RR 88 / 029

SUBSURFACE DRAINAGE

Prepared on behalf of the SOUTH AFRICAN ROADS BOARD

TITEL/TITLE: SUBSURFACE DRAINAGE.			
VERSLAG NR: **REPORT NO:** RR 88/29	**ISBN:** 1-86844-223-3	**DATUM:** **DATE:** September 1993	**VERSLAGSTATUS:** **REPORT STATUS:** Final
NOAK NR/RDAC NO: 88/29			
GEDOEN DEUR: **CARRIED OUT BY:** Division of Roads and Transport Technology, CSIR P O Box 395 Pretoria 0001		**OPDRAGGEWER:** **COMMISSIONED BY:** Director General: Transport Private Bag X193 PRETORIA 0001	
OUTEUR(S): **AUTHOR(S):** C J van der Merwe		**NAVRAE:** **ENQUIRIES:** Department of Transport Directorate: Transport Economic Analysis Private Bag X193 PRETORIA 0001	
SINOPSIS: Die verslag beskryf die bevindings van die navorsingsprojek oor ondergrondse dreinering. Die belangrikheid van dreinering is gekwantifiseer deur 'n evaluering van SVN toetse op nat plaveisels. Geotekstiele is ondersoek en verskeie toetse is uitgevoer. Grond/geotekstiel versoenbaarheid en langtermyngedrag is ondersoek. Laboratoriumtoetse en voorlopige kriteria word voorgestel om die geskiktheid van geotekstiele te bepaal. Die gebruik van findreins is in die laboratorium ondersoek en 'n veldstudie is begin om die effektiwiteit van dreineringstelsels te moniteer.		**SYNOPSIS:** The report describes the findings of the research project on subsurface drainage. The need for drainage was quantified by evaluating HVS tests on wet pavements. Geotextiles were investigated and various tests performed. Soil/geotextile compatibility and long term behaviour were evaluated. Laboratory tests for geotextile selection are recommended and tentative criteria given. The use of fin drains was evaluated in the laboratory and a field study to monitor the efficacy of drainage systems was started.	
TREFWOORDE: Ondergrondse dreinering, sintetiese materiale, geotekstiele, findrein. **KEYWORDS:** Subsurface drainage, geosynthetics, geotextiles, fin drains.			
KOPIEREG **COPYRIGHT** Departement van Vervoer, behalwe vir verwysingsdoeleindes Department of Transport, except for reference purposes			**VERSLAGKOSTE** **REPORT COST** **R60.00**

DISCLAIMER

The views and opinions expressed in this report are those of the author and do not represent South African Roads Board Policy.

SOUTH AFRICAN ROADS BOARD RESOLUTION

This report has been approved for general distribution by the South African Roads Board.

REVIEWED BY

P J Strauss
A Taute

LIST OF CONTENTS

SUMMARY

The damaging effect that water has on road pavements is well-known and has been illustrated with Heavy Vehicle Simulator (HVS) tests in South Africa. These effects are described and the severity of water-related pavement deterioration is quantified for various pavement types. Based on this information, the economic justification for effective subsurface drainage is illustrated.

Various subsurface drainage methods can be applied to intercept water originating from various sources. These include impermeable surfacings, drainage layers, side drains, etc. Synthetic materials are being used increasingly in subsurface drainage applications as filters (geotextiles), water carriers (flow nets, pipes, etc.) and impermeable barriers (membranes). The biggest shortcoming of these materials is the lack of standardized test methods, information on long term field performance and proven design criteria.

Geotextiles, the most widely-used filters in drains, were investigated. Existing filter criteria were evaluated and their shortcomings identified. These criteria are based mainly on in-isolation soil and geotextile characteristics obtained from non-standardized test methods and do not consider the long-term performance of geotextiles and soil/geotextile compatibility.

Soil/geotextile compatibility and the long-term behaviour of these materials were investigated using laboratory tests and a field evaluation. Laboratory tests included in-isolation and compatibility tests and for the field evaluation an installed side drain was monitored over a period of three years.

Based on the investigation, standard test methods and geotextile design criteria are recommended. Filter criteria are given that, in addition to the normal piping and permeability requirements, also control permeability reduction over time.

The use of drainage layers in the pavement structure was investigated and one such application evaluated. The structural capacity of the layer was evaluated with the HVS and its hydraulic properties by studying flow patterns obtained with a fluorescent dye technique in the field. Based on this investigation, recommendations are given on the use of drainage layers in the pavement structure.

Other aspects that are briefly discussed include drains in the pavement structure, vegetation, construction and maintenance of subsurface drainage systems.

The use of fin drains was evaluated. A state of the art review was conducted and laboratory studies were performed. A transmissivity tester was developed and three types of fin drains tested at varying lateral pressure and hydraulic gradients.

The durability of geotextiles was investigated by means of a literature survey. It was concluded that long-term field evaluations are best suited to evaluate geotextile durability.

A field study to monitor the efficacy of installed subsurface drainage systems was started. Sites with a variety of materials, designs and success records were identified and selected.

1. INTRODUCTION

This report summarises the research work done to date on the subsurface drainage project. A number of reports have been written during the course of the project and it is the aim of this report to put all the information together in a logical format. The problem statement, objectives, methodology, conclusions and recommendations of the research project are outlined in broad terms. More detailed descriptions are given in the various reports.

The reports are given as appendices and include the following:

- PhD thesis on road subsurface drainage
- Research report on recommended tests and criteria for geotextile selection
- Technical note on fin drains - state of the art review
- Technical note on development of laboratory fin drain apparatus
- RDAC interim report on fin drain laboratory tests
- RDAC interim report on geotextile durability - literature survey
- RDAC interim report on field monitoring of subsurface drainage systems

The PhD thesis incorporates a number of reports that are not included separately since they are set out in a more logical way in the thesis.

2. PROBLEM STATEMENT

The entire road infrastructure, both existing and future, will be at risk if adequate drainage is not provided. Both practical experience and experimental work with, for example, the Heavy Vehicle Simulator (HVS) have shown that one of the most important requirements for preventing premature pavement failure is to keep the moisture condition in a state of equilibrium. It is necessary to prevent water from entering the pavement structure and also, in the case of expansive clays, to prevent moisture from migrating out of the structure.

Surface (storm water) and subsurface drainage are interrelated; the effectiveness of each is dependent on the other. Design methods and practices for surface drainage are well-established and their effectiveness is relatively easy to assess. In the field of subsurface drainage, however, numerous questions remain unanswered. This is partly due to the difficulty of evaluating existing systems and also because new materials and systems (e.g. synthetic materials like geotextiles, geonets, fin drains, etc.) have been developed over the past decade and for which design criteria have not yet been established.

Designers of subsurface drainage systems are faced with the problem that they only have the specifications and design methods supplied by the manufacturers of these products on which to base their decisions. The specifications supplied by the manufacturers are not always based on standardized tests and therefore comparisons cannot be made.

The test methods for evaluating geotextiles and drainage systems should be such that they can be standardized and performed in most soil laboratories. The design criteria must be based on these tests, should be verified by adequate research both in the laboratory and in the field and should be relatively easy to implement in practice.

3. OBJECTIVES

The objectives of this project can be summarised as follows: to provide standard tests, design criteria and specifications for geotextiles, drainage systems and related products and also to give general guidelines for subsurface drainage practice.

These objectives include the following:

- collect relevant information from the literature
- evaluate existing design criteria
- develop and manufacture test apparatus and standardize test methods
- set design criteria for geotextiles in road subsurface drainage applications
- evaluate geotextiles as filters, as opposed granular filters
- investigate the durability of geotextiles
- investigate the use of fin drains as an alternative to conventional side drains
- provide test methods and guidelines for evaluating fin drains
- investigate the efficacy of installed subsurface drainage systems
- determine suitable designs for drainage systems, e.g. widths, depths, outlets.

4. METHODOLOGY

The investigation included literature surveys, laboratory tests and field tests as follows:-

Literature surveys:

- the damaging effect of water on road pavements
- general overview of geosynthetics and geotextiles
- filter criteria for geotextiles
- soil/geotextile compatibility tests
- geotextile durability
- fin drains

Laboratory tests:

- Geotextiles:
  - identification tests (mass, thickness)
  - permeabilities
  - pore sizes
  - tensile strength
  - penetration resistance
  - soil/geotextile compatibility
- Fin drains.

Field tests:

- Detailed monitoring of drainage system in high water table area
- Initial work on extended monitoring of installed systems
- HVS tests on drainage layer.
- Ad-hoc discussions and site visits.

5. CONCLUSIONS AND RECOMMENDATIONS

The following is a summary of the main conclusions and recommendations that were arrived at. Further information can be obtained in the various appendices as indicated.

		Further information	
		Appendix	*Pages*
5.1	SOURCES OF SUBSURFACE WATER	A	3-6
	Water can enter a pavement structure from various sources and in different ways. For the design of an effective subsurface drainage system, it is necessary that these sources be identified correctly.		
5.2	PAVEMENT BEHAVIOUR	A	17-44
	The way in which water affects pavement behaviour depends on the pavement type.		
	The traffic associated rate of deterioration of a wet pavement is between 2 and 200 times more than that of a dry pavement, depending on pavement type and material quality.	A	44
	The cost of installing a subsurface drainage system is usually justified when water is a definite threat.	A	45
5.3	GEOTEXTILES : LITERATURE SURVEY		
5.3.1	**General overview**		
	Geotextiles and other geosynthetics have developed rapidly over the past 20 years and a large variety is currently available.	A	52-58
	These products perform various functions in a number of applications, which require specific properties.	A	65-70
5.3.2	**Existing filter criteria**		
	Filter criteria for granular filters are based on grain size and include criteria that control piping, permeability and internal stability.	A	71-80
	A large number of filter criteria exist for geotextiles that are based on pore sizes and permeability.	A	80-98
	These criteria are not recommended for general use due to various problems with the test methods, widely differing results obtained from different criteria and a lack of correlation with long term performance.	A A	98-99 234
5.3.3	**Soil/geotextile compatibility tests**		
	A number of researchers have evaluated soil/geotextile compatibility using long term flow tests. The results cannot be directly compared because different apparatus was used, but similar trends were observed.	A	99-117

Appendix	Pages
A	100-102
A	232

The results of long term flow tests have illustrated that the main function of a geotextile in a filter application is to act as a catalyst in the forming of a filter zone in the soil. This results in a reduction in the permeability of the whole system. The final permeability is that of the filter zone in the soil which is less than the permeability of the original soil structure and of the geotextile. The main requirements for such a system to develop are :

- the permeability of the geotextile must be higher than that of the original soil structure;
- the effective pore sizes in the geotextile must be small enough to retain a sufficient proportion of the soil particles to form the bridging zone upstream of the geotextile;
- the pore sizes in the geotextile relative to the grain sizes of the soil must be such that soil particles will not get caught in the pores to such an extent that the geotextile will become clogged and therefore impermeable; and
- good contact between the geotextile and the soil must be maintained to ensure that the bridging and filter zones remain intact.

Appendix	Pages
A	118
A	232

Long term flow tests are recommended for the evaluation of geotextiles because the above mentioned aspects can be adequately evaluated.

5.3.4 Geotextiles durability

Appendix	Pages
F	

Durability of geotextiles is largely determined by the polymers from which they are manufactured. They must be resistant to soil-induced degradation (chemical and biological), sunlight degradation and degradation due to high temperature.

There are no tests available that can be used to quantify the durability of geotextiles rapidly and accurately. Experience that has been gained to date with geotextiles in South Africa must be relied upon. In general the polymers that are most widely used, namely polyester and UV stabilised polypropylene, have proved to be durable in commonly encountered environments.

All polymers are vulnerable to ultraviolet (UV) degradation to some degree, even if the geotextiles are UV stabilised. Exposure to direct sunlight should therefore be kept to a minimum.

5.4 LABORATORY TESTS : IN-ISOLATION GEOTEXTILE TESTS

5.4.1 Pore sizes

Appendix	Pages
A	122-131

Pore size tests were performed to evaluate existing criteria that are based on pore sizes. Pore sizes obtained were different from those quoted by the manufacturers, thus illustrating the sensitivity to different test methods.

5.4.2 Normal permeability

Appendix	Pages
A	131-140

Permeability tests were done using a falling head permeameter. However, a constant head permeameter is recommended for general use.

	Appendix	Pages
5.4.3 Mass per unit area This test is relatively simple to perform and is useful for the identification of geotextiles. There is not a good correlation between mass per unit area and strength.	A	140-141
5.4.4 Thickness and compressibility Geotextile thicknesses reduce considerably under pressure, particularly those of nonwoven needle-punched geotextiles, up to a pressure of about 200 kPa. Geotextile thickness is only important when drainage in the plane (transmissivity) is of importance.	A	141-147
5.4.5 Strip tensile test This is the most accurate and widely used method of testing geotextile strength. However, specialised equipment is required and the strength is only measured in one direction.	A	147-153
5.4.6 Penetration load test This test was found to be a relatively simple and useful test to perform. The test uses modified CBR equipment and simulates failure conditions in the field fairly closely.	A	153-160
5.5 LABORATORY TESTS : SOIL/GEOTEXTILE COMPATIBILITY		
5.5.1 Apparatus and test method An apparatus (permeameter) and a test method were developed to evaluate soil geotextile compatibility with long term testing. The apparatus was evaluated for repeatability and the most applicable parameters were selected.	A	170-180
5.5.2 Materials tested and results A variety of soil types and a number of filters (geotextiles as well as granular filters) were tested. A general pattern was observed of decreasing permeability with time for all filters including granular filters. This confirmed the theory of a filter zone in the soil.	A	181-193
5.5.3 Parameters to quantify flow test Various parameters were evaluated that could be used to quantify geotextile performance as determined with the flow test. A parameter termed K400 was found to be the most suitable. K400 relates the permeability of the system after 400 hours of testing to that at the beginning of the test.	A	197-206

		Appendix	Pages
5.5.4	**The influence of soil parameters on flow test results**	A	206-216
	The performance of soil/geotextile systems as quantified by the flow test were evaluated as a function of the soil parameters. It was found that the percentage fines (passing 0,075 mm sieve) had a definite influence on the degree of permeability reduction. However, this parameter alone could not predict the final reduction as this was also determined by geotextile characteristics.		
5.5.5	**The influence of geotextile parameters on flow test results**	A	217-224
	The influence of the measured geotextile parameters, normal permeability and pore sizes, on soil/geotextile compatibility was evaluated. It was found that none of these parameters could predict the long term performance of geotextiles.		
5.5.6	**Interface flow capacity (IFC) tests**	A	224-230
	IFC tests (also known as fine fraction filtration) were performed using the same materials that were used for the flow tests. No correlations could be found between the results of IFC tests and those of long term flow tests.		
5.6	FIELD INVESTIGATIONS		
	One subsurface drainage field installation was instrumented and monitored. The outflow and water table were recorded and the change in permeability of the system monitored over a period of 3 years.	A	237-247
	The decrease in permeability was found to be similar to that observed in the laboratory. The laboratory parameter K400 was found to be a satisfactory parameter for the evaluation of soil/geotextile compatibility.	A	247-254
	A number of sites with subsurface drainage systems were identified for future monitoring. The sites have a variety of systems incorporating various geotextiles, pipes, geonets, and granular filters. These will be monitored to evaluate the long term performance of various systems.	G	
	A number of reported failures of subsurface drainage systems were investigated through discussions with persons that had been involved. It was concluded that the majority of failures was due to either poor construction techniques or inadequate maintenance. Particular attention should be paid to testing of the system after construction by pumping water through the pipe system. The outlets should also be maintained properly to ensure an undisturbed outflow for the full design life.	A	311-312
5.7	RECOMMENDED TESTS AND SELECTION CRITERIA		
	Filter criteria that control piping, permeability and permeability reduction were developed using the results from the laboratory and field investigations. It is recommended that flow tests be used to control all three parameters. It is further recommended that the test also be used to evaluate the permeability reduction of granular filters.	A	258-276

		Appendix	Pages
	Tests recommended for the evaluation and selection of geotextiles for subsurface drainage installations are : - Mass per unit area - Constant head permeability - Penetration load - Long term flow (permeameter).	B	
	Tentative selection criteria were developed and are given.	B	20-22
5.8	DRAINAGE LAYER	A	277-298
	The use of a drainage layer in the pavement structure was evaluated. The layer was an open graded crushed stone subbase. It was found that the layer was sufficiently permeable to remove any water entering through the surfacing. However, Heavy Vehicle Simulator (HVS) tests showed that the structural strength of the layer was inadequate. Densification of the layer occurred as well as densification of the overlying crushed stone base, resulting in rutting at the surface. For a drainage layer to succeed both structurally and hydraulically, a material must be obtained that has high permeability and which is not prone to densification under traffic. Possibilities include cement stabilised open-graded materials and water bound macadams.		
5.9	FIN DRAINS		
	The cost and installation of fin drains was investigated. These drains contain synthetic cores or water carriers instead of a granular material, thus resulting in a smaller dimension. Fin drains were found to be easier to install and cheaper than conventional drains.	A C	302-309
	An apparatus was developed to measure flow rates through fin drains under lateral compression.	C D	
	Flow rates of various fin drain cores were measured under varying pressures and gradients. The following flows were measured for three types of cores under 20 kPa lateral pressure and a hydraulic gradient of 0,3 : 5 mm geonet : 0,2 l/s 10 mm cuspated core : 0,8 l/s 37 mm cuspated core : 9,0 l/s	E	

6. RECOMMENDATIONS FOR FURTHER RESEARCH

Certain areas were identified where further research is required. The areas that were identified include:

- Investigation of pore pressures in different pavement types, under different conditions and using different methods of water introduction with HVS tests and also in roads where natural wet conditions exist. Such an investigation will provide a clearer understanding of failure mechanisms and also give an indication of the relative severity of the method of water introduction used with HVS tests.

- Field studies of pavement performance under wet and dry conditions to verify severity factors obtained from HVS tests.

- Field investigations of subsurface drainage systems to obtain acceptable limits for geotextile strength, piping and permeability reduction criteria.

- Evaluation of alternative materials for drainage layers, e.g. waterbound macadams and open-graded cemented materials.

- Field investigations to evaluate various aspects of subsurface drainage systems, including designs (widths, outlets, etc.), types of drains (e.g. fin, conventional), generic types of geotextiles (e.g. woven, nonwoven) and obtain acceptable limits for geotextile strength, piping and permeability reduction criteria.

- A general analytical model to analyse flow systems needs to be developed using numerical methods such as finite elements.

APPENDIX A

SOME ASPECTS OF ROAD SUBSURFACE DRAINAGE IN SOUTH AFRICA.

PHD THESIS, UNIVERSITY OF PRETORIA

SOME ASPECTS OF
ROAD SUBSURFACE DRAINAGE IN SOUTH AFRICA

CHRISTIAAN JOHANNES VAN DER MERWE

A dissertation submitted as part fulfilment for the degree

Doctor of Philosophy
in the Department of Civil Engineering
Faculty of Engineering
University of Pretoria

Pretoria

1988

SUMMARY

The damaging effect that water has on road pavements is well known and has been illustrated with Heavy Vehicle Simulator (HVS) tests in South Africa. These effects are described and the severity of water related pavement deterioration is quantified for various pavement types. Based on this information the economic justification for effective subsurface drainage is illustrated.

Various subsurface drainage methods can be applied to intercept water originating from various sources. These include impermeable surfacings, drainage layers, side drains, etc. Synthetic materials are being used increasingly in subsurface drainage applications as filters (geotextiles), water carriers (flow nets, pipes, etc) and impermeable barriers (membranes). The biggest shortcoming of these materials is the lack of standardized test methods, information of long term field performance and proven design criteria.

Geotextiles, the most widely used filters in drains, were investigated. Existing filter criteria were evaluated and their shortcomings identified. These criteria are based mainly on in-isolation soil and geotextile characteristics obtained from non-standardized test methods and do not consider the long term performance of geotextiles and soil/geotextile compatibility.

Soil/geotextile compatibility and the long term behaviour of these materials were investigated using laboratory tests and a field evaluation. Laboratory tests included in-isolation and compatibility tests and for the field evaluation an installed side drain was monitored over a period of three years.

Based on the investigation, standard test methods and geotextile design criteria are recommended. Filter criteria are given that, in addition to the normal piping and permeability requirements, also control permeability reduction over time.

The use of drainage layers in the pavement structure was investigated and one such application evaluated. The structural capacity of the layer

was evaluated with the HVS and its hydraulic properties by studying flow patterns obtained with a fluorescent dye technique in the field. Based on this investigation, recommendations are given on the use of drainage layers in the pavement structure.

Other aspects that are briefly discussed include fin drains, drains in the pavement structure, vegetation, construction and maintenance of subsurface drainage systems.

Finally, recommendations are made regarding areas needing further research.

SAMEVATTING

Die skadelike invloed van water op padplaveisels is wel bekend en is by verskeie geleenthede in Suid-Afrika met behulp van Swaarvoertuig-nabootser (SVN) toetse geïllustreer. Hierdie invloed word beskryf en gekwantifiseer vir verskeie tipes plaveiselstrukture. Die ekonomiese regverdiging van effektiewe ondergrondse dreinering word met behulp van hierdie inligting aangetoon.

Afhangende van die bron van ondergrondse water, kan verskeie metodes aangewend word om hierdie water te dreineer, onder andere ondeurlaatbare oppervlaktes, dreineringslae, sugriole, ens. Sintetiese materiale word toenemend gebruik in ondergrondse dreineringstoepassings as filters (geotekstiele), waterdraers (vloeinette, pype, ens) en ondeurlaatbare membrane. Die grootste tekortkoming van hierdie materiale is die gebrek aan gestandaardiseerde toetsmetodes, inligting oor langtermyn gedrag en beproefde ontwerpmetodes.

Geotekstiele, wat meestal as filters in dreineringstoepassings gebruik word, is ondersoek. Bestaande filter ontwerpmetodes vir hierdie materiale is geëvalueer en tekortkominge geïdentifiseer. Hierdie metodes is meestal gebaseer op toetse wat afsonderlik op die geotekstiel en die grond uitgevoer word sonder inagneming van die grond/geotekstiel versoenbaarheid en die langtermyn gedrag van hierdie materiale.

Grond/geotekstiel versoenbaarheid en langtermyn gedrag is deur middel van laboratorium- sowel as veldtoetse ondersoek. Afsonderlike toetse asook versoenbaarheidstoetse is uitgevoer in die laboratorium en vir die veldondersoek is 'n geïnstalleerde sugriool oor 'n tydperk van drie jaar gemonitor.

Toetsmetodes en geotekstiel ontwerpmetodes word aanbeveel, op grond van die bevindinge. Filter ontwerpmetodes word aanbeveel wat pypvorming, deurlaatbaarheid en afname in deurlaatbaarheid met tyd beheer.

Die gebruik van dreineringslae in die plaveiselstruktuur is ondersoek en een sodanige plaveisel is geëvalueer. Die strukturele vermoë van die

plaveisel is met behulp van die SVN geëvalueer en die hidrouliese eienskappe van die dreineringslaag deur middel van vloeipatrone wat in die veld verkry is met fluoresserende kleurstof. Aanbevelings word gemaak oor die gebruik van dreineringslae in die plaveiselstruktuur.

Ander aspekte wat kortliks bespreek word sluit in die sogenaamde vin-drein, sugriole in die plaveiselstruktuur, plantegroei, konstruksie en onderhoud van ondergrondse dreinering.

Ten slotte word aanbevelings gedoen vir verdere navorsing.

ACKNOWLEDGEMENTS

I would like to thank the following persons and organizations for their aid and contributions:

Dr C R Freeme, Director of the Division of Roads and Transport Technology (DRTT) of the CSIR and his predecessors, Dr S H Kühn and Mr R N Walker, for the opportunity to perform the research work and for their permission to use the research findings in writing this thesis.

The various South African road authorities for their funding, support and assistance and for their willingness to implement the research findings.

Prof P F Savage, Dr L Maree and Dr A J van Wyk who supervised this thesis.

All my supervisors and colleagues at the DRTT for their support and assistance.

All those persons, both DRTT colleagues and others, who helped with the drawings and typing of this thesis.

My wife and child(ren) for their encouragement and support.

CONTENTS

CHAPTER 1

INTRODUCTION, SCOPE AND OBJECTIVES

CHAPTER 1

CONTENTS PAGE

1.1 INTRODUCTION

Drainage, according to the Oxford English Dictionary, is to leave dry by the withdrawal of water or moisture. Subsurface, according to the same dictionary, is that which lies immediately below the surface.

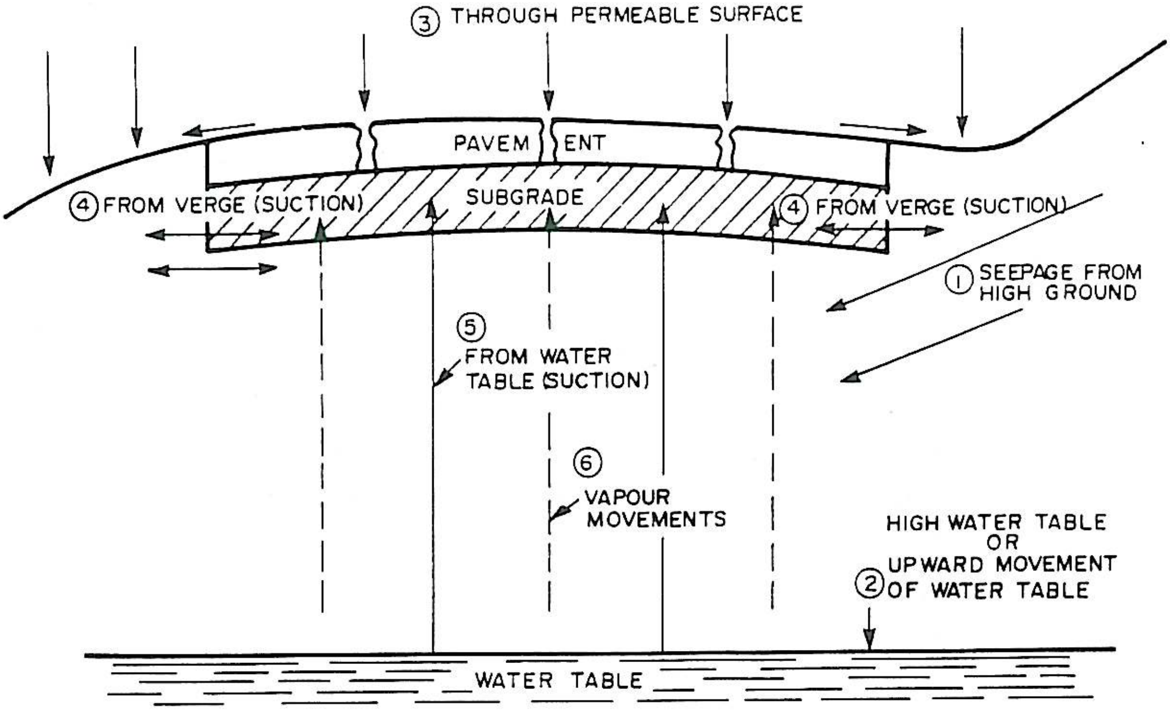
Road drainage, or that which is applied to keep a road dry, can be subdivided into two main areas, namely surface (or stormwater) drainage and subsurface drainage. Surface drainage is aimed at removing rain water in the shortest possible time and includes the design of culverts, pipes etc using hydrological inputs. The geometric design of roads is also usually included under surface drainage.

Subsurface drainage is applied to drain the water that is not removed by surface drainage or by the natural processes of evaporation and plant transpiration. This includes water that infiltrates the road pavement directly as well as that which infiltrates the ground elsewhere and reaches the pavement as ground water.

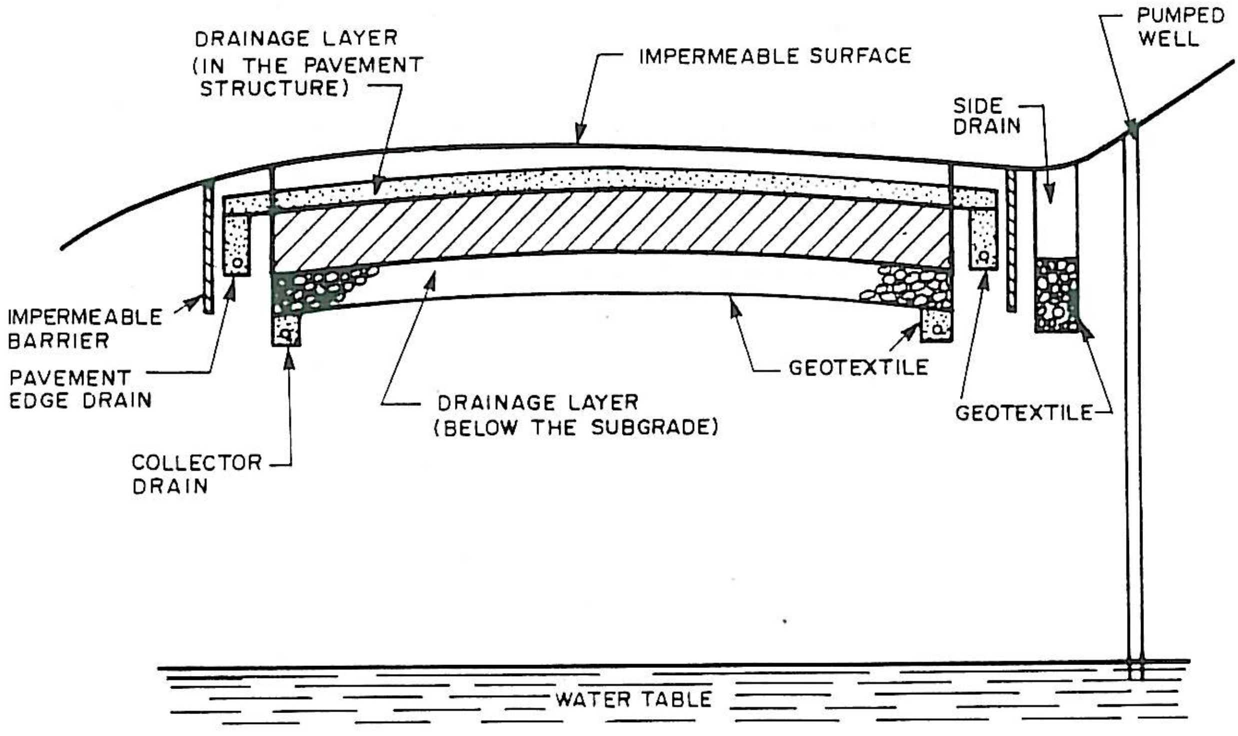
It follows then that, although surface and subsurface drainage are usually studied, designed and constructed separately, they are interrelated and an effective subsurface drainage system is dependant on adequate surface drainage.

The work described in this thesis deals with aspects related to subsurface drainage, but the damaging effect that water has on pavement structures, which is also described, can be as a result of either surface or subsurface drainage inadequacies. Furthermore, aspects dealt with in this thesis are only those related to the pavement and pavement surround, thus excluding drainage of embankments and cut-slopes (eg settlement and stability of embankments).

The ways in which water can enter the road pavement are shown in Figure 1.1 (a) (TRRL, 1952), and some measures that can be applied



(a) WAYS IN WHICH WATER CAN ENTER THE PAVEMENT STRUCTURE (TRRL, 1952)



(b) METHODS OF SUBSURFACE DRAINAGE

FIGURE 1.1 *ASPECTS OF ROAD SUBSURFACE DRAINAGE* to drain the water from these sources, in Figure 1.1 (b). The following is a brief description of each of these sources and of possible subsurface drainage methods.

(1) Seepage from high ground

This usually occurs when a layer of permeable soil overlies an impermeable layer, is common in hilly terrain and can be expected to occur in cuts. Seepage from this source can be continuous or intermittent, occurring only after a rain storm. Side drains are usually installed to intercept seepage from high ground.

(2) High water tables

High water tables are not very common in South Africa, but perched water tables do occur. These are caused by impervious banks in an otherwise permeable soil structure. Water that is en-route to the natural water table (usually 10m or more below the surface) is contained by such a bank to form an isolated, perched water table. Depending on the depth of such a water table, its presence can be a permanent threat to the pavement or only in the rainy season when the upward movement of the water table poses a threat. The water table can be permanently lowered with side drains or pumped wells or, if the threat is less severe, the pavement can be protected by a drainage layer beneath the subgrade which is supplied with collector drains.

(3) Permeable surface

The amount of rain water that will infiltrate the pavement through the surface depends on, amongst other things, the extent to which the surface is permeable (caused by cracks and joints) and the effectiveness of the surface drainage (gradients etc). This source of water can therefore be counteracted by effective surface drainage, an impermeable surfacing or a drainage layer in the pavement structure to remove water that has infiltrated through the surface.

Pavement edge drains can be installed to remove water from the drainage layer.

(4) Verge

Differences in moisture content between the subgrade and the verge usually occur when the shoulder is unsurfaced. This results in moisture movement between the subgrade and the verge which, in the case of clay subgrades, causes swell with an increase and shrinkage with a decrease in moisture content. Pavement edge drains or impermeable barriers can be installed to prevent moisture movement in either direction.

(5) Suction from water table

In fine grained materials, capillary forces can result in a negative head which is larger than the positive gravitational head and the water will rise faster than it will fall. This usually occurs with heavy clays at low moisture contents. If this is expected to occur (eg with a high water table) a drainage layer below the subgrade can be constructed.

(6) Vapour movements

Vapour movements occur when there are differences in vapour pressure which arise either from moisture content differences or from temperature gradients. Vapour movements also occur at low moisture contents. A drainage layer beneath the subgrade can be applied to intercept moisture from this source.

1.2 SCOPE

Aspects dealt with in this thesis include:

- The need for road drainage as demonstrated by the damaging effect that water has on road pavements;

- Side drains. These drains consist of a filter and water carriers. Granular filters are sometimes used but since the development of geotextiles, these synthetic materials are the most commonly used. In recent times the granular water carriers and the traditional perforated pipes have also been replaced with synthetic materials in the so-called fin drains;

- Geotextiles, which are used in side drains, pavement edge drains, collector drains and as separators in drainage layers (usually drainage layers below the subgrade). The advantages of these synthetic materials are that they are often cheaper than granular filters, their characteristics and qualities are more homogenous and can be effectively controlled during manufacture, and the installation of geotextiles is usually simpler and cheaper than that of granular filters. The main disadvantage is the lack of information on field performance of these materials, the lack of standardized test methods and the lack of proven design criteria, and

- Drainage layers in the pavement structure as an alternative to ensuring an impermeable surface.

1.3 OBJECTIVES

The main objectives of the work reported on in this thesis were:

- To investigate and quantify the need for road drainage;

- To investigate geotextiles with the following specific aims:

  - to collect relevant information from the literature
  - to evaluate existing design criteria
  - to develop and manufacture test apparatus and standardize test methods for use in the road industry
  - to test available geotextiles using standardized test methods
  - to set design criteria for geotextiles in road subsurface drainage applications;

- To investigate the use of fin drains as an alternative to conventional side drains, and

- To investigate the use of drainage layers in the pavement structure.

CHAPTER 2

THE NEED FOR ROAD DRAINAGE

CHAPTER 2

CONTENTS PAGE

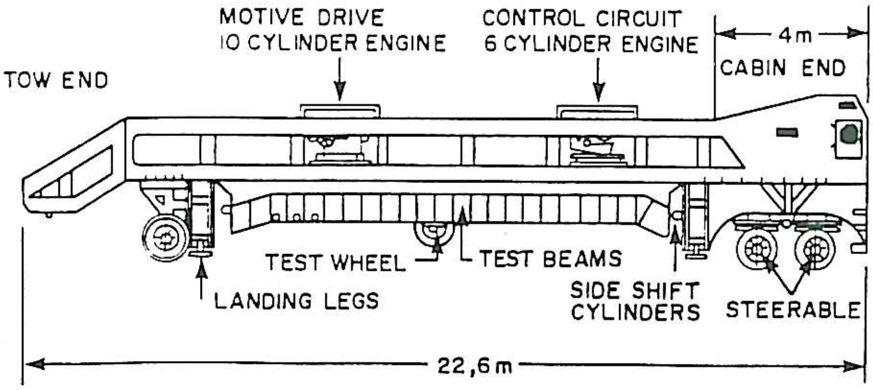
2.1 INTRODUCTION

The road builders of the Roman Empire knew of the damaging effect of water. Their roads, built in 312 BC, were kept above the level of the surrounding terrain and they included a sand layer on top of the subgrade below the first level of flat stones (Cedergren, 1974). As was the case with many other engineering skills, little progress was made in the 20 centuries after the Roman Empire to improve road drainage methods. It was only in the early 19th century that a renewed interest was shown in pavement drainage. Pioneers of that time were Tresaquet, the originator of the "French drain", Thomas Telford and John McAdam (Ridgeway, 1982). McAdam's statement to the London Board of Agriculture in 1820 is well known:

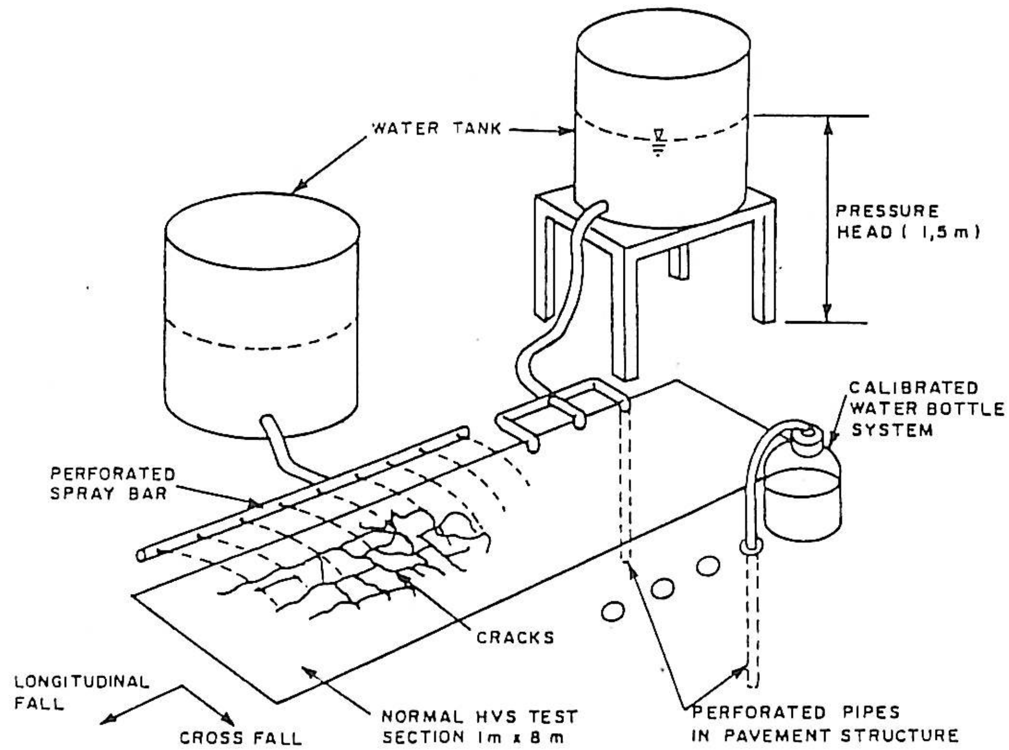
... if (the native soil) is preserved in a dry state it will carry any weight without sinking... if water pass through a road and fill the native soil, the road whatever may be its thickness, loses support and goes to pieces."

For the past one and a half centuries, considerable experience has been gained, both in practice and in research, on road drainage. The most prominent drainage engineer of modern times, Harry R Cedergren, gathered a wealth of information in his oft-referred to books (1967, 1974), where the need for drainage and the mechanisms of pavement failure due to excess water are clearly illustrated on the basis of information that was available in the early 1970's.

Since the 1970's, the understanding of pavement behaviour, both in the dry and in the wet condition, has improved considerably in South Africa with the aid of the fleet of Heavy Vehicle Simulators (HVS's). With the aid of these machines, shown diagrammatically on Figure 2.1(a), traffic loadings are applied at an accelerated rate on selected sections of road. These



(a) SCHEMATIC ILLUSTRATION OF HVS



(b) METHODS USED FOR WATER INTRODUCTION (De Beer et al, 1987)

FIGURE 2.1 *HEAVY VEHICLE SIMULATOR (HVS) TESTS* test sections are instrumented and the pavement performance is monitored while traffic loadings, equivalent to that expected during the entire life of the pavement, are applied.

The effect of water on pavements is evaluated during an HVS test by purposely introducing water into the pavement structure. The water is either introduced from the top by spraying the surface or in depth using perforated pipes that are installed at the side of the test section. For the in-depth method, water is either allowed to flow into the perforated pipes from bottles or is forced in under pressure from a water tank. The different methods used are illustrated in Figure 2.1(b) (De Beer et al, 1987).

HVS tests have been performed on a variety of pavement types and the mechanisms of failure resulting from water in the pavement structure have been identified. These mechanisms and results from selected HVS tests are described in Sections 2.2 to 2.5.

Water alone does not have a detrimental effect on the pavement structure, but the combination of water and traffic loading results in an accelerated rate of pavement deterioration. This is due to the decrease in strength of the materials caused by higher moisture content and, more importantly, due to the development of excess pore water pressures (EPWP) in the water that is trapped in the pavement structure. These pore pressures cause erosion and pumping which lead to pavement failures.

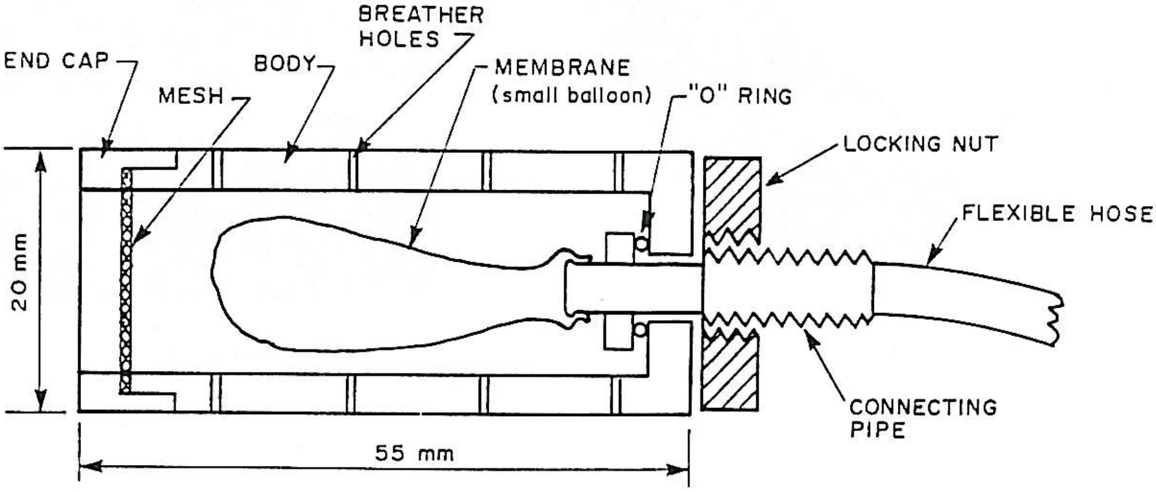
Dempsey (1982) measured pore water pressures, using a small (19mm diameter) piezometer, in a dynamically loaded pavement model in the laboratory and also in a test track investigation. The results from that investigation indicated that excess pore pressures increased with higher loads and also increased as the number of load repetitions increased. Pore pressures of 20kPa were measured in the pavement.

A small pore pressure apparatus was also developed by the author for measuring pore pressures under HVS testing. The apparatus, with an outside diameter of 20mm, is shown on Figure 2.2(a) and in Photograph 2.1(a). A mesh at the front and small holes on the sides of the apparatus allow the free movement of water into the apparatus and a membrane (small balloon) reacts to changes in the pore pressure. The balloon, connecting pipe and flexible hose are filled with hydraulic fluid and the other end of the flexible hose is connected to a pressure transducer.

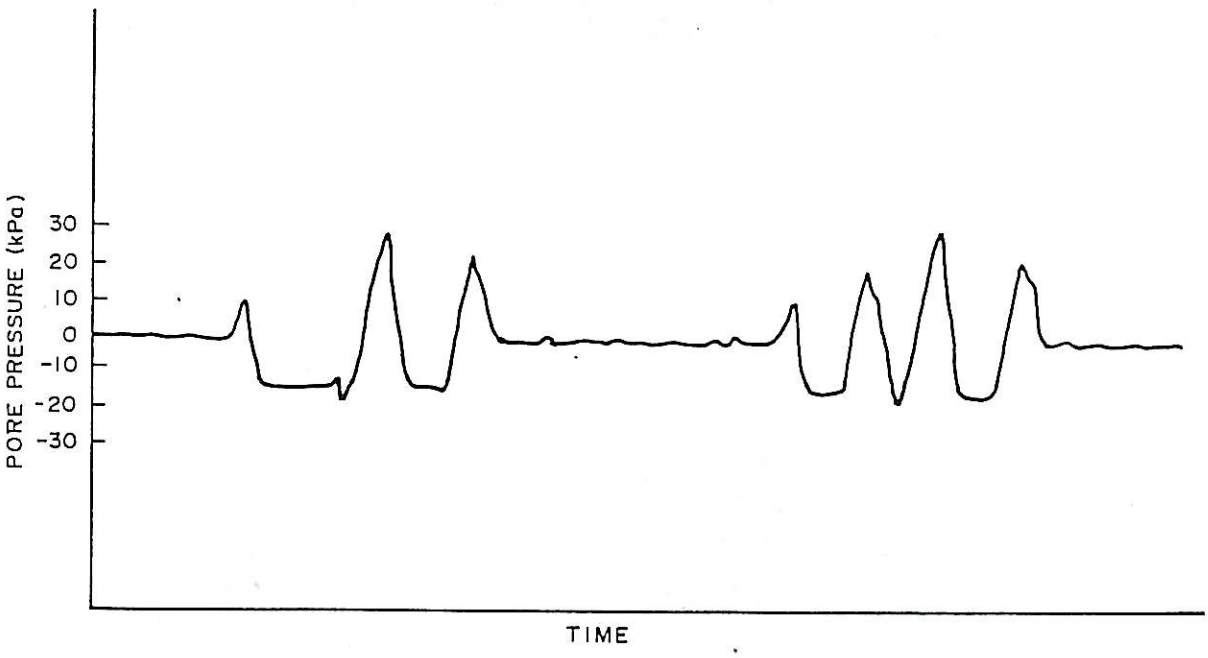
The apparatus was calibrated in the laboratory with a triaxial cell as shown on Photograph 2.1(b). After calibration it was installed in an HVS test section (Photograph 2.1(c)). The apparatus was installed at different depths in a pavement structure and pore pressures were measured while traffic loads were applied.

