the surfacing.

Permanent pavements cover a wide spectrum from unsealed gravel roads, to chip sealed, asphaltic concrete or portland cement concrete surfaced roads. While slightly greater deflections and rutting may be more tolerable in unsealed pavements (see Section 12.7) than in sealed, all permanent pavements are characterised by low plastic and elastic deformations.

12.2 REINFORCEMENT

12.2.1 Pavement Thickness Reduction

As a result of these low plastic and elastic deformations, any geotextile placed at the sub-base/subgrade interface will not be deformed sufficiently to tension it, so **the geotextile will not fulfil a reinforcing function**. This has been substantiated by Brown *et al.* (1982), and Halliday and Potter (1984), for weak and medium strength geotextiles on subgrades with CBR ranging from 1 to 4.5.

Brown *et al.* (1982) found that deformations on the pavement surface were slightly greater on some of the sections with a geotextile on top of the subgrade, than on the otherwise identical control sections. This suggests some slippage occurred between the geotextile and the adjacent granular material (see Section 12.2.2).

Theoretically a high strength, high modulus, low creep geotextile placed between the basecourse and the subgrade, and pretensioned, can provide reinforcement in permanent roads and allow a reduction in pavement thickness. To date the costs or practicalities of this prestressing, and the likelihood of loss of prestress from creep or stress relaxation, have limited the development of this technique with little work being done to quantify any benefits.

12.2.2 Basecourse Shear Prevention

Examination of Koerner's (1986) work suggests that a geotextile with a high modulus in the grab tensile test will assist in retaining the basecourse integrity under wheel loads, if the basecourse does not slip on the geotextile. The geotextile in this case provides localised reinforcement to the basecourse so restraining movement of the larger basecourse particles, and resisting shallow shear in the basecourse layer itself. While this may not permit any reduction in pavement thickness it may allow either a slightly **lower quality basecourse** to be used, or enable a basecourse to perform at a slightly **higher moisture content.**

Brown *et al.* (1982) also used a geotextile within the basecourse layer itself. This resulted in increased permanent pavement strains, again suggesting that some slip occurred between the geotextile and the basecourse. Based on this, and shear-box and pull-out tests, they noted the necessity to match aggregate particle size and geotextile pore size to increase frictional and interlock forces.

Recent work at the University of Nottingham test track has indicated that inclusion of a geogrid within the basecourse layer, at mid-depth of the basecourse, can also reduce shear within the basecourse (Brown 1988). The geogrid function in this case is reinforcement, which inhibits shallow shear in the basecourse layer. Raymond and Hayden (1983) found the optimum depth for a geogrid in sand to be in the range of 0.3 to 0.6 times the tyre contact width. Taking the width of a typical dual wheel contact area as 0.3 metres gives the optimum depth for placing the geogrid as 100 to 200mm in sand.

12.3 SEPARATION AND FILTRATION

The main benefit from using a geotextile at the subgrade/sub-base interface is separation to prevent subgrade intrusion or subgrade/sub-base mixing.

The most obvious benefit of this separation is during pavement construction where the engineer is faced with poor ground conditions and places a geotextile on top of the subgrade. The immediate economic return from the use of the geotextile is in the physical ability to get aggregate onto the site without losing it into the subgrade, i.e. as a construction expedient.

The long-term benefits of separation can also be very worthwhile, particularly on soft wet subgrades. In permanent roads this ability to prevent subgrade/sub-base mixing (subgrade pumping or intrusion) can have a variety of benefits:

- It prevents or reduces contamination of the sub-base or basecourse, and thus avoids loss of strength and/or permeability.

- It avoids or reduces the need to have an additional "sacrificial" thickness of sub-base (i.e. the portion that will become contaminated), and thus allow use of thinner granular pavement layers.

- It allows the use of a more open graded sub-base which is be more permeable, allowing improved drainage of the pavement layers.

Subgrade/sub-base mixing is only likely to be a problem with soft wet subgrades. As covered in Chapter 9, it is only likely to be a problem for wet subgrades with in situ CBR <5. Also a number of approaches to solving the problem are available, including lime and/or cement stabilisation, and improved subgrade drainage.

Any geotextile that will survive construction stresses could be used as a separator. However, as noted in Chapter 9, the geotextile should also be a filter and retain the subgrade soil particles, yet allow water from the subgrade to pass into the permeable sub-base and away, so helping to prevent a build-up of subgrade pore pressures with corresponding loss of subgrade strength. Thus it is preferable for the separating geotextile to be designed for filtration (i.e. permeability and pore size) in accordance with the criteria for one-directional flow given in Chapter 8. As explained in Chapter 9 these criteria are aimed to be a balance between allowing pore pressure dissipation and inhibiting migration of subgrade fines up into the sub-base.

Large elongation to failure is also a desirable geotextile property in cases of likely subgrade or foundation deformation or settlement.

Therefore the geotextile properties desirable for separation of sub-base and subgrade on permanent roads are, in approximate descending order of importance:

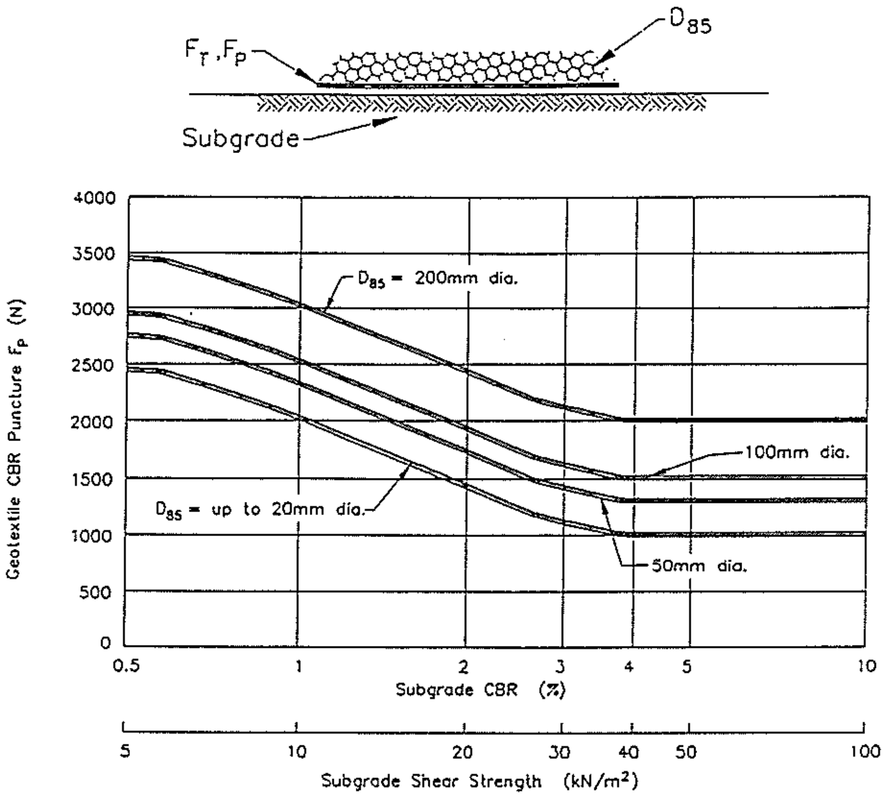
Mechanical	-	Tear resistance
Mechanical	-	Puncture resistance
Hydraulic	-	Permeability and pore size
Mechanical	-	Aggregate/geotextile friction or interlock
Mechanical	-	Large elongation to break
Mechanical	-	High modulus in grab tensile test

To some extent these are conflicting requirements suggesting that the ideal separating fabric would be a composite of two, or more, different geotextiles.

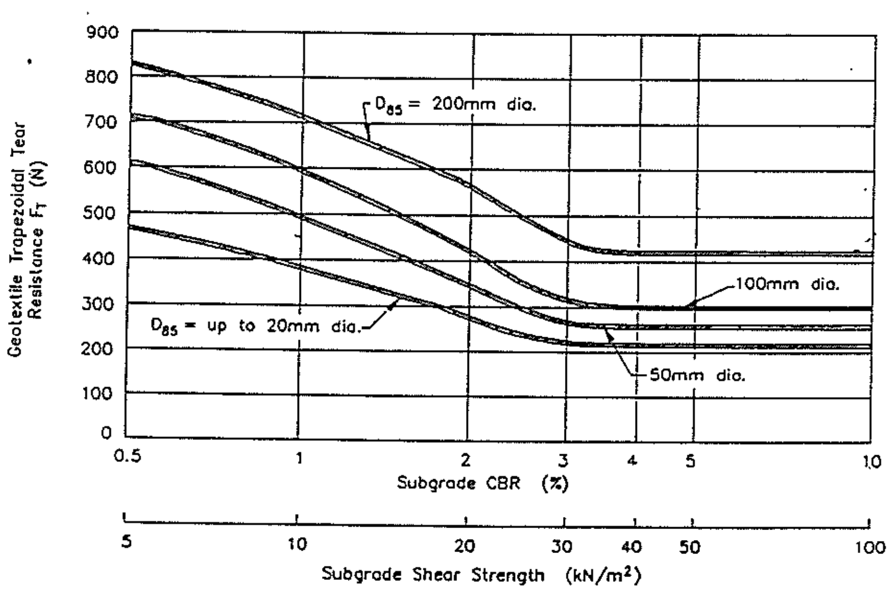
Geotextile integrity (tear resistance and puncture resistance) is a critical factor as the geotextile has to survive during installation and throughout the life of the pavement. Figure 12.1 indicates the required minimum mechanical integrity requirements (CBR Puncture and Trapezoidal Tear resistance) of a geotextile used for separation (ICI Fibres 1986). See also Chapter 11.

12.4 DIG-OUTS

Geotextiles have been used at the bottom of dig-out reinstatements in flexible pavements. The benefits of this use come mostly from separation (preventing loss of sub-base aggregate and preventing subgrade intrusion). Some benefits may also come from filtration which allows dissipation of subgrade pore pressures, if the water that finds its



(a) CBR Puncture Requirement



(b) Trapezoidal Tear Resistance Requirement

FIGURE 12.1
Mechanical Integrity Requirements for Geotextiles Used at the Subgrade-Basecourse Interface for Various Maximum Sub-base Aggregate Sizes (ICI Fibres 1986)

way into the sub-base can drain away. While no reduction in structural pavement thickness will result from use of a geotextile under dig-outs, they may encourage more durable dig-out reinstatements and may well be worthwhile over soft wet areas of subgrade. Geotextile selection is described in Section 12.3.

12.5 COMPACTION

Brown *et al.* (1982) on subgrades with CBR from 2 to 5, Potter and Currer (1981) and Halliday and Potter (1984) on subgrades of CBR 1 to 4.5, all placed a geotextile layer over the subgrade, then constructed unbound basecourse pavements on it. The pavements were typically surfaced with a bituminous mix. While the geotextile facilitated construction by preventing basecourse aggregate loss and subgrade intrusion, the presence of the geotextile did **not result in any greater compaction (i.e. density) of either the basecourse or the bituminous surfacing mix** in comparison to the control sections without geotextile.

12.6 RIGID PAVEMENTS

Rigid pavements constructed of portland-cement concrete are prone to pumping. Pumping can create voids beneath the slab near the joints. If drainage is poor, these voids tend to fill with water which is pumped out as the slab deflects under passing vehicles. This pumping tends to be worst under rigid pavements where load transfer across joints is poor, sub-base drainage is poor and the road carries many heavy vehicles. Pumping tends to pump subgrade and particularly sub-base fines up through the joint in the slab, to create larger voids which in turn increase the amount of pumping.

A geotextile at the subgrade/sub-base interface can be used to reduce this loss of subgrade fines by pumping. The geotextile filtration mechanism is for alternating or turbulent flows which are discussed in detail in Chapter 9. However in these situations, loss of subgrade fines may be a small part of the problem, as most if not all the fines lost may be from the sub-base.

Placing a geotextile directly between the concrete slab and the sub-base to retain the sub-base fines is likely to expose the geotextile to severe abrasion. The concrete may tend to adhere to the geotextile and cyclically drag the bottom of the geotextile across the sub-base surface as temperature changes causes the slab to expand and contract; this will tend to abrade the geotextile. Where the geotextile passes under the joints in the concrete the geotextile will also be exposed to abrasion, and cyclical stretching.

12.7 UNSEALED ROADS

It could be argued that gravel-surfaced roads can tolerate rutting and deflections as merely a pass of the grader or the addition of basecourse will rectify them. Design for such roads that will carry more than 10,000 vehicles over the life of the pavement is beyond the current state of knowledge if a reduced pavement thickness obtained by geotextile reinforcing is required. This suggests it may be unwise, at least at present, to design for reduced pavement thickness because geotextile reinforcing is being used in these permanent unsealed roads. However, the benefits from separation may themselves justify the use of a geotextile.

12.8 REFERENCES:

This list includes additional reading.