A major problem that was encountered was the clogging or partial clogging of the apparatus with fine material that was pumped during the test. Photographs 2.1(d) and 2.1(e) show the apparatus after being removed from the pavement structure. The apparatus was totally clogged in some cases (Photograph 2.1(d)) and partially clogged in others (Photograph 2.1(e)). In Photograph 2.1(e) the balloon is still partially visible.

In the two cases shown in Photographs 2.1(d) and (e) the apparatus was installed in a crushed stone base and the fine material from that layer was pumped into the apparatus. Pore pressures were recorded at the start of the test but after the apparatus became clogged no pressures were recorded. On the same test section, an apparatus was also installed in the stabilized subbase. Here the apparatus was not clogged and pore pressures could be recorded. Figure 2.2(b) shows typical results obtained. The HVS test wheel travels over the length of the test section and after each repetition it moves horizontally over the width of the test section to



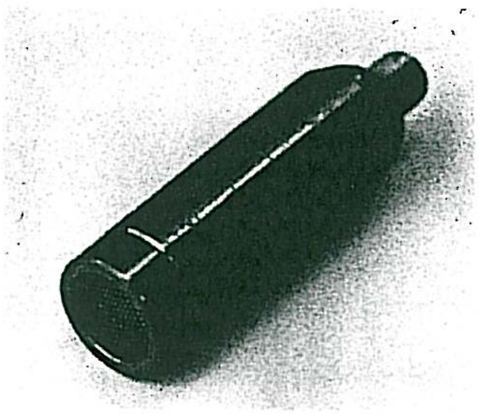
(a) APPARATUS



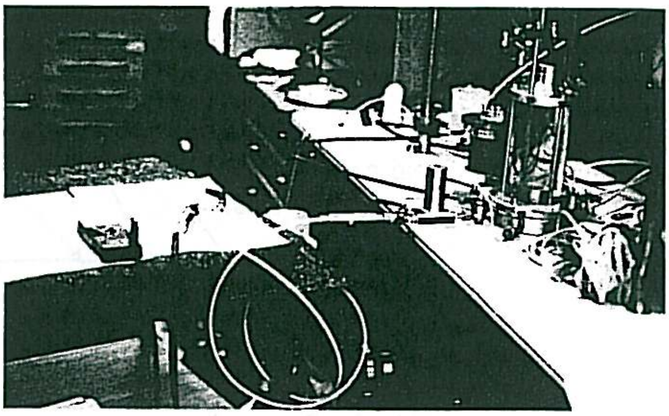
(b) TYPICAL RESULTS OBTAINED

FIGURE 2.2

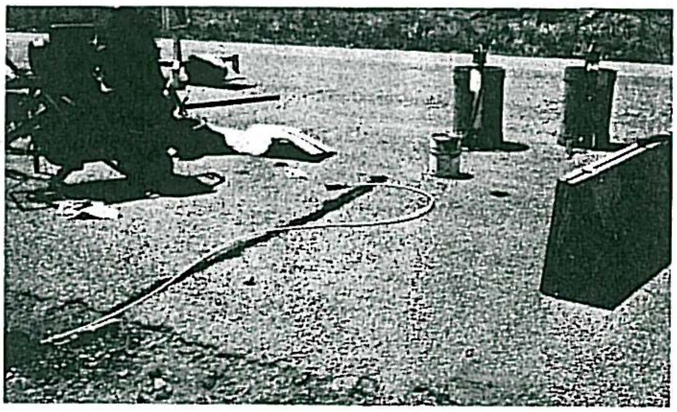
PORE PRESSURE APPARATUS



(a) Apparatus



(b) Laboratory calibration



(c) Field installation

(d) Clogged

(e) Partially clogged

PHOTOGRAPH 2.1

Pore pressure apparatus

apply the next wheel load directly next to the previous one. One cycle is completed when wheel loads have been applied to the full width (1m) of the test section. Figure 2.2(b) shows pore pressures recorded during two cycles. The apparatus was installed at the centre of the test section. The results show that the pore pressures increased until the wheel was directly above the pore pressure apparatus at which time the highest pore pressures were recorded. It is also noteworthy that the negative pressures (suction) that occurred, as the wheel moved away from the apparatus, were of similar magnitude than the positive pore water pressures. Pressures of approximately 30kPa were recorded, somewhat higher than those measured by Dempsey (1982).

Due to other priorities, no further work was done with the pore pressure apparatus. It is recommended that the work be continued, however, which should greatly improve the understanding of pore pressures and pumping, which are major factors in water related pavement damage.

The mechanisms that lead to pavement deterioration due to water are described in more detail in the remainder of this Chapter (Sections 2.2 to 2.5). The information was obtained mainly from previous HVS tests. An attempt has also been made to quantify the effect of water by calculating severity factors. Finally (Section 2.6) the economic implications of adequate subsurface drainage are discussed.

2.2 GRANULAR BASES

Pavements with granular bases are used fairly extensively in South Africa. HVS tests have been performed on a variety of these pavements, the results of which provide useful information on the influence that water has on the behaviour of these pavements. It has been illustrated (Maree et al, 1982) that there is a reasonable similarity between the performance of granular base pavements as evaluated with the HVS and that observed under actual traffic.

In Section 2.2.1 below, the general behaviour of granular layers, as determined with HVS testing, is described. In Sections 2.2.2 to 2.2.5 the influence of water on different types of granular bases is described, based on selected HVS test results.

2.2.1 General behaviour of granular layers

Based on results obtained from various HVS tests on pavements with granular layers, Freeme et al (1986) have described the general trends observed in the performance of these layers under traffic. These are summarized on Figure 2.3, showing the change, with traffic loading, of deformation, deflection, resilient modulus and strength. Deformation is the permanent deformation measured at the surface, deflection is the resilient deflection between the top and bottom of the layer, resilient modulus is the effective modulus of the layer calculated from the resilient deflection and using mechanistic analysis, and strength is the penetration per blow of the dynamic cone penetrometer (DCP) which is an indication of shear strength.

Phase 1 is often referred to as the settling in phase where the granular layer is usually densified under traffic, resulting in a high rate of deformation and an increase in the effective modulus. The extent to which this occurs is dependent on the initial density of the layer obtained during construction.

The second phase occurs if the quality of the material is adequate for the applied traffic and if water does not enter the layer. The rate of increase in deformation and the reduction in effective modulus is usually lower in this phase than in Phase 1. The strength measured with the DCP, however, increases during this phase because the changes that occur are as a result of densification rather than shear failure.

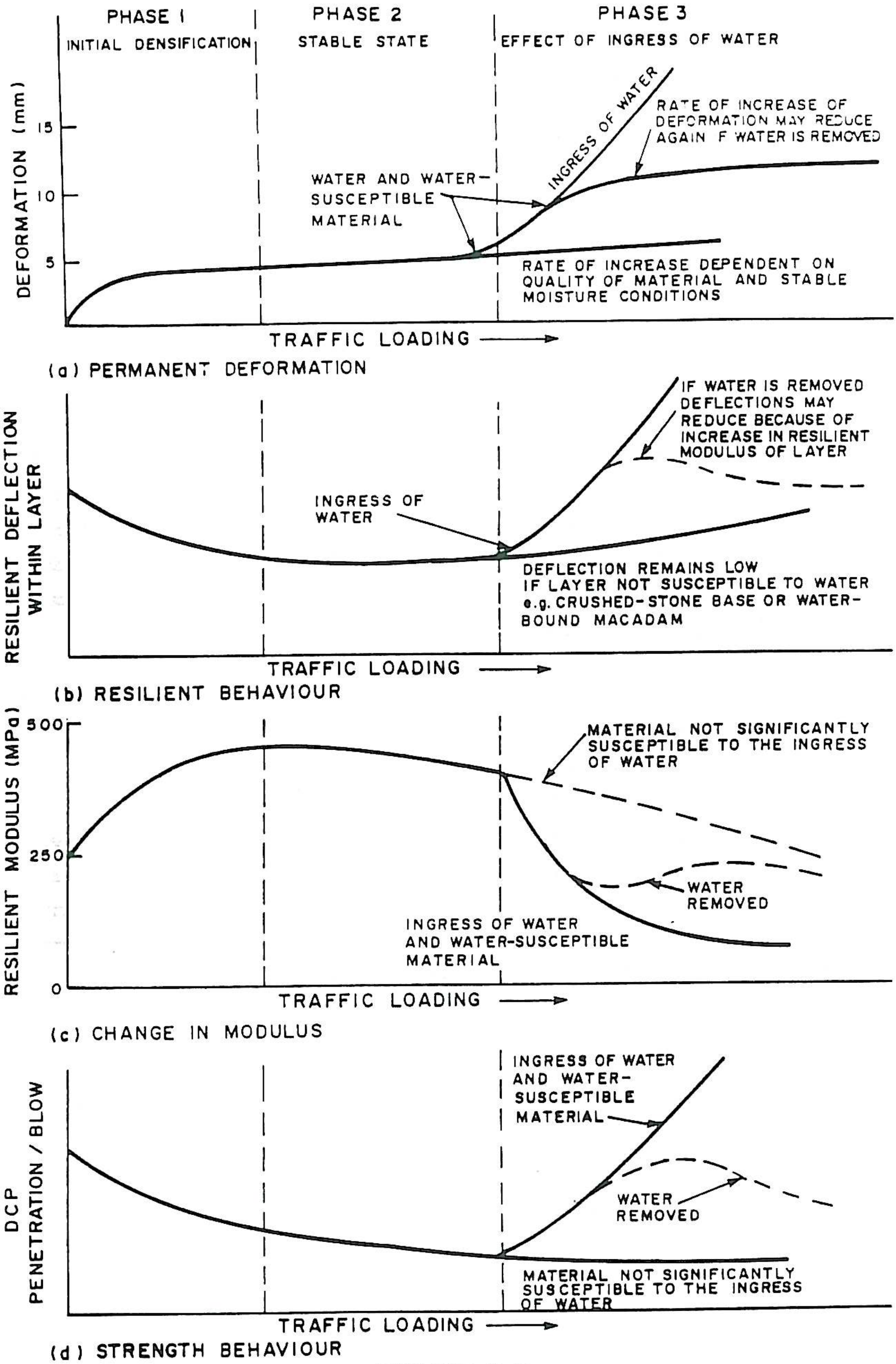


FIGURE 2.3

TYPICAL BEHAVIOUR OF GRANULAR LAYERS UNDER HVS TESTING (Freeme et al, 1986)

When water enters the granular layers, shear failure usually occurs, resulting in the changes shown in Phase 3. The rate of deformation increases rapidly and both the effective modulus and the shear strength decrease. The rate of deterioration during this phase is largely dependant on the quality of the material. When the water is removed from the layer, the rate of change becomes similar to that in Phase 2, but the actual permanent deformation at the surface is irreversible and the damage caused by the water is therefore permanent.

The rate at which the changes occur after the introduction of water, as described above, is determined by the pavement structure and the type of granular material. This is discussed in more detail in Sections 2.2.2 to 2.2.5 below, where HVS tests on different types of granular material are described.

2.2.2 Crushed stone bases

Graded crushed stone obtained from crushing solid unweathered quarried rock, is classified into three categories according to the TRH documents (DRTT, 1985(a) & (b)). The highest quality, G1, is compacted to a specified density of 86-88% of apparent density and may not contain any material other than that obtained obtained by crushing the parent rock. Natural fines not obtained from the parent rock may be included in G2 and G3 materials. Field density specifications of the latter are in terms of Mod AASHTO compaction namely 100-102% for G2 and 98% for G3 crushed stone. Specific grading requirements are given in TRH14 for these materials.

HVS tests have been performed on a number of pavements with crushed stone bases and tests from which the influence of water can be evaluated have been selected for discussion in this section.

The tests were performed between 1979 and 1981 (Freeme et al, 1985) and are described in detail by Maree (1982), Maree et al (1982) and Van Zyl et al (1983). A summary of the pavement structures tested and the permanent deformation obtained with each test is shown on Figures 2.4(a) and 2.4(b) respectively.

The permanent deformation shown is that measured at the surface, but the results indicated that for these pavement structures, most of the deformation originated in the crushed stone bases. The moisture conditions shown on Figure 2.4(b), namely dry, wet and soaked, are defined as follows: the layer is dry if the degree of saturation (s) is less than 50%, wet if s is between 50 and 80% and soaked if the layer is more than 85% saturated.

Road P157/2 near Jan Smuts Airport incorporated the highest quality G1 crushed stone base. After the road had been trafficked in a dry condition, water was introduced at various depths in the pavement structure and frequent rain resulted in water at the surface. Despite the different methods of water introduction, very little change in the rate of deformation was observed and it was concluded that the high density of this layer made it too impermeable for water to enter. The performance of this layer did therefore not reach Phase 3 as described in Section 2.2.1 above. These results indicated that, if high quality crushed stone is adequately compacted and supported (255 mm cemented layer support for this particular pavement), it is relatively insensitive to water damage.

The crushed stone base of Road P157/1 near Olifantsfontein was of G2 quality. After testing the pavement in the dry condition, water was sprayed on the surface. The water passed through fatigue cracks in the surfacing and, due to the lower density of the G2 material, entered that layer. Water reduces the shear strength of crushed stone, as has been shown in an

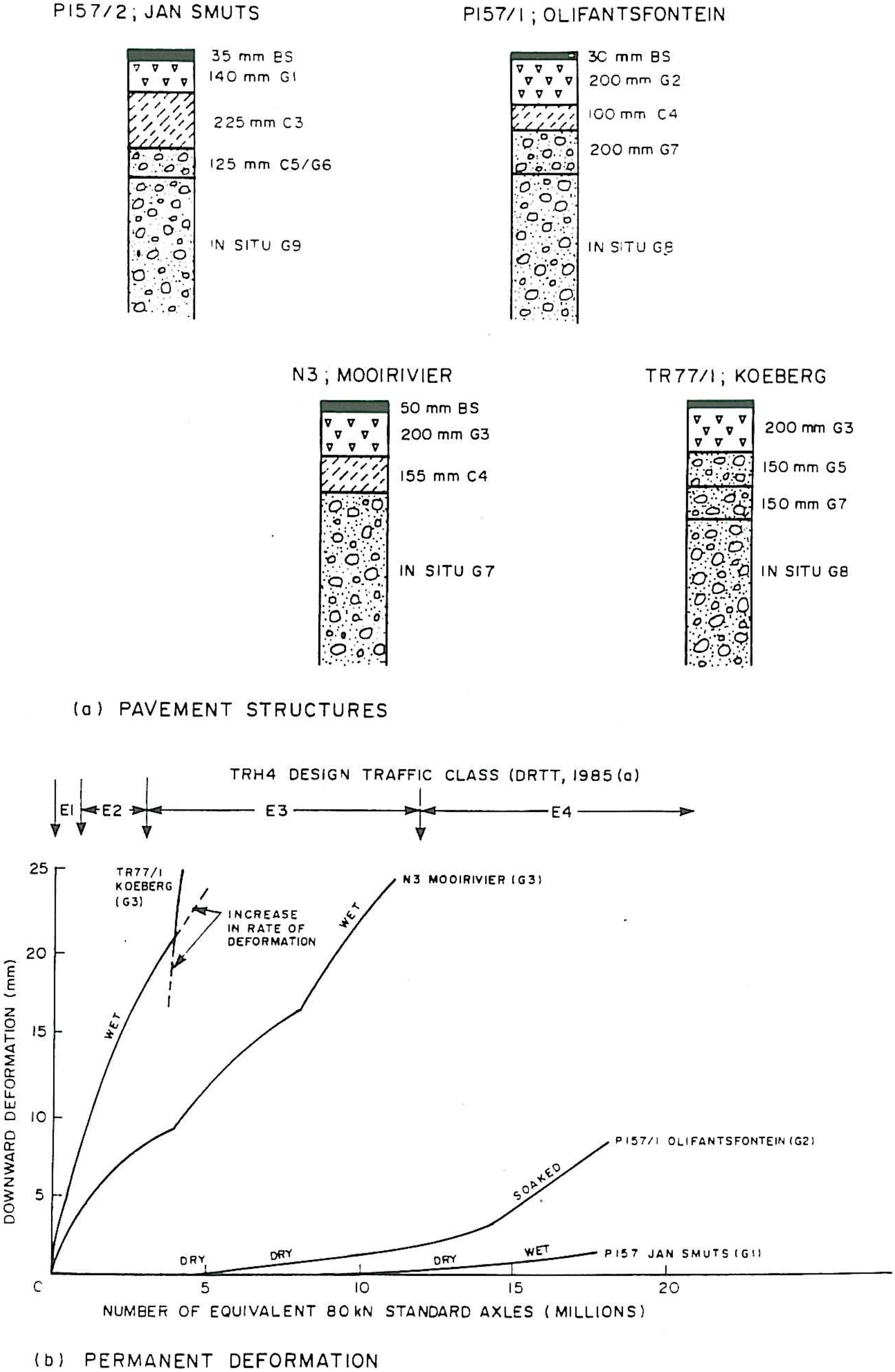


FIGURE 2.4

HVS TESTS ON PAVEMENTS WITH CRUSHED-STONE BASES (MAREE, 1982) earlier laboratory study (Maree, 1978), and consequently resulted in shear failure of this particular layer. The result was an increase in the rate of permanent deformation, as shown on Figure 2.4(b) (soaked stage). The density of this material was such that the water could penetrate sufficiently to reach a soaked (s>85%) stage. It is noteworthy that the difference in quality between this material and the G1 on P157/2 described above, only became evident after the introduction of water.

The crushed stone layer of the N3 near Mooirivier was of G3 quality with lower density than the G1 and G2 layers described above. Water was not purposely introduced into this pavement at any stage of the test, but the material was wet (s>50%) throughout, due to climatic conditions and the lower density of the material. The higher rate of permanent deformation from the beginning of the test, as shown on Figure 2.4(b), is ascribed by the authors to these factors (high moisture content and low density). It was also noted during the test that, before the surface was cracked, rain did not result in an increase in the rate of permanent deformation. After cracks had formed on the surface, however, the rate of deformation increased after a rain storm, followed by a decrease in the rate after the material had dried out to its original moisture content. The permanent deformation on this pavement was found to be as a result of the densification of the crushed stone and not due to shear failure as was the case with the G1 and G2 materials.

The G3 base layer on Road TR77/1 was also wet (s>50%) from the beginning of the HVS test and after water was introduced in depth and at the surface, the layer became soaked (s>85%). This resulted in the EPWP state (de Beer et al, 1987) and the resultant increase in the rate of deformation was dramatic, as is evident from Figure 2.4(b). The high rate of deformation on this pavement is ascribed by Maree (1982) not only to the lower density of the crushed stone but also to inadequate support of the base layer (granular subbase as opposed to cemented subbases for the other pavements described above).

Maree also points out that the tests on this pavement clearly showed the sensitivity to water of open graded bases with high permeability, which is in contrast to the view expressed by Cedergren (1974), namely that open graded bases should be used to ensure good drainage.

It is evident from the pavement performances described above that the extent to which water influences the pavement deterioration differs for the various qualities of crushed stone. In an attempt to quantify the influence of excess water, severity factors have been calculated for each type of crushed stone. These factors can only be an approximation of the expected results of excess water, since the actual behaviour is dependant on many factors that will differ from one pavement to another. These include the actual pavement structure (layers other than the base layer), climatic and material conditions that will determine the moisture conditions before and after the introduction of water (eg dry to wet on P157/2 as opposed to wet to soaked on TR77/1), the pavement state which will be determined by the amount of trafficking prior to the introduction of water and the method of water introduction (surface, in-depth, pressure etc).

A severity factor (SF) can be broadly defined as:

$$SF = \frac{\text{Rate of pavement deterioration with water}}{\text{Rate of pavement deterioration without water}}$$

For the HVS tests described above, this factor can be obtained by dividing the rate of permanent deformation immediately after the introduction of water by that immediately prior to water introduction. This is illustrated in Figure 2.4(b) (Road TR77/1). The rates of deformation are obtained by linear extrapolations of the deformation curves before and after water introduction. For the pavement indicated (TR77/1) these rates are 3,24 mm per million equivalent 80kN standard axles (3,24 mm/ME80's) and 25,72 mm/ME80's respectively. The severity factor for the G3 base with an uncemented subbase is therefore 25,72/3,24=7,9. This means that, all other factors being equal, this particular pavement will deteriorate at a rate 7,9 times higher with excess water than without. Severity factors for the G2 and G1 bases (P157/1 and P157/2) are obtained in the same way as 4,6 and 1,5 respectively. A factor can not be obtained for the G3 base supported by a cemented subbase (N3) since there was no clearly defined point at which water was introduced.

2.2.3 Natural gravel bases

A number of roads in the Transvaal that were upgraded from gravel to black-top roads have natural gravel bases of normal subbase standard. HVS tests were performed on two such roads (Maree, 1982), the bases of which were both classified as G5, according to TRH14 (DRTT, 1985(b)). The layers beneath the bases varied from G6 to G9 quality.

A number of tests were performed on these roads, at different positions and using different wheel loads. Water was introduced with some tests by surface spraying and in others summer rain resulted in surface water. In each case the rate of permanent deformation increased dramatically indicating that these base layers were extremely sensitive to excess water. Density measurements indicated that the deformation was due to shear failure rather than densification, but DCP results suggested that the material was in fact densified under traffic and that shear failure resulted in the material being de-densified to its original density before trafficking.

Severity factors can be obtained from permanent deformation curves as described in Section 2.2.2 above. These vary from approximately 10 to 28 for the various tests with an average of 21.

2.2.4 Slag bases

Slag, a byproduct obtained from the iron and steel industry, is sometimes used as an alternative to graded crushed stone. HVS tests were performed on a pavement with a slag base (170mm), supported by two cemented subbase layers (170 and 110mm) on top of a G8 quality selected layer (Horak et al, 1982). Although the grading of the slag met the specification for G1 and G2 layers as given in TRH14, the density was only 83% of apparent density.

The rate of permanent deformation of the pavement in the dry state was similar to that observed on Road P157/2 described in Section 2.2.2 above (G1 base), but this rate increased considerably after the introduction of water as indicated on Figure 2.5. The difference in the quality of the layer was, as was the case with the G2 layer (P157/1), only noticeable in the soaked stage. The permeability of the slag base was very high (250-300 times that of the G1 base) which allowed water to saturate that layer and to enter the underlying layers. The EPWP state developed which resulted in the pumping of fines from both the base and the selected layer. There was very little evidence of shear failure but all the layers were densified under traffic.

Water was only introduced in one half of the test section and two permanent deformation curves were therefore obtained; one for the dry and one for the wet section (see Figure 2.5). The severity factor can therefore be calculated using the average rate of deformation obtained from those portions of the curves after water introduction (ie after 1 million repetitions). The factor thus obtained is 10,5.

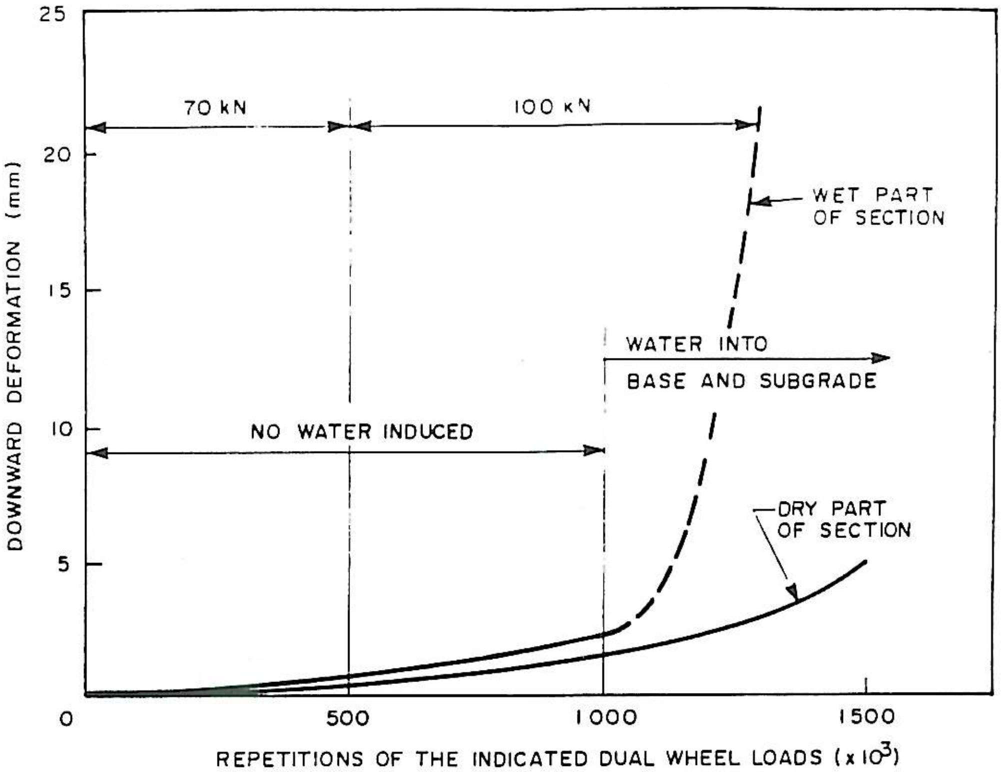


FIGURE 2.5 *PERMANENT DEFORMATION OBTAINED WITH HVS TESTS ON A SLAG BASE (Horak et al, 1982)*

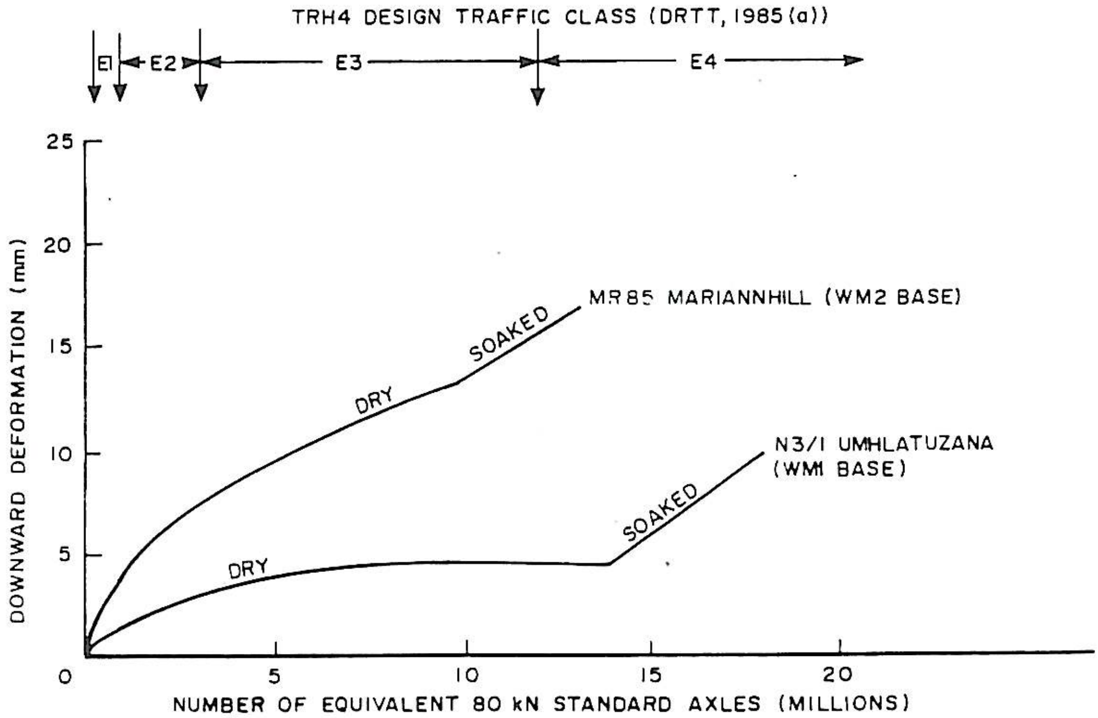


FIGURE 2.6 *PERMANENT DEFORMATION OBTAINED WITH HVS TESTS ON WATERBOUND MACADAM BASES (Horak et al, 1986)*

2.2.5 Waterbound macadam bases

Waterbound macadam consists of single sized coarse aggregate which is placed and compacted prior to the voids being filled with fines. The high cost of labour intensive construction resulted in a decrease in the use of these materials after the 1950's, but new mechanized construction methods and the good service records of pavements with waterbound macadam bases, especially in wet regions, resulted in a renewed interest in these materials.

The TRH documents (DRTT, 1985(a) and 1985(b)) distinguish between two categories of waterbound macadams namely WM1 and WM2. The only difference in the specifications for these materials are in the density requirements namely 88-90% and 86-88% of apparent density for WM1 and WM2 respectively. The coarse aggregate, which ensures granular interlock with high resistance to shear failure, has an open grading with maximum stone size of 53 to 75mm. The filler material provides stability and cohesion in the base material and a maximum stone size of 9,5mm is allowed with a fairly large degree of freedom in the grading.

Horak et al (1986) describe HVS tests on pavements with WM1 and WM2 bases that were performed on roads in Natal. In these tests the waterbound macadam bases were well supported with 300mm cemented subbases (C3 quality). Figure 2.6 shows the permanent deformation obtained with these tests. In the dry state the permanent deformation of the higher density WM1 base was similar to that of G1 and G2 bases, whereas the rate of deformation on the WM2 base was somewhat higher. After water was introduced, however, the increase in the rate of deformation was larger for the WM1 base than for the WM2 base. The good performance of the WM2 base in the wet stage is attributed to the higher permeability of that layer. The high permeability was a result of lower density as well as more open graded filler material. High permeabilities in the crushed stone bases resulted in densification and shear failure with the introduction of water, but with the particle interlock obtained with the large aggregates of the WM2, this base layer could provide resistance to shear failure and densification and at the same time act as a drainage layer to remove the excess water. Adequate side drains were also provided on this pavement to remove the water from the base layer. Further tests indicated that the permeabilities of these materials were largely determined by the grading of the filler material, and it would therefore be possible to construct a high density (WM1) base which would also have high permeability and would therefore perform well in both the wet and dry states.

Severity factors can be calculated from the permanent deformation curves in Figure 2.6. Values of 9,7 and 1,7 are obtained for the WM1 and WM2 bases respectively. These factors would in fact apply to low permeability and high permeability materials respectively, which would not necessarily be WM1 and WM2, depending on the grading of the filler material.

2.3 CEMENTED BASES

The TRH documents (DRTT 1985(a) and (b)) distinguish between four categories of cemented materials namely C1, C2, C3 and C4. The strength requirements for these materials range from 6-12 MPa unconfined compressive strength (UCS) for C1 to 0,75-1,5 MPa for C4.

The high quality cemented layers (C1 & C2) are not recommended for base layers but provision is made in the catalogues of TRH4 for C3 and C4 materials as bases. Pavements with these bases are fairly common in the rural areas of the Transvaal.

The general behaviour of cemented materials, as determined with HVS testing, is described in Section 2.3.1 below. In Sections 2.3.2 and 2.3.3 the influence of water on pavements with cemented bases is described in more detail.

2.3.1 General behaviour of cemented layers

The behaviour of cemented layers under traffic, as has been observed with various HVS tests, is described by Freeme et al (1986). Figure 2.7, which is similar to Figure 2.3 for granular layers, shows the changes that take place in cemented layers under traffic. In Phase 1, the cemented layer is uncracked and its behaviour is similar to that of a concrete slab. Water in this phase has no influence on the behaviour of the cemented layer since it is virtually impermeable. Continued traffic loading results in cracks forming in the layer, which leads to a reduction in the effective modulus. This reduction continues until the material is in a state similar to that of a granular layer. The properties of the layer are then dependant on the quality of the material that was originally stabilized. When water enters the layer at this stage, Phase 3 occurs, which is similar to that of a granular material. Densification and shear failure result in increased permanent deformation and deflections and a decrease in effective modulus and shear strength.

2.3.2 Cemented bases in a deep pavement structure

A pavement is a deep, well balanced structure when the strength of the pavement is well distributed in its depth ie there is not a large difference in strength between the base and subbase and between the subbase and selected layer (De Beer, 1986(a)). HVS tests have shown different behaviour in both the dry and the wet states between cemented bases in a deep structure and those in a shallow pavement structure. The behaviour of a deep

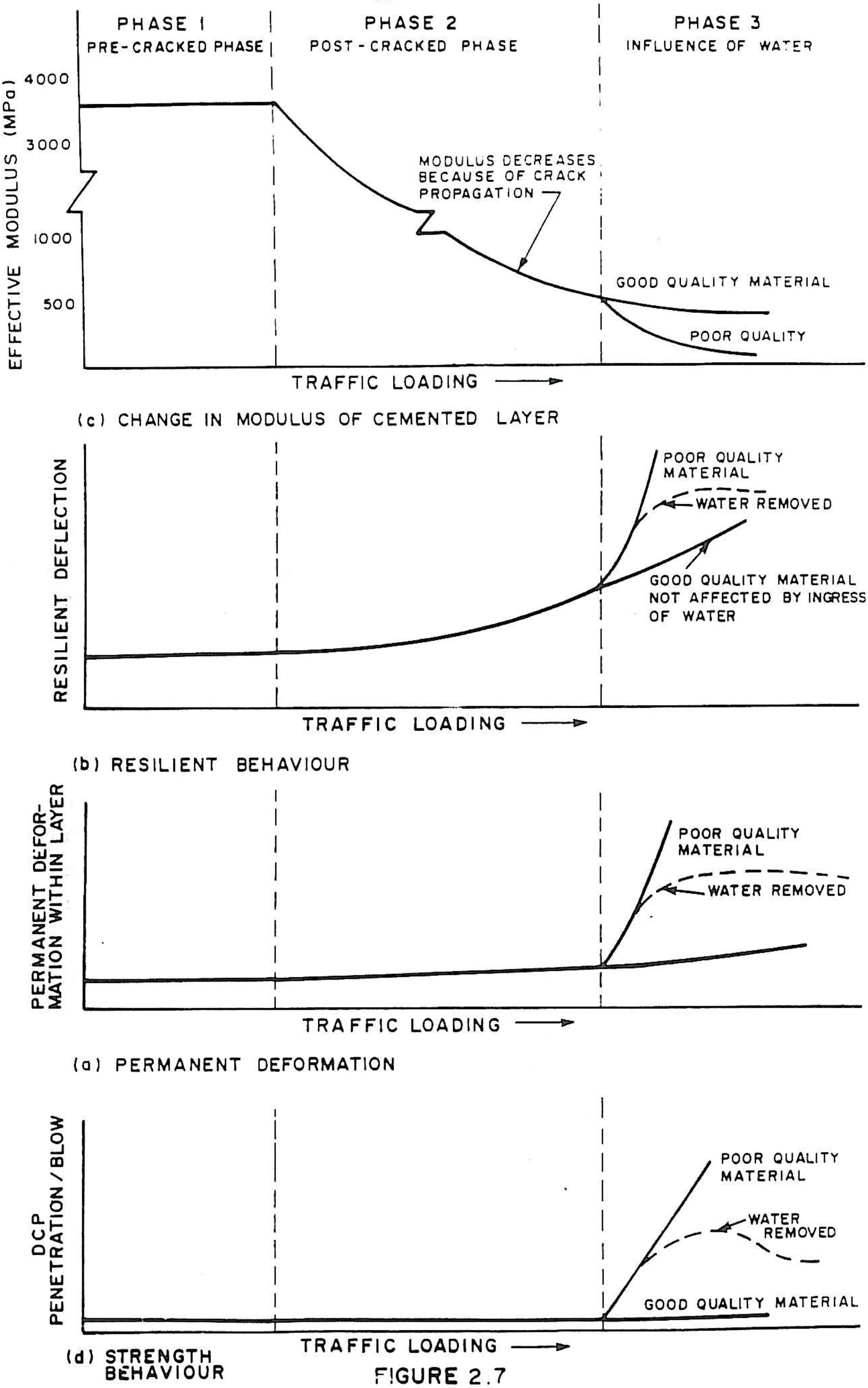


FIGURE 2.7

TYPICAL BEHAVIOUR OF CEMENTED LAYERS UNDER HVS TESTING
(Freeme et al, 1986)

structure is described here and in Section 2.3.3 below, that of a shallow structure.

HVS tests were performed on a deep, well balanced pavement with a cemented base at Rooiwal in the Transvaal (De Beer 1986(b) and 1987). The pavement structure consisted of a C3 quality base (150mm), a C4 subbase (150mm) and a G4 selected layer on G5 quality in situ material.

The permanent deformation obtained with one of the tests performed is shown on Figure 2.8. The test section was divided into two parts, one of which (MP 9-15 on Figure 2.8) had water introduced to a depth of 600mm after 1 million load repetitions. No rain fell during this particular test and water was not purposely sprayed on the surface, but spillage and leakage from the water bottles resulted in water on the surface.

The permanent deformation in both the dry and wet stages of the test was found to originate from the top 50-75mm of the base layer. Only that part of the layer became cracked due to a crushing effect of the wheel loads. The bottom part of the base as well as the subbase remained in a cemented state.

The in-depth water that was introduced had very little effect on the rate of deformation, which is ascribed by De Beer to the low permeabilities of the base and subbase. Water that was spilled on the surface, however, entered the top part of the base through surface cracks and saturated that part of the layer. This resulted in the EPWP state developing with a resultant increase in the rate of permanent deformation, pumping of fines from the top 50-75 mm of the base layer and potholes forming after approximately 1,5 million load repetitions.

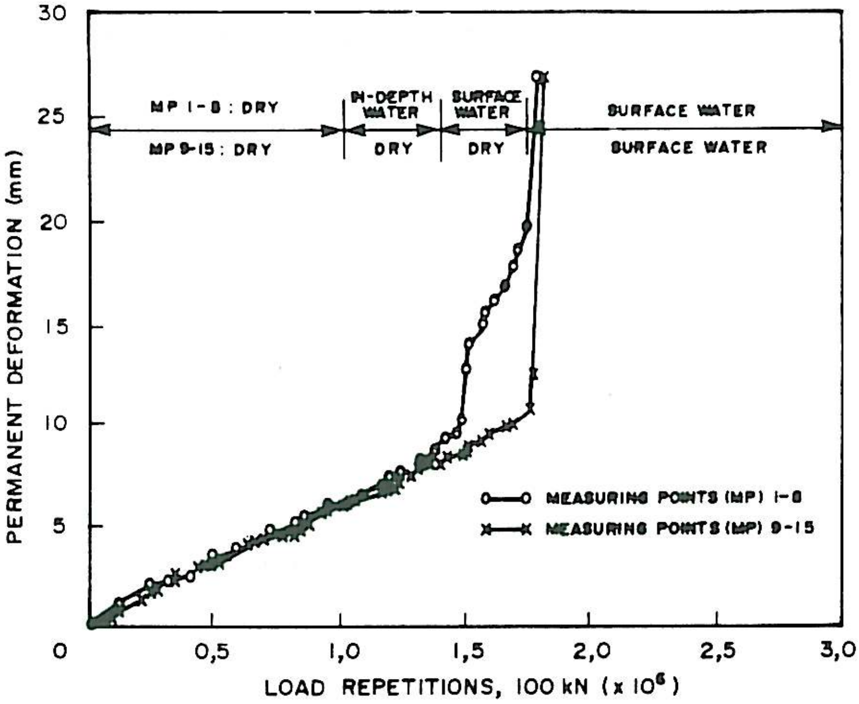


FIGURE 2.8 *PAVEMENT DEFORMATION OBTAINED WITH HVS TESTS ON A DEEP PAVEMENT STRUCTURE WITH CEMENTED BASE (De Beer, 1987)*

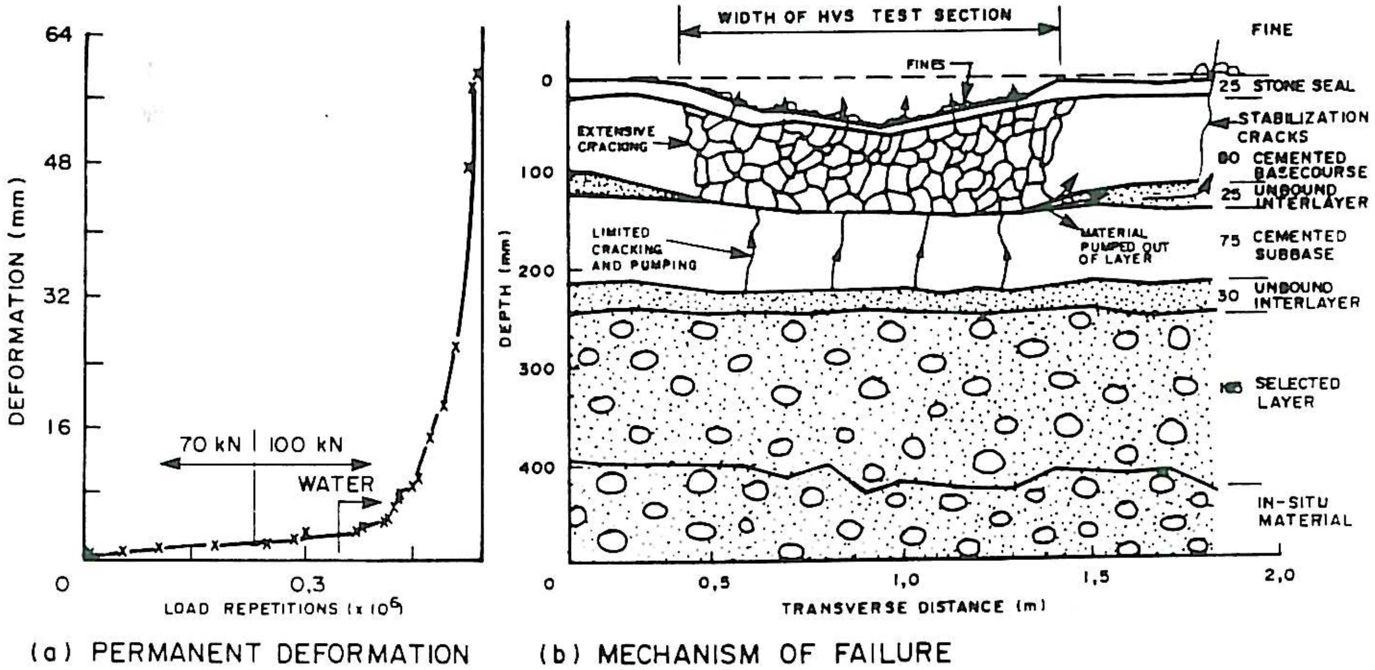


FIGURE 2.9 *RESULTS FROM HVS TESTS ON A SHALLOW PAVEMENT STRUCTURE WITH CEMENTED BASE (Kleyn et al, 1985)*

To calculate the severity factor, those portions of the deformation curves (Figure 2.8) where the wet section (MP 9-15) had surface water and MP 1-8 was dry, can be used. If the average rates of deformation during that phase is calculated for the wet and dry sections, a severity factor of 5,1 is obtained.

2.3.3 Cemented bases in a shallow pavement structure

A pavement is shallow and unbalanced when the strength is not well distributed in the depth of the pavement (De Beer, 1986(a)). This occurs when there are weak layers between the cemented layers, or if the subbase is substantially weaker than the base layer.

A shallow pavement structure with a cemented base, where the influence of water was particularly severe, was tested with the HVS near Hornsnek (Kleyn et al, 1985). The pavement consisted of cemented base and subbase layers (80 mm, C3/C4) with thin (30 mm) unbound layers between the base and subbase and also between the subbase and selected layer (See Figure 2.9(b)).

The permanent deformation obtained is shown on figure 2.9(a). Water was introduced at depth as well as on the surface, resulting in a considerable increase in the rate of deformation. The mechanism of failure of this pavement is shown on Figure 2.9(b). The entire base layer cracked severely due to insufficient support and water penetrated the base layer as well as the unbound interlayer. The EPWP state developed in these layers (de Beer et al, 1987) which resulted in fines, mainly from the unbound interlayer, being pumped out at the surface. Virtually all the material from the interlayer was pumped out, causing a subsidence of the base layer and consequently a very high degree of permanent deformation. The same mechanism occurred, to a limited extent, with the subbase and underlying unbound interlayer.

If the severity factor is calculated using linear extrapolation of the rate of deformation before and immediately after the introduction of water, as was described in Section 2.2.2, a value of 3,5 is obtained from Figure 2.9(a). It is evident, however, that the rate increased considerably as trafficking continued under wet conditions, resulting in a severity factor of approximately 100 towards the end of the test. This illustrates the fact that the influence of water is dependant on the state of the pavement structure when water is introduced. The best indications of severity factors are obtained when water is introduced to half of the test section while the other half is kept dry, as with the test described in section 2.3.2 above.

In another HVS study (De Beer, 1988) on a cemented layer in a shallow pavement structure, the method of applying water to only part of the test section was applied. The severity factor obtained for this test (deformation curves not shown) is 2,1. In this pavement, however, the interlayer was thicker (30-50mm) and stronger (cemented but partially carbonated) and pumping did not occur.

The test results described above indicate that the effect of water on cemented bases in shallow pavement structures is largely dependant on the characteristics of any unbound interlayers. The severity factor varies considerably, with high values resulting if pumping occurs.

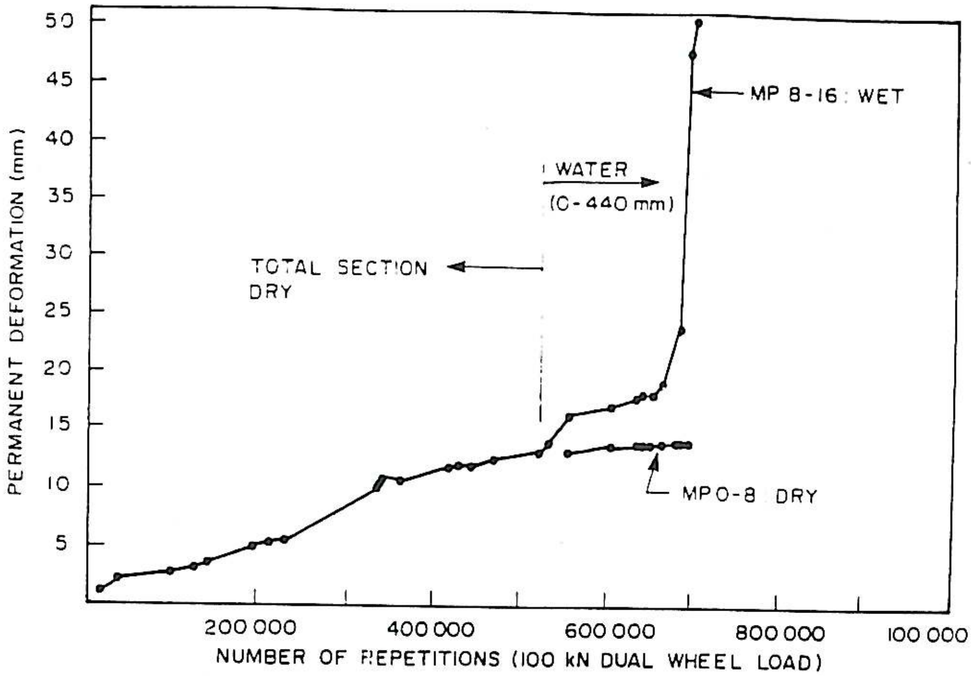
2.4 BITUMINOUS BASES

The influence of water on bituminous materials is limited, for which reason these materials are often used in base layers in the wetter regions of South Africa. Although the bituminous materials as such are not water sensitive, the effect of water on pavements with bituminous bases is often fairly dramatic.

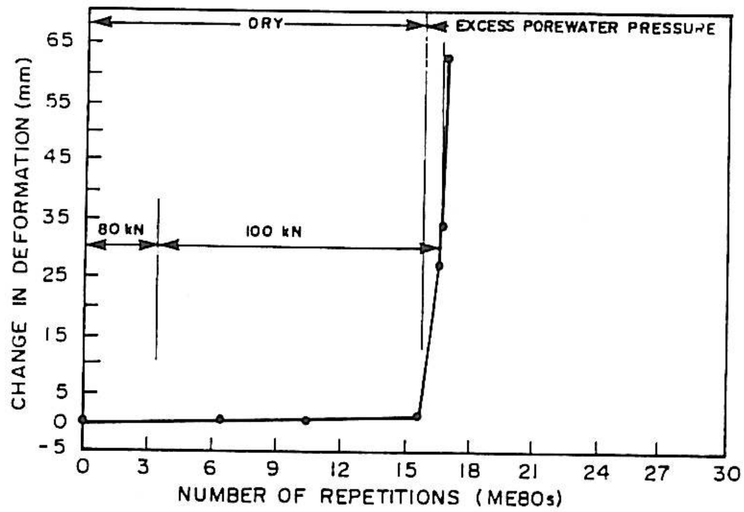
Adequate support is needed for bituminous bases and lightly cemented subbases (C3/C4) are often used for this purpose. HVS tests have been performed on such pavements (De Beer, 1985), two of which are described here. The tests were performed on roads at Mariannhill and at Umgababa in Natal. The pavement structure on both roads was similar with a continuously graded asphalt base (125mm at Mariannhill and 90 mm at Umgababa). The subbases consisted of two cemented layers (upper and lower subbase) each 150mm and of C3 quality. Below the subbases were selected layers of G4 quality (250 mm) on top of G7 quality subgrades. A significant difference in the behaviour of the two pavements resulted from the different materials used for the subbase layers. At Mariannhill, a granular material (weathered granite) was used and at Umgababa a fine grained material (Berea Red Sand).

The permanent deformation that resulted from testing at Mariannhill is shown on Figure 2.10(a). During the dry part of the test fatigue cracks occurred in the bituminous layer and the subbases were also cracked, the upper subbase more than the lower subbase.

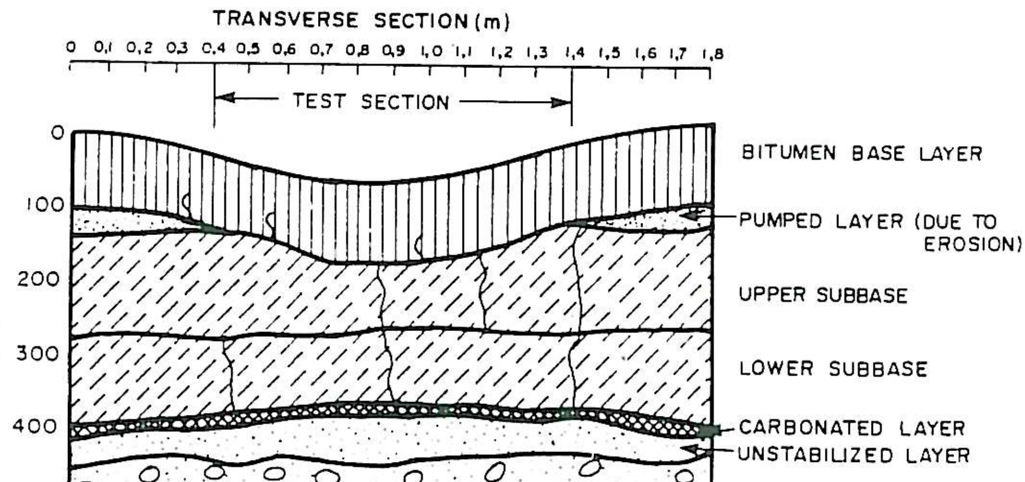
Water was introduced to one half of the test section. This was done with pipes (in depth water) and also under pressure (1 to 2m pressure head). The resultant increase in the rate of permanent deformation was considerable, as can be seen on Figure 2.10(a). The voids that were formed by the cracks in the subbase became saturated and the EPWP state developed. This resulted in shear failure, further cracking of the material, erosion and pumping. The fines that were eroded from the material were pumped through the fatigue cracks in the base resulting in a decrease in the thickness of the upper subbase. This led to a subsidence of the base, further fatigue cracks in that layer and rutting. Deflection measurements indicated that the strength of the upperlayers (base and subbases) decreased by approximately 100 per cent during the wet phase and that the effective modulus of the bituminous base also decreased as a result of increased fatigue cracking.



(a) PERMANENT DEFORMATION : MARIANNHILL



(b) PERMANENT DEFORMATION : UMGABABA



(c) MECHANISM OF FAILURE : UMGABABA

FIGURE 2.10 *RESULTS FROM HVS TESTS : BITUMINOUS BASE PAVEMENTS WITH STABILIZED SUBBASES (De Beer, 1985)*

940-4-5892/9 BD

The permanent deformation curve obtained with one of the tests at Umgababa (two tests were performed) is shown on Figure 2.10(b). Water was also introduced in depth and under pressure, resulting in an increase in deformation even more dramatic than with the Mariannhill test. The mechanism of failure was similar to the Mariannhill test, the main difference being that there was very little evidence of cracking in the cemented layers. The material (fine grained Berea Red Sand) lacked resistance to erosion and a loss of material from the upper subbase was the main failure mechanism. The EPWP state developed mainly at the interface between the base and upper subbase layer, eroding the subbase material from that area and pumping it sideways to form a layer of unbound material beneath the base next to the test section. This is shown diagrammatically in Figure 2.10(c).

Severity factors can be obtained for the two tests using average rates for dry and wet for Mariannhill (Figure 2.10(a)) and linear extrapolation for Umgababa (Figure 2.10(b)). The values thus obtained are 30 for Mariannhill and 820 for Umgababa. The second test at Umgababa (results not shown) results in a factor of 190. These values indicate that the erosion prone materials like Berea Red Sands are considerably more sensitive to water related damage than granular cemented materials in these type of pavements.

2.5 CONCRETE BASES

Concrete base pavements are limited to highly trafficked roads in South Africa and only a small percentage of roads have been constructed using these pavements. Information on their behaviour, as determined with HVS testing, is also relatively limited compared to that of other pavement types. However, the influence of water on concrete pavements is well known and well documented in literature from other countries, notably the USA.

Although concrete as such is relatively impermeable and thus not affected by water, concrete pavements are especially prone to failure resulting from the pumping of fines from the subbase. As early as the AASHO Road Test, all failures of concrete pavement sections were associated with pumping (Dempsey, 1982). In South Africa, pumping of fines from concrete pavements have also been observed with HVS tests (Viljoen, 1987 and Coetzee, 1988).

The mechanisms of pumping and the resultant failures are documented by various authors (eg van Wyk et al, 1984, Dempsey, 1982 and Cedergren, 1974). Water that enters the pavement through cracks or through the joints is trapped beneath the concrete slab either at the interface between the concrete and the subbase or within the subbase material. For pumping to occur, the water must be displaced at a high enough velocity (caused by deflections) and the material must be erodable. High deflections occur with high wheel loads, thin concrete slabs and inadequate support. When these factors are present the fine material from the subbase is eroded and transported by the water. The presence of water in the subbase also reduces its strength and therefore its resistance to erosion. The fine material is either transported horizontally beneath the concrete slab or, if the pore pressures are adequate, through cracks and joints to the surface. Dempsey (1982) distinguishes between pumping and channelling, the latter referring to the horizontal displacement of material. The displacement of material results in a change in the support conditions of the concrete slab due to voids in some areas and excess material in others. This leads to cracking (both corner or diagonal cracking and transverse cracking), corner breaking or spalling, shoulder depressions, punch-outs and faulting. Faulting occurs with relatively thick slabs as follows: water is pushed (by deflection) from the approach slab to the leave slab and returned to the approach slab at a higher velocity carrying fines from under the leave slab. This results in a void forming under the leave slab and excess material under the approach slab and hence a relative upward movement of the approach slab.

Severity factors cannot be obtained for concrete base pavements using the methods as described in Sections 2.2 to 2.4 above, since permanent deformation is not a typical method of failure for these pavements. Surface deflections do give an indication of the damaged caused to a concrete pavement, since they are dependant on the support conditions which are adversely affected by voids etc. HVS tests were performed on thin jointed concrete pavements (Coetzee, 1988) with slab thicknesses ranging from 130 to 240 mm. Figure 2.11 shows the increase in surface deflection obtained with the test on the 130 mm slab pavement. Water was introduced at the surface (through a hole in the joint sealer) at one joint on the test section from the beginning of the test (wet joint on Figure 2.11). The rate of increase in deflection at the wet joint was slightly higher than that at the dry joint until cracking and pumping was observed, at which time the rate at the wet joint increased considerably indicating the effect of a void forming in the cemented subbase. A severity factor can be calculated using the average rates of increase in deflections at the wet and dry joints respectively. A value of 4,9 is thus obtained. For the tests on the pavements with thicker slabs (results not shown), severity factors of 3,8 and 1,5 are obtained for the 150 and 250 mm slab thicknesses respectively. Although the support conditions of the various pavements differed (C2/C3) cemented layers of varying thicknesses), these results indicate that the thinner concrete slabs are more prone to water related damage. The severity factor will also be determined to a large extent by the erodability of the subbase materials.

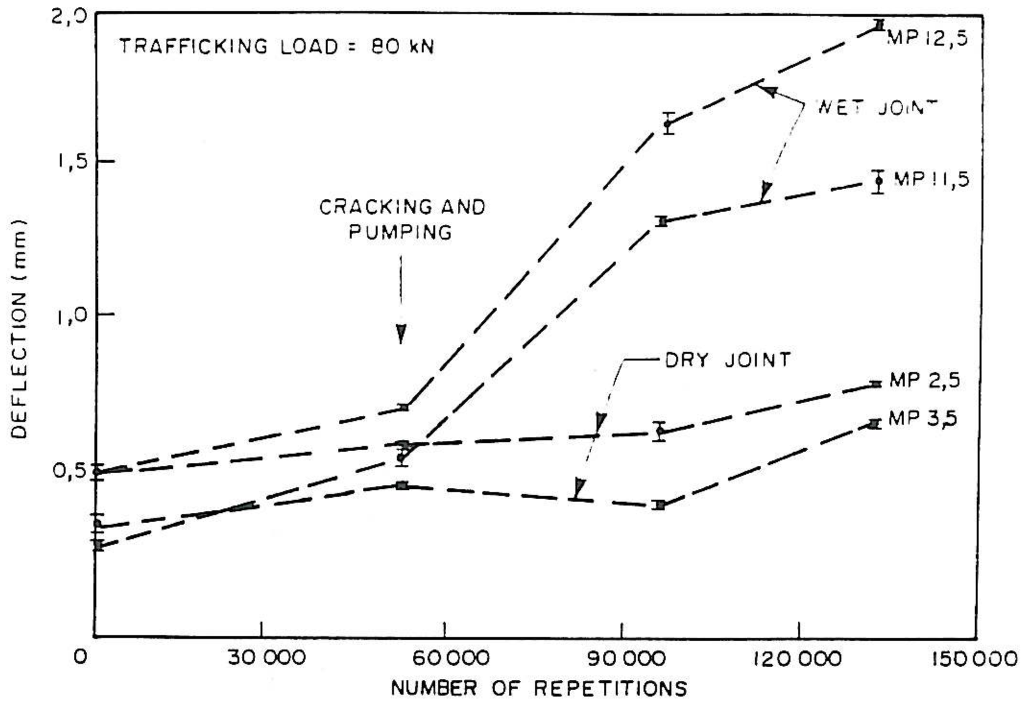


FIGURE 2.11 *HVS TESTS ON THIN JOINTED CONCRETE PAVEMENT: INCREASE IN SURFACE DEFLECTION WITH TRAFFIC (Coetzee, 1988)*

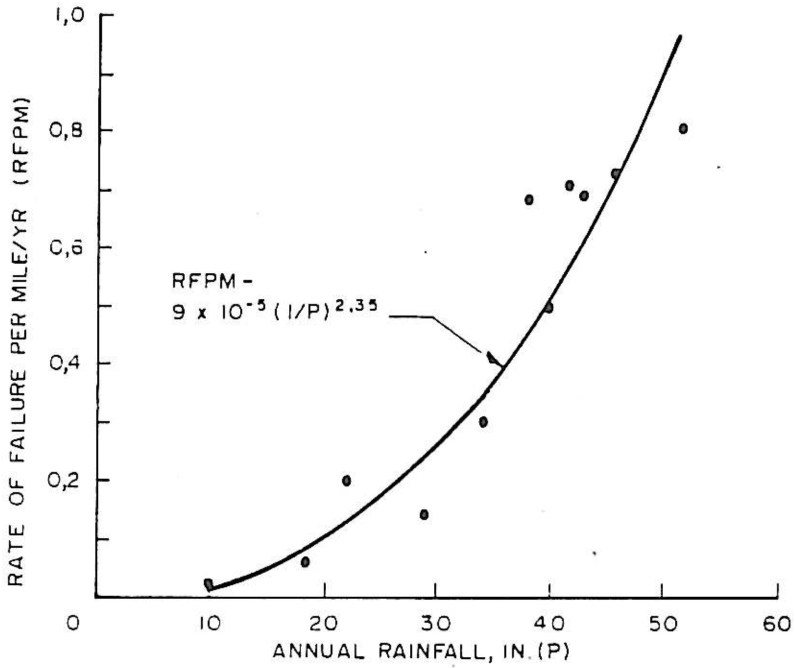


FIGURE 2.12 *EFFECT OF RAINFALL ON THE RATE OF FAILURES PER MILE PER YEAR OF CONTINUOSLY REINFORCED CONCRETE PAVEMENTS IN TEXAS (SAREF et al, 1987)*

A good indication of the severity of water in concrete pavements is given by a pavement condition study performed in Texas (Saraf et al, 1987). Condition surveys were carried out on continuously reinforced concrete pavements in various climatic regions of Texas and the total number of failures (patches and punchouts) per mile were recorded. The survey was repeated after ten years and from the two sets of results the rate of failure per mile per year (RFPM) was calculated for the different climatic regions. Figure 2.12 shows the results obtained, indicating RFPM against average rainfall. An equation is given for the relation that best fits the data points. A severity factor can be calculated from these results if it is assumed that the lowest rainfall (10 inches per year resulting in 0,02 RFPM) represents the dry condition and that the wet condition is represented by those roads in areas with more than 40 inches of rain per year. The latter all resulted in a RFPM of approximately 0,7 to 0,8. This gives a severity factor of 35 to 40. This calculation implies an assumption that the drainage conditions on both the "dry" and the "wet" roads were similar.

2.6 SUMMARY, ECONOMIC IMPLICATIONS AND CONCLUSIONS

2.6.1 Summary

The way in which water affects different pavement types was described in Sections 2.2 to 2.5 above. It was shown that when excess water is present, different failure mechanisms occur in different pavement types, which lead to an increase in the rate of pavement deterioration. The extent to which this rate increases also differs from one pavement type to another and severity factors were calculated for the various pavement types. A summary of the severity factors are given in Table 2.1 below. The values are rounded off and for those pavement types where more than one value was obtained, a range is given.

TABLE 2.1 - Summary of severity factors (SF)

Pavement type (base)		Severity factor (SF)
Granular:	Crushed stone (G1)	1,5
	Crushed stone (G2)	5
	Crushed stone (G3)	8
	Crushed stone (Slag)	10
	Waterbound macadam (high permeability)	2
	Waterbound macadam (low permeability)	10
	Natural gravel (G5)	20
Cemented:	Deep pavement structure (C3/C4)	5
	Shallow pavement structure (C3/C4)	2-100 (depending on interlayers)
Bituminous:	Granular cemented subbase	30
	Fine grained cemented subbase with low erosion resistance	190-820
Concrete:		1,5-35

As was discussed earlier (Section 2.2.2) these factors are only approximations of the expected damage that can occur when water enters the pavement structure. The actual values will depend on many factors including the following:

- The different pavement layers, layer thickness and material properties. Small quantities of unbound materials, wedged between cemented materials (interlayers) for example, will increase the severity factors considerably.
- The amount of water and the pressure under which it is forced into the pavement. Various methods of water introduction were used in HVS tests, some more severe than others.
- The state of the pavement at the time of water ingress.
- Wheel loads - higher wheel loads result in higher pore pressures in the wet state.

Water related damage can only occur when both water and traffic loads are present. In the HVS tests from which the severity factors were obtained (all except the value of 35 for concrete pavements - Texas field study) water and traffic loads were present on a continuous basis. This is not always the case in practice, however. In a high water table area, in the absence of adequate subsurface drainage, this may well be the case, but if the source of water is through a permeable pavement surfacing, excess water will only occur during and immediately after a rainstorm. For such cases the severity factors shown in Table 2.1 must be multiplied by a factor representing the expected ratio of wet conditions to dry conditions (eg 0,1 if it is expected that the road will be wet for one tenth of the year).

The only field verification obtained for a severity factor is that for concrete pavements from the pavement survey conducted in Texas. Those results indicated that the severity factors obtained from HVS tests (using increase in deflection) were in fact conservative and that the actual water related damage that occurs in practice is considerably higher than that predicted with HVS tests. It is recommended that similar pavement condition surveys be conducted on different pavement types in South Africa to determine the validity of the severity factors obtained from HVS tests.

2.6.2 Economic implications

A method is illustrated here whereby the economic justification of installing sub-surface drainage systems can be evaluated, using the severity factors given in Section 2.6.1 above. An example is given comparing the cost of rehabilitation of a pavement with and without a side drain, assuming that the side drain is totally effective and that without the side drain, excess moisture will be present in the pavement continuously.

Costs for a side drain were obtained from contracts awarded in 1988 (TPA, 1988) and calculated for a typical conventional side drain (geotextile filter and crushed stone), with the same dimensions as the side drain that was monitored for the geotextile evaluation (Chapter 6, Figure 6.2). The costs are as follows:

Item	Unit cost (R)	Cost per running meter (R/m)
Excavation	$6,14/m^3$	7,37
Backfill	$7,98/m^3$	4,31
Crushed stone	$38,88/m^3$	25,66
Pitch fibre pipe (150mm)	9,35/m	9,35
Geotextile ($210g/m^2$)	$2,28/m^2$	6,38
Total		R53,07/m
		= R53 000/km

The rehabilitation measure considered in this example is a thin (20 mm) asphalt overlay, at a cost of $R2,98/m^2$ (TPA, 1988). For a single carriageway rural road (7,4 m wide), the cost of such an overlay would therefore be R22 000/km.

The method recommended in TRH12 (DRTT, 1984(a)) to compare rehabilitation options, is that of present worth of costs (PWOC) as follows:

$$PWOC = \sum Ai\ (1+r)^{-ni}$$

where

PWOC = present worth of future rehabilitation costs

Ai = costs incurred at times ni

r = discount rate.

A discount rate of 6% is recommended for the evaluation of rehabilitation options (Jordaan, 1985).

The expected maintenance free life of a thin asphalt surfacing is between 8 and 14 years for an A-category road, according to TRH4, Table 20 (DRTT, 1985(a)). A life of 10 years is assumed. This applies to a well drained (dry) road ie with a side drain installed if the particular section of road is threatened by a high water table. The total amount of money spent on such a section of road (1 km) over a 10 year period is the construction cost at the beginning of the road's life (Time 0) plus the cost of the side drain (Time 0) plus the cost of overlay after 10 years (Time 10), if the original surfacing was a 20 mm asphalt layer. The construction cost of the road is the same for both the drained and undrained condition and can therefore be left out of the calculation.

The present worth of cost for the dry (drained) situation is then (side drain plus one overlay):

$$PWOC\ (dry) = A0 + A10\ (1+r)^{-10}$$

where

A0 = cost of side drain

A10 = cost of 20mm asphalt overlay.

Substituting the values given above:

$$PWOC\ (dry) = 53 + 22\ (1{,}06)^{-10}$$
$$= 65{,}28 \text{ thousand Rands/km}$$

If the side drain is not installed, the life of the original asphalt surfacing as well as that of any subsequent overlays will be reduced by the severity factor applicable. If a severity factor of 10 is assumed then the life of each overlay will be one year and the present worth of cost as follows (no side drain installed, A0=0):

$$\begin{aligned} \text{PWOC (wet)} &= 0+22(1{,}06)^{-1}+22(1{,}06)^{-2}+\ldots \\ &\quad +22(1{,}06)^{-9}+22(1{,}06)^{-10} \\ &= 161{,}92 \text{ thousand Rands/km} \end{aligned}$$

It is evident that, with the values used in the above example, the installation of a side drain would be the most economical solution. The PWOC (wet) can be calculated for other severity factors. A severity factor of 5 results in a PWOC of R78,61 thousand/km and if SF=4 then PWOC(wet) = R61,95 thousand/km, which is less than PWOC(dry). PWOC (dry) is equal to PWOC (wet) when SF = 4,2. The installation of a side drain is therefore economical if a severity factor of higher than 4,2 is expected.

The above example was calculated for a subsurface drainage/rehabilitation measure cost ratio of 53/22 = 2,4 and an expected life for the rehabilitation measure of 10 years. Severity factors above which subsurface drainage is more economical can be obtained for other cost ratio's representing various subsurface drainage methods and various rehabilitation measures and also for different life expectancies of the rehabilitaticn measures Table 2.2 gives a summary of various alternatives.

TABLE 2.2 - Severity factors above which subsurface drainage is more economical

Expected life of rehabilitation measure	Cost ratio = Cost of subsurface drainage / Cost of rehabilitation measure							
Years	0,5	1,0	1,5	2,0	2,5	3,0	4,0	5,0
5	1,6	2,2	2,7	3,3	3,9	4,5	5,6	6,8
10	1,7	2,3	3,0	3,7	4,3	5,0	6,3	7,6
15	1,8	2,5	3,3	4,0	4,8	5,5	7,0	8,6
20	1,9	2,8	3,6	4,5	5,3	6,2	7,9	9,6

The values to the right in Table 2.2 represent the more expensive subsurface drainage alternatives (or cheaper rehabilitation measures) and those to the left the cheaper drainage measures (or more expensive rehabilitation measures).

A fairly wide range of alternatives are included in Table 2.2 and even with the most expensive drainage alternative (cost ratio = 5,0 vs rehabilitation life expectancy of 20 years), a severity factor of 10 would justify the construction of subsurface drainage. Severity factors of above 10 were obtained for a number of pavement types (Table 2.1).

2.6.3 Conclusions

The need for adequate drainage to protect pavements from premature failure has been illustrated. The mechanisms that cause water related pavement deterioration were described for various pavement types and the influence of water was quantified by calculating severity factors. These factors are merely approximations based on available information and can vary considerably due to a number of factors. Nevertheless, economic evaluations using these severity factors indicate that the installation of subsurface drainage systems will usually be justified when the pavement is threatened by subsurface water.

CHAPTER 3

GEOTEXTILES - A LITERATURE SURVEY

CHAPTER 3

CONTENTS PAGE

3.1 INTRODUCTION

In Chapter 1 the applications of geotextiles in road subsurface drainage were illustrated. Certain advantages and disadvantages of these materials were discussed.

This chapter gives a summary of relevant information on geotextiles that was obtained from available literature. A general overview of geotextiles is given in Section 3.2. Sections 3.3 and 3.4 contain information that is directly related to the research reported on in this document, namely filter criteria and soil/geotextile compatibility tests.

It should be noted that this chapter is by no means a comprehensive literature survey on all aspects of geotextiles. The applications of geotextiles in civil engineering are numerous, including reinforcement, river protection and many others. These applications are beyond the scope of this document and are only briefly discussed in Section 3.2

3.2 GENERAL OVERVIEW OF GEOTEXTILES

This section can be seen as an introduction to geotextiles. The historical development of these relatively new materials in civil engineering is described briefly. This is followed by a discussion on the classification and terminology of both geotextiles and geosynthetics, the latter being a wider term encompassing a variety of synthetic materials which include geotextiles. The various types of geotextiles are also described.

An introductory discussion is also presented on the functions, applications and properties of geotextiles and a description is given of the geotextiles that were used in the investigations described in later chapters.

3.2.1 Historical Development

The historical development of geotextiles and geosynthetics internationally is well documented in the proceedings of the Third International Conference on Geotextiles (Giroud, 1986, p 1) as well as in some earlier works (Rankilor, 1981, p 3; Raumann, 1982, p 10).

The use of 'composite materials', which have properties different to that of the individual materials forming the 'composite', was used long before Man discovered the technique. An example is the method used by bird species such as swallows who build their nests with a combination of mud and straw. Reinforced soil was used by early civilizations such as the Babylonians and ancient Chinese who used wood, bamboo and reeds to strengthen their soil. The Romans used woven reed mats in road construction. During the first half of this century there were trials with cotton fabrics for strengthening road pavements between 1926 and 1935 (Beckham, 1935), and during World War II rolls of canvas were used to support armoured vehicles.

The development of geotextiles as they are known today, was a direct result of the major developments which occurred in the manufacture of synthetic fibres of which the durability was far superior to that of natural fibres. The first synthetic fibre was manufactured in 1913 from polyvinyl chloride (PVC) followed by a polyamide fibre ("nylon") in 1930. Then came polyester (1930's), polyethylene (1949) and polypropylene (1945).

The first fabrics made from these synthetic fibres were woven, notably those used in the packaging industry in the mid-1950's. It was not until the mid-1960's however, when the processes for non-woven fabrics were developed, that the geotextile market became substantial. By 1972 the European market became well established with manufacturers in the UK, France and Austria. The North-American and Japanese markets began their growth in 1975. From a very humble beginning in the 1950's and 1960's, with the first significant sales in the early 1970's, the geotextile market has grown strongly. In 1984 approximately 300 million square metres of geotextiles were used worldwide (Giroud, 1986, p 3).

The first known use of synthetic materials in civil engineering was in 1957 when sand bags made of nylon woven fabrics were used in the Netherlands for coastal protection. This was followed by a woven fabric between soil and rip-rap for coastal erosion control in Florida, USA in 1958. The first nonwoven fabric to be used and also the first synthetic material in road construction was used in an asphalt overlay in the USA in 1966. In 1968 non-woven fabrics were used for unpaved roads in France and in 1970/71 the applications of geotextiles for use in road embankments and filters in drains were developed.

In 1961 the first paper on geotextiles appeared in the Journal of American Society of Civil Engineers (Agerschau, 1961). One of the earliest attempts at developing design criteria was by the US Army Corps of Engineers (Calhoun, 1972). In 1977 an "International Conference on the use of fabrics in geotechnics" was held in Paris. This was followed by the "Second International Conference on Geotextiles" in Las Vegas in 1982. During this conference it was decided to form an international body for geotextiles and in 1983 the International Geotextile Society (IGS) was formed. The IGS organized the Third International Conference on Geotextiles in Vienna, Austria in 1986.

The introduction of geotextiles in South Africa only occurred in the early 1970's (Davies, 1986 p 2). In 1972 the first geotextiles were imported from France. These were nonwovens, as were most of the products that have been imported over the last decade. In 1979 the first factory was opened in South Africa, which produced the same nonwoven geotextiles that were originally imported in 1972. The introduction of wovens only occurred in 1984 when a factory producing these products was opened. The first symposium or conference dealing with geotextiles in South Africa was a Filters Symposium organized by the South African Institution of Civil Engineers in October 1986. One day of this two-day symposium was devoted to synthetic filters.

3.2.2 Classification of Geosynthetics

3.2.2.1 Existing terminology and classifications

The terminology for synthetic materials is often confusing mainly because their application in civil engineering is a relatively new science (± 30 years) and terminology and classifications still have to be standardized. Adding to the confusion is the continuous development of new products and applications.

Various terms have been used such as fabrics, filter cloths, filter fabrics, geofabrics and geotextiles. In the 1970's and early 1980's a variety of mesh- or netlike products were developed (Giroud, 1986, p 3) and new terminology was introduced: geonet, geogrid, geospacer, etc. The term "geosynthetics" was used at the last international conference (Giroud, 1986; Hoekstra et al, 1986, p 344) to encompass most of the synthetic materials used in "geo-" or civil engineering applications.

Table 3.1 (Giroud, 1986, p 17) shows a proposed classification of all products used in civil engineering applications, not only synthetic but also conventional materials like steel and glass. Giroud proposed the term "geoproducts" for this purpose.

TABLE 3.1 PROPOSED CLASSIFICATION OF GEOPRODUCTS (Giroud, 1986, p 17)

<table>
<tr><th>SCALE</th><th>DIMENSIONS</th><th colspan="2">TYPES</th><th>MATERIALS</th></tr>
<tr><td rowspan="2">Micro-geoproducts</td><td>Uni-dimensional</td><td colspan="2">Fibres, Filaments, Yarns</td><td rowspan="3">Synthetics
Steel
Glass</td></tr>
<tr><td>Bi-dimensional</td><td colspan="2">Microgrids</td></tr>
<tr><td rowspan="5">Macro-geoproducts</td><td rowspan="2">Uni-dimensional</td><td colspan="2">Cables
Strips</td></tr>
<tr><td colspan="2">Rods ("Nails")</td><td>Steel</td></tr>
<tr><td rowspan="3">Bi-dimensional</td><td rowspan="3">Geofabrics*</td><td>Grids (Geogrids)</td><td rowspan="2">Usually synthetic
("Geosynthetics")</td></tr>
<tr><td>Geotextiles
Webbings
Mats (Geomats)
Nets, waffles
Formed sheets</td></tr>
<tr><td>Geomembranes</td><td>Asphalt (usually with synthetic fabric)</td></tr>
</table>

* Here the term "geofabrics" is not limited to fabrics but is applicable to all planar structures made of polymers.

The following is a discussion of the various products:

Micro-geoproducts: The uni-dimensional products are usually the fibres or filaments from which geotextiles are made. These can be used for reinforcement by randomly placing them in the soil mass (e.g. De Garidel et al, 1986; Gray et al, 1986). The bi-dimensional products are small (± 10 x 10 mm) grids also used for reinforcement by random distribution in the soil mass (e.g. De Garidel et al, 1986).

Grids, geogrids, webbings: These are relatively stiff nets with symmetrically shaped openings used for reinforcement.

Geotextiles: Woven, nonwoven and knitted geotextiles are planar, fabric-like products used for reinforcement, separation, filtration and fluid transmission. The term "geofabrics" is often used synonymously with geotextiles, sometimes refer–

ring to nonwovens where "geotextiles" refers to wovens. The author therefore disagrees with Giroud's use of the term 'geofabrics' referring to all planar structures, as indicated in Table 3.1. Rankilor (1981, p 92) uses the terms 'felts' and 'mats' for thick nonwoven geotextiles.

Mats, geomats, formed sheets: These are usually special application products used for surface protection e.g. erosion control. They sometimes incorporate special features such as cells for establishing plant growth.

Nets, waffles: Nets and waffles usually have a definite third dimension and are typically used for fluid transmission e.g. in drains as a substitute for granular filters and/or pipes. The term "geospacer" (Hoekstra et al, 1986) has been used for these products. Nets are made of randomly distributed fibres and differ from non-woven geotextiles in that the openings between fibres are much larger resulting in a very high permeability. The fibres are also thicker and stronger resulting in a more rigid structure. Waffles usually have a larger third dimension than nets, resulting in a larger fluid capacity. They are typically made from planes or sheets of polymers, rather than fibres, which are cast into various forms. One example is the so-called "egg box" waffle.

Geomembranes: These are impermeable sheets used for linings and as separators. In an earlier classification (Rankilor, 1981, p 86) the term "membrane" was used to describe geotextiles as well as impermeable sheets.

Geosynthetics: This term refers to all synthetic materials used in civil engineering applications. The author disagrees with Giroud's use of the term as referring only to macro-products (Table 3.1). The term should also include the micro synthetic products.

3.2.2.2 Recommended classification

Based on the discussion above, a general classification of synthetic materials is recommended in Table 3.2.

TABLE 3.2 RECOMMENDED CLASSIFICATION OF GEOSYNTHETICS

Products/Terms	Function
Fibres, filaments, yarns	Elements of other synthetic products used for reinforcement
Geotextiles, geofabrics	Woven, nonwoven and knitted used for reinforcement, separation, filtration and fluid transmission
Grids, geogrids, microgrids, webbings	Reinforcement
Geospacers, nets, geonets, waffles	Fluid transmission
Geomembranes	Impermeable fluid barriers
Mats, geomats, formed sheets, and any other special products	Special applications e.g. erosion control, vegetation establishment, etc.
Composites, Geocomposites	Combination of e.g. geotextile and geomembrane, geotextile and geonet - various applications

3.2.3 <u>Classification of Geotextiles</u>

In Section 3.2.2 above, geosynthetics, which include geotextiles, were classified. This section gives a further description of geotextiles or geofabrics (Rankilor, 1981, p 84; Main Roads Department, 1983, p 2; MacLean et al, 1984, p 50; Scullion et al, 1984, p 6; Cleaver et al, 1985, p 49). The term geotextiles is used throughout. The four basic types of geotextile are:

Wovens
Nonwovens
Knitted
and Composites

3.2.3.1 Synthetic materials

The geotextile market only became significant after synthetic materials that could resist rot became available (Giroud, 1986, p 3). Synthetic materials that are used for geotextiles include:

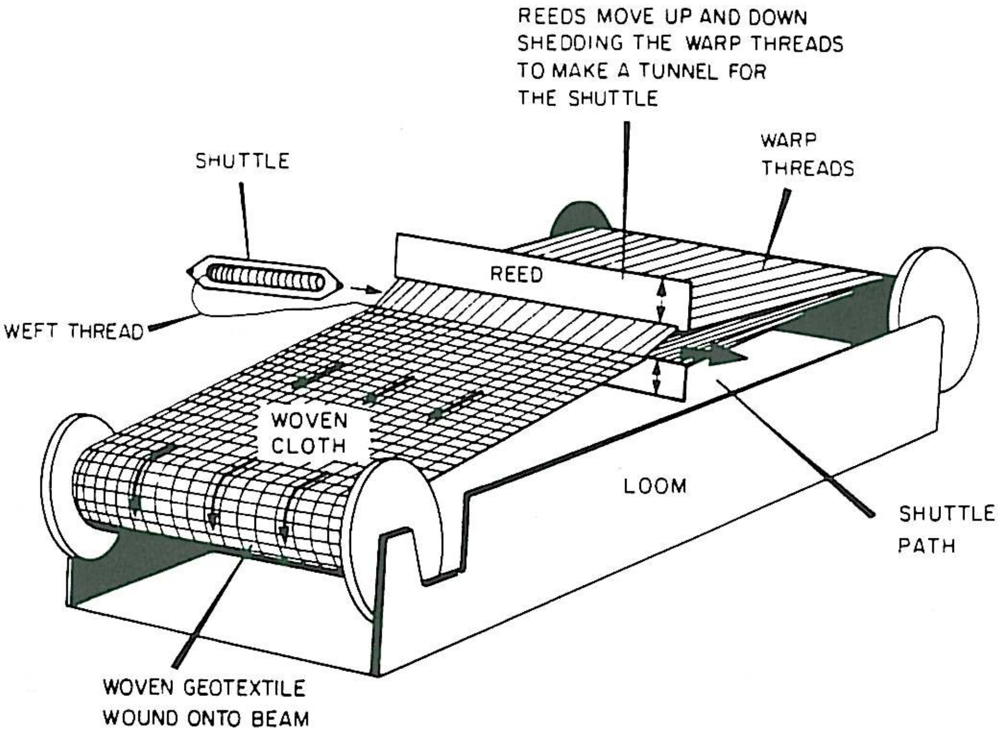
Polyolefines (polypropylene)
Polyesters
Polyamide (nylon)
Polyethylene
Polyvinylchloride.

Of these, polypropylene and polyesters are the most commonly used.

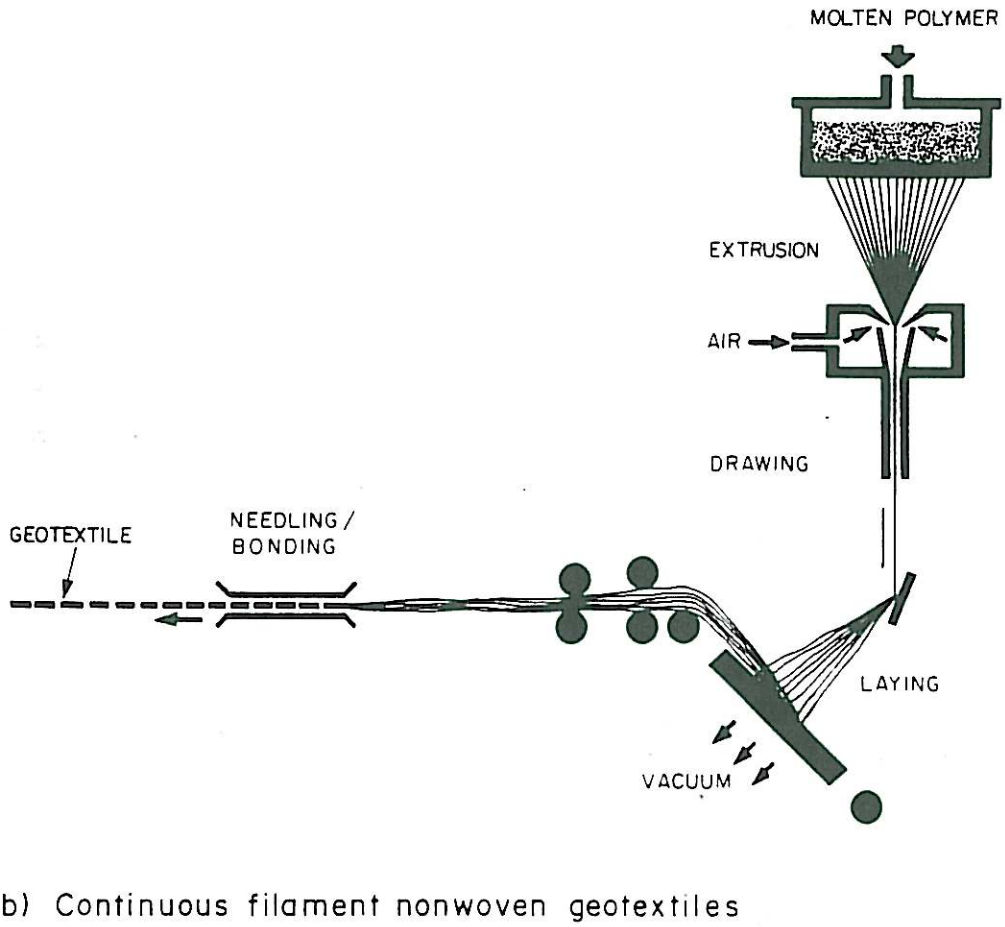
3.2.3.2 Woven Geotextiles

The manufacture of woven geotextiles is based on traditional textile weaving methods. The geotextile is manufactured on a loom, as shown in Figure 3.1(a). Warp threads run along the length of the loom and are lifted and lowered by reeds. The weft threads are inserted between the warp threads with a shuttle. Warp threads are usually stronger and stiffer than the weft which have to be flexible enough to allow the weaving process.

The end product obtained by weaving depends on the type of thread used, the method of weaving and after-treatment (stabilizing) of the woven product. A classification of woven geotextiles is given in Figure 3.2(a), and discussed below.

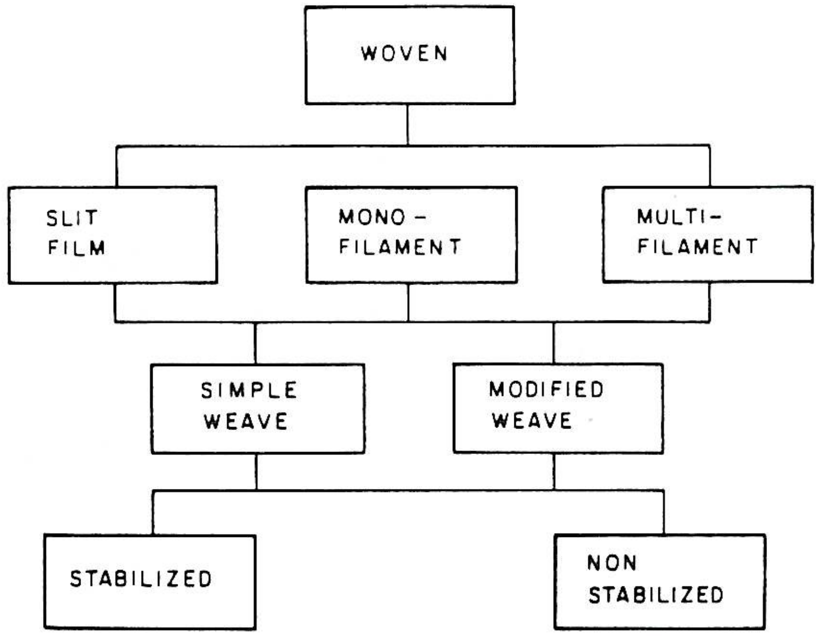


a) Woven geotextiles

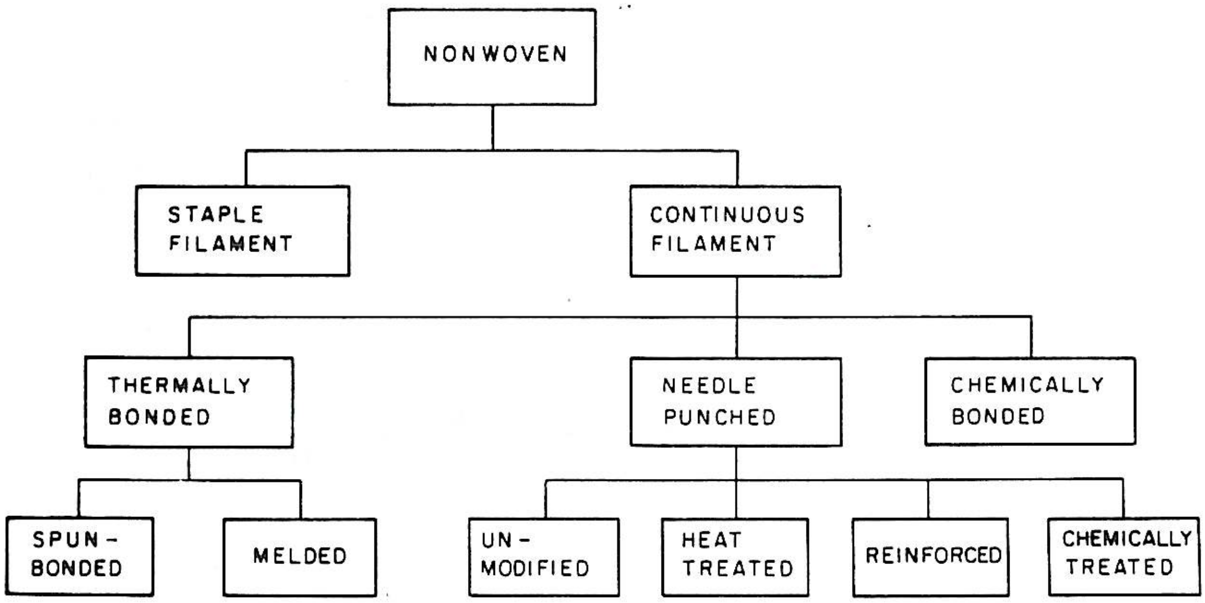


b) Continuous filament nonwoven geotextiles

FIGURE 3.1
SCHEMATIC ILLUSTRATION OF GEOTEXTILE MANUFACTURE



a) Woven geotextiles



b) Nonwoven geotextiles

FIGURE 3.2
CLASSIFICATION OF GEOTEXTILES

(a) Thread type

Slit film (ribbon filaments, thin strips, tapes): Slit film geotextiles are woven from thin strips, usually less than 5 mm wide, which are split from an extended sheet of synthetic material. These geotextiles are the most common wovens and are usually less permeable than those woven with monofilaments.

Monofilament: Monofilament threads are single fibres of constant diameter. Geotextiles that are woven with these threads can have larger openings resulting in higher permeability.

Multifilament: In multifilament woven geotextiles, the warp and/or weft threads consist of a number of single strands twisted together. These products are the most expensive and have the highest strength. Permeability is usually higher than that of slit film wovens.

(b) Weave pattern

Simple Weave is the basic weaving pattern where every warp thread intersects every weft thread and both threads are similar.

Modified weave is applied to alter the opening sizes, permeability and strength of the geotextiles. This can be done by varying the insertion patterns (e.g. a warp thread stepping over two weft threads at a time), using different types of thread for the warp and weft directions, or by simply reducing or increasing the number of warp or weft threads.

(c) Stabilizing

If a geotextile has an open weave with large openings, the threads can move relative to each other, thus distorting the openings. This is overcome by stabilizing the geotextile either by heat crimping or by chemical coating.

3.2.3.3 Nonwoven geotextiles

A classification of nonwoven geotextiles is shown on Figure 3.2(b). The first subdivision is staple filament and continuous filament geotextiles.

3.2.3.3.1 Staple filament (Drylaid)

The term staple filament refers to relatively short (50 to 100 mm) polymer filaments or fibres which are bound together by needling and secured by chemical bonding.

3.2.3.3.2 Continuous filament (Molten process)

Figure 3.1(b) shows the manufacturing process of continuous filament nonwovens. Continuous filaments are extruded from the molten polymer. The filaments or fibres are then laid in a random manner on a moving belt, prior to a bonding process. Three different processes of bonding are used namely thermal bonding, mechanical bonding (needle punching) and chemical bonding.

(a) Thermally bonded (heat bonded)

The laid fibres pass through a heating chamber and are heated to a point where the polymer is in a semi-liquid state. The fibres then bond to each other at the points of contact. When pressure is combined with heat, the term "calandering" is used. Depending on the types of fibre used, the geotextile is termed "spun bonded" or "melded".

Spun bonded: The process described above is applied to "monofilament" fibres, or fibres made of a single polymer.

Melded: The fibres are "heterofilaments", i.e. consists of two different polymers, one forming a core and the other an outer sheath. In the heat bonding process only the outer sheath reaches a semi-liquid state allowing the fibres to bond. The inner core remains in a solid state and its molecular structure is not altered.