Brown S.F. 1988. Personal communication.

Brown, S.F., Jones, C.P.D., Brodrick, B.V. 1982. Use of non-woven fabrics in permanent road pavements. *Proceedings of Institution of Civil Engineers* 73 Part 2): 541-563.

Christopher, B.R., Holtz, R.D. 1985. *Geotextile engineering manual - course text.* National Highway Institute, Federal Highway Administration, Washington, DC.

Department of Main Roads 1983. *Evaluation of geotextiles.* Main Roads Department, Queensland, Australia.

Halliday, A.R., Potter, J.F. 1984. The performance of a flexible pavement constructed on a strong fabric. *Transport and Road Research Laboratory (TRRL), Laboratory Report* LR 1123. 15p.

ICI Fibres 1986. Geotextiles or geosynthetics? *Proceedings of Third Asian ICI Fibres Geotextiles Conference,* Thailand.

Koerner, R.M. 1986. *Designing with geosynthetics.* Prentice-Hall, Englewood Cliffs, NJ.

Potter, J.F., Currer, E.W.H. 1981. The effect of a fabric membrane on the structural behaviour of a granular road pavement. *Transport and Road Research Laboratory (TRRL), Laboratory Report* LR 996. 17p.

Raymond, G.P., Hayden, F.B. 1983. Effect of reinforcement on sand overlying bases of different compressibilities subject to repeated loading. *Research Report* 79, Queens University, Ontario, Canada.

Ruddock, E.C. 1983. Fabrics and meshes in roads and other pavements: a state-of-the-art review. *Technical Note* 87, CIRIA.

Waters, T.J. 1984. Evaluation of geotextiles for drainage and reinforcement applications. *Proceedings of Australian Road Research Board Conference* 12(2).

CHAPTER 13

REFLECTIVE CRACKING

13.1 CRACKING IN EXISTING PAVEMENTS

It is common to see cracks in asphaltic concrete, cement stabilised, portland cement concrete, and basecourse/chip seal pavements. The mere presence of cracks is not necessarily detrimental to the performance of the pavement, except in portland cement concrete and structural asphaltic concrete where cracks can cause loss of slab action or load transfer. Only when water and repeated applications of heavy loads are superimposed on the cracked pavements do problems usually occur. This entry of water through cracks can lead to a loss of pavement base and subgrade strength, and pumping of base material through the cracks: these in turn lead to further distress with trafficking.

As a generalisation the types of movement across these cracks can be:

A — horizontal caused by temperature fluctuations. These cause opening and closing of the crack over a 24 hour period, plus those caused by seasonal influences.

B — vertical differential caused by heavy axles forcing down first one side, then the other side of the crack.

C — vertical, caused by heavy axles tending to force the entire cracked area down (typical of crocodile-cracked or extensively fatigue-cracked areas).

Cracks can develop for a number of reasons and the four pavement types listed are the most common. The most likely type(s) of crack movement for each of the crack types is indicated by the A, B, or C reference after each description.

Asphaltic concrete pavements:

• "Cold" joints from lack of bond between paver runs during construction	A
• Shrinkage cracks from ageing of the bitumen	A
• Crocodile cracking from fatigue of the asphaltic concrete caused by inadequate pavement strength/rigidity, and/or substandard asphaltic concrete (e.g. high air voids), possibly combined with heavy traffic loadings	C

Cement-stabilised pavements:

• Block cracking caused by shrinkage of the concrete and by thermal movements	A, B
• Fatigue cracking indicating insufficient sub-base or subgrade strength, or excessive heavy traffic causing fatigue cracking of the cemented layer	C

Portland cement concrete pavements:

• Cracking caused by shrinkage of the concrete	B
• Cracking caused by thermal movements	A
• Traffic induced fatigue cracking	C

Unbound basecourse/chip sealed pavements:

• Fatigue cracking from inadequate pavement strength/rigidity combined with ageing of the bituminous binder	C

For all these pavements, the type of cracking, its cause, and the nature and magnitude of movement at the crack need to be considered before a decision is made to include a geotextile with any bituminous overlay.

13.2 OVERLAYS

One common technique for dealing with cracking in an existing pavement is to overlay it with a bituminous mix such as asphaltic concrete (asphalt), but there is a strong tendency for the underlying cracks to reflect through this overlay. Geotextiles have been used extensively in the USA in conjunction with an asphalt overlay to try to prevent or slow this reflective cracking. This is reported to be the single largest use for geotextiles in the United States. For example, Vicelja (1989) of California notes a personal involvement in over 1000 such projects since in 1972.

The likelihood of success when using a geotextile in the overlay to prevent or slow reflective cracking is related to the type of movement of the cracks in the existing pavement. If only horizontal movement (A above) occurs there is a reasonably good chance of success. If horizontal and vertical movement (type A and B together) occurs, there is a poor chance of success. If type C movement occurs there is generally inadequate strength in the existing pavement and a relatively thin overlay even with a geotextile may have only limited success.

While at least two theoretical procedures exist (Majizadeh *et al.* 1982, 1987) to predict the performance of geotextile-reinforced overlays, verification of these procedures has been by laboratory work with little or no field trials. While many successful uses of geotextiles in overlays have been reported, there have also been a significant number of less than successful uses. There is some evidence from American experience that the technique is more likely to be successful in warmer climates than in colder (frosty) climates (Bryan 1986). It appears that the mechanism by which the geotextile benefits (or otherwise) an overlay is complex.

The reason for the original cracking, the type of movement across the crack, the bond between the overlay and the existing pavement, the thickness of the overlay, the amount and rate of any temperature changes, the slab length, gauge length across the joint or crack, and properties of the overlay material are all contributing factors to the development of reflective cracking (Lorenz 1987).

To illustrate some of the experiences recorded to date, a number of case studies are described in Section 13.12. These case studies have also provided some of the background information for the conclusions given below.

13.3 CONCLUSIONS FROM CASE HISTORIES

The main conclusions from the case histories (Section 13.12) are that the advantages of using geotextile interlayers are variable, ranging from no apparent benefits to clear benefits, and that, **even with a geotextile interlayer, cracking will still reflect through the overlay but in most cases at a slower rate than if no interlayer had been used.**

The variable results from the use of geotextiles in the case histories indicates that the actual mechanisms by which the interlayer performs (or not) is complex and not yet fully understood. Even if the geotextile itself is adequately understood the behaviour of the many types of pavement cracks is not. Because of this apparent lack of understanding, definite detailed design rules cannot be given with confidence. However it appears that the following conclusions can be drawn:

- Geotextile/bitumen interlayers do **NOT** stop reflective cracking occurring.
- Generally, geotextile/bitumen interlayers do retard the rate of appearance of reflective cracking. A typical trend is shown graphically in Figure 13.1.
- In many of the trials, sections with geotextile interlayer performed better than control sections, but in some cases the reverse was true or there was no apparent difference.
- For geotextile/bitumen interlayers, no consistent difference in performance between needle-punched and melt-bonded non-woven geotextiles is apparent.
- It is unusual to use woven geotextiles for these interlayers.

- Types of interlayers other than geotextile/bitumen ones may, in some cases at least, perform better. For example a 100mm or 150mm thick overlay of unbound basecourse can be very effective in retarding reflective cracking; in other cases Bituthene has performed well.

- The geotextile/bitumen interlayer remains intact and waterproof in most cases even after the original crack reflected through.

- Preparation of the existing pavement and installation procedures are critical to the success of the geotextile/bitumen interlayer treatment. In particular, differential vertical movements across cracks should be less than 0.05mm, and preferably zero.

- Reflective cracking with a geotextile/bitumen interlayer may well appear offset a short distance horizontally from the original crack. This offset cracking may appear as a number of smaller cracks rather than one larger crack.

- Use of a thick compressible geotextile in the interlayer may itself cause premature fatigue cracking of the asphaltic concrete overlay (Section 13.8).

- The minimum practical thickness of asphaltic concrete laid over a geotextile/bitumen interlayer is about 50mm. The thicker the asphaltic concrete overlay, the longer the time until the appearance of reflective cracking. (Overlays thinner than 50mm can be used but they are more likely to crack early or have sections that debond.)

- Use of a polymer-modified bitumen and geotextile to form the interlayer can be more successful than an unmodified bitumen and geotextile.

- 450mm wide geotextile/bitumen bandages over existing cracks can be effective, but generally not as effective as full area treatments.

- Results from trials have been somewhat variable suggesting that factors such as pavement and pavement crack type, traffic, weather conditions, and frost penetration, affected the performance of the trials.

In short, the use of geotextile/bitumen interlayers in many cases can slow the time until appearance of reflective cracks, **but the main benefit may well be from waterproofing the pavement after the reflective cracking has appeared.** The problem is that since the geotextile is some way below the surface of the crack it may be difficult to tell when it ceases to be waterproof. Also Vicelja (1989) has experienced problems, after reflective cracking has appeared, with waterproof interlayers causing debonding of the asphalt under traffic and requiring a maintenance chip seal or crack sealing.

Relating the performance of geotextiles to the three main types of crack movement identified in Section 13.1 (A, horizontal; B, differential vertical; C, vertical), geotextiles tend not to be effective on pavements with any (maximum 0.05mm) differential vertical movement (B) across the cracks. The most common use in the USA has been over shrinkage/thermal cracks with horizontal movements (A) and mostly geotextile/bitumen interlayers are beneficial in this situation. For overlays over areas of fatigue cracking (type C), geotextile/bitumen interlayers may provide some benefit but are not really a cure for lack of pavement structural strength.

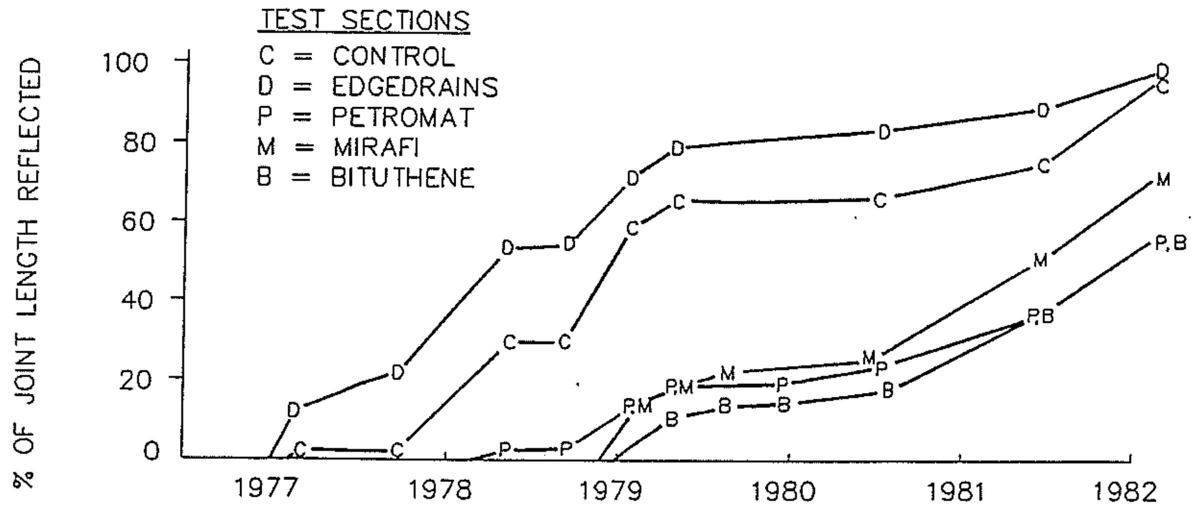


FIGURE 13.1
Example of Crack Reflection Through 100mm AC Overlay using Various Interlayers
(Gulden and Brown 1983)

13.4 LABORATORY TESTS

Many researchers have carried out laboratory tests simulating fatigue of overlays over cracks, and the effects of geotextile/bitumen interlayers. Button *et al.* (1982) found that there appeared to be some relationship between fatigue performance and the fabrics ability to retain bitumen, and also with the fabric-surface texture. Thin "slick" fabrics, not able to retain an appreciable quantity of bitumen, exhibited relatively poor fatigue performance, while thick "fuzzy" fabrics exhibited relatively good fatigue performance. They also found quite a variation in the linear shrinkage of the different types of geotextiles when immersed in hot bitumen.

Eckmann (1989) at Liège University carried out flexural fatigue tests on asphalt beams laid on an interlayer laid on plywood with a crack initiator at mid span. Different membrane interlayer systems were tested including 80/100 penetration bitumen, bitumen with "short" fibres included, bitumen with "long" fibres included, bitumen plus non-woven Typar geotextile, bitumen plus non-woven PGM 14 geotextile. Other tests included the fibres, or the geotextile, with polymer-modified bitumen. Initial findings were that inclusion of fibres or a geotextile does retard crack initiation and lowers speed of crack propagation. The type of geotextile used makes a significant difference. Polymer-modified bitumen in conjunction with fibres or geotextile gives more benefits than straight-run bitumen plus a geotextile.