(b) Mechanically bonded (Needle punched)

The "web" of filaments is passed through a needling chamber. Needles with spikes perforate the web, thus bonding the fibres by interweaving (interlocking) them. The "web" then becomes a sheet, which is the needle punched geotextile (unmodified). Further treatment is sometimes applied e.g. reinforcement by inserting continuous threads in the geotextile, a certain amount of heat treatment, or chemical treatment.

(c) Chemically bonded (resin dipped)

In this method of bonding epoxy resins or similar chemicals bond the fibres together. The geotextile is normally dipped in the chemical and fans or vacuums are sometimes used to remove the excess bonding agent.

3.2.3.4 Knitted geotextiles

Knitted geotextiles consist of a single strand systematically intertwined with itself and is manufactured with a knitting machine, instead of a weaving loom.

3.2.3.5 Composite Geotextiles (Multi-layered geotextiles)

These are manufactured by combining layers of different material. In most cases a nonwoven (usually staple filament) is needle punched onto a woven geotextile.

3.2.4 Functions, Applications and Properties

3.2.4.1 Functions

The functions performed by geotextiles in a variety of applications are given by Murray et al (1982, p 291) and Giroud (1986, p 16):

Filtration
Fluid transmission (drainage in the plane)
Separation
Protection
Reinforcement

The following is a brief discussion of each function:

(a) Filtration: The geotextile acts as a filter by allowing water to pass through and retaining the soil particles. This function is of particular importance in drainage applications.

(b) Fluid transmission: The geotextile acts as a water carrier by allowing flow in the plane. This function is important in certain drainage applications e.g. where the geotextile is wrapped around a pipe without a natural (granular) or synthetic (geonet) water carrier.

(c) Separation: The geotextile separates two different materials and ensures that one material does not contaminate the other. If a granular subbase is placed on a clayey subgrade a geotextile can be used to ensure that there is no movement of fine particles into the subbase material.

(d) Protection: The geotextile protects a material from damage e.g. where it is used for erosion control.

(e) Reinforcement: The geotextile provides tensile strength to soil or asphalt e.g. in embankments or in road structures. Giroud (1986, p 16) distinguishes between a tensioned membrane and a tensile member, indicating the two reinforcement functions. In an embankment the geotextile is a tensile member, and in reducing large deflections in an unpaved road it is a tensioned membrane.

3.2.4.2 Applications

A variety of applications exist for geotextiles in civil engineering, and new applications are found continuously. Table 3.3 (Murray et al, 1982, p 292) and Table 3.4 (Giroud, 1986, p 16) show the most important applications and the functions applicable to each application.

3.2.4.3 Properties

The method and materials used to manufacture geotextiles determine the various properties, e.g. strength, permeability, durability etc. These properties determine how well the geotextile will perform the various functions.

Table 3.5 (Murray et al, 1982, p 292) gives a summary of the different properties and their importance to the various functions.

Geotextile properties applicable to road subsurface drainage are described in more detail in Section 3.3 below and in Chapter 4.

TABLE 3.3 VARIOUS APPLICATIONS AND FUNCTIONS OF GEOTEXTILES (Murray et al, 1982, p 292)

Application	Function			
	Separation	Filtration	Drainage in the plane	Reinforcement
Roads, Railways and Area subgrade Stabilization	D	M	N.I	M
Drainage	M	D	N.I	N.I
Wet-Fill Embankments	M	D	D	M
Coastal and River protection	D	D	N.I	M
Land reclamation	M	D	N.I	M
Asphalt Reinforcement	N.I	N.I	N.I	D
Earth Reinforcement	N.I	N.I	N.I	D

D : Dominant function
M : Minor function
N.I : Not important

TABLE 3.4 RELATIONSHIPS* BETWEEN APPLICATIONS AND FUNCTIONS OF GEOTEXTILES
(Giroud, 1986, p 16)

Application category	Application area	Application type	Functions: Fluid Transmission	Filtration	Protection	Separation	Tensioned Membrane	Tensile Member
Hydraulic Applications	DRAINAGE	Geosynthetic drains without filter	X					
		Geosynthetic drains with filter (geocomposites)	X	X				
		Gravel drains, Pipes		X				
	EROSION CONTROL	Bank revetment		X				
		Erosion mat			X			
		Silt Fence, Silt curtain		X			X	
Geosynthetic Construction	CONTAINERS	Concrete forming, Sand bags (hydraulic fill)		X			X	
		Gabions, Sand Bags					X	
	GEOMEMBRANE SUPPORT	Bridging					X	
		Cushion			X			
Geotechnical Structures	ROADWAYS**	Asphalt overlay			X			
		Unpaved road (large deflection)				X	X	
		Base course (small deflection), Ballast				X		X
	SOIL REIN-FORCEMENT	Reinforced walls, slopes and embankments						X

* For each application type, only the most important functions are indicated (e.g., the table does not show that the fluid transmission function may be involved when a geotextile is placed under flat concrete slabs in a bank revetment, or when a geotextile is used in a railroad track on a saturated subgrade, etc.)

**Here, roadways are considered to include all types of traffic supporting structures such as roads, parking lots, staging areas, railroad tracks, etc.

TABLE 3.5 PROPERTIES IMPORTANT IN VARIOUS FUNCTIONS OF GEOTEXTILES (Murray et al, 1982, p 292)

GROUP	PROPERTIES	FUNCTIONS			REMARKS
		SEPARATION	FILTRATION	REINFORCEMENT	
1. Constructional and basic physical properties	Material Constituents	Not important	Not important	Minor-dominant	Constituent materials can be the dominant factor controlling stress-strain behaviour and surface friction
	Method of manufacture	Minor- dominant	Dominant	Dominant	This determines the structure which controls many properties hence it must be carefully controlled
	Mass per unit area	Dominant	"	"	A good measure of the consistency of the product quality
	Pore sizes	"	"	Not important	Of particular importance for separation and filtration and determines the capability of geotextiles to retain soil internally or at the soil-geotextile interface
	Open area	"	"	" "	
	Thickness	"	"	" "	Compression in soil and re-orientation of fibres during straining changes thickness and structure which modifies many of the geotextiles properties
	Rigidity and structure	"	"	Dominant	

TABLE 3.5 PROPERTIES IMPORTANT IN VARIOUS FUNCTIONS OF GEOTEXTILES (Murray et al, 1982, p 292) (Continued)

GROUP	PROPERTIES	FUNCTIONS			REMARKS
		SEPARATION	FILTRATION	REINFORCEMENT	
2. Mechanical and hydraulic properties	Overall stress-strain behaviour	Minor	Minor	Dominant	Reinforcement function is dependent on all these properties. Also the entire stress-strain-time relationship and not just peak load at a standard strain rate is significant
	Overall extension to rupture	Minor	Minor	Dominant	
	Creep & stress relaxation	Not important	Not important	Dominant	
	Local burst strength	Dominant	Dominant	Minor:dominant	The ability to resist and redistribute local loads are two of the principal benefits of geotextiles and must be associated with burst and tear resistance
	Tear propagation	Dominant	Dominant	Dominant	
	Surface friction/ adhesion	Not important	Not important	Dominant	To mobilise tensile strength there must be sufficient surface friction/adhesion
	Fluid flow capacity per unit area or unit thickness	Not important	Dominant	Not important	The capability of geotextiles to allow water to flow through them is fundamental to all filtration and drainage functions
3 Environmental properties	Durability Temperature stability Chemical stability U.V. light stability	Dominant	Dominant	Dominant	Geotextiles must be capable of maintaining their functional performances with time and therefore must accommodate the environment conditions in which they are required to peform.

3.2.5 Geotextiles used in investigations

The geotextiles used in the investigations described in Chapters 4 to 7 are listed below in Tables 3.6 (Nonwovens) and 3.7 (Wovens). Shown in these tables are the main characteristics that distinguish between geotextiles as described in Section 3.2.2 such as filament type and bonding method. One useful parameter that distinguishes between geotextiles is the mass per unit area. These were measured (See Section 4.3.1 for details) and the values are shown on Tables 3.6 and 3.7 for each geotextile.

3.3 FILTER CRITERIA FOR GEOTEXTILES

One of the main objectives of the research described in this document was to establish filter criteria for geotextiles. It was necessary therefore, to investigate existing criteria and to determine their validity and applicability for possible incorporation in the final criteria that are recommended.

In this section the various filter criteria that were found in the literature are described and critically evaluated. The test methods on which these criteria are based are also discussed as well as the granular filter criteria from which most of the geotextile criteria were derived.

3.3.1 Filter criteria for granular filters

Practical filter criteria for granular filters were first developed by Terzaghi in the early 1920's (Terzaghi, 1948). These criteria, with some refinements by later researchers, have been in use worldwide for the past more than 60 years. Their validity have been verified both in laboratory studies and in various field applications (eg. Cedergren, 1974 p. 182). It is not surprising then, that when synthetic filters became available, for which filter criteria had to be developed, researchers used the granular filter criteria as a basis. It is necessary, therefore, to look at the granualr

TABLE 3.6 NONWOVEN GEOTEXTILES TESTED

Geotextile No	Colour	Filaments		Bonding	Other Treatment	Mass per Unit Area*	
		Type	Polymer			(g/m^2)	CoV.(%)**
1	Grey	Continuous	Polyester	Needle punched	Unmodified	177	6,9
2						209	6,9
3						265	2,9
4						305	7,9
5						619	4,0
6	Grey	Continuous	Polyester	Needle punched	Unmodified	135	3,8
7						152	4,1
8						178	3,1
9						205	2,6
10						275	1,9
11						314	1,8
12						366	1,8
13						508	1,9
14	Brown	Continuous	Polypropylene	Needle punched	Unmodified	122	7,8
15						112	7,4

* 5 Samples of each grade were measured, average value is given
** CoV - Coefficient of Variance.

TABLE 3.6 NONWOVEN GEOTEXTILES TESTED (Continued)

Geotextile No	Colour	Filaments		Bonding	Other Treatment	Mass per Unit Area	
		Type	Polymer			(g/m^2)	CoV. (%)
16 17 18 19 20	Brown	Continuous	Polypropylene	Needle punched	Unmodified	164 197 298 455 228	3,9 7,2 8,6 3,3 6,3
21 22 23	Grey	Continuous	Polypropylene	Thermal (spun bonded)		111 136 190	2,9 1,1 1,1
24	White	Continuous	Polyester	Stiched (Reinforced)		140	5,4
25 26	Blue Brown	Staple	Polypropylene	Needle punched		556 618	5,1 7,4

TABLE 3.7 WOVEN GEOTEXTILES TESTED

Geotextile no	Colour	Filaments		Weave Pattern	Stabilizing	Mass per Unit Area	
		Type	Polymer			(g/m²)	CoV.(%)
27 28 29 30 31	Black	Slit film	Polypropylene	Modified Weft fibres over two warps Warp fibres over two wefts	None	111 137 206 361 531	3,4 0,7 0,1 1,0 1,1
32	Black	Monofilament	Polyethylene	Modified (two over two as above)	None	249	1,4
33	Grey (warp = black weft = white	Warp: Monofilament Weft: multifilament	Warp: Polyethylene Weft: Polyester			290	2,6
34	White	Multifilament	Polyester	Simple weave	None	378	0,4
35	White			Modified Weft fibre over one warp Warp fibre over two wefts		710	0,6

filter criteria before investigating the geotextile (synthetic) filter criteria.

3.3.1.1 Piping and permeability (Terzaghi criteria)

The two basic requirements for a filter are the piping and permeability requirements:

Piping requirement: The pore spaces in the filter must be small enough to prevent the soil particles from being washed through the filter.

Permeability requirement: The pore spaces in the filter must be large enough to impart sufficient permeability to avoid the build-up of hydrostatic pressures behind the filter.

It is eveident that these two requirements oppose each other and a double specification is therefore invevitable.

Different terminology is sometimes used to refer to the piping and permeability requirements eg. (Wates, 1986(a)):

Filtration and drainage: Adequate filtration implies the condition where the piping requirement is satisfied and adequate drainage refers to the condition where the permeability requirement is satisfied.

Stability and effectiveness: The filter is stable if the piping requirement is satisfied and the filter is effective if there is adequate permeability.

It was pointed out by de Mello (1977) that the piping requirement in fact implies that soil particles should not be washed through the filter in significant quantities and that a certain amount of piping is inevitable.

The original criteria that were established by Terzaghi were based mainly on emperical values obtained by laboratory experiments (Terzaghi, 1922). Bertram (1940), with the advice of Terzaghi, made further laboratory investigations and refined the criteria set by Terzaghi. The criteria were further refined and expanded by the US Army Corps of Engineers (1941) and the US Bureau of Reclamation (Karpoff, 1955).

The piping and permeability requirements discussed earlier refer to pore spaces in the filter. The criteria set by Terzaghi and others, however, use particle sizes since these are easily obtained simply by grading analyses of the filter. It should be kept in mind that, although particle sizes are used throughout in the filter criteria, it is in fact the pore spaces that determine whether a filter is effective and stable.

Although the actual criteria used today may differ slightly from one organization to another, the criteria are basically the same, as follows (Division of Roads and Transport Technology (DRTT), 1984(b)):

Piping criterion: $D_{15}f \leq 5 \times D_{85}s$

where $D_{15}f$ is the diameter of a particle of the filter where 15% of all other particles are smaller (the size of the sieve that allows 15% of the filter to pass through).

and $D_{85}s$ is the diameter of a particle of the soil where 85% of all other particles are smaller.

Permeability criterion: $D_{15}f \geq 5 \times D_{15}s$

where $D_{15}f$ is as defined above

and $D_{15}s$ is the diameter of a particle of the soil where 15% of all other particles are smaller.

These criteria are illustrated on Figure 3.3. The piping criterion gives a maximum value for the D_{15} of the filter and the permeability criterion gives a minimum value for the D_{15} of the filter. The piping criterion is relative to the D_{85} of the soil (large soil particle, 85% is smaller) and the permeability criterion is relative to the D_{15} of the soil (small soil particle, 15% is smaller).

In addition to the piping and permeability criteria, a compatibility criterion is also shown on Figure 3.3, namely:

$$D_{50}f \leq 25 \times D_{50}s$$

where $D_{50}f$ and $D_{50}s$ refer to the 50% particle sizes of the filter and soil respectively.

This additional criterion is based on the requirement of the USBR (Karpoff, 1955) that the grading curves of the filter and the soil should be somewhat parallel.

Other restrictions and criteria are also often imposed eg.:

- Percentage passing 0,075mm sieve of the filter must be less than 5% (Terzaghi, 1948)
- Maximum stone size of the filter should be 75mm (USBR)
- When the soil contains a large percentage of gravels, the filter should be designed on the basis of the grading curve of the portion of material which is finer than 25mm. (Sherard et al, 1963).

Examples of the use of these filter criteria to design granular filters are shown in Chapter 5 (Section 5.3.3).

3.3.1.2 Internal stability (Auto stability)

The internal stability refers to the requirement that the

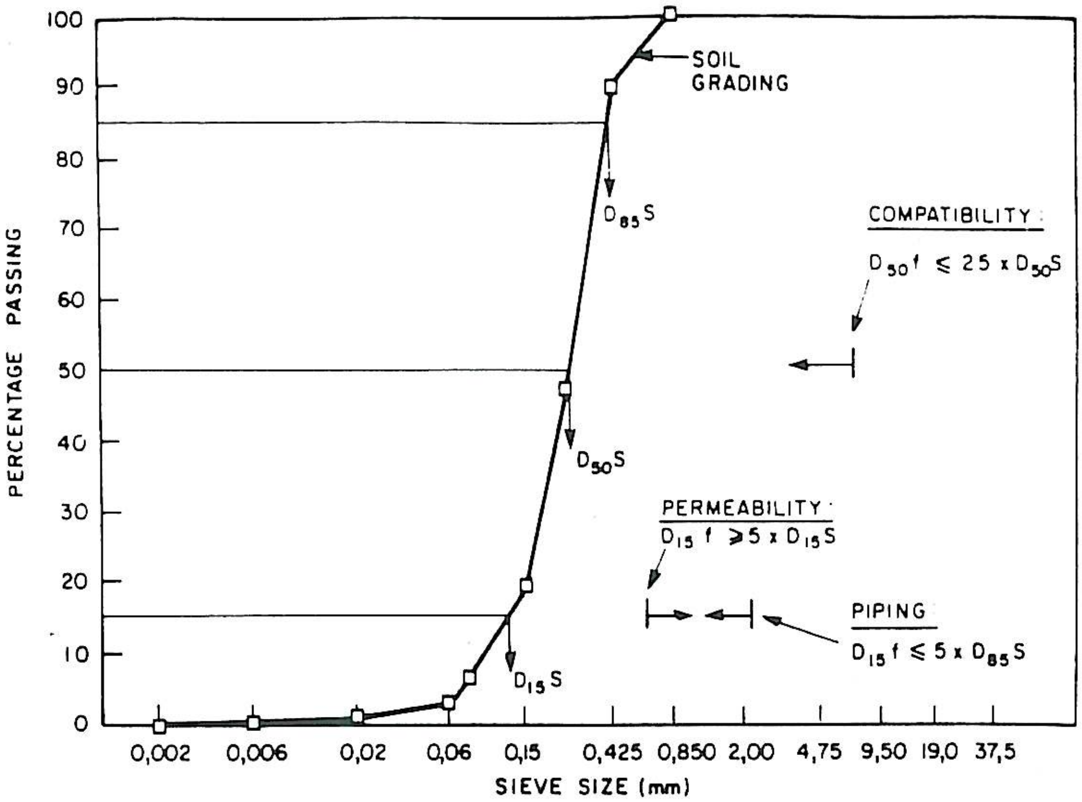


FIGURE 3.3 *GRANULAR FILTER : TERZAGHI CRITERIA*

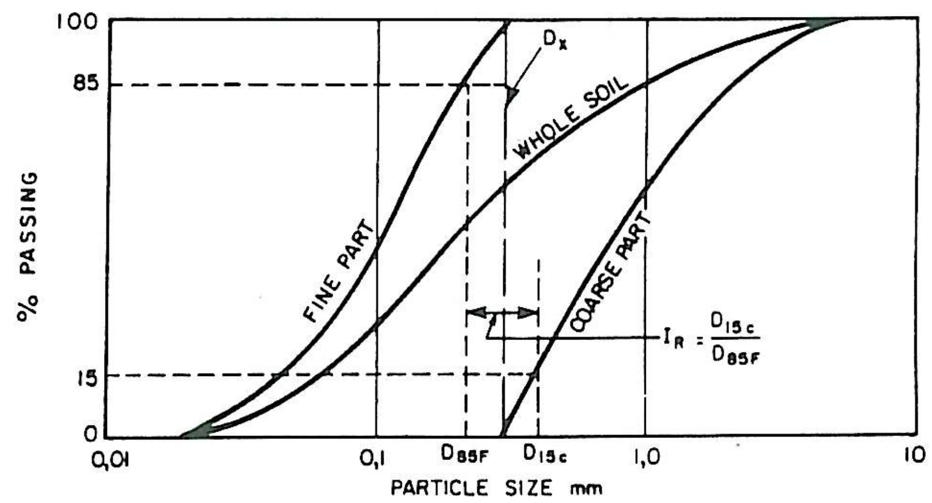


FIGURE 3.4 *INTERNAL STABILITY, DE MELLO (1975)*

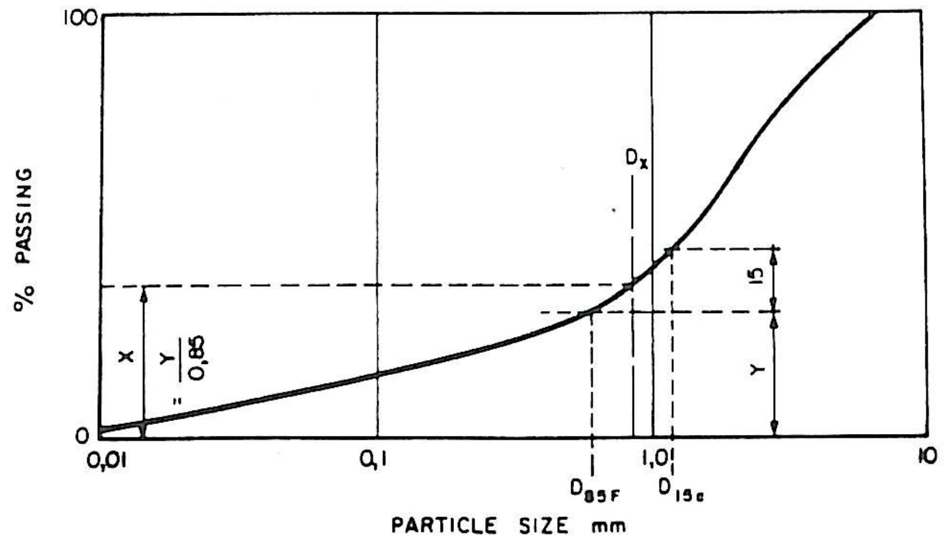


FIGURE 3.5 *INTERNAL STABILITY : SHERARD (1981)*

particles in the filter material should themselves be retained and should not migrate within the filter. The filter is instable if it cannot prevent the loss of its own small particles.

Scheurenberg (1986) gives a good overview of the various criteria available to control internal stability and the historical development of these criteria.

The best known method is where the filter is divided into coarse and fine fractions, using the piping criterion described in Section 3.3.1.1 above, assuming the coarse fraction to be the soil, as follows (de Mello, 1975):

$$D_{15} \text{ (coarse fraction)} \leq 5 \times D_{85} \text{ (fine fraction)}$$

$$\text{or} \quad D_{15}C \leq 5 \times D_{85}F$$

$$\text{or} \quad \frac{D_{15}C}{D_{85}F} \leq 5$$

The concept is illustrated in Figure 3.4.

The above method only applies to gap graded materials, but Sherard (1984) developed the method further to include any broadly graded filter material. Sherard defined the ratio $D_{15}C/D_{85}F$ as the Internal Stability Ratio (I_R) and stated that this ratio should be less than 5 throughout the grading curve.

Sherard showed that I_R can be calculated over the entire grading curve as follows (See Figure 3.5):

If the curve is split into coarse and fine fractions at a point D_x, equivalent to X% finer (where X = Y/0,85), then D_y and $D_{(y+15)}$ represent $D_{85}F$ and $D_{15}C$ respectively.

$$\text{Then } I_R = \frac{D_{15}C}{D_{85}F}$$

$$= \frac{D_{(y+15)}}{D_y} \leq 5$$

This value can be calculated at any point, X, by choosing different Y values.

An example of how this method is applied is shown in Capter 5 (Section 5.3.3).

3.3.2 Determining pore sizes in geotextiles

In Section 3.3.1 above it was shown that, although filter requirements refer to pore sizes (or opening sizes) of the filter, the criteria for granular filters are based on grain sizes. Geotextiles do not consist of grains, however, and various methods have been developed to obtain effective pore sizes of geotextiles. Existing filter criteria that are discussed in Section 3.3.4 below, are based on these effective pore sizes.

Two basic methods exist for determining pore sizes, namely direct methods and indirect methods.

3.3.2.1 Indirect methods

(a) Inverse (reverse) sieving

This is the most widely used method of determining pore sizes in geotextiles. The method involves sieving glass balls (ballotini balls) of varying sizes through the geotextile and recording the percentage of each size that is retained on the geotextile as well as that which passes through the geotextile. The various pore sizes in the geotextile and a pore size distribution can then be obtained. The basic assumption is that the sizes of the glass balls reflect the pore sizes in the geotextile.

The method is discussed in more detail in Chapter 4 (Section 4.2.1), where tests using this method are described.

There are many variations of this method including:

- The use of sand instead of glass balls
- The use of a water spray which washes the beads through the geotextile while it is vibrated
- Different frequencies, amplitudes and times of vibration.

Although this method of determining pore sizes and the use of criteria based thereon is very popular due to the apparent similarity to granular filter criteria, numerous problems exist with the method including

- The true shapes and sizes of the pores are not necessarily the same as those of the spherical glass beads
- Factors such as compression of the geotextile and the opening of the pores under stress are not taken into account (Polycroft et al, 1982, p71). These factors do not exist with granular filters
- The test method has not been standardized and the different variables that can have an influence on the results include duration of test, type of glass beads or sand, frequency and amplitude of vibration, the use of a water spray, the water pressure and type of nozzle used. Wates (1986(b), p11) showed that an increase in the duration of sieving from 5 to 25 minutes can result in a 100 per cent difference in results
- The repeatability is poor. According to Legge (1986) errors using standard procedures in different laboratories are of the order of 50 per cent. Wates and Wittstock

(1986, p86) showed that effective pore sizes differ by more than 100 per cent if three layers of geotextile are used instead of one

- The pore sizes obtained are only an approximate estimate of the larger openings (McGown et al, 1982, p168). Interparticle forces make it impractical to use beads smaller than 60 to 70 micron and extrapolation must therefore be used in the case of nonwoven geotextiles, with pore sizes as small as 10 micron (Wates, 1986(b), p2).

(b) Viscosity method

This method has not been used to date, but is merely suggested by the author as a possible indirect method of determining indicative pore sizes.

The flow of liquids through a tube is governed by the equation

$$r^4 = \frac{8 \times V \times l \times \eta}{\pi p}$$

where r is the radius of the tube, V is the volume of water escaping from the tube, l is the length of the tube, η is the absolute viscosity and p is the difference of pressure at the ends of the tube.

If a tube with a certain radius r is used, and liquids with different viscosities are allowed to flow through the tube, different volumes of flow will be recorded from which, theoretically, the same radius, r, will be calculated.

If a geotextile is mounted in the tube so as to allow the liquid to pass through the geotextile, the volume will decrease and the radius, r', thus obtained, will be an effective radius equal to the sum of the radii of all the channels in the area of geotextile through which the liquid passed.

If the procedure is repeated with a different liquid with, say, a lower viscosity, a larger volume of this fluid will pass through the larger channels in the geotextile than was the case with a high viscosity fluid and a different effective total radius r' will be obtained.

By repeating the procedure with a series of fluids with different viscosities, a "pore size distribution" or a distribution of r' values can be obtained.

The main disadvantage of such a method would be that the actual sizes of the individual pores or channels cannot be determined. The advantage would be that a higher degree of repeatability will be obtained and the test will be relatively easy to standardize.

3.3.2.2 Direct Methods

(a) Image projection (wovens)

An image of a specimen is projected on a screen and the openings are measured (Calhoun, 1972, p5). This metehod is relatively simple and accurate, but is restricted to woven geotextiles which have distinct, symmetrical openings.

(b) Photography (nonwovens)

This method was used by Wates et al (1986, p 85). Samples of geotextile were prepared by gold plating under vacuum and were then photographed after enlargement using an electroscanner. The distribution of pores was determined by either measuring the linear distance between filaments at the surface or by fitting circles to pores defined by the intersection of filaments.

The disadvantages of this method are that specialized equipment is needed and that the pore size distribution is based on measurements at the surface. Pores thus measured are not necessarily representative of pores or channels through the geotextile.

3.3.3 Permeability

Permeability usually refers to the permeability coefficient k, in Darcy's law. The units are meters per second. To calculate this the hydraulic gradient and hence the thickness of the filter, in this case the geotextile, must be known. This can be estimated by measuring the thickness under a specified pressure with a pressure loading system. This is not a true reflection of the flow path length, however, since the exact amount of compression that will occur is not known, not in the case where only water passes through the geotextile, nor in the true filter situation where the geotextile is in direct contact with soil. In the latter case, the thickness can also change due to the ingress of soil particles.

The method most widely used is to express the permeability in litres per square meter, thus ignoring the thickness of the geotextile. The head loss across the geotextile must then be quoted. The term "permittivity" is sometimes used as opposed to permeability to indicate the difference in units. Both falling head and constant head permeameters are used, but recent trends are towards constant head because accurate measurements with the falling head method are difficult due to the high permeabilities of geotextiles. In Chapter 4 (Section 4.4.2) the measurement of geotextile permeability is illustrated.

3.3.4 Existing Filter Criteria

The following is a discussion of the various criteria that exist. The criteria given are those that apply to a one way filter situation, i.e. where water only flows in one direction. Criteria have also been developed for reverse filter situations (e.g. coastal protection), but these have been omitted because they are not applicable.

In each case a summary of the criteria is given, followed by the method by which they were determined (if available) and a critical discussion of the criteria.

The following notation is used throughout.

The filter system consists of a soil (base soil, in-situ soil, soil to be filtered) plus a geotextile or a soil plus a granular filter.

Then O represents the diameter of the openings in the soil (s), geotextile (m) or granular filter (f)
D represents the diameter of the particles (grains) of the soil (s) or the granular filter (f)

For example:

$O_{15}m$ is the diameter of an opening in a geotextile where 15% of all other openings are smaller.
$O_{85}s$ is the diameter of an opening in the soil where 85% of all other openings are smaller
$D_{15}s$ is the diameter of a particle of soil where 15% of all other particles are smaller (The size of the sieve that allows 15% of the soil to pass through)
$D_{85}f$ is the diameter of a particle of the granular filter where 85% of all other particles are smaller

In most cases, the system referred to consists of a soil plus geotextile and the suffixes s, m and f have been omitted. Then

O_{15} refers to the openings in the geotextile
D_{15} refers to the particles in the soil.

3.3.4.1 Calhoun (1972)

(i) Summary

For woven geotextiles:

(a) For granular materials with less than 50% passing the 0,075 mm sieve ($D_{50} \geq 0,075$)

$$0,15\text{mm} \leq O_{50} \leq D_{85}$$
$$\text{and} \quad \%\ OA \leq 40\%$$

(b) For cohesionless materials with more than 50% passing 0,075mm ($D_{50} < 0,075$):

$$0,15\ \text{mm} \leq O_{50} \leq 0,212\ \text{mm}$$
$$\text{and} \quad \%\ OA \leq 10\%$$

(ii) Method

Calhoun determined the effective opening size (EOS) and percent open area (%OA) by projecting an image of the woven geotextiles on a screen and physically measuring the opening sizes. The variation in opening sizes are small, and EOS can be substituted with D_{50} (Rankilor, 1981, p204).

The criteria give minimum and maximum values for opening sizes of the geotextiles, relating to permeability and piping respectively.

The criteria were obtained by laboratory tests and some field observations. Laboratory tests were carried out in a permeameter with water flowing through a soil sample and the geotextile. Seven geotextiles were tested and seven different soil gradings were used.

The ability of the geotextiles to retain soil (resist piping) was determined visually by observing the colour of the water.

The resistance to clogging was determined by measuring change in hydraulic gradients over a period of 160 minutes. Piezometers were installed at different positions in the soil sample and at the outlet. The gradient ratio was defined as:

$$\text{gradient ratio} = \frac{i'}{i}$$

where i' = hydraulic gradient over the bottom 25 mm of soil plus the geotextile

i = hydraulic grandient over the entire soil sample plus the geotextile

It was assumed that a larger hydraulic gradient in the bottom portion of the sample than in the total sample (gradient ratio > 1,0) indicated clogging. In later work (US Army Corps of Engineers, 1977) a limiting value of 3,0 was placed on the gradient ratio. Work by Haliburton et al (1982, p 100) using a similar apparatus and also using the gradient ratio as an indication of clogging, indicated that the D_{50} value (or EOS) had no relationship to clogging behaviour, but the percent open area was almost directly related to clogging potential.

(iii) Discussion

The criteria were developed for woven geotextiles and for a limited variety of soils (cohesionless only). Although the laboratory tests were done on soil-geotextile systems, thus taking the actual interaction between the soil and geotextile into account, the criteria were based on results obtained after only 160 minutes. In Chapter 5 of this document it will be shown that soil/geotextile compatibility tests should be carried out over a period of approximately 400 hours.

The method of determining clogging potential by using the gradient ratio is not necessarily a true indication of clogging. Koerner et al (1982, p 93) did long term flow tests using apparatus similar to that used by Calhoun. The reduction in flow rate through the soil and geotextile with time were recorded as well as the change in gradient ratio. While the flow rate was decreasing the gradient ratio remained constant and when the flow rate reached a state of equilibrium (constant flow) after approximately 100 hours, the gradient ratio increased continuously and above the value of 1,0, indicating a non-equilibrium situation and potential for clogging. The work of Koerner et al is described in more detail in Section 3.4.1.3.

3.3.4.2 Zitsher (1975)

(i) Summary

(a) For woven geotextiles and for sands with 0,1 mm $\leq D_{50} \leq$ 0,3 mm and $U < 2$ (i.e. uniform poorly graded)

$$O_{50} \leq (1{,}7 \text{ to } 2{,}7) \times D_{50}$$

(b) For nonwoven geotextiles and cohesive soils:

$$O_{50} = (25 \text{ to } 37) \times D_{50}$$

(ii) Method

(a) The criterion for woven geotextiles was obtained from "practical investigations". No further detail could be obtained.

(b) For nonwoven geotextiles Zitscher's criteria were derived as follows by considering granular filters. He states that for granular filters $D_{50}f/D_{50}s$ = 100 to 150 and that for a soil

$O_{50}s$ = approx. ¼ of its $D_{50}s$ and hence:

$$\frac{O_{50}f}{D_{50}s} = 25 \text{ to } 37$$

To extend this to geotextiles $O_{50}f$ (granular) is substituted by O_{50} (geotextile).

(iii) Discussion

No comments can be made on the criterion for woven geotextiles, since no information is available. It is not clear whether the criterion is for piping only or for both piping and permeability.

The criterion for nonwovens was obtained by drawing a parallel between granular filters and geotextiles. This is probably an over simplification of the soil-geotextile interaction. The criterion was not verified by any laboratory tests. The exact meaning of the criterion, whether it relates to piping or to clogging, is also not clear. The Terzaghi criteria for granular filters state that $D_{50}f \leq 25 \times D_{50}s$ for compatability. Piping is governed by $D_{85}s$ and clogging by $D_{15}s$.

3.3.4.3 ICI fibres (1978)

(i) Summary

For nonwoven geotextiles

(a) For middle range soils (silts) with $0{,}02 \text{ mm} \leq D_{85} < 0{,}25$ mm

$$O_{50} = (\leq) D_{85}$$

(b) For silts and clays with $D_{85} < 0{,}02$ mm

$$D_{85} < O_{50} < 0{,}07 \text{ mm}$$

(c) For sands and gravels with $D_{85} > 0{,}25$ mm
and $D_{15} > 0{,}02$ mm

$$O_{50} \geq D_{15}$$

(ii) Method

These criteria were derived from existing granular criteria:

(a) The Terzaghi criterion to avoid piping in granular filtes states:

$$D_{15}f \leq 5 \times D_{85}s$$

Atterburg (1908) determined a relationship between the particle sizes and opening sizes for granular materials

$$O_{50}f = 0{,}2\ D_{15}f$$

or

$$D_{15}f = 5 \times O_{50}f$$

By substituting $D_{15}f$ in the piping criterion:

$$O_{50}f \leq D_{85}s$$

To relate this to geotextiles $O_{50}f$ is substituted by $O_{50}m$:

$$O_{50}m \leq D_{85}s$$

ICI recommend that the O_{50} of the geotextile should be equal to or slightly less than D_{85} of the soil, presumably to cater for piping as well as permeability.

(b) If $D_{85} < 0{,}02$ mm, acccording to ICI fibres, the large portion of clay particles assist with the formation of the filter, and the choice of a filter with O_{50} less than 0,02mm is unnecessary. The criterion given above for these soils is given without an explanation of how it was obtained in the documents available to the author (Rankilor, 1981, p206)

(c) The criterion that ensures permeability in granular filters is:

$$D_{15}f \geq 5 \times D_{15}s$$

The relationship between particle size and opening size for granular filters as given in (a) above:

$$D_{15}f = 5 \times O_{50}f$$

By substituting $D_{15}f$ in the permeability criterion, and $O_{50}f$ with $O_{50}m$ for geotextiles:

$$O_{50}m \geq D_{15}s$$

(iii) Discussion

These criteria have been calculated using the granular filter criteria as basis. Once again the interaction between the soil and the geotextile has not been considered and no laboratory tests were performed to evaluate the criteria.

3.3.4.4 Ogink (1975)

(i) Summary

The criteria were developed for sands.

(a) For woven geotextiles:

$$O_{90} = D_{90}$$

(b) For nonwoven geotextiles:

$$O_{90} = 1,8 \times D_{90}$$

(ii) Method

The criteria were developed from laboratory studies. The method, however, is not given in the available literature (Rankilor, 1981, p208).

(iii) Discussion

From the available literature, it seems that these criteria are mainly aimed at preventing piping, but the fact that a single value is given, indicates that this value is an optimum for both piping and permeability.

3.3.4.5 Giroud (1982)

(i) Summary

For cohesionless materials

(a) Permeability:

k (geotextile) > 0,1 x k (soil)

(b) Piping

	$1 < U' < 3$	$U' > 3$
Loose soil	$O_{95} < U' \times D_{50}$	$O_{95} < \frac{9}{U'} \times D_{50}$
Medium dense soil	$O_{95} < 1{,}5 \times U' \times D_{50}$	$O_{95} < \frac{13{,}5}{U'} \times D_{50}$
Dense soil	$O_{95} < 2 \times U' \times D_{50}$	$O_{95} < \frac{18}{U'} \times D_{50}$

Where U' = Linear uniformity coefficient of soil. This parameter is similar to uniformity coefficient (D_{60}/D_{10}), but is obtained by drawing a straight line as close as possible to the central portion of the grading curve, thus eliminating the coarsest and finest fractions of the soil.

(ii) Method

(a) The permeability criterion is calculated using Darcy's law of flow and by calculating the increase in pore pressure due to a filter with a different permeability than that of the soil. In the calculations, a thickness of 10 mm is assumed for the geotextile (a conservative value; the thicker the geotextile the larger the increase in pore pressure). An increase in pore pressure of 10 per cent is allowed and a further safety factor of 10 is used.

(b) The retention criteria are based on geometric calculations assuming spherical particles and round openings in the geotextile. It is further assumed that:

(i) If U' < 3, all the particles are interlocked, forming a stable structure. The only requirement is that the filter retains the coarsest particles. If U' > 3, the soil particles are not stable and it is not sufficient to retain only the coarsest particle.

(ii) If the soil is dense two particles must move simultaneously to leave the soil structure unstable, but if the soil is loose the soil will be left unstable if only one particle moves.

(iii) Discussion

(a) The criterion assumes that the permeability of the geotextile remains the same throughout its life. The potential for clogging with time is not considered at all.

To determine the permeability coefficient of the geotextile, its thickness must be known. Although its thickness under a certain pressure can be determined, the effective thickness and effective permeability will be different due to ingress of soil particles into the openings.

The soil/geotextile interaction is thus not taken into account in this criterion.

Furthermore, McGown et al (1982, p 168) showed that the permeability coefficient of a geotextile cannot be determined using Darcy's law and that pressure should be taken into account.

(b) The calculation of the retention criteria is based on numerous assumptions. Giroud recognises the fact and mentions all the assumptions and simplifications.

3.3.4.6 Heerten (1982)

(i) Summary

For static load conditions.

(a) Piping (Sand tightness)

non cohesive soils:

$$U > 5 : O_{90} < 10 \times D_{50}$$
$$\text{and} \quad O_{90} \leq D_{90}$$
$$U < 5 : O_{90} < 2,5 \times D_{50}$$
$$\text{and} \quad O_{90} \leq D_{90}$$

Cohesive soils:

$$O_{90} < 10 \times D_{50}$$
$$\text{and} \quad O_{90} \leq D_{90}$$
$$\text{and} \quad O_{90} \leq 0,1 \text{ mm}$$

(b) Permeability (Permeability reduction)

<u>wovens</u>

$$\eta_G \times kn \geq k$$

where η_G = permeability reduction factor
kn = permeability coefficient of geotextile at expected load conditions
k = permeability of the soil

η_G is given on a figure as a function of kn and D_{10} of the soil.

<u>Non-wovens</u>

$$O_{90} < 0,5 \times D_{10}$$

If this criterion cannot be met then

$$\eta_v \times kn \geq k$$

where η_v is the permeability reduction factor for non-wovens and is a function of

$$\frac{k\eta^2}{n.Tg.O_{90}}$$

where n = porosity of geotextile
Tg = thickness under 2kPa

(ii) Method

Heerten's criteria were developed from a combination of field and laboratory tests. A total of 16 geotextiles were dug out of existing revetments on seadikes and inland waterways after a number of year's service. These geotextiles were examined in the laboratory together with new samples of the same geotextiles to determine the change in properties that occurred. The changes in permeabilities, porosity and thickness of the geotextiles were determined and from these the criteria were developed. The soils used in the investigation had the following ranges of characteristics:

D_{10}: 0,003 - 0,10
D_{50}: 0,09 - 0,30
U: 1,5 - 60,0

(iii) Discussion

In the development of these criteria, Heerten considered the interaction between soils and geotextiles, quantified the permeability reduction rather than initial permeability and considered actual field applications. It is the author's opinion that this attempt was a considerable step forward in determining realistic filter criteria.

Some comments on the test method are valid, however. The tests in the laboratory were carried out on the geotextiles plus the soil that had infiltrated the material, without considering the soil immediately above or below the geotextile. The reduction in permeability is not only due to the clogging or blocking of the geotextile as such, but is also dependent on the permeability of the filter zone that builds up directly upstream of the geotextile.

Although the time of installation and hence the age in years of the installations are known, the amount of water that actually passed through the filters is not, and it is quite possible then that soil-geotextile combinations that were found to be successful in certain applications would fail in other, morc critical applications. This fact was not considered in determining the filter criteria.

3.3.4.7 Lawson (1986)

(i) Summary

$$O_{90} = C \; Dn$$

C is a constant of which different values are given for different values of Dn. Three values are given representing the piping limit, the permeability limit and an intermediate limit above which a certain amount of piping will occur, but a state of equilibrium will still be reached. The values shown are limited to one soil type which was investigated.

(ii) Method

The work by Lawson was based on laboratory experiments where long term flow and piping was monitored using 6 different geotextiles.

(iii) Discussion

The approach taken by Lawson is, in the author's opinion, the correct one. The criteria are based on actual performance that was determined with long term flow tests, thus taking into account the soil-geotextile interaction as well as the time factor.

The method of quantifying the opening sizes of the geotextile is, however, not entirely satisfactory, as discussed in Section 2 above. The scope of the investigation was also limited to only one soil type and 6 geotextiles. The geotextiles, however, covered a wide range of opening sizes. Lawson's work is described in more detail in Section 3.4.1.4.

3.3.4.8 Ragutzki (1973)

(i) Summary

$$O_{90} < (0{,}5 \text{ to } 0{,}7) \times D_{50} \text{ (piping)}$$

(ii) Method

Unknown

3.3.4.9 Schober & Teindl (1979)

(i) Summary

$$O_{90} < B \times D_{50} \text{ (piping)}$$

where B depends on uniformity

(ii) Method

Unknown

3.3.5 CONCLUSIONS

It is evident that there are numerous shortcomings in the criteria that exist for synthetic filters. The most important shortcomings can be summarized as follows:

- There are widely differing criteria that were developed from different (non-standardized) tests
- In most cases the soil and the geotextile were evaluated separately with so-called in-isolation tests with no attention given to the soil-geotextile interaction. The in-isolation tests (pore size determination and permeability) have serious shortcomings.
- The criteria control piping and permeability and little or no attention is given to long term performance.
- The comparisons made between a geotextile and a granular filter is an oversimplification since geotextiles do not consist of grains and the opening sizes can change due to stresses.
- Very little information exists on the long term field behaviour of geotextiles to confirm the validity of any criteria.
- In most cases the criteria are based on oversimplified models and assumptions.

- The criteria are based on work done overseas with geotextiles and soil types available in those countries. South African materials therefore need to be tested locally.

3.4 SOIL/GEOTEXTILE COMPATIBILITY TESTS

In Section 3.3 the shortcomings of existing filter criteria for geotextiles were illustrated, the most important being that the soil/geotextile interaction is not considered.

A number of researchers have, however, attempted to determine the behaviour of geotextiles as filters by considering the interaction between the soil and the geotextile. The methods and results obtained in these studies are discussed in this section.

Two types of tests have been performed with soil/geotextile combinations. These are long term flow tests with permeameters and accelerated tests using the so-called interface flow capacity (IFC) tests. The two types of tests are discussed below in sections 3.4.1 and 3.4.2 respectively.

3.4.1 Long term flow tests

In these tests a geotextile is mounted in a cylinder or permeameter. A soil sample is placed on top of the geotextile and water from a constant head supply is allowed to flow through the soil and geotextile. The rate of flow through the system is measured at regular intervals. This flow or the calculated permeability coefficient is then plotted against time. A number of researchers have performed these tests and their test apparatus, results and conclusions are discussed below.

3.4.1.1 Rankilor (1981)

Information given by Rankilor is based on research by McGown et al (1973) and Marks (1975).

(i) Apparatus, method and materials tested

A schematic illustration of the apparatus used by Marks (1975) is shown on Figure 3.6(a). No detail is given regarding the dimensions, method or range of materials tested.

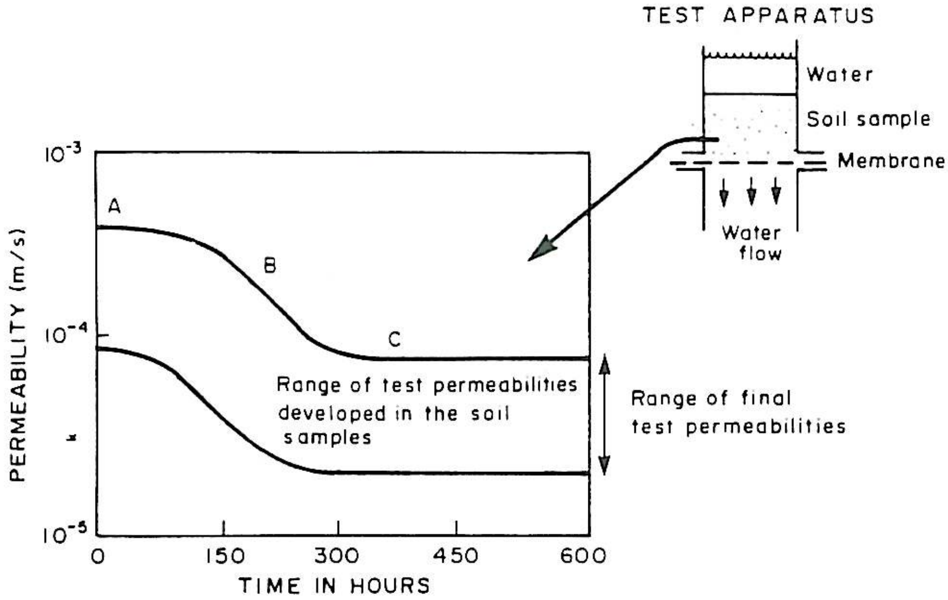
(ii) Results

Typical results are shown on Figure 3.6(a), indicating that the permeability coefficient, k, decreased initially and then stabilized after about 300 hours. The decrease in permeability is attributed to the build-up of a filter zone in the soil sample behind the geotextile, rather than clogging of the geotextile.

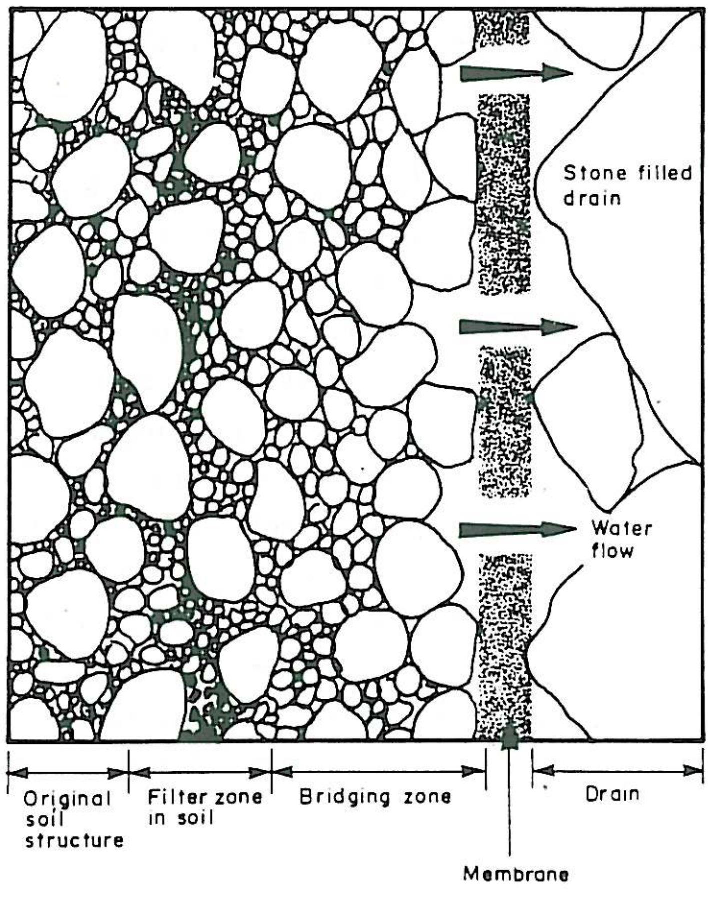
The forming of the filter zone is described by Rankilor, referring to Figure 3.6(b). At the start of the test the original soil structure is intact up to the soil/geotextile interface. The permeability is then that of the soil (A in Figure 3.6(a)). As the test continues, water washes the fine soil particles out of the bridging zone through the geotextile, resulting in a certain amount of piping. At the same time, fine particles build up in the filter zone, resulting in a decrease in permeability in that zone (B in Figure 3.6(a)). This decrease in permeability continues until the forming of the filter zone is completed after which the permeability remains constant (C in Figure 3.6(a)). The filter zone is sometimes called the "filter cake", but the author is of the opinion that the term can be misleading.

(iii) Conclusions

Rankilor concludes as follows:



(a) TEST APPARATUS AND TYPICAL RESULTS



(b) MAGNIFIED CROSS-SECTION THROUGH SOIL / GEOTEXTILE

FIGURE 3.6

LONG-TERM TESTS REPORTED ON BY RANKILOR (1981)

(a) The final permeability of the system is that of the filter zone, which is less than the permeability of either the soil or the geotextile

(b) Since it is not the geotextile, but the filter zone that filters the soil, the exact size of the holes in the geotextile are not important.

(c) The geotextile supports the bridging network, and if it is removed the bridging network will break away, followed by the filter zone and the soil will recommence piping. Good contact between the geotextile and the soil is therefore important.

3.4.1.2 Polycroft and Jones (1982)

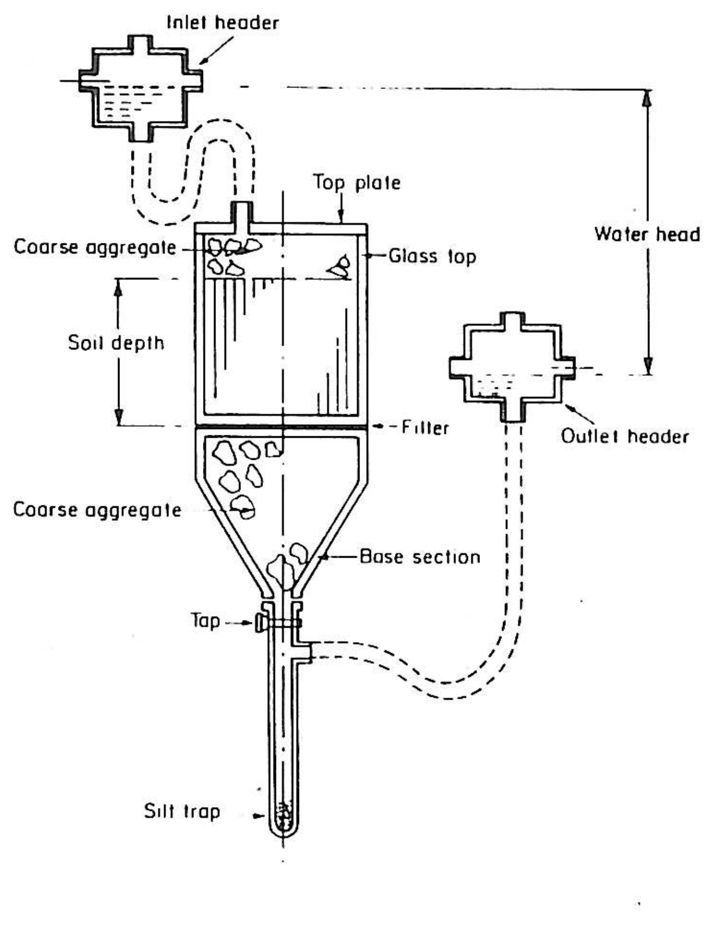
(i) Apparatus, method and materials tested

The apparatus used by Polycroft and Jones is shown in Figure 3.7(a). The geotextile sample with a 200 mm diameter was supported by aggregate. The water head was controlled by an outlet header, thus resulting in a "full flow" condition on the downstream side of the geotextile. The soil was placed in layers and the permeameter was filled from the bottom to minimise air entrapment and to eliminate surface tension effects. Ordinary tap water was used.

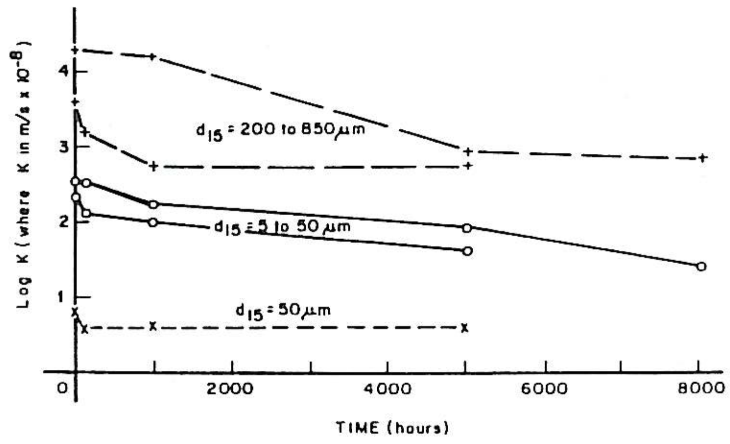
The range of soils tested varied from D_{15} = 0,005 mm to D_{15} = 0,350 mm and the geotextiles had nominal opening size (possibly O_{95}) from 0,05 to 0,350 mm. A mesh with opening size of 2,00 mm was also tested.

(ii) Results

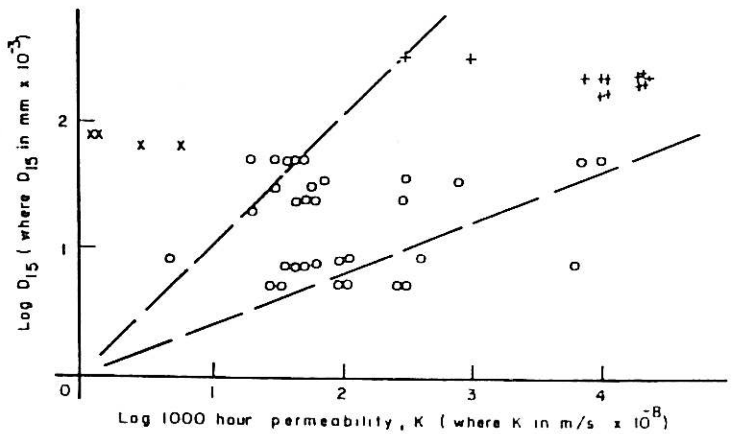
In general the flow rate declined rapidly in the initial stages and more slowly in the later stages. Figure 3.7(b) shows typical results. A decrease in the permeability by a factor of 10 was common. The authors state that the results were erratic and inconsistent and attribute this to the fact that ordinary water was used instead of de-aired water. They



(a) Apparatus



(b) Typical results



(c) 1000 hour permeability vs D_{15} soil

FIGURE 3.7 *LONG-TERM TESTS BY POLYCROFT AND JONES (1982)* believe that for this reason, the relationship between the D_{15} of the soil and the permeability of the soil/geotextile system after 1 000 hours of testing, is poor, as shown on Figure 3.7(c).

In general there was no evidence that the fabrics had become blocked, but in certain situations fines migrated to the surface of the geotextile, forming a layer of reduced permeability.

Although some piping was evident during the initial phase of each test, significant piping only occurred on two occasions namely with the 2 000 μm mesh and with a geotextile when a soil was used with D_{15} = 0,01 mm and uniformity coefficient (U) = 4.

(iii) <u>Conclusions</u>

(a) Results obtained with these type of tests can vary in an inconsistent manner.

(b) The permeability of soil/geotextile systems is controlled almost wholly by the permeability of the soil.

(c) The migration of fines in certain gap-graded soils lead to a build-up of material above the geotextile, resulting in a reduced permeability.

3.4.1.3 Koerner and Ko (1982)

(i) <u>Apparatus, method and materials tested</u>

In this study a 95 mm diameter apparatus was used. A soil sample of 720 g was placed directly onto the geotextile and ordinary water from a constant water head of 380 mm was allowed to flow through the soil/geotextile system.

Four soil types were used, as shown on Figure 3.8(a). The D_{10} values of the soils varied from 0,30 mm for the sand to <0,001 mm for the silty clay and uniformity coefficients from 3 to 17. Permeability coefficients varied from $1,2x10^{-1}$ to $6x10^{-7}$ cm/sec.

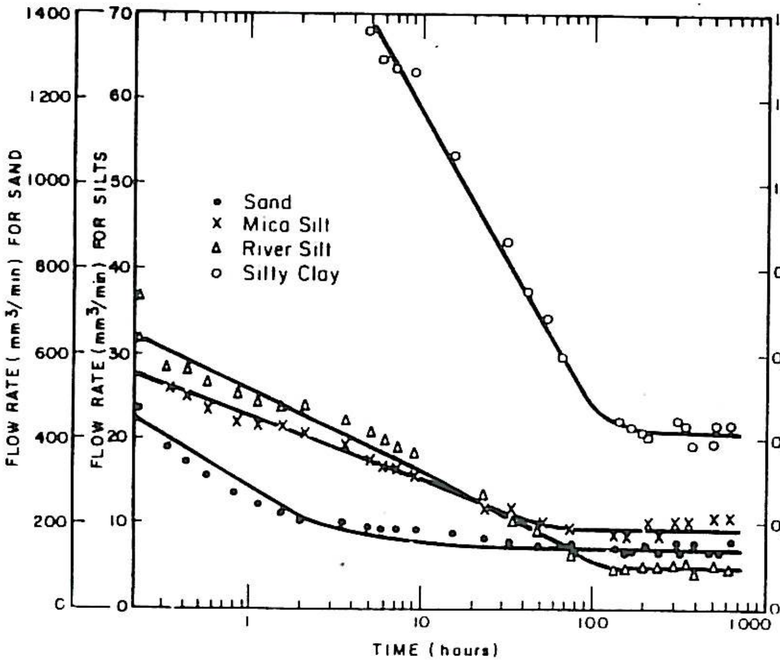
A total of five geotextiles - woven, nonwoven and knitted - were evaluated with permeability coefficients, k as indicated on Figures 3.8(b) and (c).

(ii) Results

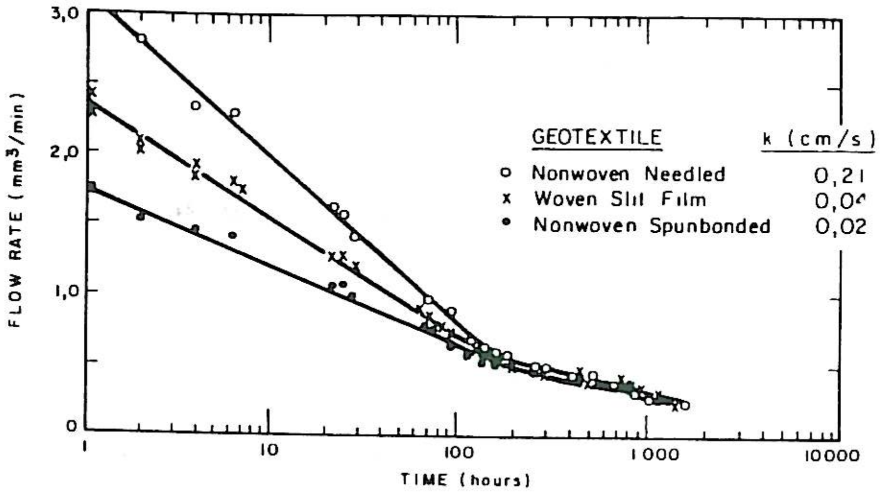
Figure 3.8(a) shows the results of testing one type of geotextile with various soils and Figures 3.8(b) and (c) the results of a particular soil type with various geotextiles. The authors ascribe the initial decrease in flow to compaction of the soil. The long term shape of the curves are ascribed to soil/geotextile interaction and two possibilities are mentioned namely a stable soil filter structure at a finite distance from the geotextile or the developing of a stable arch over the geotextile openings at the soil/geotextile interface. Various attempts using image analyzers and other methods to determine the soil/geotextile interaction, were unsuccessful.

The slope of the final (long term) part of the curves were calculated after a transition between the slopes of the curves had occurred. In the case of a fine soil (silty clay), this state was reached after approximately 180 hours, and with sand after 4 hours. The slope was 0,2 for the silty clay and 0,0 (accurate to one decimal place) for the other soils.

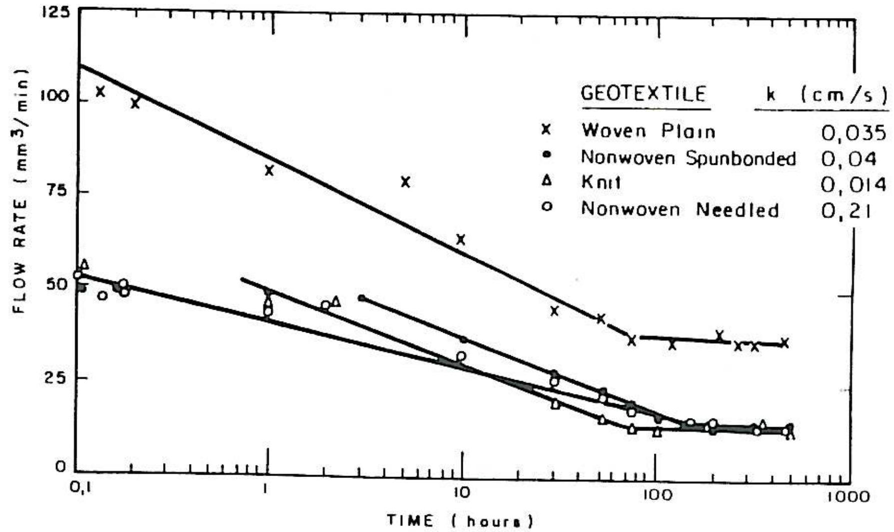
A slope of zero (or close to zero) indicates, according to the authors, a state of equilibrium between the soil/geotextile system and the applied hydraulic conditions. A slope greater than zero indicates a non-equilibrium condition due to blinding or partial clogging of the voids in the geotextile.



(a) Nonwoven needled geotextile and various soils



(b) Silty clay and various geotextiles



(c) Mica silt and various geotextiles

FIGURE 3.8 *RESULTS OF LONG-TERM TESTS BY KOERNER AND Ko (1982)*

The authors also investigated the so-called gradient ratio (Calhoun, 1972) on a long-term basis. The findings were discussed in Section 3.3.4.1 (iii).

(iii) Conclusions

(a) The flow through a soil/geotextile system is a complex phenomenon and can only be definitely examined by long-term flow tests. The minimum time required for such tests is up to 200 hours for soils with high clay content.

(b) The initial range of the curve obtained during these tests is governed by the soil, and the final range by the soil/geotextile interaction.

3.4.1.4 Lawson (1986)

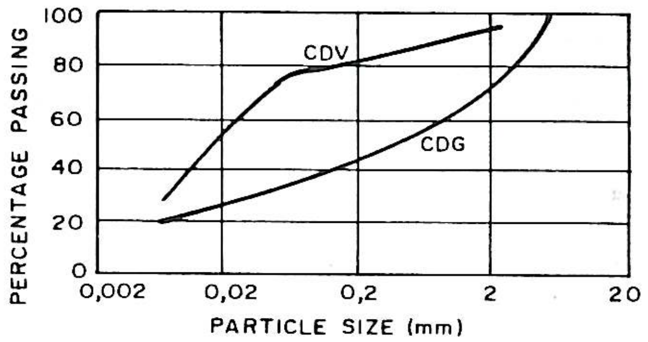
(i) Apparatus, method and materials tested

The apparatus used in this study was identical to that used by Polycroft and Jones (1982), discussed in Section 3.4.1.2 above.

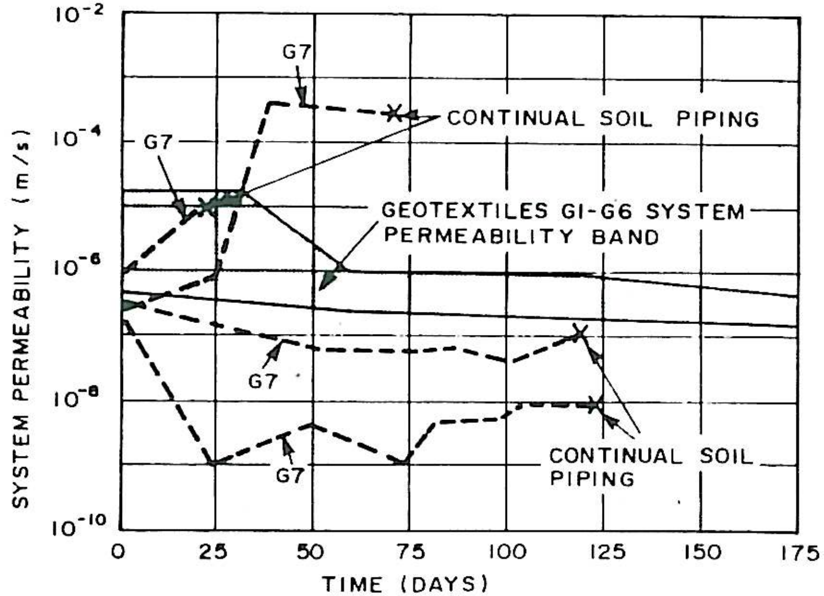
Two residual soils from Hong Kong were tested namely a completely decomposed granite (CDG) and a completely decomposed volcanic (CDV) soil. The gradings of these soils are shown on Figure 3.9(a). Six geotextiles and a geogrid were evaluated, the hydraulic properties of which were as follows:

PRODUCT CODE	CONSTRUCTION	PORE SIZE (mm)		PERMEABILITY
		O_{90}	O_{50}	L/m^2/sec*
G1	Nonwoven, meltbonded	0,40	0,02	30
G2	Nonwoven, meltbonded	0,10	0,07	50
G3	Nonwoven, meltbonded	0,18	0,12	80
G4	Nonwoven, meltbonded	0,35	0,20	150
G5	Woven, monofilament	0,20	0,17	90
G6	Woven, monofilament	1,5	1,5	>500
G7	Geogrid	7,0	7,0	>>500

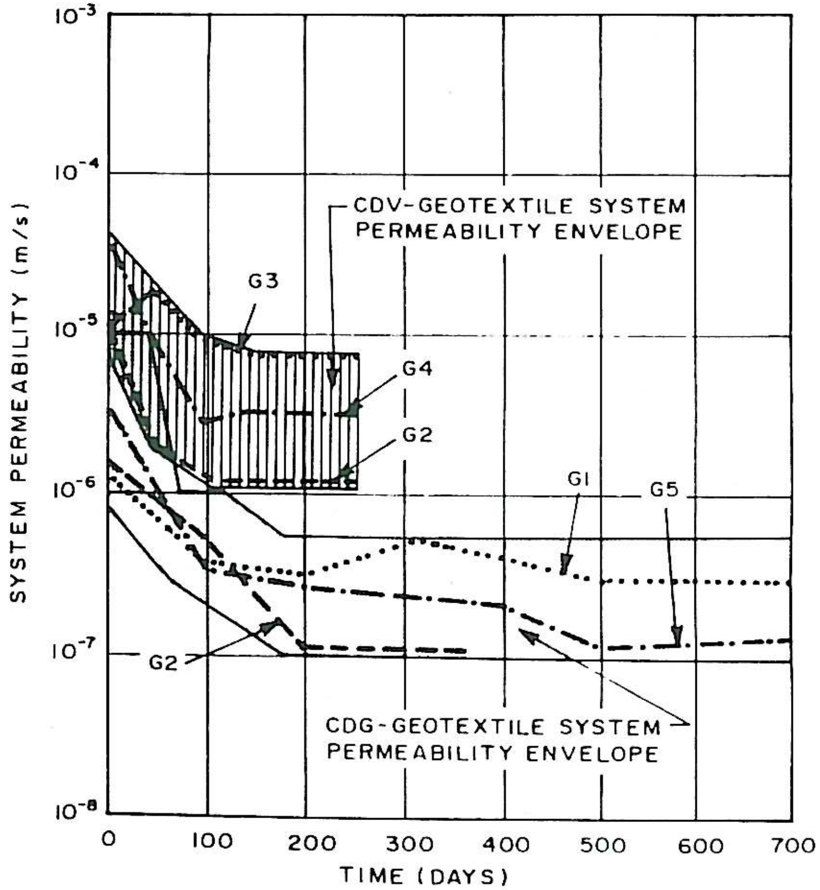
* 100 mm constant head.



(a) GRADING OF SOILS



(c) RESULTS OF CDG SOIL WITH GEOGRID



(b) RESULTS OF TWO SOIL TYPES WITH DIFFERENT GEOTEXTILES

FIGURE 3.9

LONG-TERM TESTS BY LAWSON (1986)

(ii) Results

Figure 3.9(b) shows the results obtained with the two soil types and the geotextiles. The results of the geogrid and CDG soil are shown on Figure 3.9(c).

The tests were run over long periods, up to 700 days (17 000 hours). After initial decrease in permeabilities, equilibrium conditions were reached with all the geotextiles tested. The geogrid with nominal opening size of 7,0 mm, resulted in continuous soil piping. Based on these tests, filter criteria were developed for the CDG soil. (See Section 3.3.4.7).

(iii) Conclusions

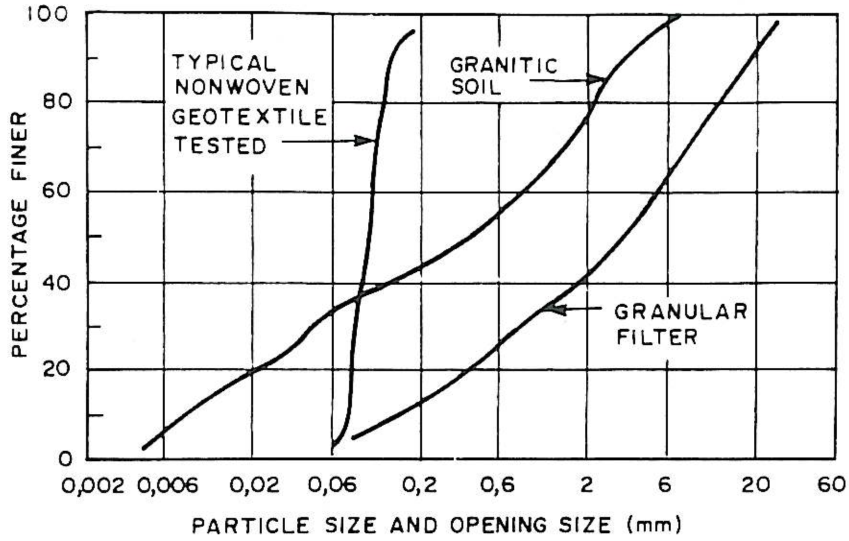
The actual mechanics of soil/geotextile filtration is complex, but filtration performance requirements are quite simple. For optimal filter performance the permeability of the soil/geotextile system must reach equilibrium and the soil must be prevented from continuous piping.

3.4.1.5 Greenway, Ho and Brand (1986)

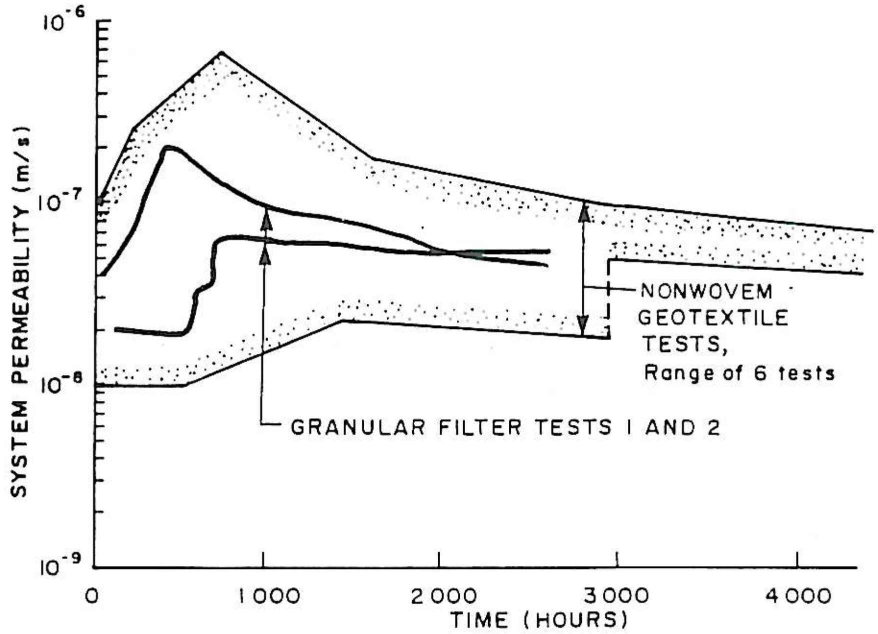
(i) Apparatus, method and materials tested

Permeameters with 300 mm diameter were used. The soil was compacted to 95% of Proctor density and was supported by uniform 20 mm crushed aggregate which was also compacted. The soil was saturated from below, followed by continuous flow from the top. A constant water head of 1,88 m was maintained and the sample height was 260 mm, resulting in a hydraulic gradient of approximately 7.

One soil type, a granitic soil, also described as a widely graded silty sand, was used, the grading of which is shown on Figure 3.10(a).



(a) GRADING OF THE SOIL, GRANULAR FILTER AND GEOTEXTILES



(b) SUMMARY OF TEST RESULTS

FIGURE 3.10

LONG-TERM TESTS BY GREENWAY et al (1986)

Six non-woven geotextiles were tested, which varied in weight between 105 and 210 g/m^2. The opening sizes O_{90}, varied between 0,10 and 0,30 mm. A typical opening size curve is shown on Figure 3.10(a). Two tests were also performed where the geotextile was replaced with a granular filter, which conformed to the common filter design rules. The grading of the granular filter is also shown on Figure 3.10(a).

(ii) Results

A summary of the results obtained is shown on Figure 3.10(b). According to the authors the change in permeability with the geotextiles (increase as well as decrease in permeability) occurred in the early part of each test until a filter zone had developed within the soil near the geotextile interface. The permeabilities obtained after the tests had stabilized, were equal to or greater than that of the original soil permeability.

The results obtained with the granular filters, were very similar to those obtained with the geotextiles, as shown on Figure 3.10(b). The final permeabilities obtained fell within the range obtained with the geotextiles.

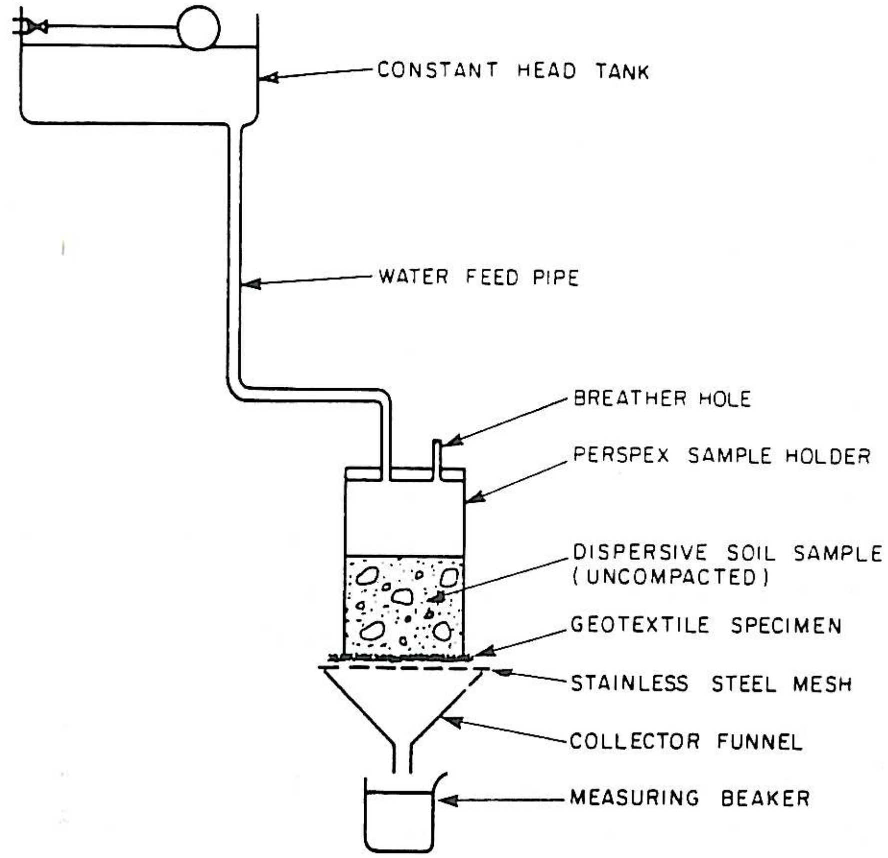
(iii) Conclusions

The geotextiles tested proved to be successful filters and their behaviour was directly comparable to that of granular filters tested under the same conditions.

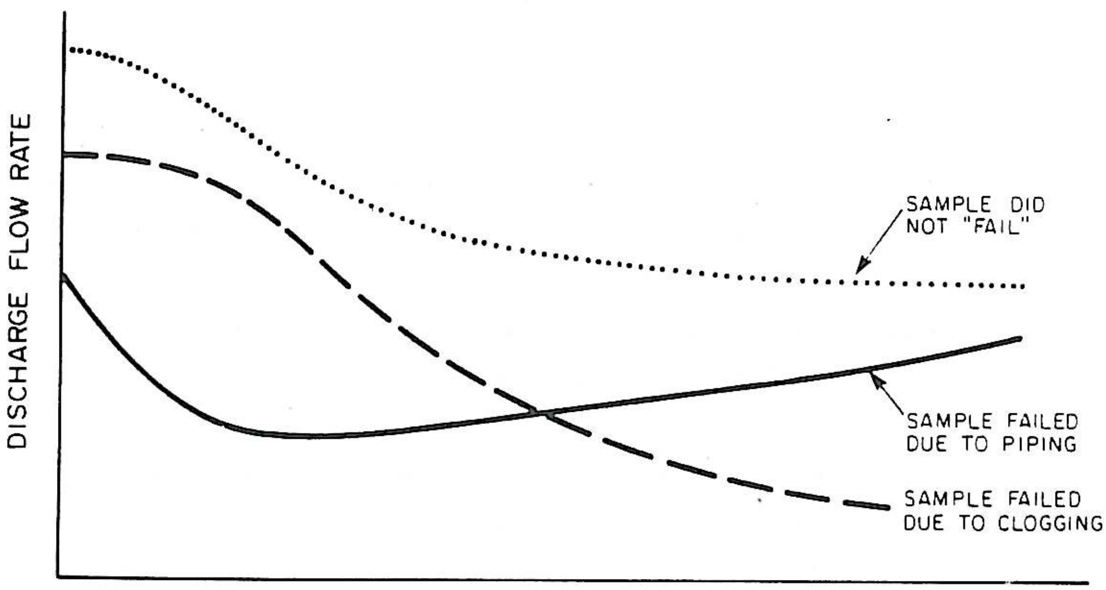
3.4.1.6 Legge (1986)

(i) Apparatus, method and materials tested

A diagrammatic representation of the apparatus is shown on Figure 3.11(a). The soil was uncompacted and ordinary tap water was used from a constant head tank.



(a) Diagrammatic representation of apparatus



(b) Typical results

FIGURE 3.11

LONG TERM TESTS BY LEGGE (1986)

The study was restricted to dispersive soils and a wide range of woven and non-woven geotextiles were tested.

(ii) Results

Typical results (not quantified) are shown on Figure 3.11(b). The tests were run for three months and, according to the author very little variation in flows were recorded after three weeks with geotextiles that did not "fail". Some geotextiles failed due to clogging, resulting in a continuous decrease in permeability. Others failed due to piping, resulting in a continuous increase in permeability.

Visual inspection after the tests had been completed, showed that abnormally high proportions of finer soil fractions were present at the soil/geotextile interface where the systems had clogged. Systems that had not clogged, appeared to have lost the fine fraction at the interface.

(iii) Conclusions

The author concludes that soil/geotextile compatibility tests are essential for evaluating geotextiles as filters.

3.4.1.7 Summary

A summary of long term flow test results obtained by the various researchers as discussed above, is shown on Figure 3.12. From this, and from the conclusions drawn by the different researchers, the following general remarks can be made regarding long term flow tests:

(a) The various tests were not performed with standardized equipment or procedure. Different apparatus, methods, duration of tests etc were used. The large variation in results shown on Figure 3.12 can probably be attributed to these differences and to the different soil types that were used.

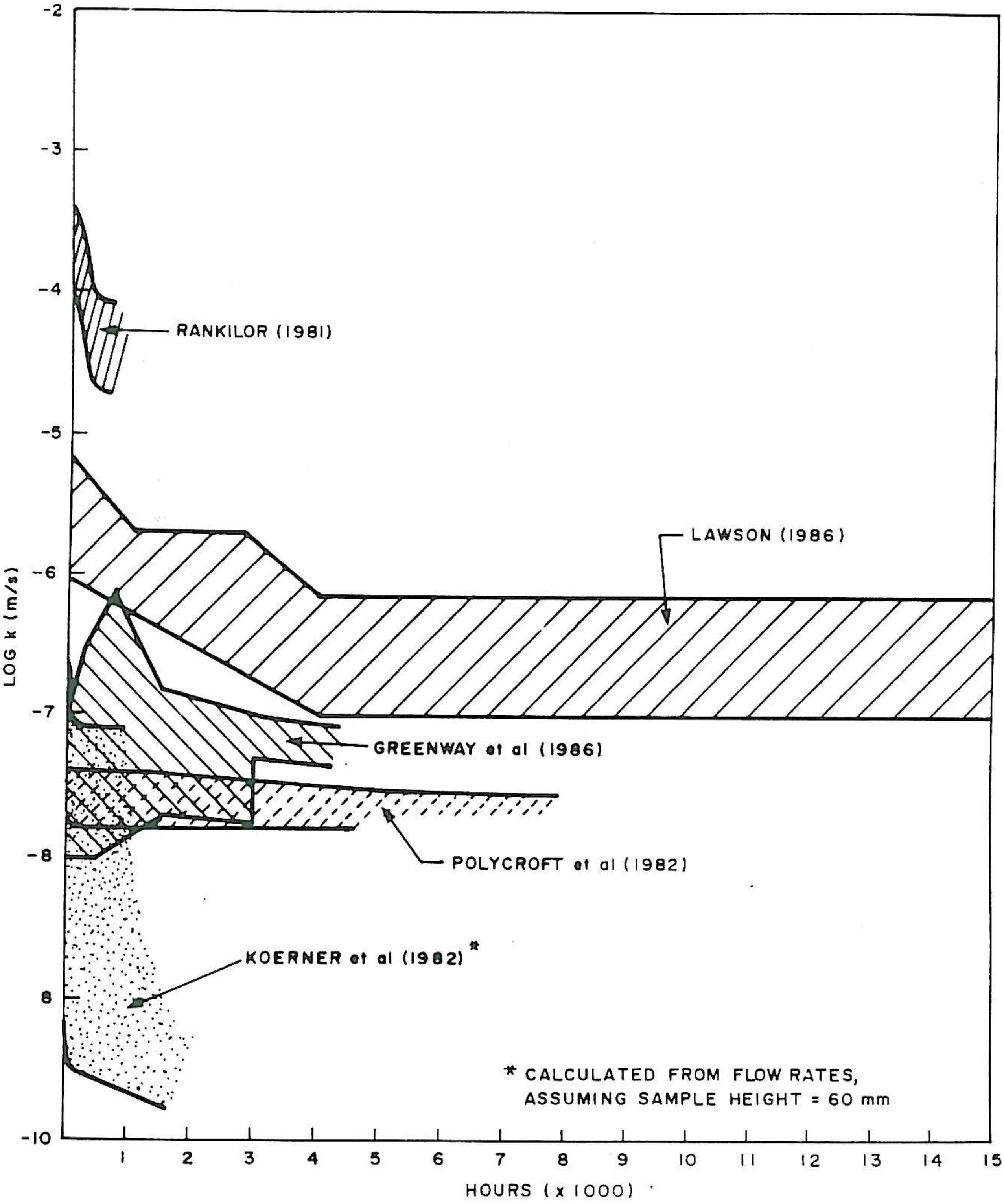


FIGURE 3.12

SUMMARY OF LONG-TERM FLOW TESTS BY OTHER RESEARCHERS

(b) Despite the differences, certain aspects of the results were similar. Typically, there was an initial decrease in permeability, probably due to the forming of a filter zone, as described in Section 3.4.1.1 (ii) above. The magnitude of decrease was typically by a factor of 10. This was followed by a relatively constant flow, indicating a state of equilibrium in the soil/geotextile system and a stable filter zone in the soil.

(c) The final permeability was usually less than that of the soil or the geotextile and was a function of the soil (filter zone) permeability, rather than the geotextile permeability.

(d) The filtration function was performed by the filter zone in the soil and not by the geotextile. The exact pore sizes of the geotextile were therefore not important.

(e) Continuous piping only occurred where opening sizes were 2 mm or larger.

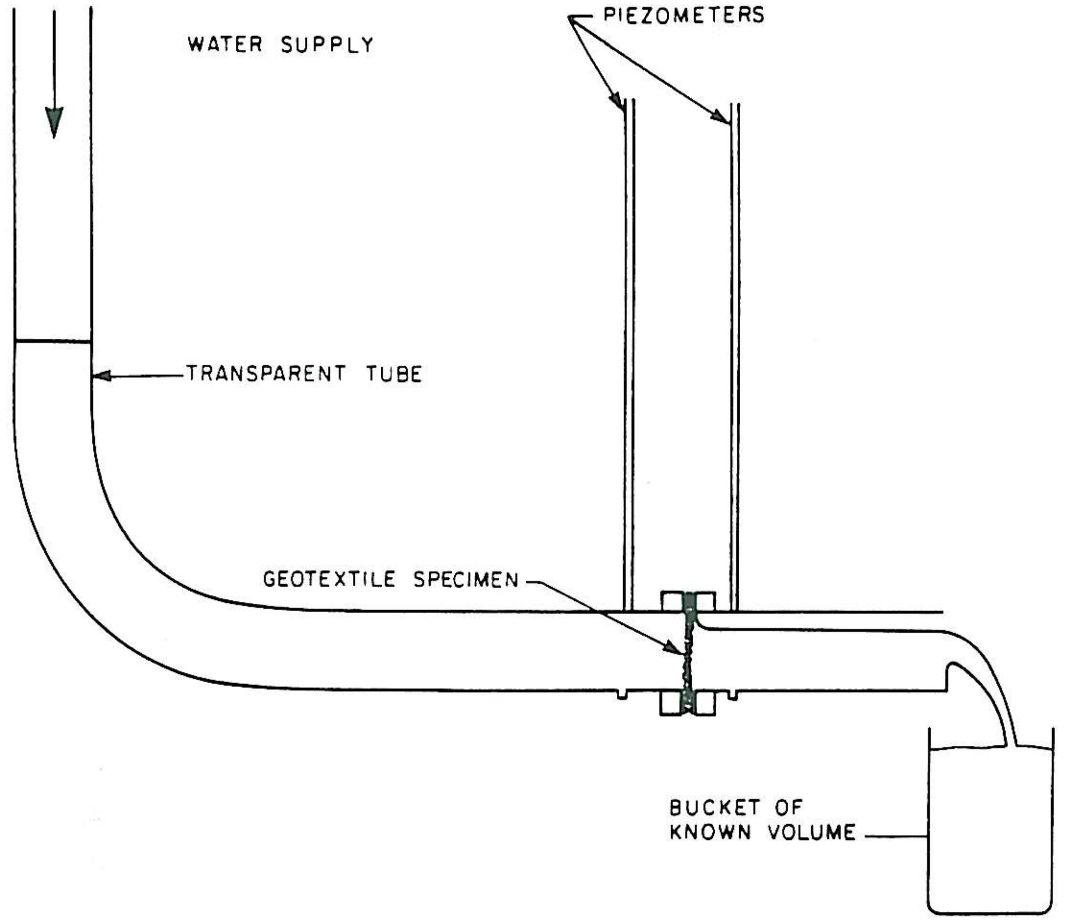
(f) Where natural (granular) filters were tested, their performance was similar to that of geotextiles.

(g) The flow through a soil/geotextile system is a complex phenomenon, and can only be evaluated by long term flow tests.

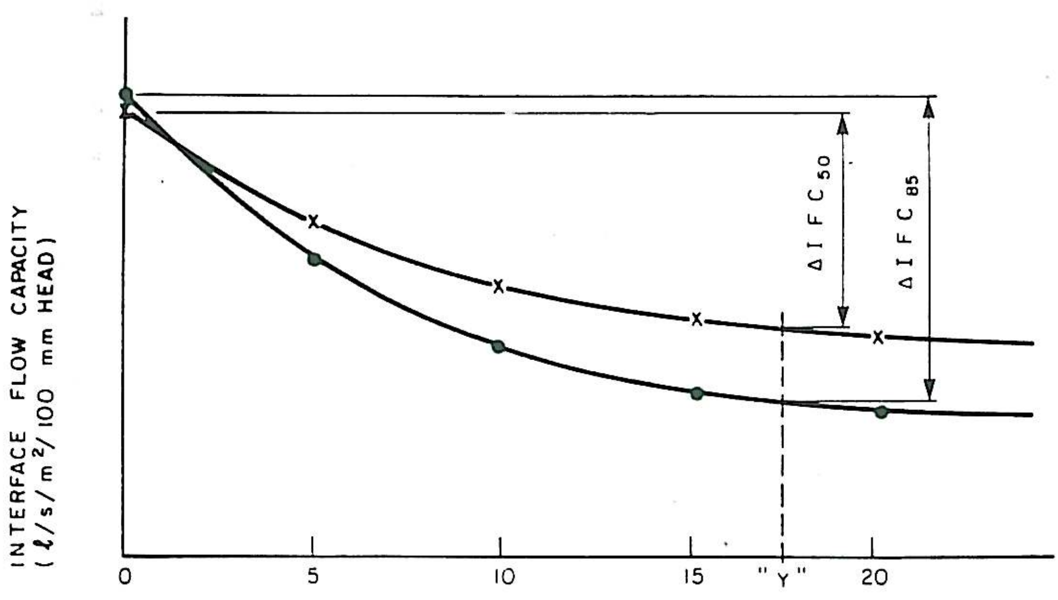
3.4.2 Interface flow capacity (IFC) tests

This method of evaluating soil/geotextile compatibility has been recorded by Legge (1986) and a similar method called the plugging flow capacity (PFC) by Hoover (1982).

The test is performed by submerging a geotextile in a flow path, using the apparatus shown in Figure 3.13(a). The through-flow capacity of the geotextile is determined by



(a) Apparatus



(b) Typical results

FIGURE 3.13

INTERFLOW CAPACITY (IFC) TESTS BY Legge (1986) recording the flow rate through and the head loss across the specimen.

Thereafter, 20 g of the 85% soil-particle size (D_{85}) is introduced into the intake in 5 g increments. The flow capacity is recorded and plotted on a graph of flow capacity versus mass of soil particles introduced as shown on Figure 3.13(b). This is repeated for the 50% soil particle size (D_{50}).

The interface flow capacity (IFC) is defined as the flow capacity for "y" grams of soil introduced where y is the mass of soil required for one layer of D_{85} particles to cover the specimen.

The drop in IFC for the x particle size (Dx) is then the difference between the flow capacity for no soil introduced and the interface flow capacity for "y" grams of the Dx soil introduced.

If the drop in IFC for the D_{50} soil is larger than that for the D_{85} soil a build-up of the D_{50} soil particles on the interface will cause clogging. Similarly, the drop in IFC owing to the D_{15} soil should be less than that owing to the D_{50} particle size.

No test results are given by the authors reporting this test method, but Legge (1987) reports that a good correlation was obtained between clogging potential as measured with this method and actual clogging observed with long-term permeameter tests, as described in Section 3.4.1.6 above.

3.4.3 Conclusions

Test results are limited and difficult to compare due to different test apparatus and procedures. The tests do, however, take into account the interaction between the soil and the geotextile and therefore give more meaningful results than filter criteria based on in-isolation tests.

3.5 CONCLUSIONS

Relevant conclusions were given at the end of each section. The most important conclusions are the following:

- In-isolation tests on geotextiles and existing filter criteria that are based on these tests have various shortcomings. Soil/geotextile compatibility and long-term performance, including the possibility of clogging, are not considered or catered for in the design criteria. The various criteria differ widely, resulting in the situation that, given a particular soil, any geotextile can be chosen as a filter, depending on which set of criteria is used.

- Soil/geotextile compatibility tests take into account the interaction between the soil and the geotextile. Filter criteria cannot be developed from the research results available, due to different test methods and apparatus used by the various researchers, but the results show similar tendencies.