13.5 WATERPROOFING

As previously noted (Section 13.3), one of the recognised benefits of using bitumen-impregnated geotextiles in overlays is the waterproofing which continues after the overlay cracks. The benefits from the waterproofing function of the geotextile after the overlay cracks are indirect, i.e., water is prevented from weakening the basecourse, sub-base, subgrade, etc.

For overlays on portland cement concrete pavements the elimination or reduction of surface water available under the slab should reduce pumping and faulting, and increase the strength of the subgrade.

For overlays over asphaltic concrete pavements, the elimination or reduction of the surface water available should increase the strength of base and subgrade materials leading to reduced deflections, and hence to reduced fatigue of the asphalt layers. This in turn can lead to thinner structural overlay requirements, or a longer life for any given overlay thickness.

For overlays over fatigue-cracked chip seals or fatigue-cracked asphaltic concrete, waterproofing should increase the strength of the base and subgrade materials leading to reduced rutting and shear of these layers, and to reduced deflections and hence fatigue of the overlay.

Waterproofing can be beneficial in areas of frost action.

However, the use of a geotextile or other surfacing layer to limit the penetration of surface water should not be a substitute for good drainage design (NCHRP 1983).

13.6 GEOTEXTILE FUNCTIONS

The means by which a geotextile influences the performance of an overlay is a complex mechanism. It can be regarded as occurring in two stages. First, the appearance of reflective cracking is generally slowed. The mechanism for this appears to include some form of stress absorption (and maybe crack re-direction) in the vicinity of the original cracks, probably related to the bitumen impregnation. Second, once a crack appears it remains waterproof (at least for some extended period), presumably because the bitumen saturates the geotextile.

Section 13.1 considered the different types of movement of cracks in the existing pavement. Consideration of the mechanisms by which a crack propagates through an overlay over an existing cracked pavement, will assist in the understanding of how geotextile interlayers may function. For cracks with only temperature-induced horizontal movement (type A, Section 13.1) the cracks propagate upwards from the existing cracked pavement (Figure 13.2). The stresses are fatigue stresses. Because a thicker overlay insulates the existing road, these thermally induced fatigue stresses are likely to be most significant with overlays of less than 100mm. Reducing adhesion between the existing pavement and the overlay by a geotextile/bitumen interlayer has been shown to be effective in reducing these stresses (Nunn 1989, Foulkes 1989). This suggests the geotextile provides a benefit that arises from the **stress absorbing function of the interlayer** which presumably **relates mainly to the retained bitumen**. If cracking does occur the geotextile will offer waterproofing.

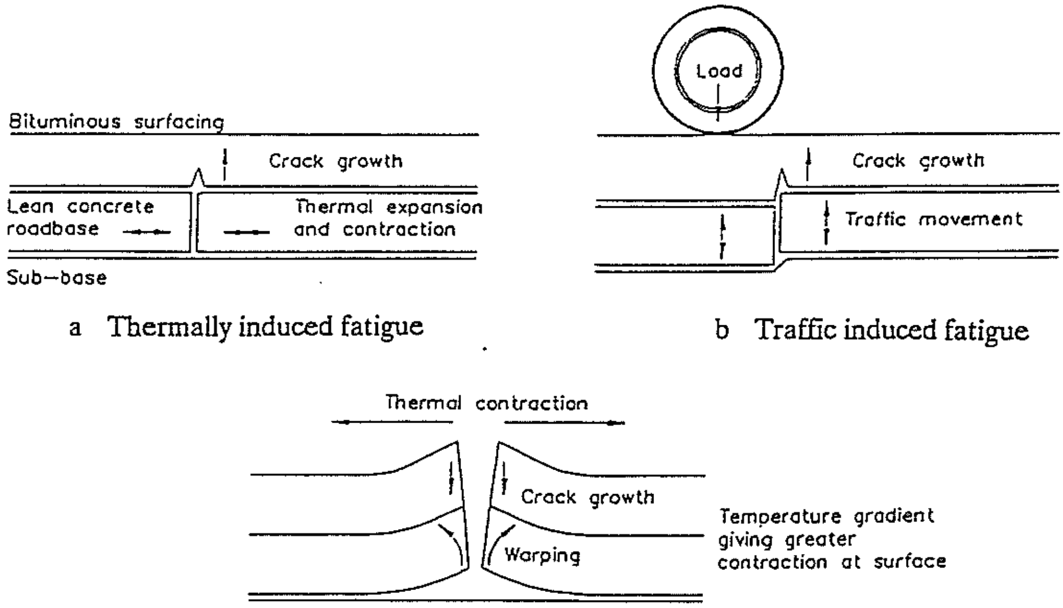
For cracks with differential vertical movement (type B, Section 13.1) traffic-induced shear stresses cause rapid fatigue cracking of the asphalt overlay (Figure 13.2).

Temperature movements of the existing cracked pavement also occur, adding their tensile fatigue stresses to the asphalt overlay. So in practice it is rare to have cracks with only type B (vertical differential) movement, and it is more usual to have a combination of types A and B (horizontal and vertical differential) (Nunn 1989, Foulkes 1989). A bitumen-saturated geotextile interlayer can act by absorbing the horizontal stresses (type A movement) but is unlikely to influence the vertical differential shear stresses (type B). This suggests the geotextile will be only of limited benefit, that arises from the **stress absorbing function of the interlayer** which presumably **relates mainly to the retained bitumen.** If cracking does occur the geotextile will offer waterproofing.

Nunn (1989) and Foulkes (1989) also considered a further type of cracking: surface-induced cracking for asphalt overlays over existing cracked concrete pavements. Cracks starting at the surface of the asphalt overlay can be caused by a combination of thermal contraction and warping of the pavement. The thermal contraction is caused by a cold pavement in winter conditions when the upper pavement layers are at a lower temperature than the underlying layers and the consequent differential contraction causes the existing concrete pavement to warp. (It is not known if this also applies to a lesser extent over existing cracked asphalt pavements.) Since these cracks start from the surface a geotextile/bitumen interlayer is unlikely to be of much benefit, except possibly to dissipate the differential thermal movements (Chemie Linz 1988) and to offer waterproofing after cracking (Figure 13.2).

The fourth type of cracking of the overlay occurs over existing pavements with extensive fatigue cracking (type C). The type of cracking of the existing pavement suggests inadequate pavement strength for the loads carried, which has led to excessive deflections and fatigue cracking. A thin asphalt overlay is itself unlikely to strengthen and stiffen the pavement, so it also will be prone to fatigue cracking which probably will start from the bottom of the overlay. The geotextile/bitumen interlayer will **waterproof and so strengthen** a cracked pavement by enabling underlying pavement layers to dry. This itself can lead to reduced pavement deflections and correspondingly increased overlay life before reflective cracking appears. Thick asphalt overlays will themselves strengthen a pavement and reduce deflections and hence fatigue cracking. If any cracking should occur, the geotextile/bitumen interlayer will provide waterproofing.

As noted in Section 13.7.2, excess tack coat bitumen on the geotextile can penetrate the overlaying asphalt mix. In addition to any stress absorption around the geotextile, this excess bitumen can increase the fatigue resistance of the overlay, so retarding reflective cracking.



a Thermally induced fatigue

b Traffic induced fatigue

c Surface initiated cracking

FIGURE 13.2
Mechanisms of Reflective Cracking (Nunn 1989)

The tensile strength and modulus of the geotextile appear not to affect the performance of the interlayer/overlay system. This is evidenced by the similar performances in the trials quoted in Section 13.12 of various types of geotextile. It is further evidenced by the ability of the glass fibre bandages (Section 13.12.3) to perform even if the width of bandage is too narrow to ensure anchorage for development of its full tensile strength.

In mechanical terms geotextiles with a modulus of elasticity of, at best, 5 GPa are not likely to reinforce bituminous mixes having a modulus of the order of 40 GPa. To reinforce in the structural sense the reinforcement should sustain the load at least as soon as, and preferably before, the material which it is reinforcing. Since the modulus of geotextiles is an order of magnitude less than asphaltic concrete it seems that reinforcing is, at most, a minor role in reflective crack prevention (Perfetti and Sangster 1988). Once the overlay has cracked then the reinforcing function of the geotextile may well limit the amount the cracks open hand; this can then lead to a number of smaller cracks appearing rather than one large one.

It thus appears that the prime benefits of geotextile interlayers arise from the geotextile's ability to retain bitumen.

13.7 TACK COAT

13.7.1 Tack Coat Application Rate

The general quantity of the tack coat can be calculated by using the equation (Button *et al.* 1982):

$$Qd = 0.36 + Qs \pm Qc \qquad \ell/m^2 \text{ residual bitumen where:}$$

Qd = design tack coat quantity
Qs = fabric bitumen saturation content
Qc = correction to tack coat quantity based on bitumen demand/surplus of the old surface.

The 0.36 ℓ/m^2 is based on usual field practice for overlays with no fabric.

Qc can range from +0.15 for a very coarse, existing surface to -0.15 for a very smooth, dense, bitumen-rich existing surface.

Therefore for example:

$$Qd = 0.36 + 0.9 \pm (0 \text{ to } 0.15) \ \ell/m^2 \text{ residual bitumen}$$

The test for Qs is Task Force 25 Method 8 (Section 13.8) for which 100mm by 200mm samples of geotextile are submerged for 30 minutes in asphaltic cement (the bituminous binder as specified for the job) maintained at 135°C. The samples are then hung to drain for 30 minutes at 135°C. Bitumen content is determined by weight.

The Task Force 25 method measures the bitumen content with the uncompressed geotextile. This suggests that the method and above formula tends to over-estimate the tack coat requirements (indicated by excess bitumen), particularly for a geotextile in a chip seal (Section 14.1.4).

An alternative test for Qs is that used by Button *et al.* (1982). This method involves the saturation of a piece of fabric in AC-10 asphaltic cement at 121°C for 1 minute. The saturated fabric is allowed to cool and is then pressed with a hot iron between two absorbent papers to remove the excess bitumen.

13.7.2 Interface Shear Strength

For a pavement performance, it is important to have sufficient shear strength at the interface between the old pavement and the new overlay to prevent traffic-load induced slippage failures, yet simultaneously allow some horizontal movement caused by temperature effects. The magnitude of the required shear strength is dependant on the traffic - imposed braking, turning and accelerating stresses, the ambient temperature, and the thickness of the overlay over the geotextile. The thinner the overlay the more critical is the adequacy of the interfacial shear strength.

Button *et al.* (1982) carried out laboratory shear strength tests at a range of temperatures (20°C-60°C) on different fabrics, each with three different tack-coat application rates (low, optimum, high). Results were compared to "Control 1" which simulated old pavement-new overlay interfaces with 0.23 ℓ/m^2 of tack, and no geotextile (Table 13.1). "Control 2" were asphaltic concrete specimens with no construction interface.

The interfacial shear strengths of all the fabrics were found to be similar to the controls at 60°C. While the shear strengths were less than the controls at 40°C and 20°C, shear strengths seemed more than adequate.

They concluded from these laboratory tests that, given enough tack coat to approximately saturate the compressed geotextile, the potential for slippage at the interface is no greater than for conventional overlays.

Button *et al.* (1982) also found that a fabric with a "fuzzy" surface offered much better shear strength than the other fabrics tested.

The results of these shear strength tests are given in Table 13.1. The optimum tack coat in this table is that obtained from the equation in Section 13.7.1, using Button's test for Qs. The low tack is half the optimum, and the high tack is double the optimum application rate.

By increasing the tack-coat application rate, interfacial adhesion was increased. Presumably at this high application rate, tack-coat bitumen will penetrate the asphaltic mix increasing its fatigue resistance, but possibly decreasing its stability and also causing flushing on the surface.

Also of interest are the shear values obtained at low application rates. While generally lower than "control 1", they are not dramatically so and may well offer sufficient shear resistance for most situations. As a general guide it seems that adequate interfacial shear strength can be obtained, and this shear strength will be adequate over a wide range of tack-coat application rates.

Specimen Identification	Test Temp (°C)	Shear Strength (psi)		
		Low Tack	Optimum Tack	High Tack
A	20	230	240	—
	40	160	190	240
	60	—	120	—
D	20	200	280	—
	40	190	290	190
	60	—	120	—
E	20	310	390	400
	40	240	265	—
F	20	185	340	390
	40	170	170	—
G	20	150	260	—
	40	130	180	170
Control 1	20	—	350	—
No fabric	40	—	180	—
0.23 Tack	60	—	130	—
Control 2	20	—	440	—
Asphaltic	40	—	290	—
Concrete with no interface	60	—	150	—

TABLE 13.1
Shear Strength Test Results (Button *et al.* 1982)

13.7.3 Tack Coat Type

The tack coat can be either hot "straight run" bitumen or bitumen emulsion. Cut-back bitumens, or emulsions containing solvents should be avoided with some geotextile types as the petroleum solvents may weaken the fibres.