One of the main objectives of the research described in this document was to establish standardized, practical filter criteria. The available information summarized in this chapter and the conclustions given above indicate that soil/geotextile compatibility tests should form the basis of such criteria, and that the validity of existing criteria should be tested against results obtained with the soil/geotextile compatibility tests.

CHAPTER 4

GEOTEXTILE IN-ISOLATION TESTS

CHAPTER 4

CONTENTS PAGE

4.1 INTRODUCTION

All existing design criteria (as discussed in Section 3.3) and geotextile specifications are based on in-isolation tests. Selected in-isolation tests were performed as part of the investigation reported on in this document for the following reasons:

- To examine the influence of those in-isolation parameters that are usually applied in filter design (eg. pore sizes) on the results obtained with soil geotextile compatibility tests. This would indicate which of the existing criteria, if any, could be incorporated in the final criteria.

- To develop, manufacture and evaluate test apparatus and test methods that can be used as standard tests for evaluating geotextiles for road subsurface drainage applications.

- The selected tests were performed on all geotextiles available at the time of testing. These results were made available to road authorities which enabled them to make comparisons between geotextiles based on identical tests. Prior to these results being available, comparisons had to be made on the basis of specifications supplied by the geotextile manufacturers, each of which used their own testing apparatus and methods.

In-isolation tests can broadly be categorized into hydraulic tests and mechanical (or physical) tests. Hydraulic and mechanical tests that were performed are described in Sections 4.2 and 4.3 respectively. In Section 4.4, in-isolation tests (both hydraulic and mechanical) that were not performed as part of this investigation, are described briefly.

4.2 HYDRAULIC TESTS PERFORMED

Hydraulic tests include all in-isolation tests that determine the properties of geotextiles related to their behaviour in hydraulic applications eg. drainage, erosion control etc., and as such are directly related to road subsurface drainage.

Two hydraulic tests were performed, namely the determining of pore sizes and normal permeability.

These tests were briefly discussed in Chapter 3 (Section 3.3.2 and 3.3.3) and their use in filter criteria was illustrated in Section 3.3.4.

4.2.1 Pore Sizes

The method used in this investigation was inverse (or reverse) sieving. The test basically involves sieving glass beads through the geotextile and recording the percentage of each size that is retained on the geotextile as well as that which passes through. The various pore sizes and a pore size distribution can then be obtained, assuming that the sizes of the glass beads reflect the opening sizes in the geotextile.

4.2.1.1 Apparatus

Apparatus that is typically used for these tests is shown on Figure 4.1. A spray nozzle is often included and water is sprayed over the geotextile to wash the glassbeads through while the whole apparatus is vibrated on a vibrating table.

It has been shown that the test is very sensitive to such factors as test duration, water pressure, frequency and amplitude of vibration etc. For these reasons, it was attempted in this investigation to use apparatus that can easily be standardized.

The apparatus that was manufactured and used in this investigation is shown on Figure 4.2. It consists of two metal cylinders between which the geotextile is clamped. An 'O'-ring prevents the glass beads from escaping through the sides.

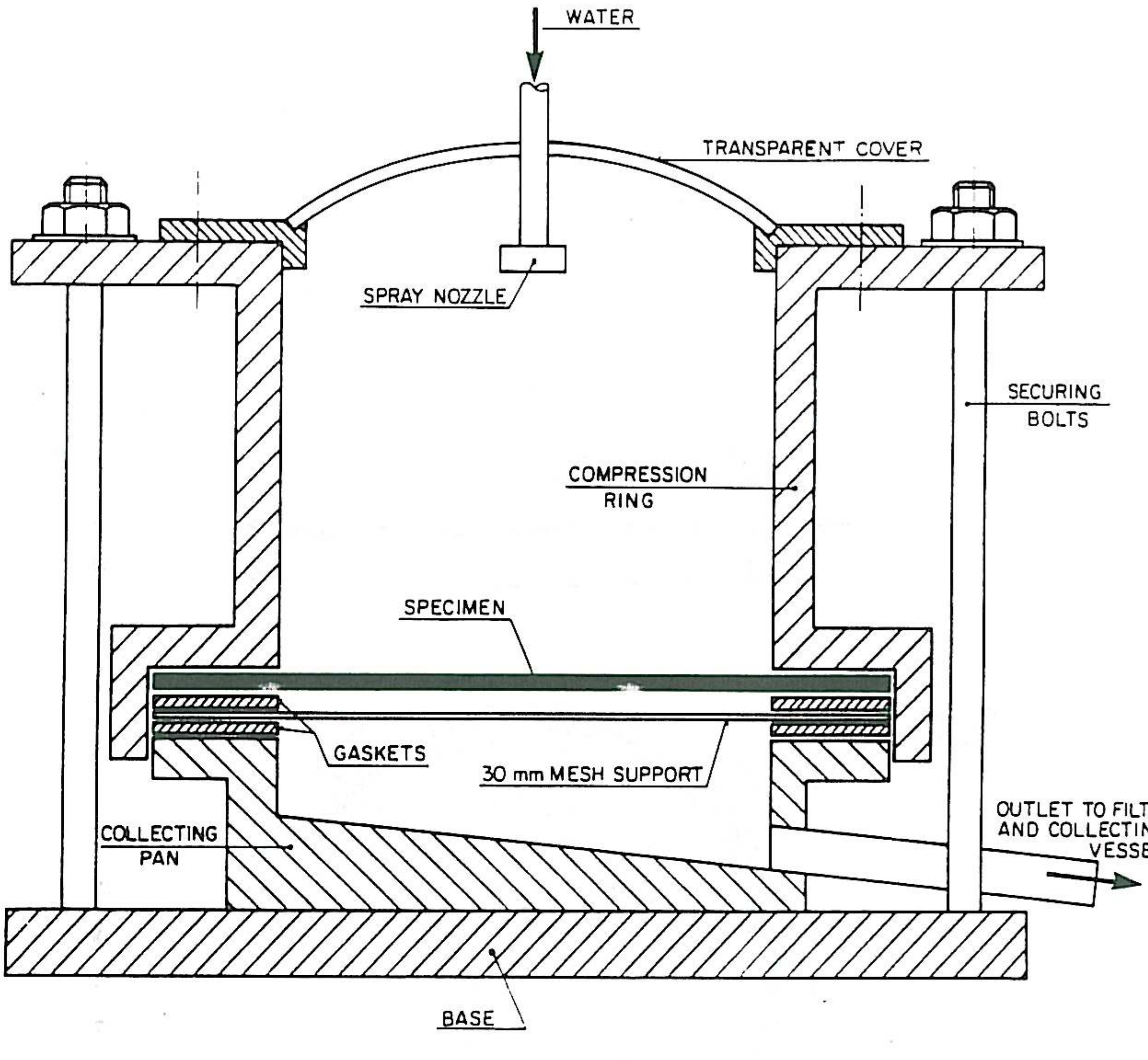


FIGURE 4.1
TYPICAL SIEVING APPARATUS

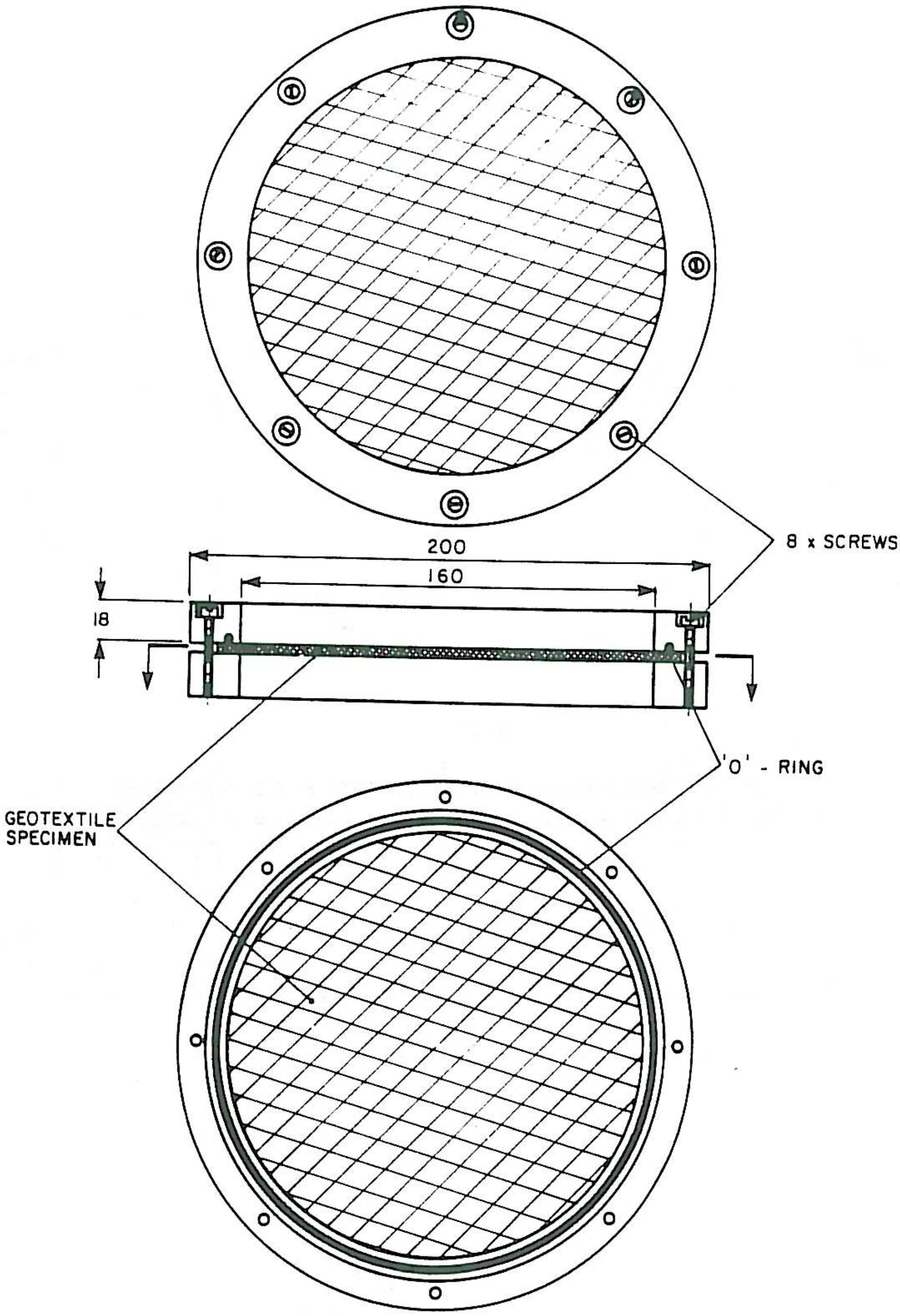


FIGURE 4.2

SIEVING APPARATUS USED IN THIS INVESTIGATION

The apparatus fits tightly into a standard 200 mm laboratory soil sieve, which allows the actual sieving to be done on a standard vibrating apparatus. Photograph 4.1 shows the apparatus with the geotextile and the glass beads prior to sieving.

The glass beads that are normally used for the test are the so-called "Balotini balls". These are true spheres ranging in size from approximately 50 to 2 000 μm. These need to be imported however, at a cost of about R2/gram. The glass beads that were used in this investigation are normally used in the paint industry and, although not true spheres, they are almost spherical, relatively cheap and easy to obtain. The beads are available in various size ranges and three different ranges were combined to make up the grading shown below (Table 4.1).

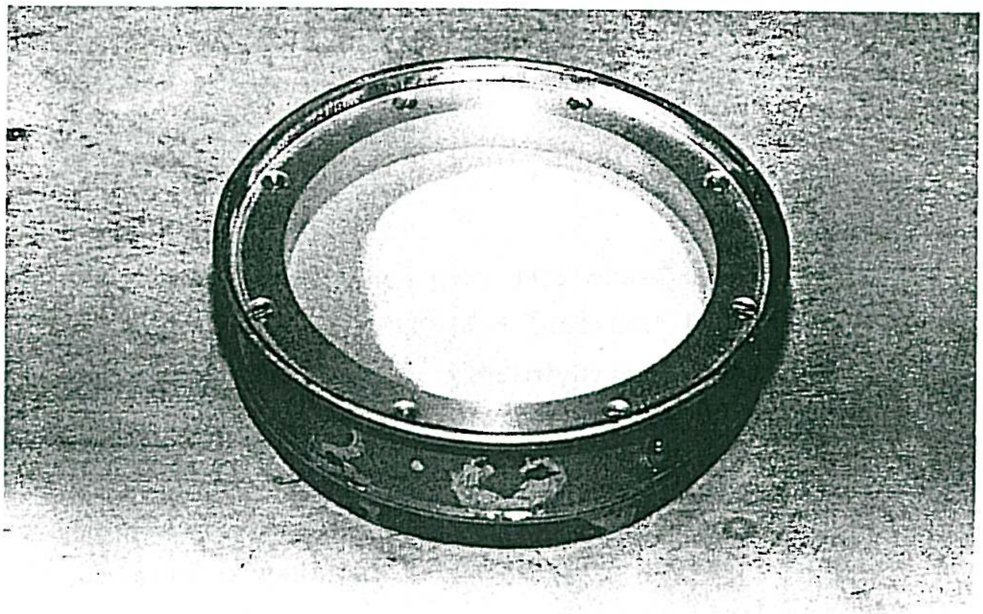
TABLE 4.1: Grading of glass beads used for sieving test

SIZE (mm)	PERCENTAGE PASSING
2,000	100
0,850	99,4
0,425	68,9
0,250	62,7
0,150	46,4
0,075	25,6
0,060	12,9

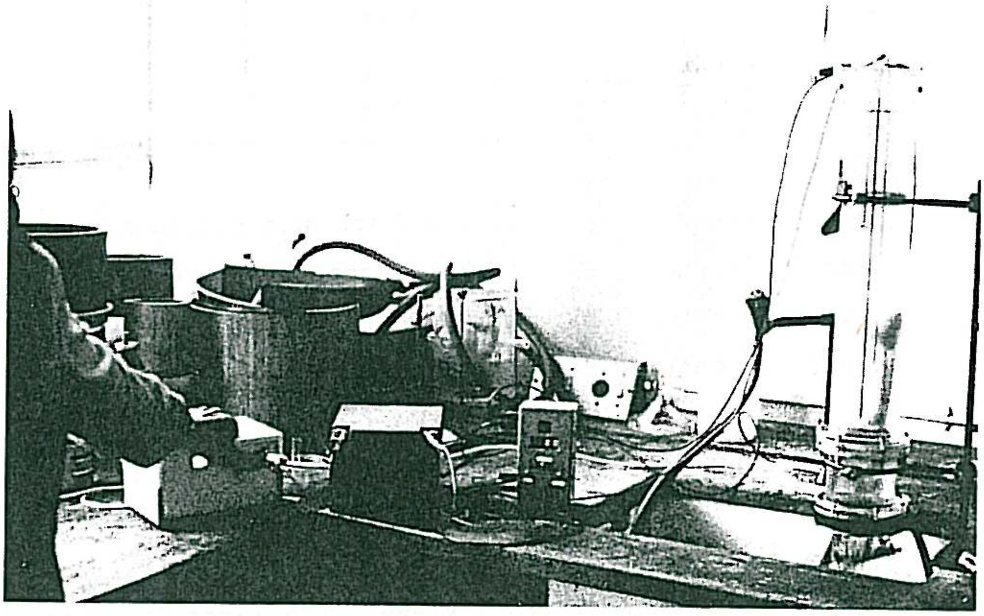
The percentage passing 0,060 mm was obtained by hydrometer analysis and in all further tests this percentage was assumed to be constant; hydrometer analyses were not carried out for each geotextile tested.

4.2.1.2. Test Procedure

The geotextile was mounted in the apparatus shown in Figure 4.2 and placed in a sieve with large (> 2 mm) openings which was in turn placed on top of a set of sieves from 2,00 to 0,075 mm. The sieving of the glass beads through the



PHOTOGRAPH 4.1
Inverse sieving test



PHOTOGRAPH 4.2
Normal permeability test

geotextile and the grading analysis of those beads that passed through, could therefore be done simultaneously. A 300 g sample of glass beads, with grading as shown on Table 4.1, was then placed on the geotextile and a lid was placed on the top sieve. The whole set of sieves was then placed in the shaker or vibrator and allowed to vibrate for 15 minutes.

After the vibrating was completed, the beads that had passed through the geotextile and were retained on the various sieves, were weighed and recorded. A grading analysis was then performed on the beads retained on the geotextile and these values were also recorded.

The test was repeated three times using a new geotextile sample for each test.

4.2.1.3 Results

An example of results obtained is shown on Table 4.2. The grading of the beads passing the geotextile (mass passing in gram) and that of the beads retained on the geotextile (mass retained in gram) were recorded for the three tests (samples 1, 2 and 3). From these the average values G_D and G_R were calculated as well as G_T, P and X as shown on Table 4.2. The values in the last column, X, represent the cumulative opening size distribution of the geotextile. The opening size is defined as follows:

O_x is the diameter of an opening in a geotextile where X % of all other openings are smaller.

For example, from Table 4.2:
$O_{97,9}$ = 0,425 mm
$O_{0,6}$ = 0,150 mm.

The opening sizes that are normally used in filter criteria are O_{10}, O_{50}, O_{90} etc. These values can be obtained by linear interpolation eg.:

O_{90} = 0,25 + (0,425 - 0,25)/(97,9 - 40,6) x (90,0 - 40,6)
= 0,401 mm.

Similarly O_{10} = 0,173 mm etc.

TABLE 4.2: Example of Results obtained from sieving test

GEOTEXTILE NO: 32

Sieve (mm)	Mass passing (g) 1*	2	3	G_D #	Mass retained (g) 1	2	3	G_R #	G_T $ (g)	P § (%)	X @ (%)
2,000	0	0	0	0	0	0	0	0	0	0	100
0,850	0	0	0	0	3,3	3,4	3,0	3,2	3,2	0	100
0,425	0	0	5,3	1,7	82,4	81,7	77,0	80,4	82,1	2,1	97,7
0,250	9,4	9,5	19,8	12,9	11,4	13,6	1,3	8,8	21,7	59,4	40,6
0,150	51,2	51,3	50,5	51,0	0,2	0,4	0,3	0,3	51,3	99,4	0,6
0,075	59,8	54,9	63,5	59,4	0	0	0	0	59,4	100	0
0,060	78,2	84,4	79,8	80,8	0	0	0	0	80,8	100	0

* Sample numbers 1, 2 and 3

\# G_D and G_R: averages of 3 samples for mass passing and mass retained respectively

$ $G_T = G_D + G_R$

§ $P = G_D/G_T * 100$

@ $X = 100-P$

From the opening size distribution in Table 4.2, a distribution curve can be drawn. Examples of such curves, including the example discussed above (Geotextile 32), are shown in Figure 4.3.

A summary of the results obtained for all the geotextiles that were tested is given in Table 4.3. Shown on Table 4.3 is the cumulative opening size distribution (X) and the opening sizes (O_x) as well as the coefficient of variance (CoV) for the three samples tested. Cov was calculated at three values of O_x. The last column of Table 4.3 shows the opening sizes (O_{90} and O_{50}) that are published in manufacturers' documents.

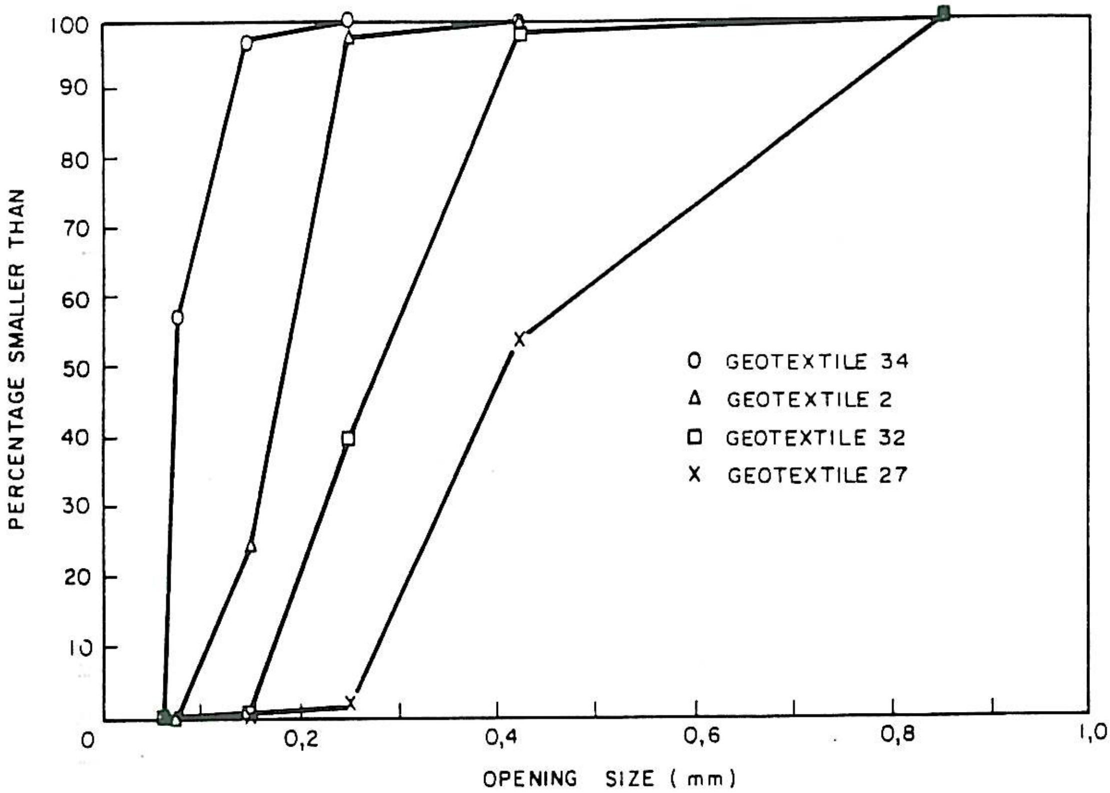


FIGURE 4.3

EXAMPLES OF OPENING SIZE DISTRIBUTION CURVES

940-4-5685/3 BS

TABLE 4.3: Summary of results from sieving tests

GEOTEX-TILE NO:	X = PERCENTAGE OPENINGS SMALLER THAN						O_x = DIAMETER OPENINGS (IN µm), WHERE X % OF ALL OTHER OPENINGS ARE SMALLER								CoV (%) FOR THREE SAMPLES			O_x (µm) (PUBLISHED)	
	0,850 mm	0,425 mm	0,250 mm	0,150 mm	0,075 mm	0,060 mm	O_{95}	O_{90}	O_{85}	O_{60}	O_{50}	O_{15}	O_{10}	O_5	CoV at O_{90}	O_{50}	O_{10}	O_{90}	O_{50}
1	100	100	100	87,8	6,7	2,5	209	168	147	174	115	83	78	69	18,1	4,4	0,7	214	50/100
2	100	100	100	98,0	24,8	0,7	147	142	137	111	101	69	66	63	1,1	5,1	2,3	214	50/100
3	100	100	100	96,4	32,3	1,3	148	142	137	107	96	67	64	62	0,4	5,3	3,3	208	70/100
4	100	100	100	99,3	34,9	0,9	145	139	133	104	93	66	64	62	1,2	6,0	0,9	150	100/140
5	100	100	100	100	88,4	17,0	118	85	74	69	67	53	35	17	11,8	20,2	57,4	129	100/140
6	100	100	100	95,3	13,2	0,3	150	145	140	118	109	77	71	65	2,9	4,6	7,1		90
7	100	100	100	81,1	4,6	0,2	223	197	171	129	119	85	80	75	17,8	4,9	3,8		90
8	100	100	100	63,6	2,7	0,2	236	222	209	146	113	90	84	78	5,8	11,5	4,2		130
9	100	100	100	84,1	6,7	1,2	218	187	156	127	117	83	78	70	7,8	1,8	1,3		160
10	100	100	100	83,0	7,8	0,9	221	191	162	127	117	82	77	69	5,3	1,7	1,5		120
11	100	100	100	98,2	27,5	1,2	147	141	136	109	99	68	65	62	2,1	6,8	2,3		120
12	100	100	100	95,7	16,3	1,9	149	145	140	116	107	74	68	63	1,0	3,0	3,8		110
13	100	100	100	100	70,6	6,2	137	124	112	72	70	62	61	48	2,0	0,8	0,9		90
14	100	100	79,1	14,2	1,4	0	383	341	299	221	205	151	125	96	22,8	11,5	25,5		
15	100	100	98,6	62,6	6,7	1,2	240	226	212	146	133	86	79	70	24,3	21,8	13,1		
16	100	100	86,5	23,3	2,4	0,6	360	295	248	208	192	120	102	84	27,5	14,0	33,3		
17	100	100	100	95,3	27,0	2,6	150	144	139	111	100	68	64	61	2,9	9,3	3,1		
18	100	100	100	100	54,7	2,4	142	133	125	84	74	64	62	61	0,5	0,8	1,2		
19	100	100	100	100	100	99,7	57	54	51	36	30	9	6	3	0,7	1,6	0,6		
20	100	100	100	100	61,5	5,4	140	130	121	75	72	63	61	53	4,5	2,0	1,9		
21	100	100	95,8	34,5	0,6	0,3	249	240	232	192	175	107	96	85	2,1	9,9	11,9	195	145
22	100	100	100	98,	27,5	0,9	147	141	136	110	99	68	65	62	0,4	2,7	0,9	150	110
23	100	100	100	100	65,9	2,6	139	128	117	74	71	63	62	61	8,5	2,9	0,9	105	75
24	100	100	100	100	100	100	<60												
25	100	100	100	100	100	100	<60												
26	100	100	100	88,4	11,6	0,5	207	164	147	122	112	78	73	67	10,3	3,2	7,1		
27	100	54,1	1,4	0,2	0,2	0,1	804	757	711	480	411	295	279	262	3,2	6,3	2,7	325	
28	100	80,5	7,1	0,2	0	0	741	632	523	376	352	269	257	220	13,2	4,0	6,9	130	
29	100	100	95,8	46,4	1,6	0,3	248	238	228	177	157	97	89	81	6,0	16,8	10,6	280	
30	100	100	95,1	36,9	1,6	0,7	250	241	233	190	172	103	93	82	2,2	8,2	8,9		
31	100	100	99,0	92,3	48,2	8,8	190	146	137	95	78	62	60	34	26,2	22,3	30,2		
32	100	97,9	40,6	0,6	0	0	416	401	386	309	279	186	173	161	5,0	21,3	26,6	360	
33	100	100	97,1	12,0	0,3	0,2	247	242	236	206	195	153	137	105	0,6	0,8	3,4	250	
34	100	100	100	99,8	97,2	57,4	74	72	70	61	52	16	10	5	2,9	25,0	47,2		
35	100	100	100	82,0	17,1	0,7	222	194	167	125	113	73	68	64	28,8	31,5	16,2		

4.2.1.4. Conclusions

- The apparatus used in this investigation was relatively cheap and easy to manufacture and could be used in conjunction with standard laboratory apparatus.

- The repeatability varied from one geotextile to another, indicating a variation due to geotextile differences rather than test method.

- The opening sizes obtained differ from those given by manufacturers which illustrates the sensitivity to different test methods.

4.2.2 Normal Permeability

The term "normal permeability" refers to the permeability in a direction perpendicular to the face of the geotextile as opposed to "in-plane permeability", which refers to the ability of the geotextile to carry water within itself.

Whereas the units for soil permeability are usually m/s, normal permeability for geotextiles is usually expressed in $\ell/m^2/s$. This is due to the fact that the length of the flow path, i.e. the geotextile thickness, cannot be determined accurately. The thickness varies with pressure and, in practical applications where the geotextile is in contact with soil and aggregates, soil ingress and pressure by the aggregates alter the effective geotextile thickness.

The term "permeability" is therefore not entirely correct since the permeability coefficient, k, cannot be determined. For this reason the terms "permittivity" and "water percolation" are often used to refer to normal permeability. In this document, unless otherwise stated, the terms "normal permeability" or "permeability" are used, meaning normal permeability, permittivity or water percolation.

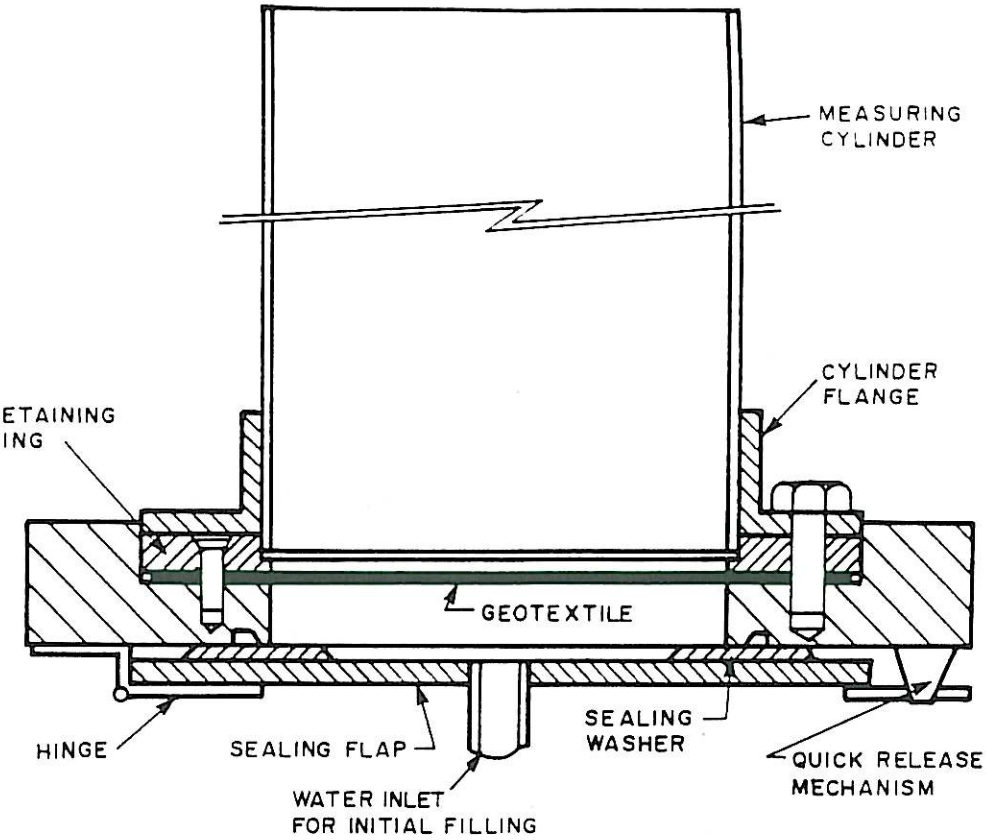
4.2.2.1 Apparatus

Permeabilities of geotextiles can be measured using either a falling head or a constant head permeameter. Typical apparatus used for these two types of tests are shown on Figure 4.4. The falling head permeater (Figure 4.4(a)) consists of a cylinder of known volume under which the geotextile is mounted. The cylinder is filled with water and a sealing flap is released, allowing the water to flow through the geotextile. The time required for the cylinder to empty is measured and the permeability can be calculated.

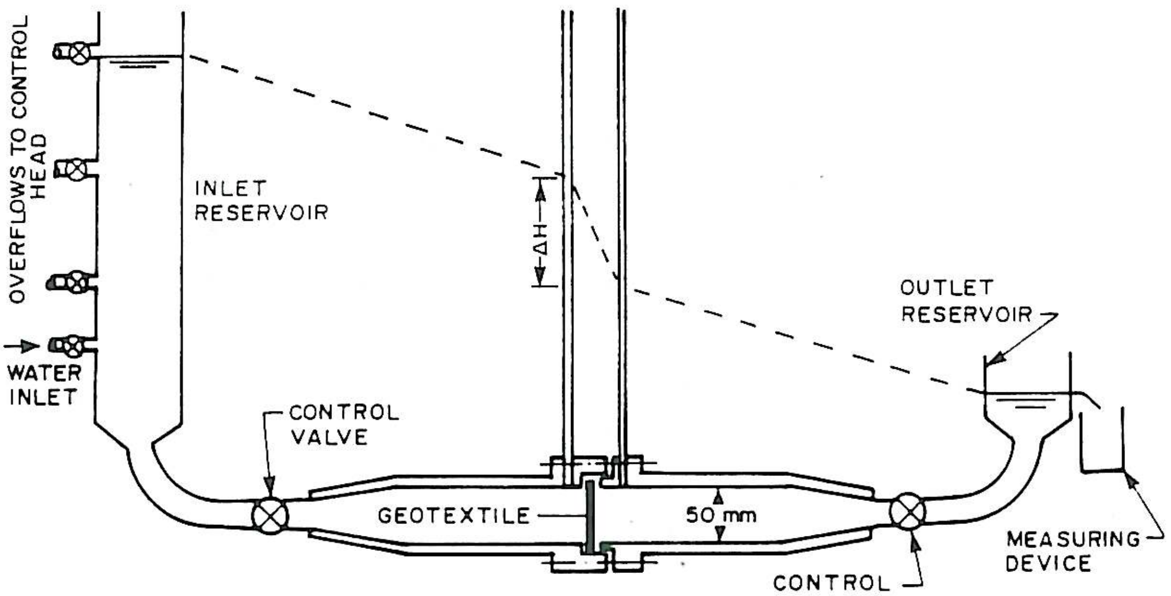
With the constant head permeameter, water is allowed to flow through the geotextile continuously and the head loss across the geotextile (ΔH in Figure 4.4(b)) is observed using two piezometers. The head loss is adjusted, using the inlet and outlet reservoirs, to a predetermined standard value and the through flow is then measured with a container at the outlet reservoir.

The apparatus for the falling head method is less complicated than that of the constant head method, but the method has certain disadvantages. Most geotextiles are highly permeable and the time for the cylinder to empty is almost impossible to measure with a hand held stop watch. For the same reason the sealing flap must be released instantaneously so as not to interfere with the water flow. With a mechanical method such as the one shown on Figure 4.4(a), this is not possible.

Despite its shortcomings, the falling head method was adopted for this investigation mainly because of the less complicated apparatus but also because the apparatus was to be used for the interface flow capacity tests, which are discussed in Chapter 5. In manufacturing the apparatus, the shortcomings of this method were taken into account, and devices were included to alleviate these. These devices are described below.



(a) FALLING HEAD



(b) CONSTANT HEAD

FIGURE 4.4

TYPICAL APPARATUS FOR DETERMING NORMAL PERMEABILITY OF GEOTEXTILES

Subsequent to the apparatus being manufactured and the tests performed, the SABS Technical Committee for geotextile specification (1987), decided on the constant head method as a standard test. For the sake of uniformity and standardisation, future tests should therefore be carried out using the constant head method as proposed by the SABS. In Section 4.2.2.3 a correlation is given between results obtained with the falling head permeameter and those obtained with the SABS constant head permeameter.

The apparatus that was manufactured and used in this investigation is shown on Figure 4.5. It consists of two perspex cylinders between which the geotextile is clamped, an electromagnet and an electronic timing device.

The circular electromagnet was introduced as an alternative to the mechanical release system shown in Figure 4.4(a). When electrical current passes through the mild steel cylinder, the magnetic field holds the disk firmly to the underside, thus supporting the water column. The release switch breaks the current, resulting in the disk falling away instantly. The water then flows freely, without interference from the supporting system.

As discussed earlier, the high permeability of most geotextiles makes it virtually impossible to measure the time manually. For this reason an electronic system was introduced. Electrical currents flow through the water between probes A and C and between probes B and C. When the water level reaches the bottom of probe A, the current between A and C is broken which triggers the timer. Similarly, the current between probes B and C is broken when the water level reaches the bottom of probe B, and the timer stops. The time has then been accurately recorded (to the nearest one hundredth of a second) for the water to pass between points A and B. The distance is known, or can be measured, and the permeability can be calculated.

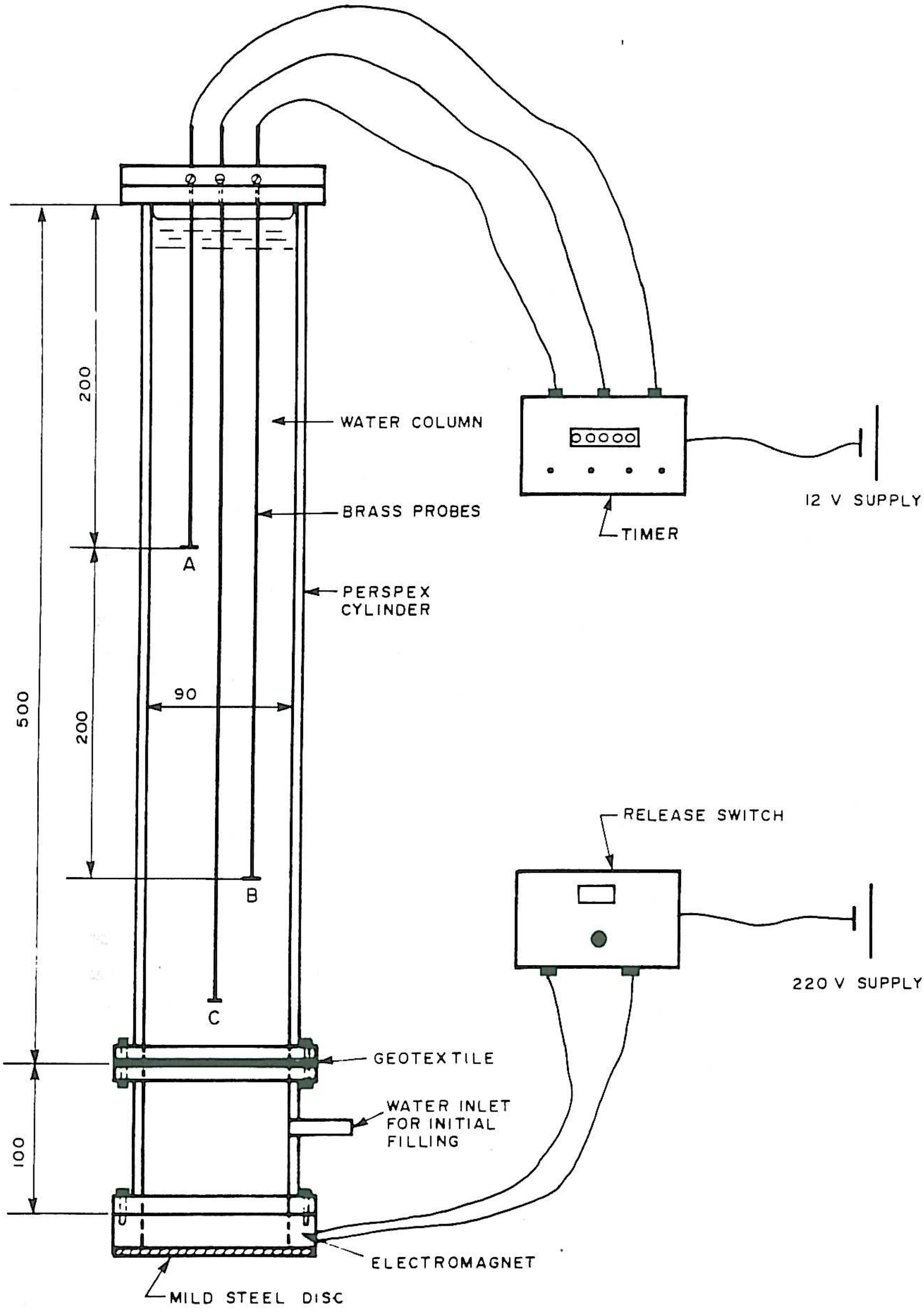


FIGURE 4.5

APPARATUS USED FOR DETERMINING NORMAL PERMEABILITY

Details of the electromagnet and the electronic timing device are documented elsewhere (van der Merwe, 1987(a)).

4.2.2.2 Test Procedure

Using the apparatus described above, the following procedure was followed to measure the normal permeability of geotextiles.

The vertical distance between the bottom of probes A and B was kept constant at 200 mm, with probe A at a distance of 200 mm from the top of the cylinder, as shown on Figure 4.5. A circular geotextile sample was clamped between the two cylinders, using nuts and bolts and silicone grease to ensure a watertight seal. With the electromagnet switched on and the supporting disk in position, the cylinder could be filled with water. Using the water inlet in the bottom cylinder, water was allowed to flow slowly into the cylinder up to a level just above the geotextile, thus removing any air entrapped in the sample. The cylinder was then filled from the top to minimise any distortion of the geotextile. Results showed that the initial height of the water level above the first probe did have some effect on the results. A difference of approximately 4 per cent was observed in the time measured when the initial water level was slightly above probe A, as opposed to when the cylinder was filled to the top. All further tests were performed with the initial water level being at the top of the cylinder.

Having filled the cylinders, the water temperature was recorded, the timer was reset to obtain a zero reading and the release button of the electromagnet was pressed. Photograph 4.2 shows the test immediately after releasing the support disk. After the cylinders had emptied, the time was recorded.

Once the sample was mounted between the cylinders, the actual testing was very simple and fast and a number of repetitions could be performed. The test was repeated five times for each sample, and five samples of each geotextile were tested, resulting in 25 values for each geotextile.

4.2.2.3 Results

Typical test results are shown on Table 4.4. The average of 25 measurements was used to obtain the permeability, γ, as follows:

$$\gamma = \text{normal permeability}$$
$$= \frac{V}{At}$$

where V = Volume of water passing through area A in time T

V = Ah

where h = height of water column = 200 mm

then $\gamma = \frac{0,20}{t}$ $m^3/m^2/S$ (t in seconds)

$= \frac{200}{t}$ $\ell/m^2/S$

TABLE 4.4. Typical results of normal permeability (Geotextile No 3)

Sample	Water temperature (^oC)	Time $\frac{1}{100}$ sec: Test number 1	2	3	4	5	Ave[2]	CoV[3]	γ[4]
1	18	46	44	46	45	39	44	6,6	
2	18	36	35	36	34	36	35,4	2,5	
3	19	49	48	47	48	40	46,4	7,9	
4	19	37	36	34	40	36	36,6	6,0	
5	19	37	35	35	35	35	35,4	2,5	
Average[1]							39,56	13,2	505,6

t = 0,3956 sec

[1] Average of 25 tests
[2] Average of 5 tests (1 sample)
[3] COV = Coefficient of variance (%)
[4] γ = Permeability ($\ell/m^2/S$) = 200/t

The coefficient of variance (CoV) was calculated for each sample as well as for the test to obtain an indication of the repeatability of the test method and the variability in the material respectively. The CoV obtained with each sample (variation due to the test method) was, with almost every test, less than 10 per cent.

A summary of all the results obtained is given in Table 4.5. The measured permeability, γm, is the average of 25 measurements and the coefficient of variation, CoV, is an indication of the variability of the geotextiles. The permeability which is published by the manufacturers of geotextiles is given as γp. The value that is expected to be obtained with the SABS constant head permeameter is given as γs. This value was obtained by a correlation between values obtained with the two methods. Only five geotextiles are known to have been tested with the SABS method, however, and these results are tentative.

It is evident that there are large differences between the values obtained with different test methods. The permeabilities quoted by the manufacturers were also determined with various test methods.

The non-woven needlepunched geotextiles were mounted with the smooth side i.e. the needled side, facing upwards in all the tests. Preliminary tests showed that when they were mounted this way the resulting permeabilities were up to 20 per cent higher than with the fluffy side facing upwards.

TABLE 4.5. Summary of normal permeabilities.

Geotextile	γm[1] (ℓ/m²/s)	CoV[2] (%)	γp[3] (ℓ/m²/s)	γs[4] (ℓ/m²/s)
1	689	0,7	150	145
2	503	11,5	120	125
3	506	13,3	120	125
4	329	7,8	110	105
5	179	8,2	90	89
6	642	15,4		140
7	577	7,0		133
8	575	8,4		133
9	555	4,0		130
10	548	8,2		130
11	496	13,9		124
12	357	16,2		109
13	245	10,0		97
14	781	4,6	400	155
15	668	23,6	380	143
16	752	11,4	312	152
17	505	10,4	238	125
18	333	19,5	192	106
19	144	18,5	133	86
20	454	5,2	121	119
21	329	16,5	130	106
2	196	22,9	90	91
23	127	10,5	78	84
24	165	8,3	47	88
25	215	11,7	47	93
26	551	3,4		130
27	56	5,4	15	76
28	69	3,0	5	77
29	36	4,8	17	74
30	50	3,5		75
31	17	10,8		72
32	1 131	25,8	400	193
33	315	11,5		104
34	6	13,8	15	71
35	15	6,3	2	72

1) γm = Permeability measured with falling head permeameter

2) CoV = Coefficient of variation

3) γp = Normal permeability according to manufacturers published data

4) γs = Normal permeability with SABS constant head permeameter, ΔH = 100 mm (based on correlation obtained with known values)

4.2.2.4 Conclusions

- The normal permeabilities of geotextiles are very dependent on the method and apparatus used and the standardisation of a test method is essential.

- The falling head permeater used in this investigation proved to be reliable and easy to use, but, to ensure uniformity of test method by all organizations, it is recommended that the SABS constant head permeameter be adopted as standard.

4.3 MECHANICAL (PHYSICAL) TESTS PERFORMED

Mechanical properties of geotextiles are not directly related to filtration or drainage functions but apply to a lesser extent to any application. In road subsurface drainage applications the largest stresses exerted on the geotextile is during construction, and a minimum strength is required to avoid tearing or puncturing of the geotextile. Physical parameters like mass per unit area are useful for identifying different geotextiles.

Mechanical tests that were performed are : mass per unit area, thickness and compressibility, strip textile tests and penetration load (CBR) test.

4.3.1. Mass per unit area

This parameter is used mainly to identify geotextiles and is especially useful to distinguish between various grades of a particular brand. The price of geotextiles is to a large extent determined by the mass per unit area.

4.3.1.1 Test procedure

Five samples of each geotextile were measured and weighed. The samples were 250 mm square (0,0625 m^2). Square samples were used rather than circular because woven geotextiles tend to lose the short fibres that result from circular samples. Square samples can also be cut and measured more accurately.

The mass of each sample was recorded to an accuracy of 0,01g, but in calculating the average mass per square meter, the values were rounded off to the nearest 1g.

4.3.1.2 Results

The results are shown on Table 4.6. The average value of five samples as well as the coefficient of variance (CoV) is shown. Also given are the values published by the manufacturers of the various geotextiles.

The higher value of CoV obtained with the needle punched nonwovens indicates that a larger variation can be expected with these geotextiles than with wovens and spun-bonded nonwovens.

4.3.2. <u>Thickness and Compressibility</u>

Geotextiles are highly compressible and, as such, a thickness cannot be quoted without specifying the pressure under which it was measured.

When a geotextile is used as a filter, the thickness is the length of the flow path in the Darcy equation to calculate permeability, as explained earlier (Section 4.2.2).

In certain applications, geotextiles act as water carriers in their longitudinal (or in plane) direction. In such applications the thickness determines the volume of water that can be carried.

Table 4.6 Summary of results: Mass per unit area.

Geotextile No	Mass per unit area		
	Measured*		Published#
	Avg. (g/m^2)	CoV (%)	(g/m^2)
1	177	6.9	150
2	209	6.9	210
3	265	2.9	270
4	305	7.9	340
5	619	4.0	550
6	135	3.8	130
7	152	4.1	150
8	178	3.1	180
9	205	2.6	200
10	275	1.9	270
11	314	1.8	300
12	366	1.8	360
13	508	1.9	500
14	122	7.8	90
15	112	7.4	110
16	164	3.9	140
17	197	7.2	200
18	298	8.6	280
19	455	3.3	350
20	228	6.3	400
21	111	2.9	112
22	136	1.1	136
23	190	1.1	190
24	556	7.4	350
25	618	5.1	450
26	140	5.4	130
27	111	3.4	110
28	137	0.7	135
29	206	0.1	220
30	361	1.0	325
31	531	1.1	500
32	249	1.4	250
33	290	2.6	300
34	378	0.4	370
35	710	0.6	750

* Avg = Average of 5 samples.

CoV = Coefficient of variance

= Standard deviation/average * 100

\# Manufacturer's published data.

The compressibility of geotextiles determines their change in thickness under pressure and as such is a useful parameter. Compressibility is obtained by measuring thickness under various pressures.

4.3.2.1 Test procedure

The test set-up is shown in Figure 4.6(a). A geotextile sample was placed under a circular loading plate with diameter 210 mm (A = 0,034636 m^2). The samples were slightly larger than the loading plate. Pressure was applied and the thickness measured, using the dial gauge, at the following pressures: 2 kPa, 5 kPa, 10 kPa, 20 kPa, 100 kPa, 500 kPa and 1000 kPa.

For the 2 kPa pressure, only the loading plate was applied. This plate was machined to weigh exactly 7,070 kg, resulting in a 2 kPa pressure (diameter = 210 mm). For the other pressures, the weight of the loading plate and spacer (M2 + Ms = 107,52 N) were deducted from the load applied by the press to obtain the desired pressures.

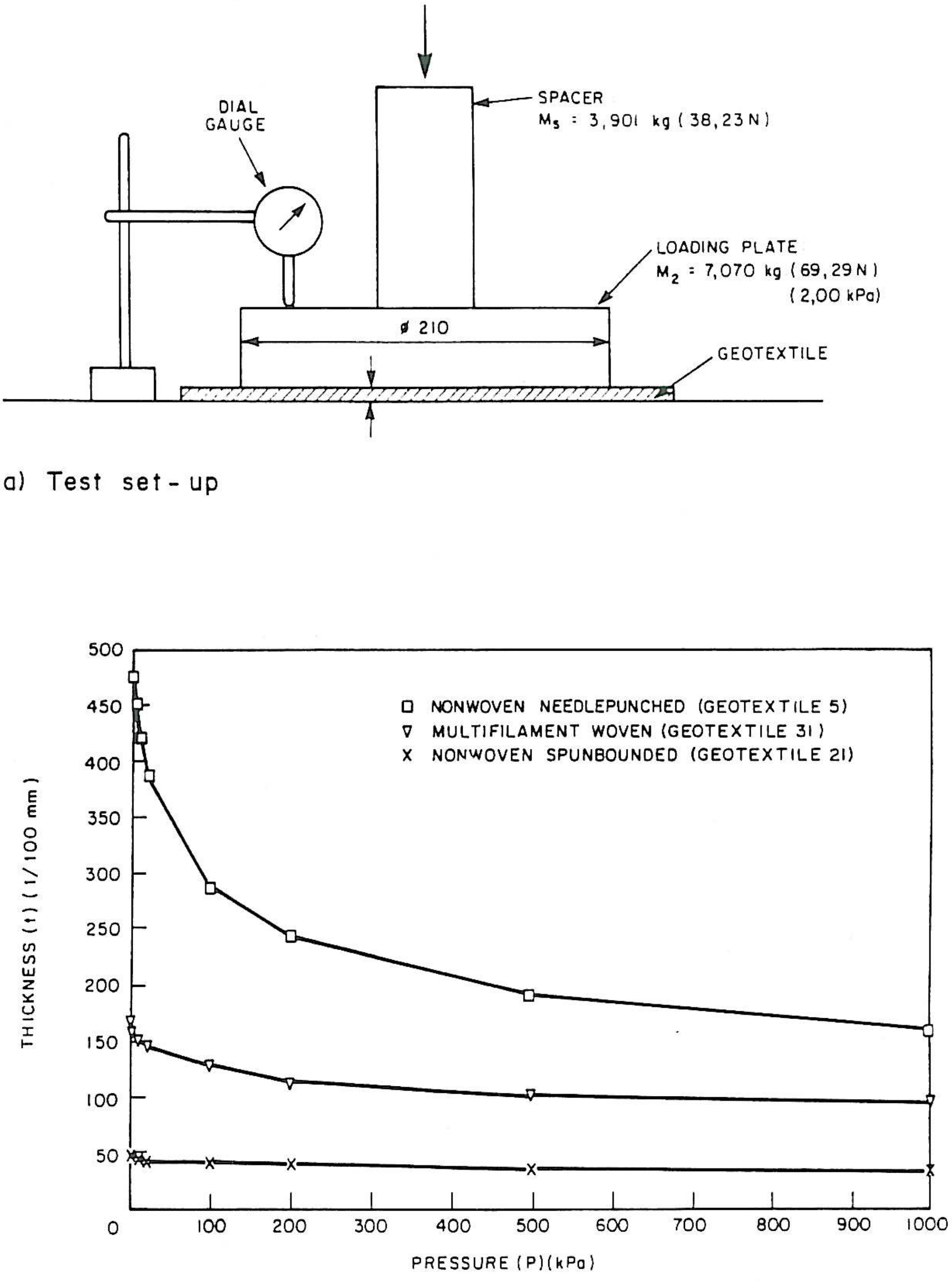
Three samples of each geotextile were tested and the average values obtained.

4.3.2.2 Results

A summary of the results is shown on Table 4.7. These include thickness and compressibility.

Compressibility is usually calculated between 2 and 20 kPa and between 20 and 200kPa, and is defined as follows:

$$C_{2-20} = \frac{t_2 - t_{20}}{t_2} \times 100$$



a) Test set-up

b) Example of results

FIGURE 4.6

THICKNESS & COMPRESSIBILITY

Table 4.7 Summary of Results: Thickness and Compressibility

Geotextile No	Measured Results*										Published $		
	Thickness, t (1/100mm)								Compressiblity		Thickness, t(1/100mm)		
	Pressure, P (kPa)										Pressure, P(kPa)		
	2	5	10	20	100	200	500	1000	C_{2-20}	C_{20-200}	2	20	200
1	137	130	119	106	74	63	51	43	22.6	40.6	160	100	60
2	214	193	169	146	100	84	67	55	31.8	42.5	200	40	80
3	257	236	213	185	126	107	86	72	28.0	42.2	240	170	100
4	305	269	244	216	149	126	100	84	29.2	41.7	280	210	130
5	475	450	421	387	287	242	190	159	18.5	37.5	420	330	200
6	142	123	110	96	67	57	45	38	32.4	40.6	150		
7	160	138	122	106	75	63	50	41	33.8	40.6	170		
8	191	172	153	131	93	78	61	51	31.4	40.5	210		
9	212	194	171	149	104	87	69	57	29.7	41.6	210		
10	259	239	221	202	139	118	95	79	22.0	41.6	280		
11	280	258	236	211	148	125	99	83	24.6	40.8	300		
12	331	309	287	258	186	158	127	106	22.0	38.8	340		
13	414	386	358	326	240	201	159	133	21.3	38.3	430		
14	133	119	109	99	75	65	53	46	25.6	34.3	120		
15	114	107	98	88	66	57	48	41	22.8	35.2	130		
16	168	154	142	131	95	82	67	57	22.0	37.4	160		
17	176	163	153	140	107	93	76	65	20.4	33.6	210		
18	208	196	186	175	143	128	108	93	15.9	26.9	260		
19	332	320	307	290	238	09	173	150	12.6	27.9	300		
20	195	184	173	161	126	109	89	76	17.4	32.3	330		
21	46	46	45	44	41	39	36	34	4.3	11.4	45	42	36
22	53	52	51	50	47	45	42	40	5.7	10.0	48	44	39
23	58	58	58	57	54	53	50	48	1.7	7.0	56	52	48
24	429	412	389	364	301	270	221	179	15.1	25.8			
25	359	347	331	313	261	236	200	167	12.8	24.6			
26	64	60	58	55	48	45	40	37	14.1	18.2			

Table 4.7, <u>Summary of Results: Thickness and Compressibility</u> continued

Geotextile No	Measured Results*										Published $		
	Thickness, t (1/100mm)								Compressiblity		Thickness, t(1/100mm)		
	Pressure, P (kPa) 2	5	10	20	100	200	500	1000	C_{2-20}	C_{20-200}	Pressure, P(kPa) 2	20	200
27	64	56	50	44	32	29	25	22	31.2	34.1			
28	71	67	61	56	43	38	33	32	21.1	32.1			
29	76	72	70	67	58	52	45	40	11.8	22.4			
30	144	138	134	130	116	108	86	74	9.7	16.9			
31	166	159	153	145	126	112	102	95	12.6	22.8			
32	81	81	81	80	78	77	75	70	1.2	3.8			
33	78	77	76	75	73	71	67	62	3.8	5.3			
34	63	61	59	57	53	51	48	46	9.5	10.5			
35	119	117	115	113	105	101	93	89	5.0	10.6			

* Average of 3 samples
$ Manufacturer's published data.

$$C_{20-200} = \frac{t_{20} - t_{200}}{t_{20}} \times 100$$

where: C_{2-20} = Compressibility between 2 and 20 kPa

C_{20-200} = Compressibility between 20 and 200 kPa

t_2 = thickness at 2 kPa

t_{20} = thickness at 20 kPa

t_{200} = thickness at 200 kPa

Also shown on Table 4.7 are the thickness values as published by the manufacturers where these values were available. They are usually given at 2, 20 and 200 kPa.

The decrease in thickness under increasing pressure is shown in Figure 4.6(b) for three geotextiles. These are typical curves for the three most common types of geotextiles, namely nonwoven needle-punched, woven and nonwoven spun-bonded. The nonwoven needle-punched show the highest degree of compressibility and the nonwoven spunbondeds remain relatively thin irrespective of the applied pressure.

It is evident from the results that the compression under pressures larger than 200 kPa is relatively small. This is true for all geotextiles tested and it is therefore unnecessary to test at pressures higher than 200 kPa.

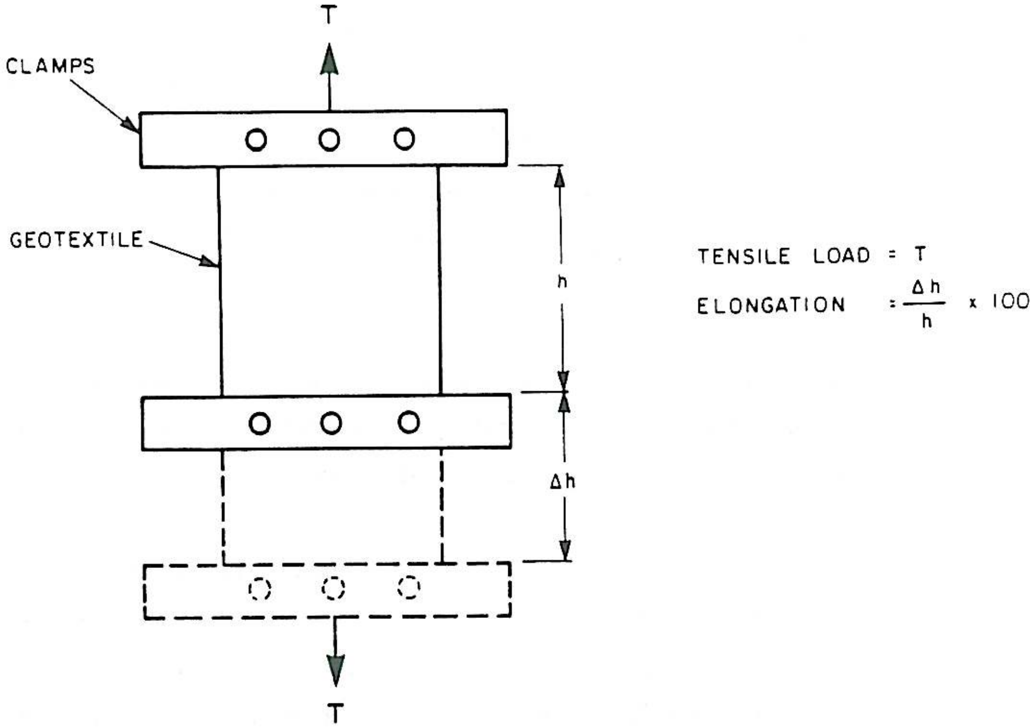
4.3.3 Strip tensile test (tensile strength)

The strip tensile strength is the most widely used test for determining the strength of geotextiles.

4.3.3.1 Test procedure

The test set up is shown on Figure 4.7(a) and the dimensions of the samples on Figure 4.7(b). The width to height ratio is of importance. It has been shown (Baudonnel et al (1982), Myles et al (1986)) that, in the case of nonwoven geotextiles,

a) TEST SET UP



b) SAMPLE DIMENSIONS

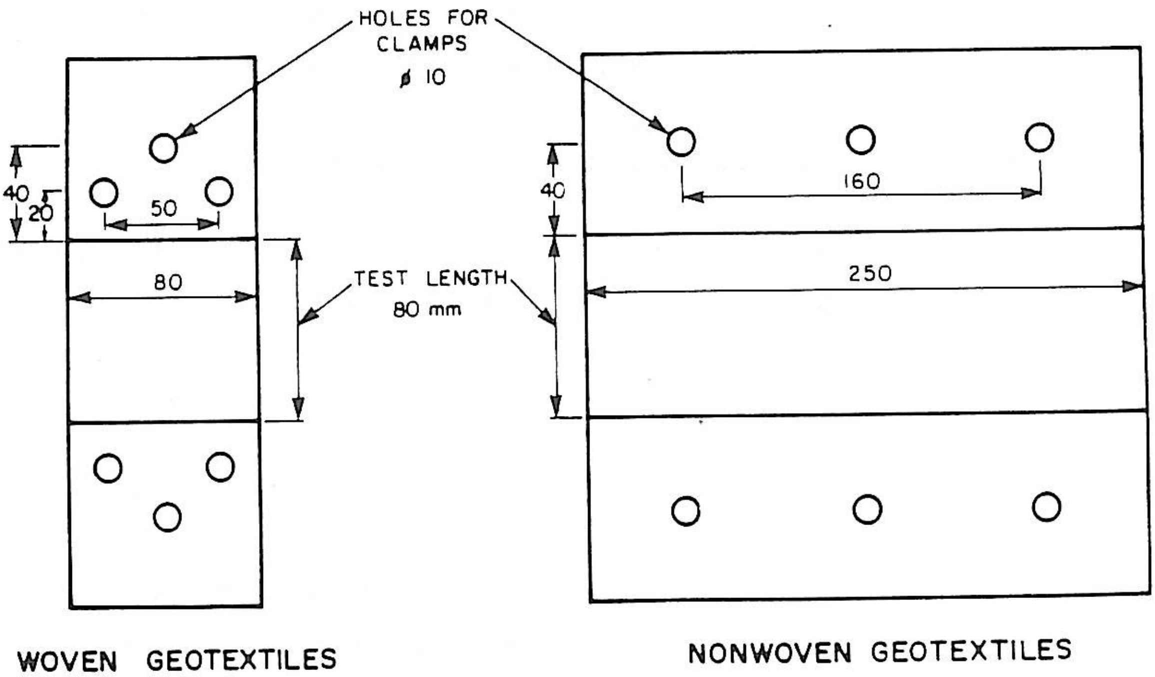


FIGURE 4.7

STRIP TENSILE TEST

940-4-5769/1 BS

the strength is under-estimated if the width to height ratio is less than 3. For woven geotextiles this aspect is not very critical and the width can be reduced. A reduced width for wovens is often necessary to enable the testing equipment to cope with the higher strengths of these geotextiles. Based on these facts, an SABS technical committee on geotextiles (SABS (1987)) decided on the dimensions shown on Figure 4.7(b) as standard dimensions for the strip tensile test (i.e. 80 mm x 80 mm for wovens and 80 mm x 250 mm for nonwovens).

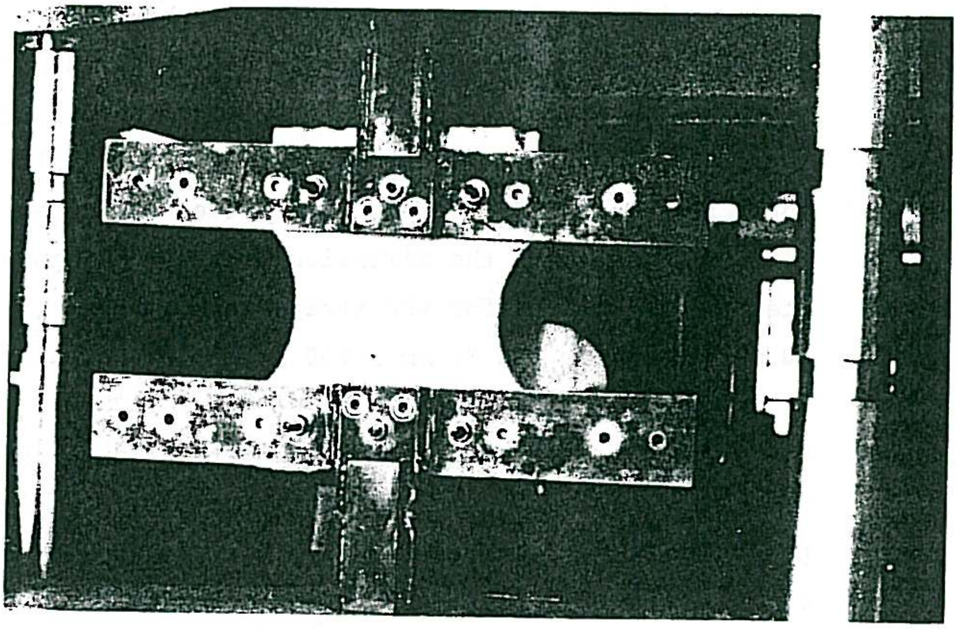
To perform the test, the samples were held over their full width at both ends by clamps. The clamps consisted of two pieces of flat iron between which the geotextile was held, using nuts and bolts. Holes were provided in the samples for this purpose.

A tensile load was applied, and the maximum load attained, as well as the elongation at the maximum load, were recorded. Photograph 4.3 shows the test being performed. Three samples of each geotextile were tested and the average values calculated.

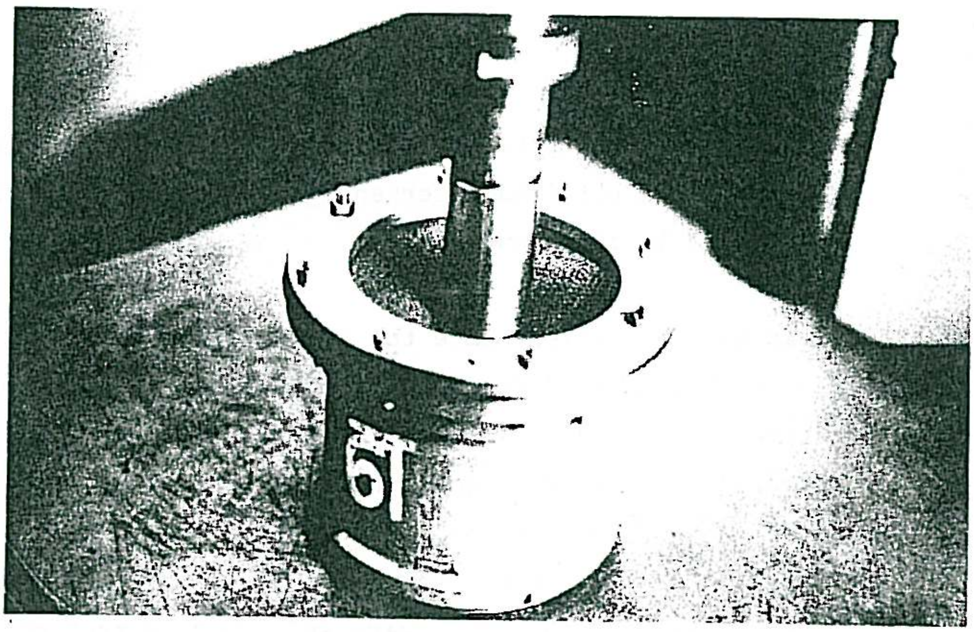
4.3.3.2 Results

A summary of the results is given in Table 4.8, showing the maximum load obtained, percentage elongation and the values published by the manufacturers.

Nonwoven geotextiles have the same strength in all directions, but wovens have different strengths in their warp and weft directions. To evaluate these differences, the tensile strength was measured in both the warp and weft directions, as well as in a direction 45° to the warp and weft directions. These values are also shown in Table 4.8, as well as the average values for the warp and weft directions and the average for all three directions measured.



PHOTOGRAPH 4.3
Strip tensile test



PHOTOGRAPH 4.4
Penetration load (CBR) test

Table 4.8 SUMMARY OF RESULTS: STRIP TENSILE TESTS

Geotextile No	Test direction	Measured Results: T(max) [1] Avg [2] (kN/m)	Measured Results: T(max) [1] CoV [3] (%)	Measured Results: Elongation [4] Avg (%)	Measured Results: Elongation [4] CoV (%)	Published [5] T(max) (N/m)	Published [5] Elongation (%)
1		17.9	6.0	66	28.5	7.0	60-80
2		18.3	12.5	64	14.1	10.4	60-80
3		20.6	19.2	61	4.7	14.6	60-80
4		27.3	13.6	75	5.8	19.0	60-80
5		30.5	11.7	63	9.2	33.0	60-80
6		5.9	6.4	78	15.1	7.2	70-75
7		7.2	9.2	77	5.2	9.0	75-80
8		8.9	6.5	76	1.0	10.4	70-75
9		9.5	3.8	63	2.3	11.2	70-75
10		15.3	9.7	64	8.5	17.0	70-75
11		17.9	3.1	69	8.4	18.0	70-75
12		17.9	11.1	64	12.2	22.0	65-70
13		22.2	12.7	72	12.3	31.0	65-70
14		6.4	3.8	60	49.7	4.5	50-80
15		5.8	5.3	51	40.3	6.0	50-80
16		7.8	13.9	43	23.9	7.0	50-80
17		10.3	25.7	41	26.2	10.0	50-80
18		14.7	29.6	52	27.9	14.0	50-80
19		20.0	14.3	100	32.6	16.0	50-80
20		9.5	6.9	91	8.4	19.0	50-80
21		6.0	18.5	52	2.4	4.2	30
22		7.3	3.0	50	1.5	6.0	33
23		12.0	9.7	58	10.6	9.0	38
24		18.1	1.5	140	2.9		
25		25.0	26.6	131	21.8		
26	Warp	8.1	16.0	25.0	2.8		
	Weft	6.9	9.0	55.0	7.9		
	45deg [6]	4.1		24.0			
	Avg.Wa/We [7]	7.5		40.0			
	Av.W/W/45 [8]	6.4		34.7			
27	Warp	14.2	4.3	13.0	10.8	22.0	15
	Weft	14.6	3.3	13.0	3.5	16.0	15
	45deg	0.5		25.0			
	Avg.Wa/We	14.4		13.0			
	Av.W/W/45	9.8		17.0			
28	Warp	18.8	1.6	14.0	9.8	22.0	15
	Weft	16.3	8.5	13.0	18.4	22.0	15
	45deg	5.3		25.0			
	Avg.Wa/We	17.6		13.5			
	Av.W/W/45	13.5		17.3			
29	Warp	32.4	7.3	18.0	12.4	34.0	
	Weft	33.7	2.4	19.0	4.6	38.0	
	45deg	11.4		30.0			
	Avg.Wa/We	33.1		18.5			
	Av.W/W/45	25.8		22.3			

Table 4.8, SUMMARY OF RESULTS: STRIP TENSILE TESTS (continued)

Geotextile No		Measured Results				Published 5)	
		T(max) 1)		Elongation 4)			
	Test direction	Avg 2) (kN/m)	CoV 3) (%)	Avg (%)	CoV (%)	T(max) (N/m)	Elon-gation (%)
30	Warp	55.4	4.2	13.0	1.0		
	Weft	36.5	0.9	21.0	6.9		
	45deg	15.9		31.0			
	Avg.Wa/We	46.0		17.0			
	Av.W/W/45	35.9		21.7			
31	Warp	73.2	4.4	16.0	13.3		
	Weft	68.8	6.9	18.0	4.9		
	45deg	7.4		20.0			
	Avg.Wa/We	71.0		17.0			
	Av.W/W/45	49.8		18.0			
32	Warp	47.7	6.6	24.0	3.7	68.0	25
	Weft	29.4	5.1	20.0	5.3	36.0	20
	45deg	11.6		34.0			
	Avg.Wa/We	38.6		22.0			
	Av.W/W/45	29.6		26.0			
33	Warp	46.7	0.9	22.0	8.6	68.0	25
	Weft	37.0	6.1	12.0	1.0	60.0	15
	45deg	14.7		34.0			
	Avg.Wa/We	41.9		17.0			
	Av.W/W/45	32.8		22.7			
34	Warp	55.0	4.2	13.0	6.0		
	Weft	51.0	11.7	12.0	8.6		
	45deg	21.7		23.0			
	Avg.Wa/We	53.0		12.5			
	Av.W/W/45	42.6		16.0			
35	Warp	95.8	2.5	28.0	12.0		
	Weft	67.5	20.4	23.0	11.1		
	45deg	18.4		29.0			
	Avg.Wa/We	81.7		25.5			
	Av.W/W/45	60.6		26.7			

Notes:

1. T(max) = maximum tensile load = tensile strength
2. Average of 3 samples
3. CoV = Coefficient of variance
   = standard deviation/average x 100
4. Elongation at maximum load (see Figure 4.7(a))
5. Manufacturer's published data
6. 45° to the warp and weft directions
7. Average of warp and weft directions
8. Average of warp, weft and 45° directions.

It is evident that there were definite differences in the strengths obtained in the various directions. The strengths obtained in the 45° direction were markedly lower than those in the other directions. These results are analysed and discussed in more detail in Section 4.3.5 below.

4.3.4 Penetration Load (CBR) test

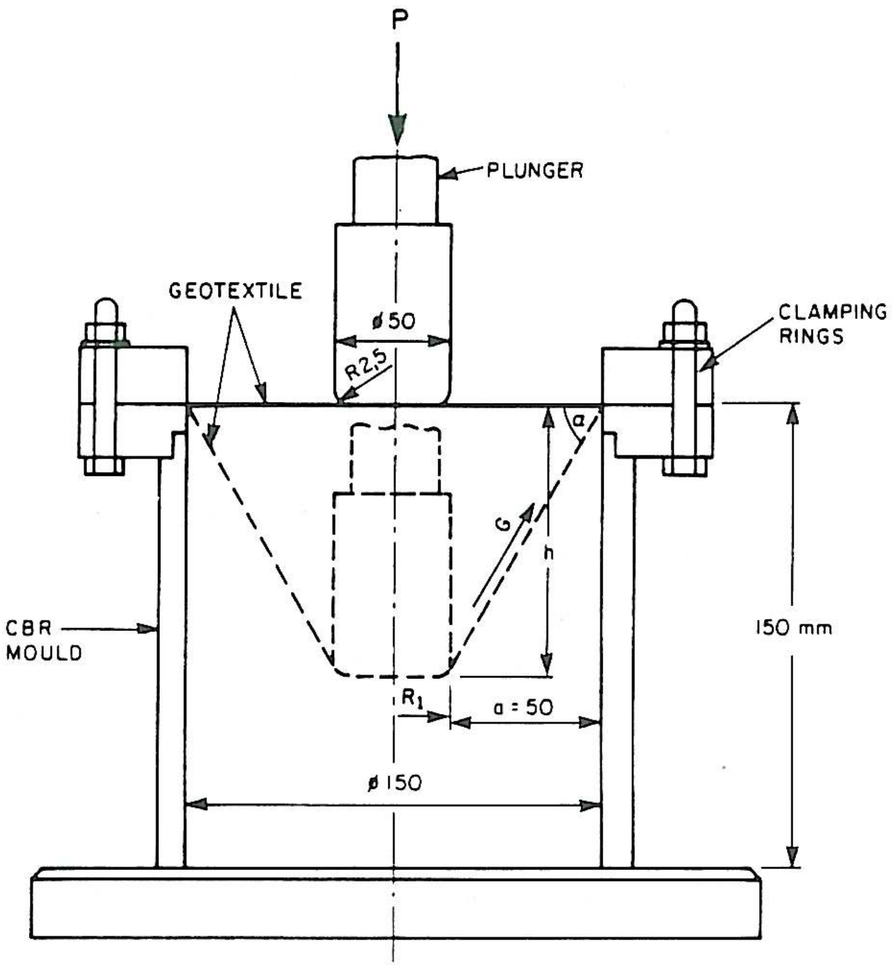
This test is used to measure the resistance of the geotextile to penetration by a loaded plunger. Although the test does not measure tensile strength directly, an indication of the tensile strength can be obtained from the results, as indicated in Section 4.3.5 below.

The test does, however, have certain advantages if compared to the strip tensile test described in Section 4.3.3 above. These include:

- The method of failure of the geotextile, i.e. by penetration or puncture, is a likely method of failure in subsurface drainage applications.
- The test equipment is standard CBR equipment, found in most soil laboratories, with a modified plunger. The equipment needed for the strip tensile strength is specialised equipment which is used in very few laboratories.
- The strength is measured in all directions simultaneously, as opposed to one direction with the strip tensile test. This is important for woven geotextiles, as shown above (Table 4.8).

4.3.4.1 Test procedure

The apparatus is shown on Figure 4.8. A standard CBR mould, a modified CBR plunger (rounded edges) and special clamping rings were used. Details of the clamping rings that were manufactured for this test are shown on Figure 4.9.

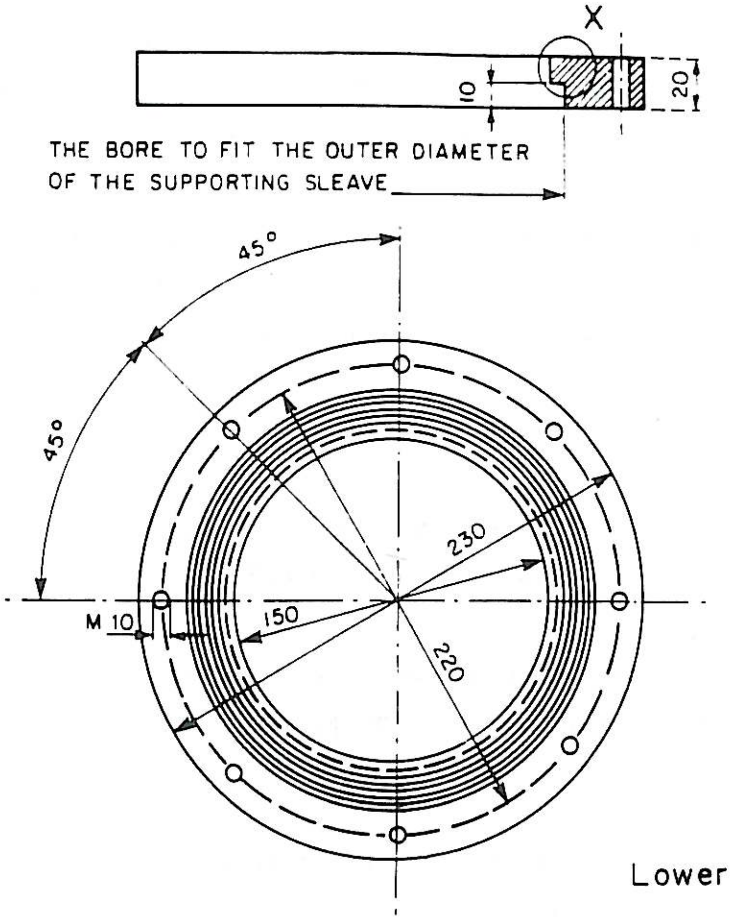


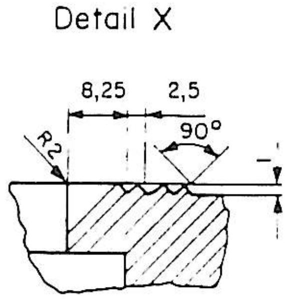
PENETRATION LOAD = P

ELONGATION $= \dfrac{\sqrt{a^2 + h^2} - a}{a} \times 100$

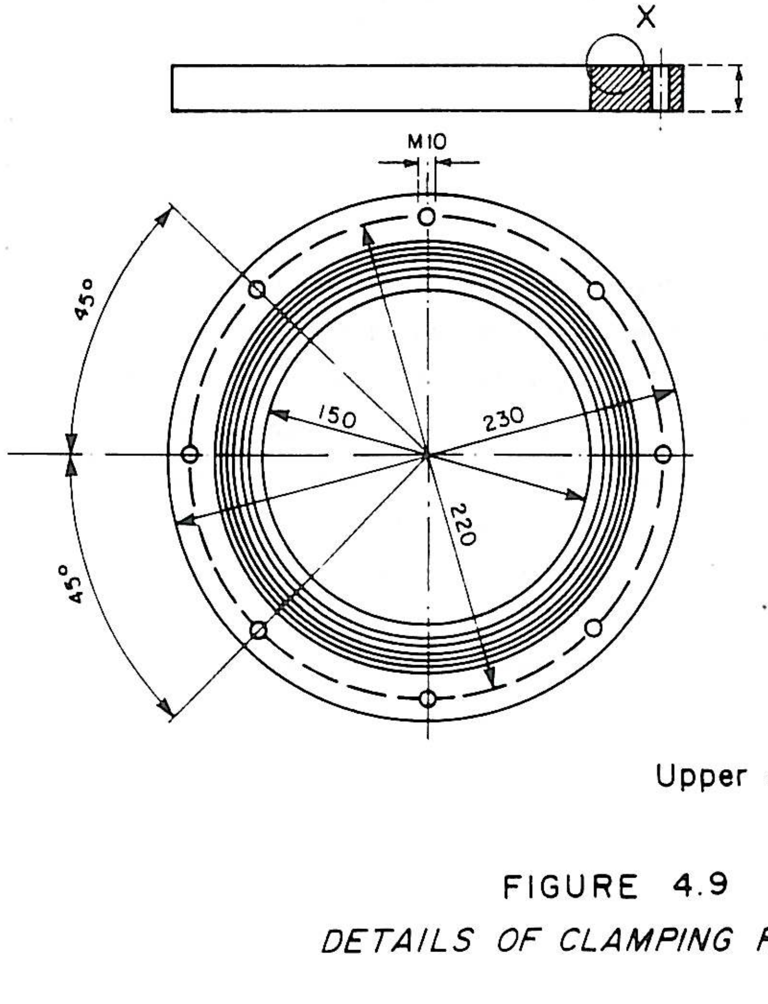
FIGURE 4.8

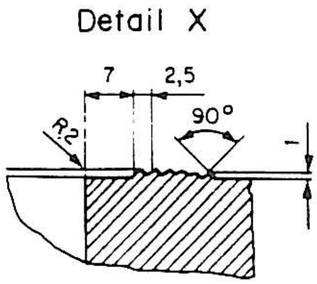
PENETRATION LOAD (CBR) TEST





Lower clamping ring





Upper clamping ring

FIGURE 4.9

DETAILS OF CLAMPING RINGS

The geotextile samples were cut to fit in the clamping rings, using these rings as stencils. The clamps with the sample were placed on the mould and the load applied at the rate specified in the CBR test. Photograph 4.4 shows the test being performed. The maximum load attained and the elongation at that load were recorded. Three samples of each geotextile were tested and the average values calculated.

4.3.4.2 Results

A summary of the results is given in Table 4.9. The elongation was calculated as shown in Figure 4.8.

Although the tensile strength is not measured directly, it can be calculated, using the following theory (Alfheim et al, 1977):

Referring to Figure 4.8, the vertical force exerted by the plunger is P. If the force in the geotextile is G, the vertical component is G Sinα. At any stage of the test, P is equal to the vertical force in the geotextile:

$$P = g \operatorname{Sin}\alpha$$

$$\text{or } G = P/\operatorname{Sin}\alpha$$

If G = total stress in the geotextile

and $2\pi R$ = the circumference of the geotextile at a distance R from the centre of the plunger

then T = $G/2\pi$ = the stress per unit length in the radial direction, if it is assumed that the stress in the geotextile is uniformly distributed over the circumference of the geotextile and that the tensile stress (T) is the same in all directions (warp, weft, etc.)

Table 4.9

Summary of Results: Penetration Load (CBR) Test

Geotextile No	P(max) 1)		Elongation 4)		T(max) 5)
	Avg 2) (N)	CoV 3) (%)	Avg (%)	CoV (%)	(calculated) (kN/m)
1	2343	8.8	43.3	1.5	20.8
2	2833	9.8	39.3	3.7	25.9
3	3273	9.7	32.9	1.7	31.6
4	3463	5.6	37.0	1.6	32.3
5	6590	4.1	48.2	2.6	56.8
6	1370	7.4	53.1	1.7	11.5
7	1625	6.7	60.9	7.6	13.2
8	1982	5.7	54.4	1.1	16.6
9	2263	3.4	52.4	3.8	19.1
10	2940	8.1	44.7	5.6	25.9
11	3840	13.8	42.8	5.2	34.2
12	3707	5.0	43.3	2.2	33.0
13	5573	4.6	54.4	3.2	46.6
14	1194	8.2	31.2	0.7	11.7
15	1148	9.9	28.9	3.8	11.6
16	1303	1.1	26.4	3.2	13.6
17	2073	5.2	29.7	4.2	20.7
18	2127	11.4	24.2	6.3	22.8
19	3423	3.2	35.2	1.9	32.4
20	2117	4.6	31.5	7.5	20.7
21	1036	12.7	34.9	1.3	9.8
22	1220	12.2	29.7	5.7	12.2
23	1917	9.3	42.4	8.2	17.1
24	3933	7.4	59.3	2.8	32.2
25	4563	3.9	64.8	2.5	36.5
26	992	5.3	20.7	2.3	11.3
27	2313	1.1	15.4	5.3	29.5
28	2833	2.7	21.2	3.0	31.9
29	5093	0.3	23.8	1.4	55.0
30	6050	2.1	11.4	2.7	87.5
31	7387	6.3	15.3	4.0	94.5
32	4803	5.2	25.8	0.8	50.4
33	2233	7.8	8.5	4.1	36.7
34	3520	20.8	9.8	6.7	54.2
35	7973	3.6	20.9	2.5	90.3

Notes:

1) P(max) = maximum penetration load
2) Avg = average of 3 samples
3) CoV = Coefficient of variance
   = standard deviation/average load x 100
4) Elongation at maximum load (see Figure 4.8)
5) T(max) = calculated tensile strength (see text, Section 4.3.4.2)

by substituting G:

$$T = P/2\pi R \ \mathrm{Sin}\alpha$$

The maximum stress occurs at the point of penetration, i.e. at the edge of the plunger

Where $R = R_1$

Then $T = P/2\pi R_1 \ \mathrm{Sin}\alpha$

Substitute in the above: $\sin\alpha = h/x$

$$x = \sqrt{a^2 + h^2}$$

Then $T = P \sqrt{a^2 + h^2}/2\pi R_1 h$

The tensile strengths were calculated, using the above, and are also shown on Table 4.9. A comparison between these values and the tensile strengths measured with the strip tensile test is made in Section 4.3.5 below.

4.3.5. Analysis and Discussion

It is not always practical to perform all possible tests on geotextiles and correlations are often sought.

4.3.5.1 Mass per unit area and penetration load (CBR)

Figure 4.10 shows the correlation between mass per unit area and penetration load, P(max). These values were obtained from Tables 4.6 and 4.9 respectively. The correlation is relatively poor ($r^2 = 0{,}6926$). This is not surprising, since the strength is not only dependant on the amount of synthetic material, but also on the type of fibre and the method of construction.

4.3.5.2 Penetration Load (CBR) and strip tensile

Correlations were obtained between penetration load, P(max) and three different values of the tensile strength, T(max). The three values were T(max) in the warp direction, the average of the warp and weft directions, and the average of the warp, weft and 45° directions. These values were obtained from Tables 4.8 and 4.9 for T(max) and P(max) respectively.

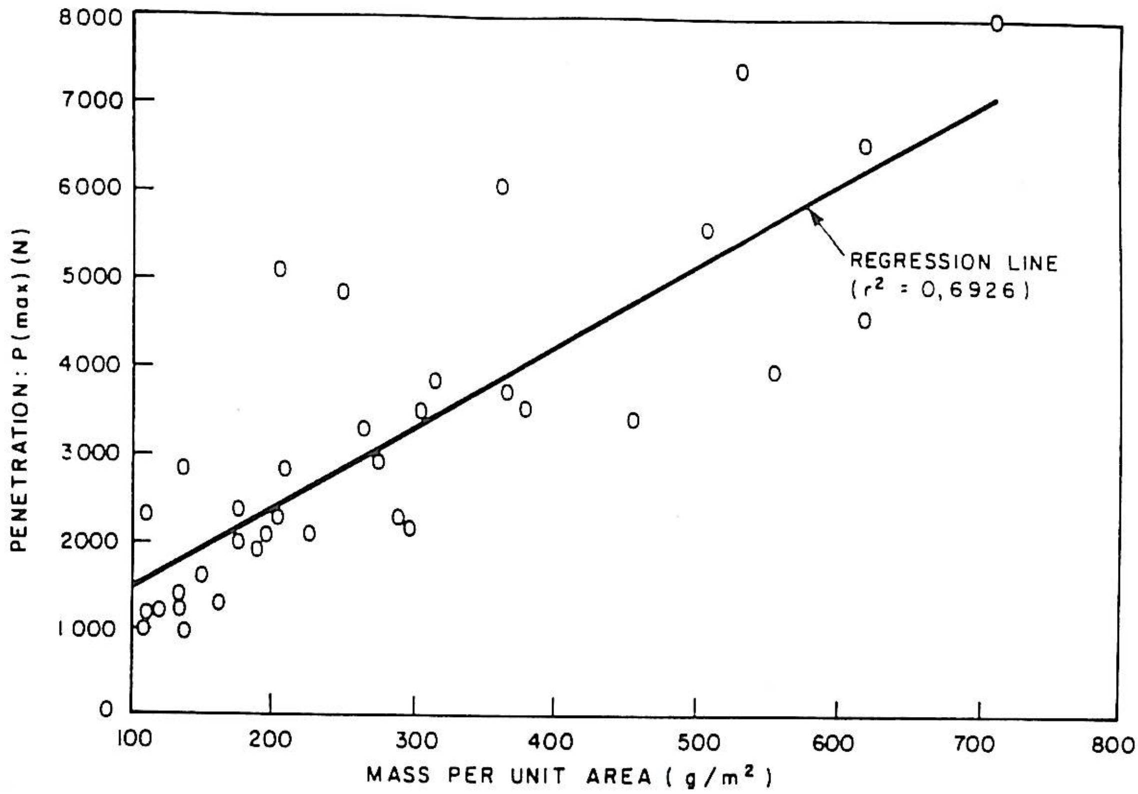


FIGURE 4.10

MASS PER UNIT AREA versus PENETRATION

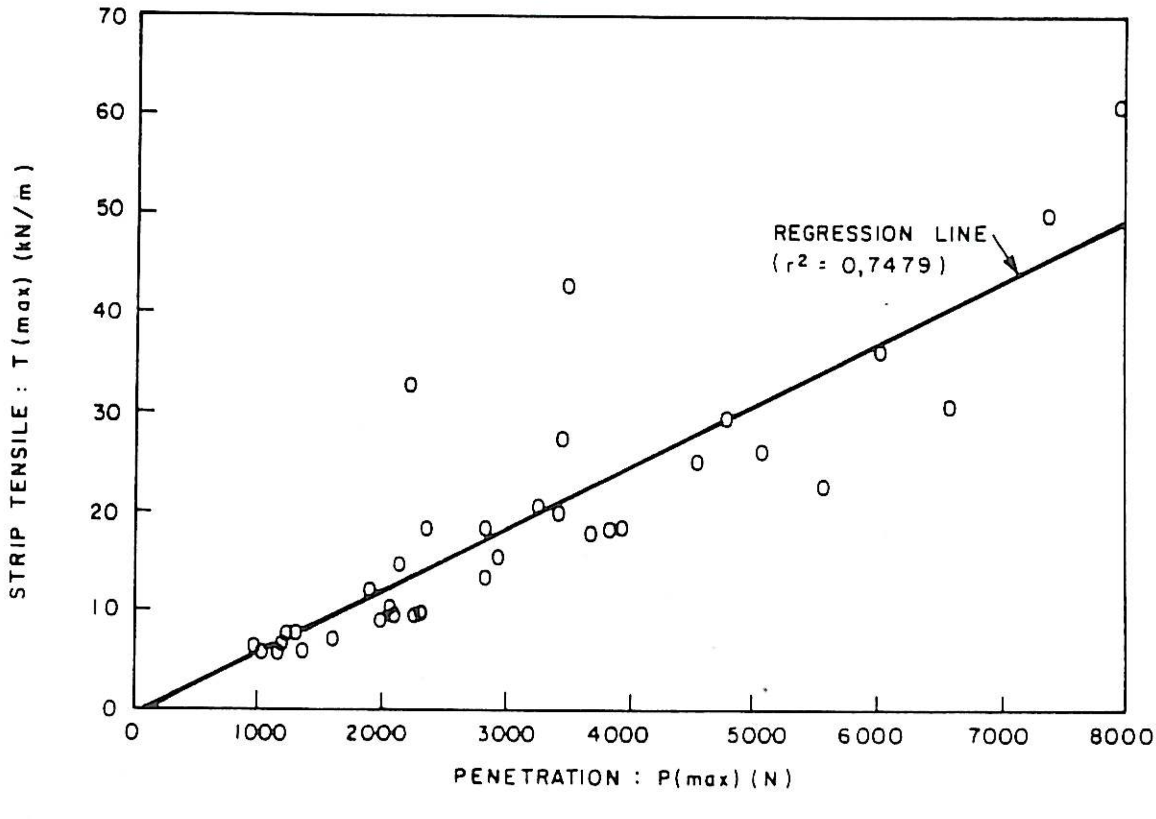


FIGURE 4.11

PENETRATION versus STRIP TENSILE (AVG. 3 DIRECTIONS)

The correlation between P(max) and the average for T(max) in all three directions is shown in Figure 4.11. The correlation co-efficient is r^2 = 0, 7479. For T(max) in the warp direction only and for the average in the warp and weft directions, r^2 was 0,6701 and 0,6932 respectively. This indicates that the correlation improves if the average values of more directions are used in the correlation. This was expected since the penetration load test measures strength in all directions. The best correlation obtained (r^2 = 0,7479), however, is still relatively poor.