Recommendations of the geotextile manufacturers differ regarding the type of tack coat. Some recommend straight-run bitumen, others recommend emulsified bitumen, while yet others recommend either hot bitumen or emulsion. As well some recommend the use of polymers in the bitumen.

While emulsified bitumens can have the advantages of no cut-back, and spray temperatures much lower than the geotextile's softening point, they do have some disadvantages. First, to apply sufficient emulsion to saturate the geotextile and tack both sides there is a risk of the emulsion running off the road, particularly one with a gradient. This could be overcome by the use of high bitumen-content emulsions. Second it is usual to wait for the emulsion to break before laying the geotextile. This relates to the chemistry of a bitumen emulsion which usually requires either an added chemical, or evaporation of water, or contact with stones (or sand) to cause the emulsion to break. If the geotextile is laid on the unbroken emulsion and then overlaid with asphalt, the emulsion may take a very long time to fully break. (This also applies to emulsion used with a geotextile in chip seals. The very slow break in this case may show up either as bitumen emulsion washing out of the seal coat if rain falls within several days of the seal construction, or as an initial lack of bond of the seal to the underlying pavement and slippage in areas of traffic stress.)

It is of interest that Kent County Council in England use 200-penetration straight-run bitumen as a tack coat for the geotextile interlayer on their roads. Alternatively Perfetti and Sangster (1989) recommend a hot bitumen with 500 ± 60 penetration at 25°C and polymer-modified; or similar in emulsion form. Excess tack coat of a high penetration bitumen is more likely to bleed through the overlay than a low penetration bitumen. (Refer also to Section 13.4.) Possibly the viscosity or penetration of the bitumen used for the tack coat influences the stress absorption properties of the interlayer.

13.8 GEOTEXTILE PROPERTIES

The United States Federal Highways administration assigned a joint committee (Task Force 25) the task of developing standard guidelines for special geotextile applications. The specifications they developed for asphalt overlays are shown in Table 13.2.

Geotextile Characteristic	Testing Method	Unit	Minimum Requirements
Tensile Strength	Task Force 25	N	365
Grab Elongation	Task Force 25	%	50
Bitumen Saturation	Task Force 25	ℓ/m^2	0.9
Melting Point	ASTM D276	°C	170

TABLE 13.2
Task Force 25 Requirements

The minimum bitumen saturation requirement in Table 13.2 favours non-wovens and may well eliminate woven geotextiles.

A melting temperature of 170°C allows the use of polypropylene geotextiles. As these soften near 140°C, they could be expected to cause shrinkage, wrinkling, etc., as the hot asphalt is laid. In practice generally, this does not seem to be a problem, which is contrary to some of the results from laboratory testing (Button *et al.* 1982), but may be explained by laboratory testing by Courard *et al.* (1989) which showed an asphalt overlay laid at 150°C caused the temperature in the geotextile to rise to less than 100°C, not enough to melt polypropylene but possibly enough to cause some loss of that fibre's strength. Laboratory tests of Button *et al.* also suggest that linear shrinkage under hot asphalt can cause cracking of the overlay for some geotextiles, particularly if there are cuts or wrinkles in the laid geotextile.

Table 13.2 suggests that there may be some requirements of a geotextile other than those defined. As covered elsewhere in this chapter, other factors include resistance to traffic abrasion, ease of laying, limiting of geotextile compressibility to reduce fatigue of the overlay mix, etc.

Using thick fabrics will increase laying difficulties and construction costs; if compressible they may also exacerbate fatigue-cracking problems of any subsequent asphaltic-concrete overlay. Geotextiles or paving felts manufactured specifically for inclusion in interlayers under asphalt are now becoming available. These have low compressibility and may be left needle-punched on one face to encourage bond to the bitumen and calendered on the other face to facilitate movement of the asphalt for stress release (Perfetti and Sangster 1989).

A different view is given by Vicelja (1989) who prefers heat-bonded fabrics because they resist pickup and vehicle tyre delamination caused by construction truck traffic better than other fabrics.

13.9 GLASS-FIBRE GEOTEXTILES

The comments made so far in this chapter refer to the use of non-woven polypropylene, polyester, or similar, materials because nearly all reported experience relates to these. An alternative approach could be to use woven glass-fibre geotextiles. While their bitumen retention could be relatively low, the modulus of glass fibre is similar to that of asphalt so reinforcement may be provided by such a geotextile. This suggests that glass fibre may offer benefits in slowing the rate of reflective cracking, but one may suppose that, once the asphaltic concrete overlay has cracked, the glass-fibre geotextile may not ensure lasting waterproofing.

13.10 CONSTRUCTION

Good construction procedures have a very significant effect on the performance of the interlayer overlay system. The following gives guidance on some of the construction procedures recommended (Button and Epps 1983):

1. Patch potholes. Clean and then fill cracks wider than 3mm with joint filler material. Eliminate across-crack vertical movements. If the existing pavement is uneven apply a levelling course. Sweep the existing road surface. (Geotextiles can be used on highly textured existing pavements, but any sharp projections should be removed.)

2. Spray a uniform application of bitumen. Cut-back bitumens should not be used as the petroleum-based solvents in cutbacks are damaging to some synthetic geotextiles (e.g. polypropylene) at higher temperatures. Excessive tack coat can lead to geotextile slippage and bitumen bleeding through the overlay.

3. Lay the geotextile. Simple mechanical laying equipment (e.g. pipe frame and weighted brooms on front end loader) tends to give a better job with fewer wrinkles than hand laying. It is not usual to pre-tension the geotextile.

4. Do not lay the geotextile on a damp pavement. Even though the pavement surface appears dry, moisture in any small openings can vaporise later with the sun's heat and cause bubbles to form under the geotextile.

5. Geotextiles may be butt-jointed or overlapped at transverse joints. If overlapped, the top layer should point in the direction of travel of construction equipment. Overlaps should be tacked with bitumen or bitumen emulsion. Overlap of transverse joints should be at least 300mm, whereas overlap of longitudinal joints can be 150mm.

6. Wrinkles can be a source of premature cracking of the overlay. Brooming from the centre out may remove small wrinkles. Large wrinkles should be cut, then butt-jointed. Geotextiles can be laid on straight sections of roads with few wrinkles, using purpose built equipment, but this is not so for laying around curves. Some geotextiles resist wrinkling during installation better than others.

7. Pneumatic-tyred rolling of the geotextile immediately after application will increase its adhesion to the pavement and minimise geotextile movement by traffic, construction equipment or wind. Pneumatic-tyred rollers can cause slippage downhill of the fabric. Fabrics with a "fuzzy" surface next to the bitumen tack coat offer more resistance to slippage under the tyres of construction equipment. Over-rolling can cause bitumen to "bleed" through into the surface of the geotextile causing problems when moving construction equipment.

8. If the bitumen is "bleeding" through the geotextile, sprinkle sand to prevent tyres sticking.

9. Construction traffic should not make sudden stops, starts or sharp turns on the geotextile. Nor should it be parked on the geotextile.

10. Exposure of the geotextile to traffic should be minimised. Traffic will abrade away geotextile fibres to varying degrees depending on the type of fabric; tyres will punch or wear holes at the peaks of the larger aggregate in the old surface, and pick up "bleeding" bitumen. Bitumen-saturated geotextiles wet with rain water tend to offer traffic a lower skid resistance than asphalt if trafficked before the asphalt is applied.

11. Lay asphaltic-concrete overlay. Care may be needed to limit the temperature of the asphalt being laid. If using the tack coat application rate given in Section 13.7.1, possibly a second bitumen tack coat on top of the geotextile may not be necessary.

13.11 CONCLUSIONS

The use of geotextile/bitumen interlayers under asphaltic concrete overlays can be beneficial in slowing the appearance of reflective cracking. After cracking does reappear, the bitumen-saturated geotextile will continue to waterproof the crack for a period of time.

Construction procedures can have a marked effect on the performance of these interlayers. For example, wrinkles in the geotextile can cause premature cracking of the overlay, yet wrinkles can be hard to avoid especially when laying geotextiles around curves in the road.

These geotextile/bitumen interlayers are most likely to be effective over existing cracks having only horizontal movements. Over fatigue-cracked areas the main benefit of these interlayers is likely to be related to waterproofing, rather than crack retardation. These interlayers are likely to offer only a short life over cracks with differential vertical movements under traffic.

After 17 years experience, Vicelja (1989) considers that by carefully following good procedures a success rate of 99% can be experienced using geotextile interlayers.

13.12 REFLECTIVE CRACKING THROUGH OVERLAYS : CASE HISTORIES

13.12.1 Introduction

Geotextiles have been used extensively for interlayers under asphaltic concrete overlays constructed over existing cracked pavements. The literature contains many overseas case histories detailing these uses and monitoring performance. The performance of the geotextile/asphalt overlay systems has been variable; not all uses have demonstrated benefits. These case studies have formed the basis for many of the comments made earlier in Chapter 13. A selection of these case histories are summarised in this Section.

13.12.2 New Mexico Trials

Lorenz (1987) reports on four experimental projects on highways in New Mexico, USA. Two of these projects were on Highway I-25, and two on Highway I-40. These projects outlined in Sections 13.12.2.1 to 13.12.2.2, used selections of the following interlayers:

(a) 85/100 penetration bitumen combined with 20% reclaimed rubber and 2% extender oils. Applied at 2.7 ℓ/m^2 then lightly chipped.

(b) Open-graded bituminous mix with 3% bitumen, placed in a thickness of 90mm.

(c) Heating existing asphaltic concrete, scarification to 20mm depth, application of Reclaimite, then roll to compact.

(d) Non-woven heat-bonded polypropylene/nylon geotextile (135g/m^2) laid on an 85/100 penetration-bitumen tack coat applied at 0.8 ℓ/m^2.

(e) Non-woven needle-punched polypropylene geotextile laid on an 85/100 penetration-bitumen tack coat applied at 1.1 ℓ/m^2.

(f) 120/150 penetration bitumen with 25% vulcanised granulated rubber, diluted by some 6.5% kerosene. Applied at 2.6 ℓ/m^2 then lightly chipped.

(g) 19mm down unbound basecourse laid to 100mm compacted depth.

13.12.2.1 Highway I-25 (Project 1)

On this I-25 project the existing pavement included 150mm of cement-treated basecourse. Extensive longitudinal and transverse cracking was evident as well as some localised crocodile cracking. All cracks were first blown clean and filled with 120/150 penetration bitumen.

Six sections were overlaid and monitored for five years. The overlay treatments were 50mm of bituminous mix over the heat-bonded geotextile (d, Section 13.12.2 above); similarly for the needle-punched geotextile (e, Section 13.12.2 above); a 100mm bituminous mix control, and three 65mm bituminous mix control sections.

From best to worst the performance of the sections after 5 years was:

- 65mm control (20% of original cracks reflected through),
- 65mm control (23%), needle-punched geotextile (27%),
- 65mm control (28%), heat-bonded geotextile (28%), and
- 100mm control worst.

The varying performance of the control sections themselves indicates the difficulty of comparing the performance of overlay systems.

13.12.2.2 Highway I-40 (Project 2)

The existing pavement exhibited extensive transverse, longitudinal and crocodile cracks. Crack movements were attributed to traffic loads, temperature and moisture changes.

Six sections were overlaid then monitored for five years. From best to worst the performance of the sections after five years was:

- Needle-punched geotextile (e above) with 140mm bituminous mix overlay (22% of original cracks reflected through),
- Open graded mix (b above) with 140mm bituminous mix overlay (25%),
- Bitumen/vulcanised rubber mix (f above) with 140mm bituminous mix overlay (28%),
- Heat-bonded geotextile (d above) with 140mm bituminous mix overlay (30%),
- Bitumen/vulcanised rubber mix (f above) with 140mm bituminous mix overlay (44%),
- Control section with 140mm bituminous mix overlay (51%).

Problems during and after construction emphasised the need for adequate preparation of the existing pavement and the practical difficulties of spreading, holding, and tack coating the geotextiles.

Core samples of trial sections indicated the interlayers remained intact even though the original crack propagated through.

13.12.2.3 Highway I-25 (Project 3)

The existing pavement consisted of various bitumen treated layers and exhibited numerous transverse cracks. The trial sections were monitored for just over four years.

From best to worst the performance of the sections after four years was:

- Heated scarified (c above) with 50mm bituminous overlay (77% of original cracks reflected through),
- Heat-bonded geotextile (d above) with 50mm bituminous overlay (80%),
- Bitumen/vulcanised rubber mix (f above) with 50mm bituminous mix overlay (100%),
- Needle-punched geotextile (e above) with 50mm bituminous overlay (100%), and
- Control section with 50mm bituminous mix overlay.

Core samples of trial sections showed the bitumen/rubber and geotextile membranes to be intact. Both horizontal and vertical movements appeared to have contributed to the reflective cracking.

13.12.2.4 Highway I-40 (Project 4)

On this I-40 project the existing pavement included 150mm of cement-treated basecourse. Transverse and longitudinal cracks were evident.

Two sections were constructed and then monitored for four years. The best performing of the two sections was the unbound basecourse interlayer (g above) (only 5% of original cracks reflected through), while the other section using an open-graded mix interlayer (b above) had 37% of the original cracks reflected through.

13.12.3 Polymer/Glass Fibre Trials

Twenty-one states of the USA have trialed a woven glass-fibre geotextile/polymer bitumen interlayer beneath an asphaltic-concrete overlay. The polymer/geotextile was laid as a bandage across the crack with geotextile widths between 0.3 and 1.1 metres.

This system promises the combined benefits of a polymer-bitumen stress-absorbing layer and a geotextile with high modulus (230 MPa), medium strength (175 N/mm tensile strength) and high melting point.

The results of field trials involving 30,500 metres of cracks are summarised in Table 13.3. The results are divided in four categories to emphasise the variability between reflective cracking from the different types of cracks and joints. In particular transverse joints in concrete pavement are difficult to repair because of vertical deflections, and in many cases because of the severe deterioration of the concrete around the joint. This difficult situation is shown by the wide range of concrete transverse joint results in Table 13.3 (Rowlett and Uffner 1985).

Examination of Table 13.3 suggests the polymer bitumen/glass fibre bandage system is effective in reducing the rate of appearance of reflection cracks.

13.12.4 LCPC France Trial

The LCPC laboratory at Autun, France, monitored trials using geotextile interlayers impregnated with bitumen to prevent reflection of the shrinkage cracks from aggregate bases cemented with hydraulic binders. Five trials were constructed and monitored for some four years.

The two main conclusions of the trials were:

- The bitumen-impregnated geotextile interlayer slowed reflective cracking, so the geotextile trials were at least as good as the control sections had been a year earlier.

- Use of a thick compressible geotextile (needle-punched geotextile of $340 g/m^2$) caused premature fatigue cracking of the asphaltic-concrete overlay.

Kind of Crack Repair Method	Percent of Cracks Reflected		Length Repaired (m (ft))	No. of Trials	No. of States	Time Span	Average Age (months)	Range* of Ages (months)
	Average	Range						
1. Bituminous Concrete								
Asphalt polymer/glass fibre reinforcement system	12	0-17.5	20,000 (65,638)	13	6	1976 to 1980	24	10 to 56
Control	88	0-100						
2. Concrete Interslab Cracks								
Asphalt polymer/glass fibre reinforcement system	1	0-1	4,295 (14,092)	3	1	1978 to 1980	19	6 to 32
Control	75	48-100						
3. Concrete Longitudinal Joints								
Asphalt polymer/glass fibre reinforcement system	0	0	2,794 (9,167)	11	6	1978 to 1980	15	6 to 30
Control	5	0-15						
4. Concrete Transverse Joints								
Asphalt polymer/glass fibre reinforcement system	29	0-75	2,244 (7,362)	13	6	1978 to 1980	17	6 to 30
Control	86	0-100						

* All trials were exposed to a minimum of one winter

TABLE 13.3

Summary of Field Trial Performance of Asphalt Polymer/Glass Fibre Reinforcement System for Reduction of Reflection Cracks (Rowlett and Uffner 1985)

13.12.5 South Africa Trials

These trials are reported in full by Hugo *et al.* (1982) and are worth reading as they illustrate the thoroughness and long-term commitment needed to research this topic.

The existing pavement included 300mm of cement-stabilised aggregates. Severe longitudinal and transverse cracking was evident.

Before full scale field trials were begun, laboratory tests were carried out by cyclically loading across a crack. Some overlay types allowed considerably greater crack widths at the base of the overlay before cracks appeared in the pavement surface, as indicated in Table 13.4.

Overlay Type	**Maximum Crack Width before Reflection (mm)**
Asphaltic Concrete (AC)	3
AC with bond breaking	4
Reinforced AC without bond breaking	9
Reinforced AC with bond breaking	18

TABLE 13.4
Behaviour of Overlay Systems (Hugo *et al.* 1982)

Following the laboratory study they constructed 80,000m^2 of geotextile-reinforced asphalt with 300mm wide bond breaking material at every crack. The method of construction was:

1. Clean cracks using compressed air.

2. Apply a tack coat of bitumen emulsion at 0.16 ℓ/m^2 residual bitumen.

3. Spread fine sand 5mm thick and 300mm wide over each crack, immediately in front of the paver.

4. Paver lays 20mm asphaltic concrete.

5. Apply a tack coat.

6. Lay a woven polyester geotextile.

7. Paver lays 50 mm of gap-graded asphaltic concrete. Pre-coated chips were rolled into the surface.

The overlay was monitored for eight years and carried a traffic load of some 2 x 10^6 EDA. Reflection cracking developed slowly over the first five years reaching 30%. The rate of appearance of reflective cracking then increased, reaching 60% after eight years.

In general the reflected crack pattern was strongly related to the cracks in the original pavement. Upon coring many cracks were found to have deflected horizontally, with the reflective crack appearing at the surface some distance from the original crack. Hugo *et al.* (1982) notes that the geotextile-reinforced asphalt overlay successfully served its prime function of **preventing ingress of water into the pavement structure.**

13.12.6 Georgia, USA, Trials

The Georgia Department of Transport, USA, monitored 16 asphalt-overlay test sections with different treatments on an existing jointed concrete pavement (Gulden and Brown 1983). The percentage of crack length that had reflected through the overlay after six years is summarised in Table 13.5.

Treatment	**Joint Length Reflected (%)**		
	Overlay Thickness (mm)		
	50	**100**	**150**
Heavy duty Bituthene	83	55	6
Melt-bonded polypropylene geotextile	97	70	24
Needle-punched polypropylene geotextile	91	54	27
Edge drains	98	92	73
Control	100	89	29

TABLE 13.5
Percentage of Joint Length Reflected after Six Years (Gulden and Brown 1983)

While the best performance in terms of retarding reflective cracking was the bituthene (450mm-wide strip of self-adhesive rubberised bitumen with woven fabric reinforcing), the geotextile sections performed better than the control sections for the 100 and 150mm overlays.

The 50mm overlays over geotextile performed better than the control for two years but showed little or no difference after four or more years. This suggests there is little point in applying overlays less than 50mm thick.

Many of the treatments would provide, however, a waterproof barrier that prevents surface water from entering the pavement.

13.12.7 Virginia, USA, Trials

The results of two projects in Virginia, USA, have shown that fabric interlayers perform poorly on cracked portland cement-concrete pavements if the vertical differential deflection across the cracks is more than 0.05mm (Rust 1986).

13.12.8 Trials over AC Pavements in USA

The use of geotextile interlayers over asphaltic-concrete pavements has been tested in many places in the USA. The results of many of the tests have not been reported, and those that have been differ. The most favourable results in the USA have been reported from California, Colorado, Florida and Texas. Tests with unfavourable results were reported from Arizona, California, Colorado, Maine, Vermont and Wyoming (NCHRP 1983).

The tests in Colorado were made on pavements with both transverse and longitudinal cracking as well as crocodile cracks. After five years the Petromat sections showed the best results with less than 5% reflection cracks compared to 70% in the controls.

Conversely, Wyoming reported that Petromat failed to produce any benefit, with cracking in the control and Petromat sections about the same.

13.12.9 Australian Trial

The Main Roads Department, Queensland, has established a crack control trial to test a range of treatments intended to prevent cracking in a cement-treated pavement reflecting through to the surface. The early findings of this trial indicate that full area interlayer treatments are more effective than bandage treatments (Atkinson and Gordon 1988).

13.13 GEOGRID-REINFORCED OVERLAYS

13.13.1 University of Nottingham Tests

Polymer meshes or geogrids have been used to control reflective cracking through asphalt overlays. Tests at the University of Nottingham on 1.2m-long slabs subject to cyclic loading showed that reflective cracking can be retarded when a geogrid is included in an overlay.

In addition researchers at the University of Nottingham investigated the fatigue performance of geogrid-reinforced asphalt. They found that the elastic stiffness of an asphalt layer is not significantly increased by the geogrid, so presumably the initiation of reflective cracking upwards through the overlay is not affected. The rate of propagation of the crack, however, appeared to be significantly slowed by the grid, giving the geogrid-reinforced asphalt an increased fatigue life compared to the asphalt without a grid.

The third area investigated at the University of Nottingham in laboratory tests on 1.2m-long slabs, and in trials on the Pavement Test Facility, was the rate of rutting in asphalt. Inclusion of the geogrids within the asphalt significantly reduced the rate of rutting.

13.13.2 Installation

The geogrid is pre-tensioned usually before placing the overlay by stretching it about 5% of its length then nailing the free end to the existing road.

Paver-laying asphaltic concrete directly over the geogrid is not practical as the paver tends to pull and kink the grid. The two most common methods adopted to cover grids are:

1. Hand lay a 20mm-thick protective coat of asphaltic concrete over the grid, before overlaying, or

2. Chip seal over the grid before overlaying.

13.13.3 Field Experience

Field experience of using geogrid-reinforced overlays is limited. A number of projects have been completed over the last five years, mostly in Europe. Some of these have been monitored by the University of Nottingham and are reported to be performing well.

It is, as yet, too early to draw any firm conclusions from any recent uses in Australia or New Zealand (Oliver 1988), but initial findings from a Queensland Crack Control Trial indicate that geogrid interlayers under asphalt are not as effective as geotextiles in slowing the reappearance of reflective cracking (Atkinson and Gordon 1988).

13.14 REFERENCES

This list includes additional reading.

Atkinson, D.J., Gordon, R.G. 1988. *Evaluation of measures for prevention of reflective cracking.* Main Roads Department, Queensland, Australia.

Brown, S.F., Brunton, J.M., Hughes, D.A.B., Brodrick, B.V. 1985. *Polymer grid reinforcement of asphalt.* Association of Asphalt Paving Technologists, Texas, USA.

Bryan, D. 1986. *Improving upon nature with geotextiles.* Public Works.

Button, J.W., Epps, J.A. 1983. Field evaluation of fabric interlayers. *Texas Transportation Researcher.*

Button, J.W., Epps, J.A., Lytton, R.L., Harmon, W.S. 1982. Fabric interlayer for pavement overlays. *Proceedings of Second International Conference on Geotextiles,* Las Vegas, USA.

Chemie Linz 1988. *Explanation of the general behaviour of bitumen and the effects of Polyfelt PGM 14 paving felt on the life of asphalt road surfacing.* Chemie Linz AG, Austria.

Colombier, G., Astesaw, A., Gouacolou, H. 1982. Using a geotextile to prevent shrinkage crack of rigid pavements. *Proceedings of Second International Conference on Geotextiles,* Las Vegas, USA.

Courard, L., Rigo, J.M., Degeimbre, R., Wiertz, J. 1989. Compatibility between fibres and modified bitumen. *Proceedings of First International Conference on Reflective Cracking in Pavements,* Liège, Belgium.

Eckmann, B. 1989. *Reflective cracking studies at Liège University.* Esso SAF.

Foulkes, M.D. 1989. Reflection cracking of flexible composite pavement surfacings. *Municipal Engineer* 6(5): 289-297.

Gulden, W., Brown, D. 1983. Treatments for reduction of reflective cracking of asphalt overlays on jointed concrete pavements in Georgia. *Transportation Research Record* 916: 1-5. Transportation Research Board, Washington DC.

Halim, A.O., Haas, R., Phang, W.A. 1983. Geogrid reinforcement of asphalt pavements and verification of elastic theory. *Transportation Research Record* 949: 55-65. Transportation Research Board, Washington DC.

Hayward, D. 1988. Road-holding grid cuts down repairs. *New Civil Engineer 1988.*

Houghton, L.D. 1988. Personal communication. Lower Hutt City Council, 22 April 1988.

Hugo, F., Strauss, P., Schnitter, O. 1982. The control of reflection cracking with the use of a geotextile. A ten year case history. *Proceedings of Second International Conference on Geotextiles,* Las Vegas, USA.

Kennedy, L.J. 1984. Stanley Airfield, Falkland Islands 1982. *Proceedings of Symposium on Polymer Grid Reinforcement in Civil Engineering.*

Kroshus, J.B., Varcoe, B.E. 1984. Geogrid applications in the construction of oilfield drilling pads and access areas in muskeg regions of Northern Alberta. *Proceedings of Symposium on Polymer Grid Reinforcement in Civil Engineering.*

Lorenz, V.M. 1987. New Mexico study of interlayers used in reflective crack control. *Transportation Research Record* 1117: 94-103. Transportation Research Board, Washington DC.

Majidzadeh, K., Luther, M.S., Skylut, H. 1982. A mechanistic design procedure for fabric-reinforced pavement systems. *Proceedings of Second International Conference on Geotextiles,* Las Vegas, USA.

Majidzadeh, K., Abdulshafi, A., Ilves, G.J., McLaughlin A. 1987. A mechanistic model for thermally induced reflection cracking of portland cement concrete pavement with reinforced asphalt concrete overlay. *Transportation Research Record* 1117: 83-93. Transportation Research Board, Washington DC.

New Civil Engineer 1986. Textile trials underway on Kent Road cracks. *New Civil Engineer 1986.*

NCHRP 1983. Minimizing reflection cracking of pavement overlays. National Co-operative Highways Research Program (NCHRP). *Synthesis of Highways Practice* 92.

Nunn, M.E. 1989. An investigation of reflection cracking in composite pavements in the United Kingdom. Pp. 146-153 in *Reflective cracking in pavements: assessment and control,* J.M. Rigo and R. Degiembre (Eds). Proceedings of CEP-LMC Conference, Liège, Belgium.

Oliver, J.W.H. 1988. New materials for old problems: innovative developments in highways engineering using high strength polymer grid reinforcement. *Proceedings of Australian Road Research Board Conference.*

Perfetti, J., Sangster, T. 1988. The use of a textile-based system to control pavement cracking. *Geotextiles and Geomembranes* 7(3): 165-178.

Perfetti, J., Sangster, T. 1989. The function of geotextiles in pavement structures. *Proceedings of Geosynthetics Conference,* San Diego, USA.

Pooley, G.R. 1984. Construction of a reinforced asphalt mix overlay at Canvey Island, Essex. *Proceedings of Symposium on Polymer Grid Reinforcement in Civil Engineering.*

Rowlett, R.D., Uffner, W.E. 1985. The use of an asphalt polymer/glass fibre reinforcing system for minimizing reflection cracks in overlays and reducing excavation before overlaying. Pp. 65-73 in *Pavement maintenance and rehabilitation,* B.K. Kallas (Ed.). *American Society for Testing and Materials STP* 881. 69p.

Rust, F.G. 1986. State of the art of rehabilitating reflection cracking in cemented pavements. *CSIR Technical Report* RP/29, South Africa.

Smith, R.D. 1983. Laboratory testing of fabric interlayers for asphalt concrete paving: interim report. *Transportation Research Record* 916: 6-18. Transportation Research Board, Washington DC.

Vicelja, J.L. 1989. Pavement fabric interlayers. In *Reflective cracking in pavements: assessment and control,* J.M. Rigo and R. Degiembre (Eds). Proceedings of CEP-LMC Conference, Liège, Belgium.

Walsh, I.D. 1986. Geotextiles in highway surface dressing. *Proceedings of Third International Conference on Geotextiles,* Vienna, Austria.

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

CHAPTER 14

PAVEMENTS: OTHER USES

14.1 CHIP SEALS

14.1.1 Introduction

Wise incorporation of geotextiles into chip seals promises improvements to pavements relative to conventional seals, as follows:

- Delay the appearance of reflective cracking.
- Delay fatigue cracking of the seal coat.
- Delay age-related cracking of the seal coat.
- Resist chip penetration into underlying soft basecourse or asphaltic concrete (Section 14.1.6).
- Allow thinner pavements, or lengthen pavement design life.
- Use of smaller sealing chips.

Pavement design is primarily concerned with limiting subgrade strain to limit in turn the depth of accumulated wheel track rutting. However one of the considerations in selecting a terminal rut depth implicit in the pavement design charts is traffic-induced cracking of the seal coat. Increased resistance of the seal coat to cracking therefore may allow the use of a deeper "terminal" rut depth than is now the norm, and hence a longer pavement life.

The use of geotextiles in chip seals also allows the two main functions of a seal coat — skid resistance and waterproofing — to be separated. If high cracking resistance is required then a bitumen-saturated geotextile appears to offer this possibility, while only requiring say a grade 5 chip to provide skid resistance with relatively low tyre/pavement noise.

Experience worldwide with these geotextile seals appears to be very limited. Trials that have been monitored over more than some five years appear to be non-existent. It is therefore very important that careful thought should be given to the likely costs and outcomes of using geotextiles for any of these applications, and any applications should be monitored and reported.

To the authors' knowledge there were very few applications of this type in New Zealand before 1988 but since then there has been a surge of interest.

While geotextiles in chip seals promise many benefits, like anything new, they should not be regarded as a magic cure-all. Rather an understanding of how they work and the limitations to the technology should be sought. The sections below will give some guidance but this is limited by lack of field experience to date.

14.1.2 Geotextile Functions

For all the improvements listed in Section 14.1.1, except the resistance to chip penetration, the purpose of the geotextile is to inhibit cracking. Based on the comments in Section 13.6 for geotextile interlayers, this inhibition of cracking is likely to be a complex mechanism, with the tensile strength and modulus of the geotextile unlikely to have much effect on performance, i.e. the geotextile offers little benefit from reinforcement.

As for interlayers under asphaltic concrete it may well be that the prime benefit of incorporating a geotextile in a chip seal relates to the **geotextile's ability to retain bitumen.**

The type and properties of the geotextile used influence the functions the geotextile performs. The comments in this chapter apply particularly to non-woven polypropylene or polyester geotextiles. Woven glass-fibre geotextiles for example may function differently, and provide some significant reinforcement.

14.1.3 Geotextile Properties

The geotextile needs to readily absorb bitumen and be able to accommodate pavement movements in several directions (e.g. randomly orientated cracks). Non-wovens then would seem appropriate.

The geotextile needs to be relatively easy to lay and be resistant to traffic abrasion. Using thick fabrics will increase laying difficulties and construction costs; if compressible they may also exacerbate fatigue cracking problems in any subsequent asphaltic concrete overlay. Recently, geotextiles or paving felts manufactured specifically for inclusion in chip seals or interlayers under asphalt are becoming available; these have low compressibility and may be needle-punched on one face to encourage bond to the bitumen yet calendered on the other face to facilitate movement of the asphalt for stress release (Perfetti and Sangster 1989).

As examples of the geotextiles used in chip seals, the properties of the two geotextiles used by Walsh (1986) are:

- 100% polypropylene continuous fibre heat-melded needle-punched non-woven. Weight 140g/m^2, thickness 1.5mm.

- 100% polypropylene continuous fibre heat-melded needle-punched non-woven. Weight 110g/m^2, thickness 1.1mm.

The properties of the geotextile used by Houghton (1988) in New Zealand are:

- 100% polypropylene continuous filament needle-punched non-woven.
  Weight 140g/m^2.

14.1.3.1 Binder Temperature Effects

Hot bitumen is normally sprayed at temperatures of around 165°C, and if polymer-modified bitumens are used, spraying temperatures can be as high as 185°C. Polypropylene softens near 140°C and melts at 170°C so problems of shrinkage, wrinkling, melting, etc. of the fabric would appear likely after being sprayed with hot bitumen.

Walsh (1986) reports no problems in three trials when hot cut-back bitumen was sprayed onto a polypropylene geotextile, but reports minor shrinkage (which removed creases) of the 110 g/m^2 geotextile in another trial. Houghton (1988) reports no problems with geotextile shrinkage after spraying bitumen at 165°C onto the geotextile.

It is usual to lay the geotextile onto cool or cooling bitumen and use several passes of a pneumatic-tyred roller to press the geotextile into the bitumen. When the second spray of bitumen is applied the geotextile should already be partially saturated with cool bitumen, which presumably reduces any temperature rise of the geotextile itself. In addition the geotextile should be effectively glued to the road at this stage making shrinkage difficult, but not impossible (e.g. curling of edges may occur).

14.1.4 Binder Application Rate

Application of the correct amount of binder is important. As for any chip seal too little will result in chip loss, and too much in bleeding. It is usual to apply the bitumen in two spray runs, one before and one after the geotextile is laid.

Walsh (1986) sprayed a lighter application of bitumen first to adhere the geotextile to the existing road and to almost saturate the geotextile. This was followed by a heavier second application, presumably a little for absorption by the geotextile, but most for chip retention. His approximate spray rates including cut-back are listed in Table 14.1.

Geotextile Weight (g/m^2)	Chip Size (mm)	First Binder Application (ℓ/m^2)	Second Binder Application (ℓ/m^2)
110	10	0.98	1.20*
110	10	0.98	1.15* 1.20 1.40
140	14	0.63	1.25*
140	14	0.98	1.36 1.46 1.65
*Chip loss occurred later on these sections of road			

TABLE 14.2
Spray Rates for Bitumen Binders using Geotextiles of Different Weights (Walsh 1986)

Walsh subsequently used laboratory tests to ascertain the "theoretical" volume of bitumen required to saturate the 140 g/m^2 fabric as 0.97 ℓ/m^2. The magnitude of this suggests that Walsh used the Task Force 25 Method (Section 13.8) or similar with the geotextile that was uncompressed.

Houghton (1988) used a slightly different approach. His first application of bitumen was intended to saturate the geotextile in its compressed state after rolling and traffic compaction. The second application was that required for the chip size, traffic volume, etc., if no geotextile was present. This second application rate can therefore be calculated as for a normal chip seal using the texture depth of the underlying surface in the application rate algorithm.

Houghton developed his own method for calculating the bitumen required to saturate the compressed (by rolling, then traffic) geotextile. The essence of this method can be demonstrated by an example:

> Compress tightly eight layers of the geotextile between thumb and forefinger and measure that thickness:
>
> Thickness of compressed layers = 5.08mm
>
> Thickness of one layer = 5.08/8
> = 0.635mm
>
> Volume of 1 m^2 compressed = 0.000635 m^3

If this was solid polypropylene of assumed density 990 kg/m^3, then weight of 1 m^2 would = 990 x 0.000635
= 0.629 kg/m^2

but, from manufacturer's data, 1 m^2 of fabric weighs 140 g = 0.140 kg/m^2

Therefore volume of voids in compressed fabric = (0.629 - 0.140)/990
= 0.489 ℓ/m^2

Therefore residual bitumen application rate of first spray run of bitumen (to saturate geotextile in compressed state) = **0.49 l/m^2**

This suggests Houghton's method gives a slightly lower total residual bitumen application than does Walsh's. It is not known if Houghton and Walsh used the same brand of geotextile.

The Task Force 25 (see Section 13.8) method gave the asphalt saturation 0.9 ℓ/m^2 for the geotextile Houghton used (Chemie Linz 1986), considerably more than Houghton's method. It is noted that the Task Force 25 method does not compress the geotextile, so may over-estimate the required bitumen quantity in a geotextile seal. In a chip seal, pneumatic-tyred rollers then traffic will compress the geotextile. Hence it seems the bitumen application rate should be tested using a compressed geotextile. The amount the geotextile will finally compress may be related to traffic volumes, hence the bitumen application rate required to saturate the geotextile presumably should be related to traffic volume. This suggests that quite a lot of development work is still needed for use of geotextile seals, and both the Task Force 25 method and Houghton's method need to be used wisely.

The actual application rate applied under the geotextile on Houghton's method was 0.51 ℓ/m^2. After some 18 months use, the seal coat was still in very good condition, although to date most of Houghton's seals over geotextiles have been two-coat seals. This reduces the risk of chip loss. A recent single-coat chip seal by Houghton used an application rate under the geotextile of only 0.4 ℓ/m^2; and early chip loss suggests that this application rate was too low.

14.1.5 Construction

Good construction procedures have a significant effect on the performance of the geotextile chip seals. In particular the residual bitumen application rate is very important. The following gives guidance on construction procedures:

1. Improve edge drainage, if necessary. Patch potholes. Apply a levelling course if roughness is a problem. Existing areas of fatigue cracking are likely to fail first in the new geotextile seal; if these are only a small portion of the existing surface they should be dug out and reinstated before sealing.

2. The surface should be clean and dry.

3. Apply first bitumen layer using a distributor that meets E/2 certification requirements. It is preferable for the bitumen spray width to be slightly wider than the geotextile.

4. Lay the geotextile. Mechanical laying equipment tends to give a better job with fewer wrinkles than hand laying. Hand laying is not easy and tends to produce wrinkles, especially on road corners. It is not usual to pre-tension the geotextile. The authors suspect wrinkles in the geotextile are not as detrimental to chip seals as to asphalt overlays in which wrinkles can initiate cracking of the asphalt. Large wrinkles can be cut with a sharp knife and butt-jointed, otherwise they may be apparent in the finished seal.

5. Roll the surface of the geotextile with a pneumatic-tyred roller. A steel drum or rubber-coated drum roller could be used but this increases the risk of punching underlying stones through the geotextile. Also experience (Walsh 1986) indicates that pneumatic rollers bring the binder through the geotextile sooner.

6. Apply the second bitumen layer using an E/2 certified distributor driven over the laid geotextile.

7. Spread stone chips and roll. The number of roller passes is greater than that needed for a conventional chip seal, especially outside the traffic lanes (Chemie Linz 1986).

8. Open pavement to slow traffic.

While either a hot bitumen or an emulsified bitumen may be used, hot bitumen is usual. The reasons for this are as given in Section 13.7.3. If a hot bitumen is used with no cutter or flux then pre-coated chips may be needed.

Laboratory work such as that reported by Eckmann (1989) suggests use of polymer-modified bitumen (e.g. styrene butadiene styrene polymers) with a geotextile in a chip seal offers significant advantages over straight bitumen/geotextile seals.

On curved sections of road, laying a geotextile without wrinkles may not be practical. Perfetti and Sangster (1989) suggest placing the geotextile in bands 0.5 to 1.0m wide. This would however create problems in butt-jointing the adjacent edges without gaps or overlaps.

14.1.6 Performance

Walsh (1986) reported both the 110 g/m^2 and 140 g/m^2 fabrics were performing well, and any chip loss was attributed to insufficient binder application on some sections. While cracks in the non-geotextile areas reappeared within five months, no cracks were visible in the geotextile/chip seal sections after 16 months.

Walsh also noted that the geotextile chip seal is far less tolerant of lateral traffic forces caused by turning or braking, than conventional chip seals. The thicker (140 g/m^2) fabric is more tolerant than the thinner (110 g/m^2) fabric. Until more is known, use of geotextile/chip seals in areas of high traffic stress requires particular thought. They may require more binder, more compaction, better traffic control, or a chip-locking coat.

After 22 months trafficking, no problems have become apparent with Houghton's (1988) geotextile/chip seals, although they were not constructed on areas subjected to high stresses from traffic turning or stopping.

One of Houghton's geotextile seals was constructed over newly laid asphalt on a heavily trafficked road, normally a recipe for bitumen flushing. Monitoring by sand circle over the 20 months since construction suggests less texture loss in the geotextile seal than in the non-geotextile control seal section alongside. This suggests that **geotextiles may be effective in resisting chip penetration into soft asphaltic concrete.**

If it becomes necessary to rip and reconstruct the road in the future it is not known what problems the geotextile will cause. However American experience in cold milling then recycling asphalt with geotextile interlayers indicates that some geotextile types at least cause few problems.

14.1.7 Australian Trials

The Australians have been seeking cost-effective methods of providing all-weather pavements for lightly trafficked roads in remote areas. The Roads and Traffic Authority of New South Wales propose to construct trial chip seals incorporating polymer-modified bitumen and a geotextile directly on the prepared subgrade. These subgrades are commonly poor quality, expansive clays.

The basic premise behind these trials is to provide a waterproof membrane over the subgrade. By keeping traffic away from the edge of the seal, particularly during wet weather, an all-weather pavement with a substantial service life should be provided.

They propose to use their Accelerated Loading Facility (ALF) to evaluate these trials in the second half of 1990 (RTA 1989).

A completely different trial was carried out by the Road Construction Authority, Victoria, incorporating a geotextile in a two-coat chip seal. This geotextile seal was over an existing, failed, sealed road. A polyester fibre non-woven geotextile was used. This trial was constructed in 1979 and performance after seven years was reported as very good for the section incorporating the geotextile, whereas the adjacent areas without a geotextile had deteriorated.

14.2 EMBANKMENTS ON WEAK FOUNDATIONS

There are at least two categories of weak foundations: fibrous peats and soft cohesive soils.

Fibrous peats are characterised by a high fibre content and high water content. They are highly compressible and permeable, particularly during the early stages of embankment construction.

Soft cohesive soils, such as marine silts, can also be very difficult materials on which to build highway embankments. At usual embankment construction rates these materials can be considered to exhibit undrained behaviour during and immediately after construction.

Both the preceding materials provide weak foundations upon which to build road embankments. If construction is to take place on a site underlain with a uniformly weak soil deposit such as soft clay or peat the role of a reinforcing geotextile is to reduce the risk of a slip circle failure of the embankment and foundation soil, and to reduce lateral spreading and cracking of the embankment. But the geotextile reinforcement will not significantly reduce embankment settlements associated with time-dependant consolidation of uniformly weak foundation soils.

If the foundation is of non-uniform weakness, for example it contains lenses of clay or peat, the geotextile reinforcement is incorporated to bridge the weak spots to reduce the risk of localised failure, and/or to reduce differential shrinkage.

Geotextiles can also aid construction and longer term stability of embankments. The most common application of geotextiles in these situations is at the underside of the embankment, i.e. on top of the weak foundation. Geotextiles can be used for one of two reasons: as a separation and/or reinforcement layer, or as a separation and filter layer at the base of the embankment. These two uses are shown in Figure 14.1.

(a) Reinforcing Use (b) Separation and Filter Use

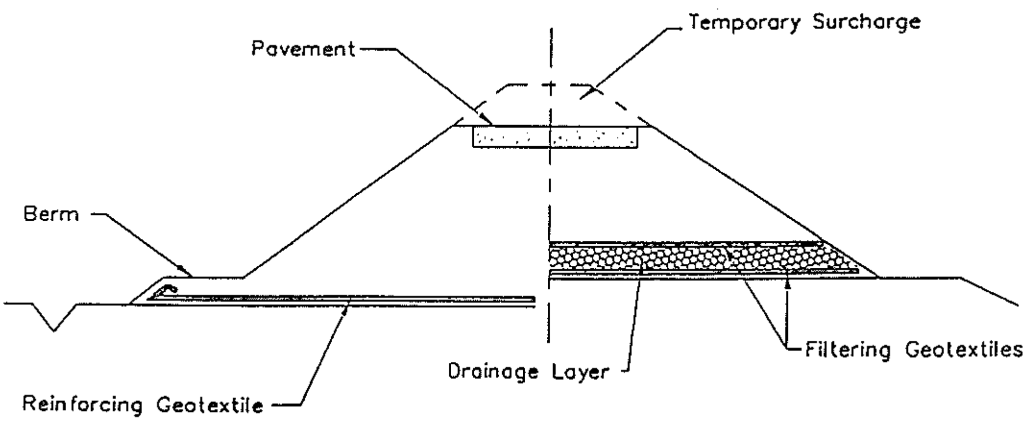


FIGURE 14.1
Use Of Geotextiles In Embankment Construction (ICI Fibres 1986)

In the separation and filter layer use (Figure 14.1(b)), the granular drainage layer allows relatively quick release of excess pore water pressures from the foundation soil. If the geotextile is considered only as a separator and filter, and not to contribute structurally to the design of the embankment, then the properties of importance are the same as those required for permanent road pavements shown in Chapter 12, Figure 12.1. In addition, elongation to break needs consideration to allow for consolidation of the foundation.

For embankments to be constructed quickly to their full height on soft foundation soils, techniques are required to ensure stability. This may be achieved using a geotextile as a reinforcing layer (Figure 14.1(a)) at the base of the embankment. Usually the critical time for embankment stability and the time of maximum geotextile stress are during or immediately after construction of the embankment. Later the foundation material consolidates and gains in strength. This reduces or eliminates the stresses in the geotextile eventually, hence the design life of the geotextile may well be much less than the design life of the embankment, and creep less of a problem. The rate of consolidation of the foundation material can be increased by the use of sand drains or geotextile-wrapped waffle core tubes pushed into the foundation to allow more rapid water drainage. Long-term consolidation is likely to result in deformation and settling of the embankment and roadway.

Ruddock (1977) reports that a road built over peat using fabric formed waves of 100 to 125mm depth after 18 months of main road traffic and, while the waves may be smoother and more regular than if the fabric had not been used, the road will require shape correction in some years time. However, use of the fabric in that instance was considerably cheaper than complete excavation and removal of the peat.

Design methods exist for the use of strong high-modulus geotextiles to enhance the stability of these poor foundation soils until they consolidate, or to enhance stability of embankments constructed over these soils. These methods are outside the scope of this report but some relevant references are listed at the end of this Chapter in Section 14.8, for additional reading.

The best known method for design of reinforced embankments is the limit equilibrium analysis method used by the US Corps of Engineers (ICI Fibres 1986). This method considers five aspects of embankment stability, i.e. bearing capacity, embankment sliding laterally, rotational slips of embankment, excessive deformation of the geotextile, and curvature of the embankment in the longitudinal direction.

14.2.1 Bearing Capacity

The overall bearing capacity of the embankment and foundation must be satisfactory. For unreinforced embankments constructed on soft foundation soils, the critical bearing capacity is based on localised bearing stability. For embankments reinforced at the base with geotextiles, the mode of bearing failure is changed from localised to overall bearing failure mode. This change affords an improved bearing capability (ICI Fibres 1986) somewhat similar to that described in Section 11.4 for temporary road design.

14.2.2 Embankment Sliding

For unreinforced as well as reinforced embankments, the driving forces result from the lateral earth pressures exerted by the embankment which, for equilibrium, must be transferred to the foundation soils by shearing stresses. For reinforced embankments the reinforcement must have sufficient frictional resistance to resist sliding on the geotextile, and the tensile strength of the geotextile must be sufficient to resist tearing (ICI Fibres 1986). This failure condition is illustrated in Figure 14.2.

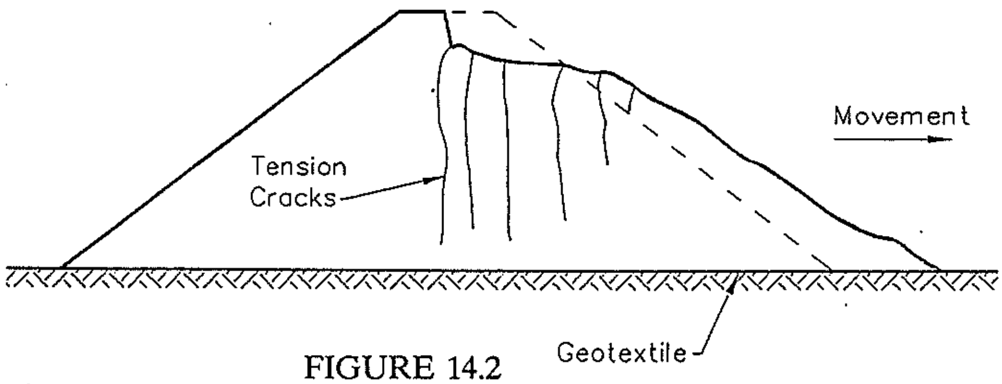


FIGURE 14.2
Embankment Sliding (ICI Fibres 1986)

14.2.3 Rotational Slip

Once the overall bearing capacity of the embankment has been assessed as satisfactory, then the stability of the edge of the embankment should be analysed. Rotational slip failure is shown in Figure 14.3. For stability the tensile strength of the reinforcement must be sufficiently high to resist rupture at the intersection of the potential failure surface (ICI Fibres 1986).

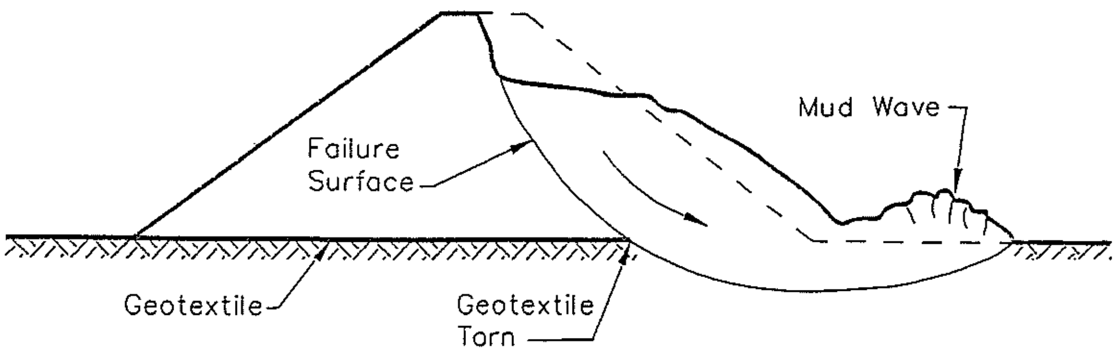


FIGURE 14.3
Rotational Slip (ICI Fibres 1986)

14.2.4 Excessive Elongation Of Geotextile

To develop the tensile forces in the geotextile requires straining of the geotextile, so some movement of the embankment must occur. Limitations on the geotextile stress/strain properties are required to control elongation and so prevent the possible failure mode shown in Figure 14.4.

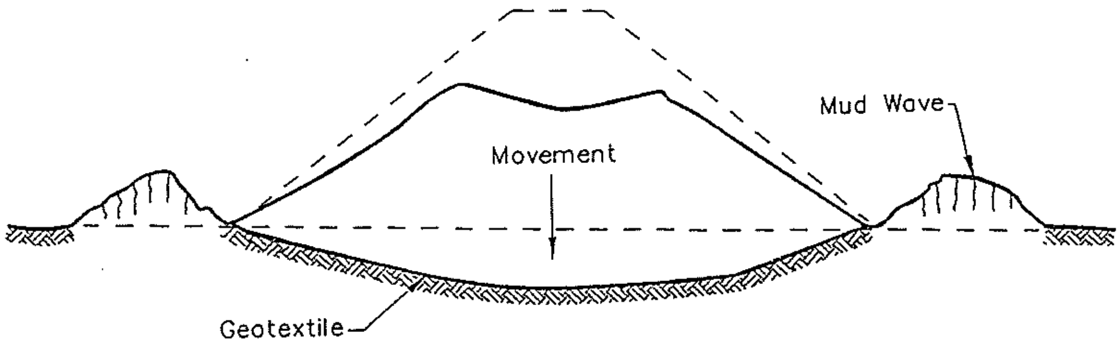


FIGURE 14.4
Excessive Elongation Of Geotextile (ICI Fibres 1986)

14.2.5 Curvature Of Embankment In Longitudinal Direction

All the four preceding aspects of stability on weak foundations dealt with the perpendicular to the direction of the embankment, which is the direction where maximum stresses occur. ICI Fibres (1986) recommended that the geotextile strength in the longitudinal direction should be at least 25% of the geotextile strength required in the direction across the width of the embankment.

14.3 CONCRETE BLOCK OVERLAYS

Existing portland cement concrete pavements that exhibit substantial spalling and cracking may still possess significant structural strength yet require resurfacing to improve the ride for vehicles, etc. One technique that has been used at container terminals is to overlay with interlocking concrete blocks laid on a thin sand bedding. There is a risk of this sand being washed into cracks in the existing underlying concrete. Geotextiles laid over the existing concrete and beneath the sand and blocks have been used to prevent this sand loss.

However, geotextiles used in this manner may be subjected to quite severe abrasion, particularly if the bedding sand has sharp particles (Knapton 1988a). The geotextile should be selected accordingly.

14.4 GEOTEXTILES/GEOMEMBRANES AND MESL*

In the majority of cases where geotextiles are used in pavements, free movement of liquid water is allowed and encouraged. There are, however, cases where the prevention of moisture movement is important for the performance of the pavement.

Saturation of a geotextile with a waterproofing agent such as bitumen can give a "waterproof" membrane with the strength of geotextile. These membranes however can allow movement of water vapour, i.e. they are not totally waterproof.

The rate at which water vapour migrates through a geomembrane is called its **membrane permeance.** Lawson and Ingles (1984) from tests have shown the relationship between bitumen retention and permeance when using bitumen on an 140 g/m^2 melt-bonded non-woven geotextile (Figure 14.5).

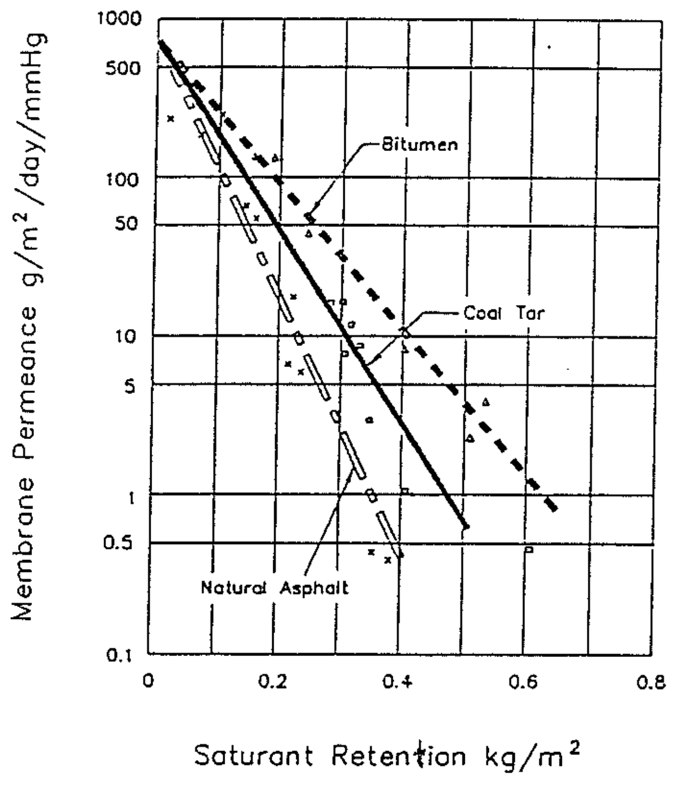


FIGURE 14.5
Relationship between Membrane Permeance and Bitumen Retention for 140 g/m^2 Heat Bonded Non-woven Geotextile (Lawson and Ingles 1984)

*(Membrane Encapsulated Soil Layer)

One of the uses of a geomembrane is as a moisture cut-off on either side of a pavement to maintain a relatively constant moisture content in the subgrade below the pavement. This technique has proved very effective in controlling volume changes in expansive clay subgrades.

Another use of geomembranes is to lay them under the entire pavement to prevent the rise of moisture into the basecourse layer. In frost-susceptible soils the purpose is to prevent the rise of moisture into the pavement, and so prevent frost heave.

A third use of a geomembrane is to completely encapsulate a soil layer to protect it from excessive moisture changes. This technique is known as "Membrane Encapsulated Soil Layer" (MESL), and has been used successfully to stabilise frost-susceptible soils, to control expansive clay subgrades, to use sub-standard basecourse by keeping it dry, and as protection against flooding. This technique could be attractive in "papa" (e.g. sand stone) country as it would allow dry papa to be encapsulated for use as a sub-base.

14.5 FROST

Frost action can cause severe pavement damage due to formation of ice lenses and frost heave. In the spring, downward melting of ice lenses causes a super-saturated condition in the pavement layers and subgrade as the underlying, as yet unmelted, ice impedes drainage. As well, roadways are then prone to severe traffic-induced rutting, because subgrade material may be pumped into the base or to the surface, and on unsealed roads the surface aggregate may be pushed into the base or subgrade.

Results (Hoover *et al.* 1981) of a three-year laboratory and field evaluation in Iowa suggest:

- The geotextile appeared not to provide a subgrade temperature insulating effect.

- Benkleman Beam deflections and plate-bearing tests indicate that a geotextile provides beneficial reinforcement during spring when the geotextile is placed on top of a frost-susceptible subgrade. However beam deflections were still higher than considered acceptable according to several methods of flexible pavement design.

- The best location for the geotextile was between the aggregate base/sub-base and the frost-prone subgrade.

- Use of a geotextile did not appear worthwhile above a non-frost-susceptible subgrade, or directly under a bituminous mix.

- The geotextile may have reduced frost heave and associated distress but this is not clear. On at least some of the sections with a geotextile, crocodile cracking, rutting, shoving, boil, and heave were apparent.

- No evidence was obtained to suggest the geotextile prevented migration of fine soil particles.

This last conclusion could result from the non-woven geotextile having pore sizes too large to contain the soil particles, particularly in the likely alternating flow conditions. Andersson (1977) reports from a trial in Stockholm, Sweden, under freeze-thaw conditions, that the geotextile between the sub-base and subgrade had a considerable filtering effect and prevented the migration of fine particles.

Field tests over two winters in Sweden were carried out to evaluate the benefits of using geotextiles to reduce frost effects on pavement repairs in unsealed roads (Lindh 1979). The geotextile was placed on top of an existing pavement made of frost-susceptible gravels, over which 100mm of gravel was laid. After two thaws the geotextile had not shown any decisive influence.

14.6 IN-PLANE DRAINAGE

Thick non-woven geotextiles are capable of providing drainage within their plane. Given adequate crossfalls this suggests they will provide drainage for the pavement layers even if the subgrade, sub-base, or basecourse have low permeabilities. While some geotextiles will provide drainage, many are compressible under a surcharge (such as a pavement and traffic) which can vastly reduce their in-plane permeability. Accordingly care is needed in selecting a geotextile for in-plane drainage.

14.7 CRACK-SEALING STRIPS

Self-adhesive strips are available to seal cracks in an existing pavement. Typically these are 250mm-wide strips of geotextile impregnated with some type of rubberised bitumen.

14.8 REFERENCES

This list includes additional reading.

Andersson, O. 1977. The use of plastic fabric for pavement protection during frost break. *International Conference on the Use of Fabrics in Geotechnics,* Paris.

Bonaparte, R., Holtz, R.D., Giroud, J.P. 1987. Soil reinforcement design using geotextiles and geogrids. Pp. 69-116 in *Geotextile testing and the design engineer,* J.E. Fluet Jr (Ed.). *American Society for Testing and Materials STP* 952. 183p.

Brakel, J. et al. 1982. Stability of slopes constructed with polyester reinforced fabric, test section at Almere-Holland 79. *Proceedings of Second International Conference on Geotextiles,* Las Vegas, USA.

Broms, B.B. 1987. Stabilisation of very soft clay using geofabric. *Geotextiles and Geomembranes* 5(1): 17-28.

Chemie Linz 1986. *Polyfelt TS. Design and practice.* Chemie Linz AG, Austria.

Chemie Linz 1987. *Geotextile World No. 3.* Chemie Linz AG, Austria.

Eckman, B. 1987. *Reflective cracking studies at Liege University.* Esso SAF.

Fabian, K., Fourie, A. 1986. Performance of geotextile-reinforced clay samples in undrained triaxial tests. *Geotextiles and Geomembranes* 4(1): 53-63.

Fowler, J. 1982. Theoretical design considerations for fabric reinforced embankments. *Proceedings of Second International Conference on Geotextiles,* Las Vegas, USA.

Hannon, J. 1982. Fabrics support embankment construction over bay mud. *Proceedings of Second International Conference on Geotextiles*, Las Vegas, USA.

Hoover, J.M., Pitt, J.M., Handfelt, L.D., Stanley, R.L. 1981. Performance of soil-aggregate-fabric systems in frost-susceptible roads, Linn County, Iowa. *Transportation Research Record* 827. Transportation Research Board, Washington DC.

Houghton, L.D. 1988. Personal communication. Lower Hutt City Council, April 1988.

ICI Fibres 1986. Geotextiles or geosynthetics? *Proceedings of Third Asian ICI Fibres Geotextiles Conference,* Thailand.

Ingold, T.S. 1986. Analysis of geotextile reinforced embankments over soft clays. *Highways and Transportation* 33(3): 3-8.

Jewell, R.A. 1982. A limit equilibrium design method for reinforced embankments on soft foundations. *Proceedings of Second International Conference on Geotextiles,* Las Vegas, USA.

Knapton, J. 1988a. A review of concrete block pavements at ports in the UK. *Proceedings of Third International Conference on Interlocking Concrete Block Paving,* Rome.

Knapton, J. 1988b. Pavement rehabilitation using concrete blocks. *Proceedings of Third International Conference on Interlocking Concrete Block Paving,* Rome.

Koerner, R.M. 1986. *Designing with geosynthetics.* Prentice-Hall, Englewood Cliffs, NJ.

Lawson, C.R., Ingles, O.G. 1984. Water repellency requirements for geomembranes. *Proceedings of International Conference on Geomembranes,* Denver, USA.

Lindh, E. 1979. Frost heave repair with fibre fabric as a separating layer between materials, field experiments in C- and Y- County. *National Road Administration, Sweden, Bulletin* 276.

Lord, A.E., Koerner, R.M. 1986. Diffusion of water from soils encapsulated by impregnated geotextiles (MESLs). *Geotextiles and Geomembranes* 3(1): 3-27.

Luipen, C., Lefebvre, G., Rosenberg, P., Paré, J.J., Lavallé, J.G. 1984. Use of fabrics for improving the placement of till on peat foundation. *Transportation Research Record* 916: 54-59. Transportation Research Board, Washington DC.

Perfetti, J., Sangster, T. 1989. The function of geotextiles in pavement structures. *Proceedings of Geosynthetics Conference,* San Diego, USA.

Road Traffic Authority (RTA) 1989. *Proposal for the Bourke ALF trial.* Roads and Traffic Authority of NSW, Australia.

Rowe, R.K., Soderman, K.L. 1986. Reinforced embankments on very poor foundations. *Geotextiles and Geomembranes* 1(4): 65-81.

Ruddock, E.C. 1977. Fabrics and meshes in roads and other pavements : a state of the art review. *CIRIA Technical Note* 87. London.

Valsangkar, A.J., Holm, T.A. 1987. Model tests on peat-geotextile-lightweight aggregate system. *Geotextiles and Geomembranes* 5(4): 251-260.

Walsh, I.D. 1986. Geotextiles in highway surface dressing. *Proceedings of Third International Conference on Geotextiles,* Vienna, Austria.

Watari, Y., Higuchi, Y. 1986. Behavior and analysis of geotextiles used on very soft ground for earth filling works. *Geotextiles and Geomembranes* 4(3 & 4): 179-189.

Yamauchi, H., Kitamori, I. 1985. Improvement of soft ground bearing capacity using synthetic meshes. *Geotextiles and Geomembranes* 2(1): 3-22.