A much higher correlation (r^2 = 0,8548) was obtained between T(max) measured with the strip tensile test and T(max) calculated from the penetration load test, as discussed in Section 4.3.4.2 above. These results are shown on Figure 4.12. Once again the average values in all three directions were used for the strip tensile test. All the values in Figure 4.12 are below the line of equality, however, indicating that higher values of T(max) were obtained with the penetration load test than with the strip tensile test. This is probably due to the assumptions made in deriving the theoretical formula for calculating tensile strength from the penetration load test and also because only three directions were measured in the strip tensile test.

Figure 4.13 shows the correlation between elongation obtained with the two tests. The correlation coefficient (r^2 = 0,6852) is considerably lower than that obtained with the strength values, and again the line of equality does not coincide with the regression line.

4.3.6. Conclusions

(a) Mass per unit area is a useful test for distinguishing between different geotextiles. It is also a relatively simple test to perform. It does not give an adequate

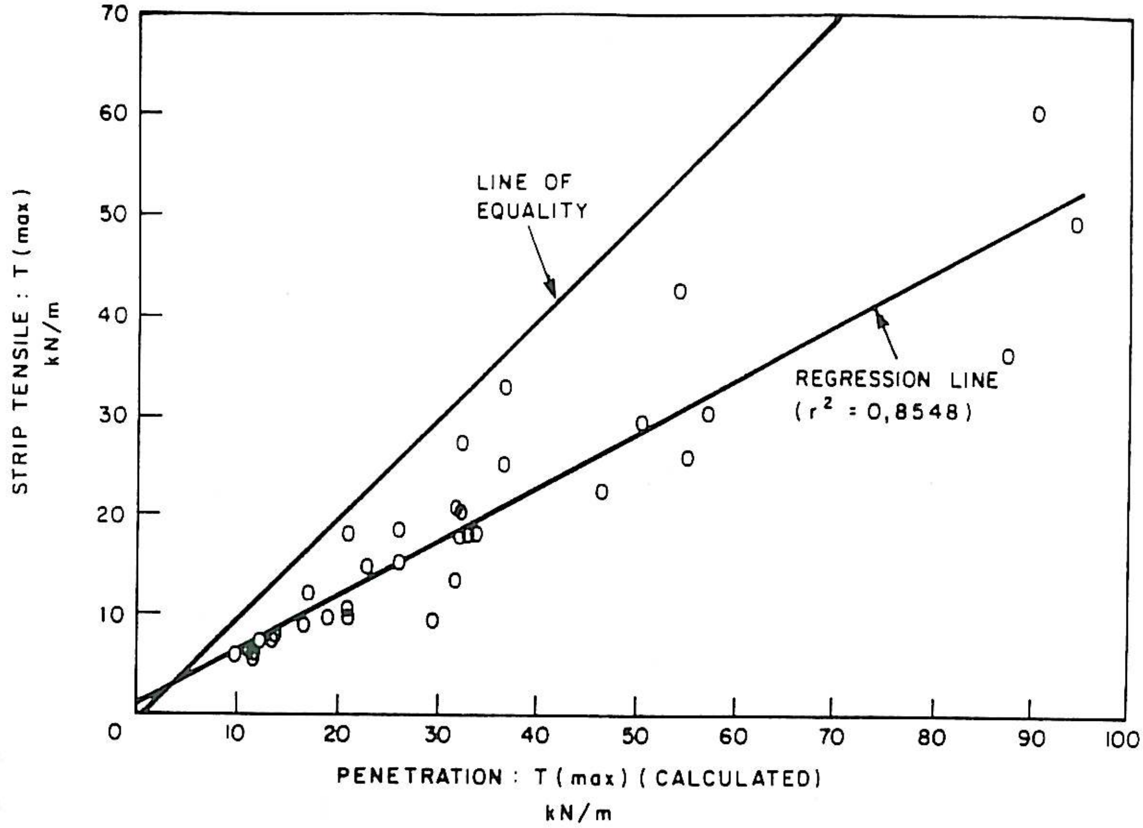


FIGURE 4.12

PENETRATION (CALCULATED TENSILE) versus STRIP TENSILE (AVG. 3 DIRECTIONS)

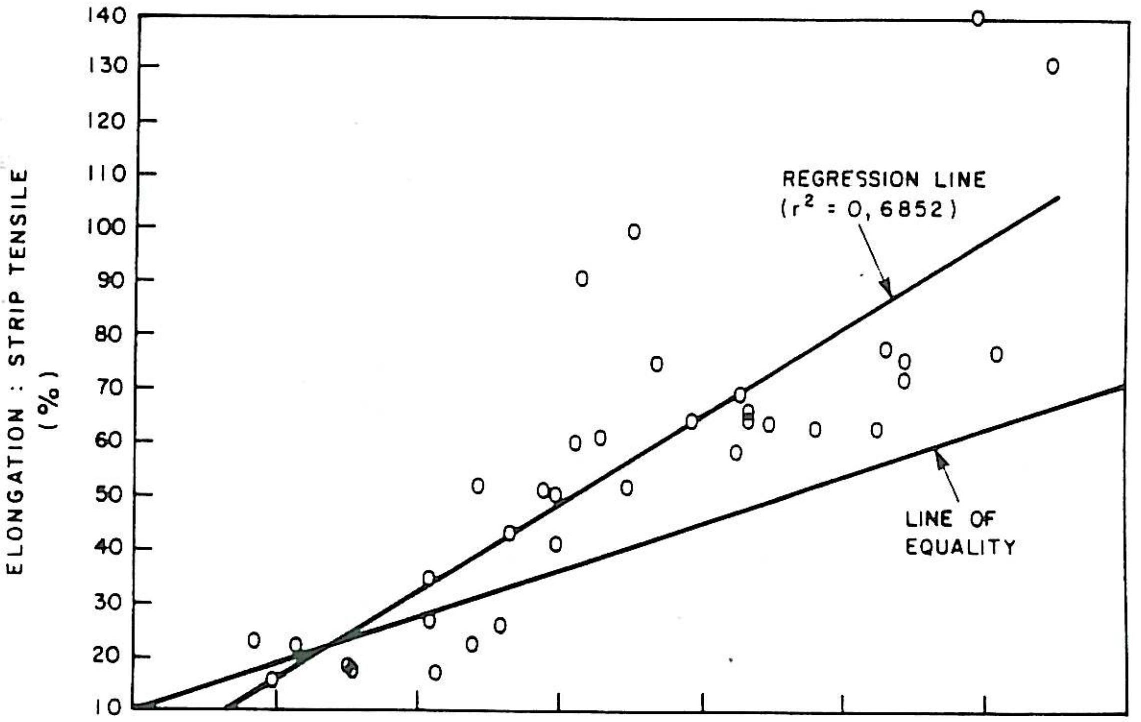


FIGURE 4.13

ELONGATION (PENETRATION) versus ELONGATION (STRIP TENSILE) (AVG. 3 DIRECTIONS) indication of the strength of the geotextile and strength tests must therefore be performed.

(b) Thickness and compressibility tests indicated that the thickness of geotextiles can reduce considerably under pressure, especially with nonwoven needle-punched geotextiles. This should be taken into account in calculating hydraulic parameters. It is not envisaged that this test will often be used to quantify geotextiles for road subsurface drainage applications. If the test is performed, pressures exceeding 200 kPa need not be applied.

(c) The strip tensile test is the most accurate method of obtaining tensile strengths of geotextiles. The test has disadvantages, however, in that the equipment needed is specialised and not found in every soil laboratory. Furthermore, the tensile strengths of woven geotextiles vary in different directions (e.g. warp, weft) and, to obtain a true indication of the geotextile's strength, tests should be performed in all directions, resulting in more tests and higher cost of testing.

(d) The penetration load (CBR) test has definite advantages if compared to the strip tensile test, namely,

- CBR equipment which is found in most soil laboratories can be used with only minor modifications.
- Strength is measured in all directions simultaneously and only one test is therefore needed.
- The method of failure (by penetration or puncture) is likely to occur in practice.
- A reasonable correlation ($r^2 = 0,85$) exists between tensile strength calculated from penetration load tests and actual tensile strength. An indication of tensile strength can therefore be obtained.

4.4 OTHER IN-ISOLATION TESTS

Numerous in-isolation tests exist that are used for geotextile evaluation worldwide. Many of these tests were inherited from the textile industry and their applicability to geotextiles in general, and subsurface drainage applications specifically, are in many cases very limited or non-existent. The most widely used of these tests are described briefly in this section.

4.4.1 In-plane Permeability

In-plane permeability (or transmissivity) is a measure of the geotextile's ability to transport fluid in it's in-plane (or longitudinal) direction. The test is performed by holding the geotextile between two plates under various pressures (similar to the thickness and compressibility test described in Section 4.3.2), and measuring the volume of water that passes through the sample.

The problems associated with the test method are similar to those with the normal permeability, namely that the soil/geotextile interaction is not taken into account. In practical applications the geotextile is in contact with soil and the ingress of soil particles into the geotextile will change its permeability characteristics.

The test is not applicable to road subsurface drainage applications, where geotextiles are used as filters and not as water carriers.

4.4.2 Tear Strength

This test measures the ability of the geotextile to withstand the propagation of tear once the geotextile has been damaged. An incision is purposely made in the geotextile before it is subjected to a tensile test, as illustrated in Figure 4.14.

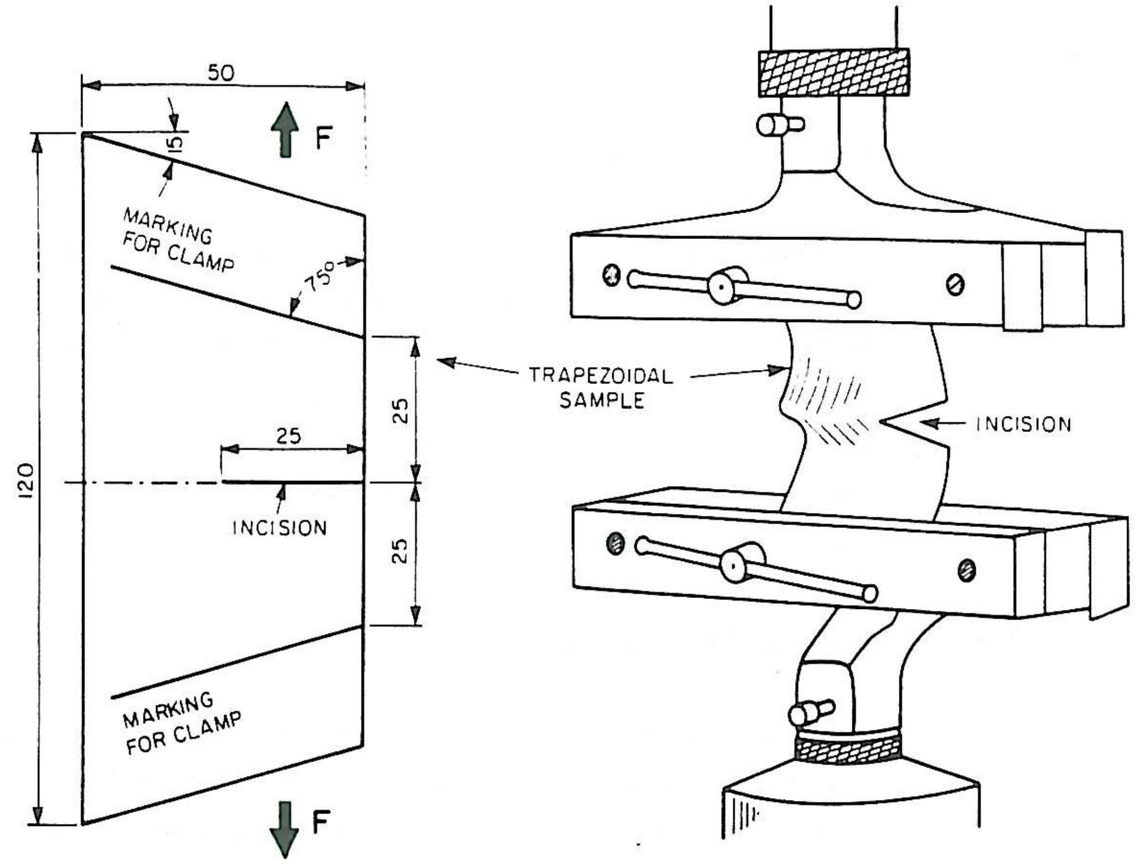


FIGURE 4.14
TEAR TEST

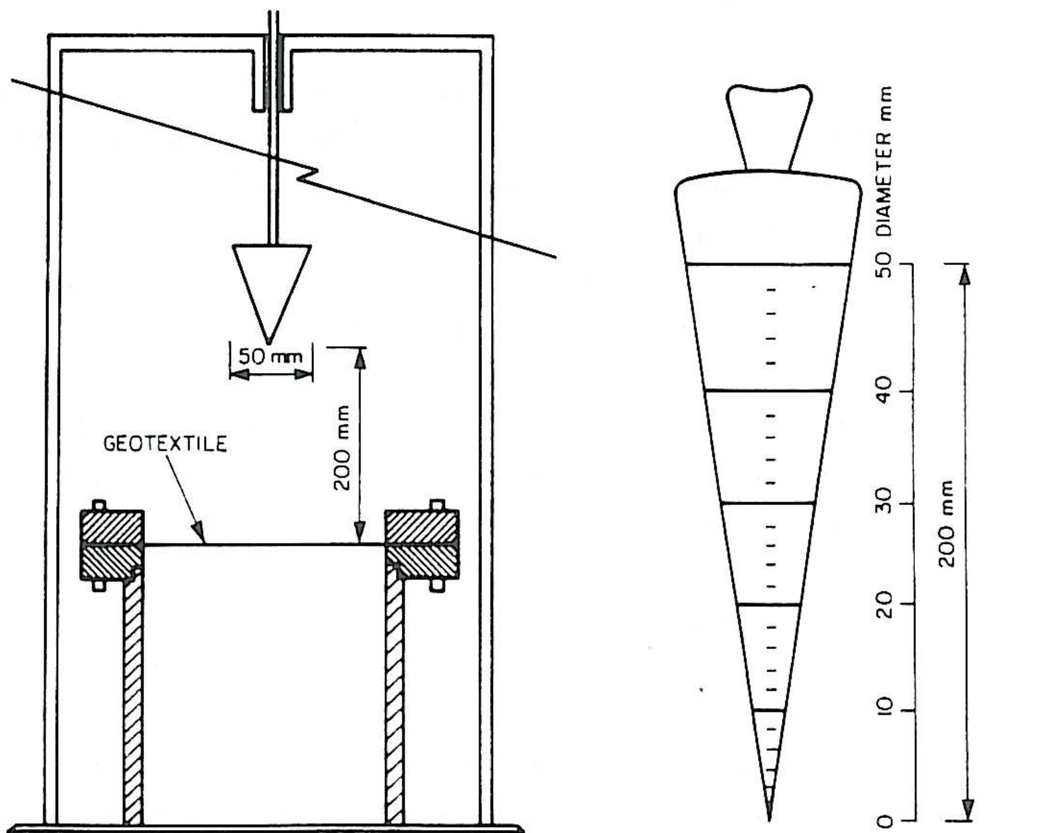


FIGURE 4.15
DROP CONE TEST

As mentioned earlier, the highest stresses exerted on a geotextile in road subsurface drainage applications are during construction. If the geotextile is damaged at that stage, the damaged part should be replaced and its ability to withstand tear propagation is therefore not relevant.

4.4.3 Drop cone test

Figure 4.15 shows a typical drop cone test apparatus. The test is performed by dropping a cone with certain mass and dimensions from a fixed height and recording the damage in the geotextile (depth of penetration of the cone).

There are similarities between this test and the penetration load, or CBR, test (described in Section 4.3.4), in that both give an indication of puncture resistance. The penetration load test has definite advantages however. An indication of tensile strength can be obtained with the penetration load test and the equipment and test method used for that test (CBR test apparatus) is standardized and available in most laboratories. The drop cone test, on the other hand does not give a strength parameter and several versions of the test exist, none of which are as yet standardized.

4.4.4 Creep test

In the creep test, the geotextile is subjected to a constant tensile stress, which is less than its tensile strength, and the elongation with time is monitored. The creep characteristics of geotextiles are dependant on the polymer used and polyproylene is the only polymer normally used in geotextiles that shows significant creep.

Creep is of considerable significance in reinforcement applications. In filtration applications, such as subsurface drainage, the creep characteristics could possibly influence the performance of the geotextile if the creep is adequate to alter the pore sizes of the geotextile. The influence of pore sizes on filtration characteristics have not been adequately quantified, however, and the influence of creep was not considered in this investigation.

4.4.5 Durability

Durability tests fell beyond the scope of this investigation, but the durability of geotextiles is an important factor and should be taken into consideration. Geotextiles should be durable against chemical, biological and ultra voilet attack. Of these, ultra violet attack is the most common problem and geotextiles should not be exposed to sunlight for periods longer than a few weeks.

Various durability tests exist and these need to be evaluated and applicable tests standardized. It is recommended that this be done in future research.

4.5 CONCLUSIONS

Based on the results obtained as discussed in this chapter, as well as results from soil/geotextile compatibility tests (Chapters 5 and 7), the following in-isolation tests are recommended for testing geotextiles for road subsurface drainage applications:

- Constant head normal permeability (SABS apparatus)

- Mass per unit area

- Penetration load (CBR) test.

CHAPTER 5

LABORATORY EVALUATION OF SOIL/GEOTEXTILE COMPATIBILITY

CHAPTER 5

CONTENTS PAGE

CHAPTER 5

CONTENTS (continued) PAGE

5.1 INTRODUCTION

The information obtained from the literature (Chapter 3) indicated that soil/geotextile compatibility tests are needed to evaluate geotextiles and should form the basis of design criteria. The soil/geotextile compatibility tests that were performed in the laboratory as part of this investigation are described in this chapter.

The tests were performed for the following reasons:

- To develop, manufacture, evaluate and standardise test apparatus and test methods.

- To evaluate soil/geotextile compatibility and the long term behaviour of such systems.

- To determine the influence of geotextile and soil in-isolation characteristics on soil/geotextile compatibility and long term behaviour.

- To gather information that would serve as a basis for design criteria.

Two types of soil/geotextile compatibility tests were performed, namely, flow tests using permeameters and interface flow capacity (IFC) tests.

5.2 FLOW TEST APPARATUS AND TEST METHOD

In section 3.4 flow tests were discussed that were performed by various researchers. Those results showed similar tendencies, but comparisons between the tests could not be made because different apparatus and test methods were used. If flow tests are to be used as a standard method for geotextile evaluation, it is imperative that the apparatus and method be standardised.

This fact was taken into account in designing and manufacturing the apparatus. Standard materials that are easily obtainable were used (e.g. a standard diameter perspex tube) and the workshop manufacturing process was kept as simple as possible to ensure repeatability.

In this section the apparatus and test method are described, as well as the first series of tests that were used to evaluate and standardise the test method.

5.2.1 Description of Apparatus and Test Method

5.2.1.1 Apparatus

The apparatus is shown on Figure 5.1 and on Photograph 5.1. It is a permeameter which consists of two 90 mm (inner diameter) perspex cylinder sections between which the geotextile is clamped. The geotextile is supported by a brass mesh and the soil sample is placed on top of the geotextile. The cylinder sections are held together by three brass rods which pass through perspex caps at the free ends of the respective cylinder sections. Water flows from a constant head reservoir through the soil and geotextile continuously. A breather hole is located in the bottom cylinder section to ensure a constant (atmospheric) pressure on the downstream side of the filter. The bleeding hole in the perspex top is used to remove entrapped air when the cylinder is filled with water initially. In some tests a mesh was placed in the top cylinder at the water inlet to distribute the water flow more evenly. This is discussed in more detail in Section 5.2.2 below.

Because of the long term nature of the tests (up to 2000 hours) and to enable a large variety of soil/geotextile combinations to be tested, a total of 30 permeameters were manufactured. A test bench (shown in photograph 5.2) was also manufactured to mount the 30 permeameters. This bench consists of a metal framework with a 150 ℓ water tank. The water tank,

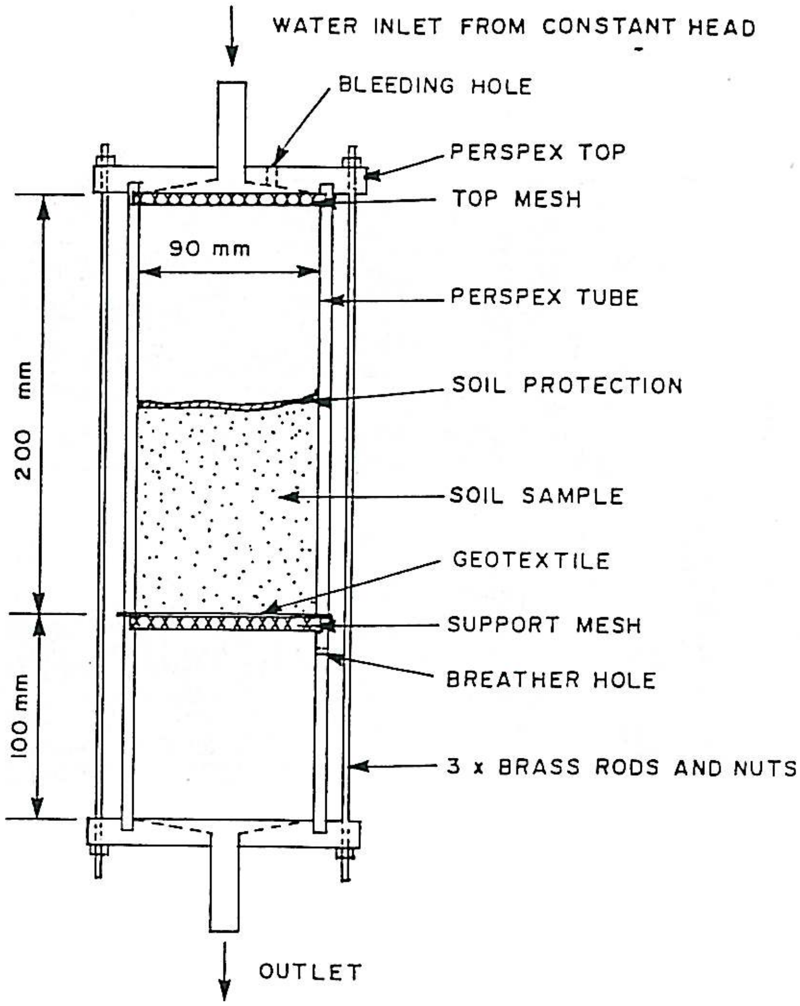


FIGURE 5.1 *APPARATUS FOR FLOW TESTS*

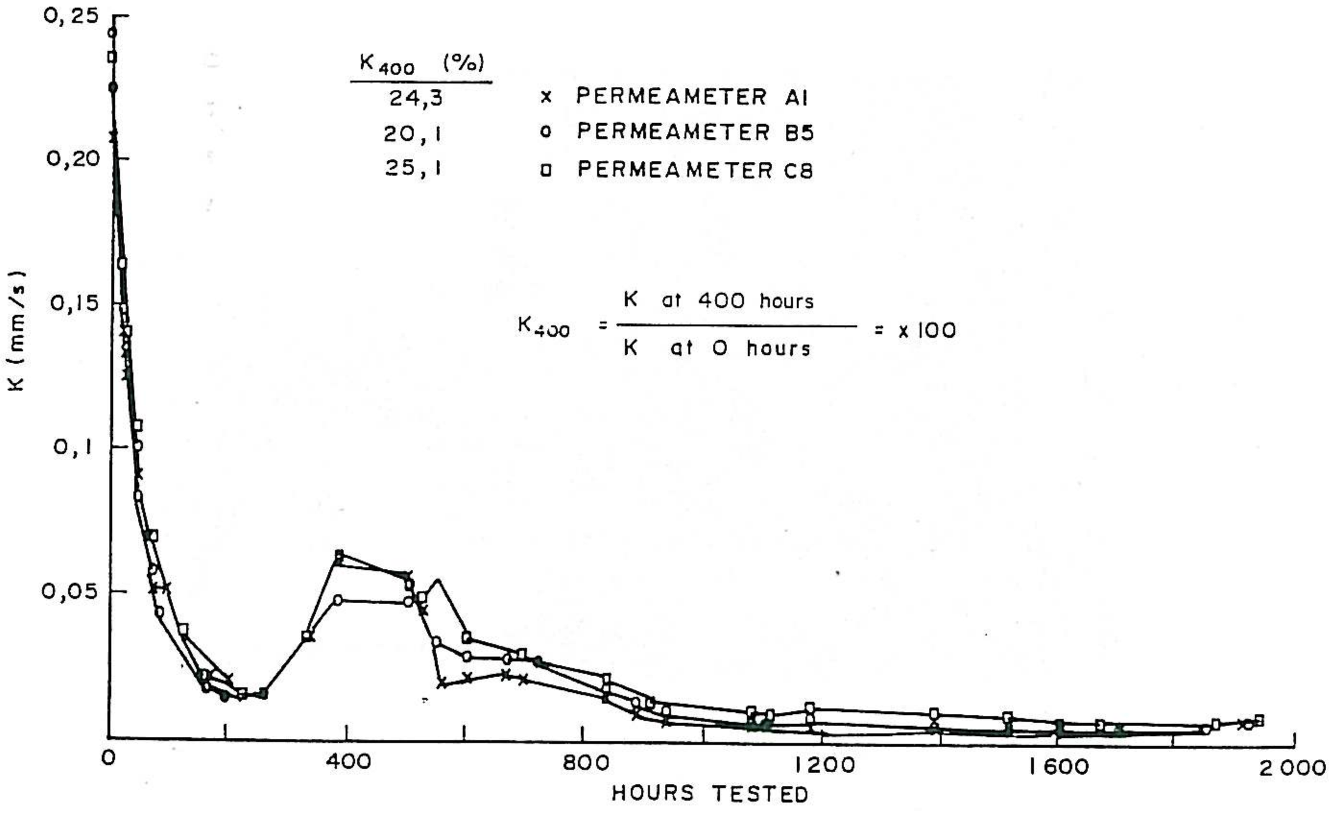
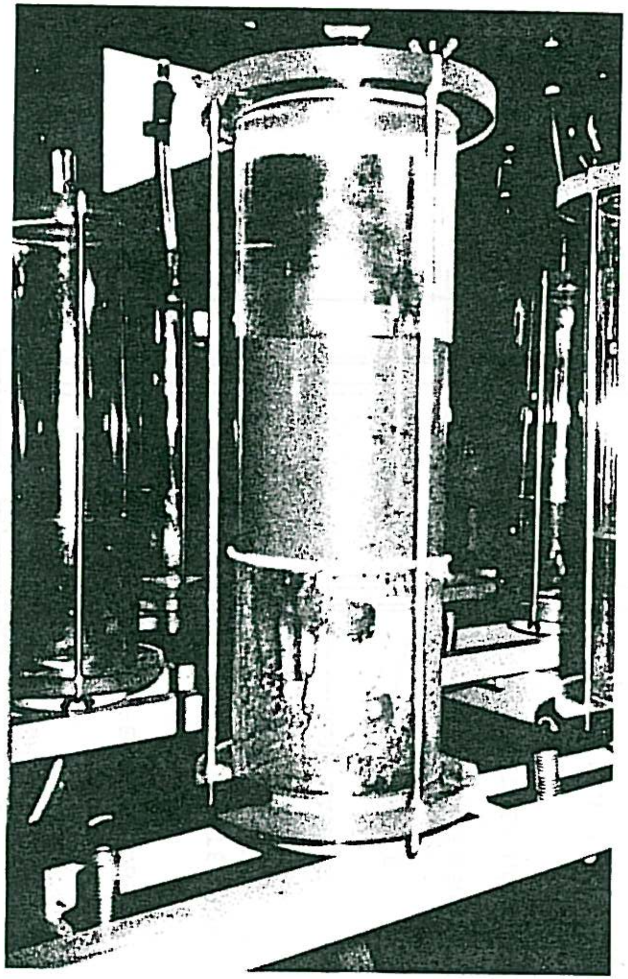
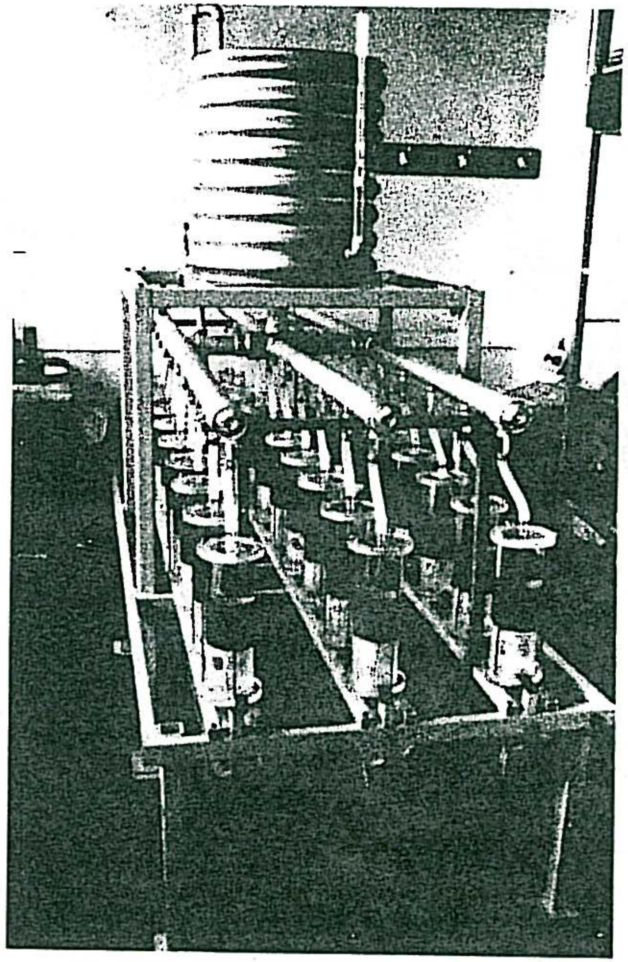


FIGURE 5.2 *LABORATORY FLOW TEST RESULTS: REPEATABILITY*



PHOTOGRAPH 5.1
Flow test permeameter



PHOTOGRAPH 5.2
Test bench with 30 permeameters and constant head water supply

mounted above the permeameters, is supplied with water from a laboratory tap (CSIR borehole water) and maintains a constant head by means of a ball valve of which the height can be adjusted. Water from this tank is distributed to the three rows of permeameters by 50 mm galvanised pipes. The permeameters are connected to these by 10 mm flexible hoses, each supplied with a valve, thus allowing each permeameter to be used independently.

The distance between the permeameter and the water supply determines the amount of head loss, and because the outer permeameters are 1,3 m further from the supply than the permeameters at the centre, a large diameter pipe was used to minimise the difference in head loss between the permeameters. The head loss is given by:

$$H_f = \frac{32 \nu VL}{gD^2}$$

Where ν = viscosity of water = $1 mm^2/s$

L = distance = 1 300 mm

g = 9 800 mm/s^2

D = pipe diameter = 50 mm

and V = velocity = $\frac{Q}{A}$

The total flow measured at the start of the first test series (see Sec. 5.2.2 below) was:

Q = 150 litres in 7,35 min

= 340 136 mm^3/sec

A = Area of 50 mm pipe

= 1 963,5 mm^2

Therefore V = 173,2 mm/sec.

Using these values, the head loss was

$$h_f = 0,29 \text{ mm}$$

A 1m head is used, resulting in a head loss difference between the permeameters of 0,03 per cent.

The flexible 10 mm hoses are the same length for each permeameter, and therefore do not result in a differential head loss.

5.2.1.2 Test method

The placing of the soil sample and initial filling of the permeameter with water was a fairly tedious procedure to ensure that there were no leakages or air entrapped and to obtain a smooth surface on top of the soil sample. Details of the procedure are documented elsewhere (van der Merwe, 1987(b)).

After the samples were placed, the valves were opened and water from the constant head water tank was allowed to flow through the soil and geotextile samples continuously. The flow rates were measured at regular intervals using stop-watches and containers. From these measurements the permeability coefficients, K, of the soil/geotextile systems were calculated at the different time intervals. Examples of these calculations are shown in Section 5.4. Typical examples of the change in permeability coefficient with time are shown in Figures 5.2 to 5.6. These results are discussed in more detail in Section 5.2.2 below.

5.2.2 <u>Evaluation of test</u>

The first series of tests was performed to evaluate the repeatability of the test method and the influence of such factors as support meshes, soil protection, initial soil moisture content, soil compaction and hydraulic gradient. For this purpose a total of 29 tests were performed using one soil (Warmbad soil, described in Section 5.3) and one geotextile (Geotextile No. 3) and by varying the factors mentioned above. Selected results from these tests are shown on Figures 5.2 to 5.6. The value K_{400}, shown with the results, is the permeability coefficient after 400 hours testing, as a percentage of the initial permeability coefficient at the start of the test. This parameter was used to quantify permeability reduction and is described in more detail in Section 5.4.

5.2.2.1 Repeatability

An indication of repeatability can be obtained from the results shown in Figures 5.2 and 5.3. The various parameters were identical for the three tests shown in each case. The variation in K_{400} resulted in a coefficient of variance (CoV) of 11,7 and 4,6 per cent for the two sets of tests shown on Figures 5.2 and 5.3 respectively.

5.2.2.2 Support mesh

Meshes with different opening sizes were used to support the geotextiles in these tests to determine whether this had an influence on results. Figure 5.4 shows the results obtained from tests where all parameters, except mesh size, were kept constant. It is evident that there was some difference in the initial behaviour of the systems. After 400 hours the difference was less marked and after 1200 hours there was virtually no difference between the three tests. The increase in permeability after 300 hours obtained with the 3,0 mm mesh was typical of all permeameters where a 3,0 mm mesh was used. This can probably be ascribed to fines being washed through the geotextile at that point, resulting in a higher flow.

5.2.2.3 Top mesh

In some tests a mesh was placed at the top of the permeameter to ensure an even flow of water so as not to disturb the surface of

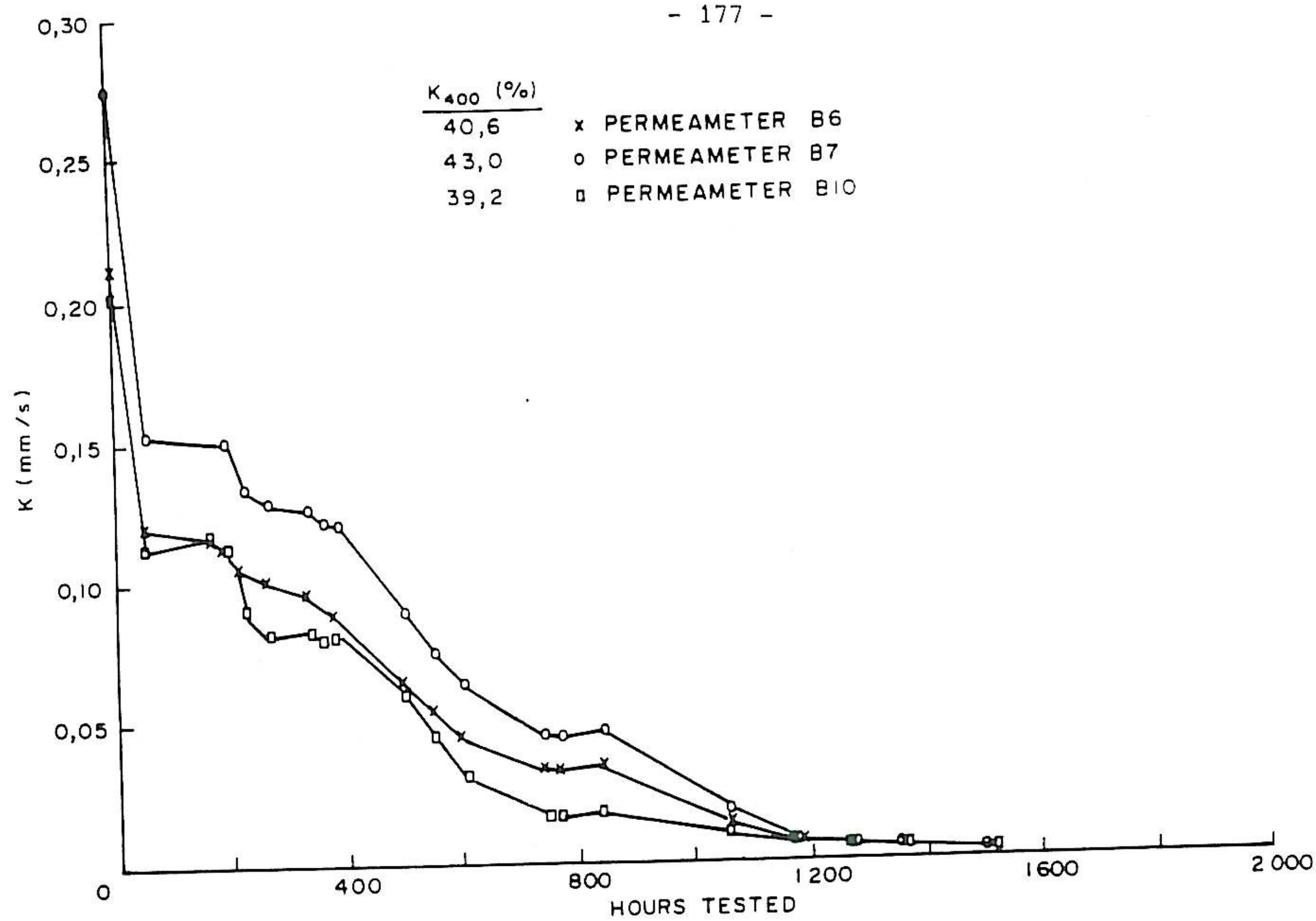


FIGURE 5.3 *LABORATORY FLOW TEST RESULTS: REPEATABILITY*

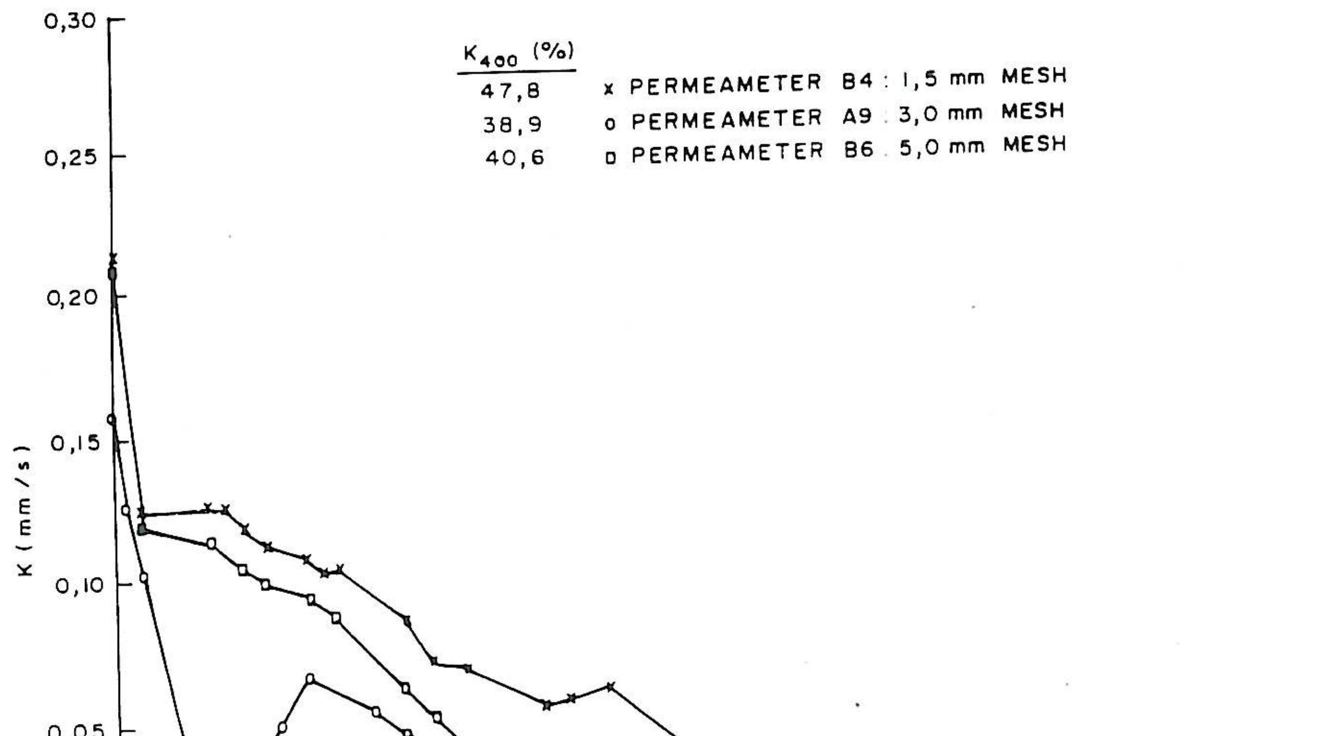


FIGURE 5.4 *LABORATORY FLOW TEST RESULTS: INFLUENCE OF SUPPORT MESH SIZE*

the soil sample. The mesh alone did not ensure a smooth sample surface and a small circular plate of galvanised iron was placed directly beneath the brass mesh. This resulted in a deposit forming on the surface of the soil sample, probably resulting from an electrolytic reaction between the brass and the galvanised iron. The result obtained from one of these permeameters (permeameter B9) is shown in Figure 5.5. The whole system was virtually clogged after 200 hours and K_{400} was 0,8%. This was true for each of the five tests that were carried out in this manner.

5.2.2.4 Soil protection

In an attempt to keep the surface smooth, a thin geotextile was placed on top of the soil sample in some tests. The results of two identical tests, one with and the other without the geotextile protection are shown in Figure 5.5. There was very little difference evident throughout the test.

5.2.2.5 Compaction, initial moisture and hydraulic gradient

Figure 5.6 shows the results obtained by varying the initial moisture and density of the soil samples. Samples were placed dry without compaction, at optimum moisture content (OMC) without compaction, and at OMC with compaction (87% Mod. AASHTO).

It is evident from Figure 5.6 that there was very little difference in the results obtained from the compacted and uncompacted samples. It was noted visually that in the test without initial moisture (A9), more fine material was washed through at the start of the test than was the case with the two moist samples. K_{400} was again similar for all three tests.

In a subsequent series of tests using a heavy clay (86% passing 0,075 mm sieve; PI = 64), it was observed that initial compaction of the sample had a fairly significant influence on the

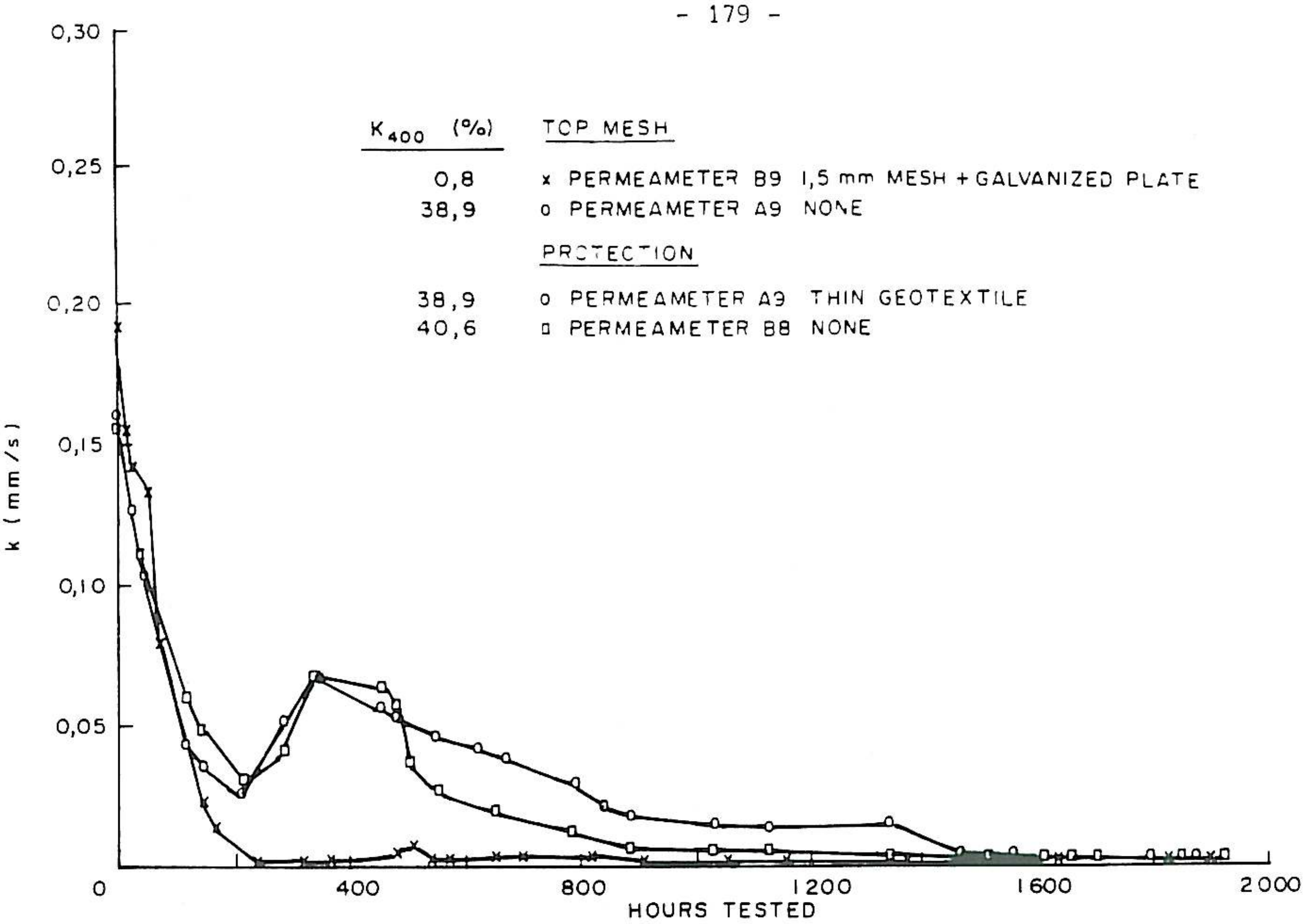


FIGURE 5.5 *LABORATORY FLOW TEST RESULTS: INFLUENCE OF TOP MESH AND SOIL PROTECTION*

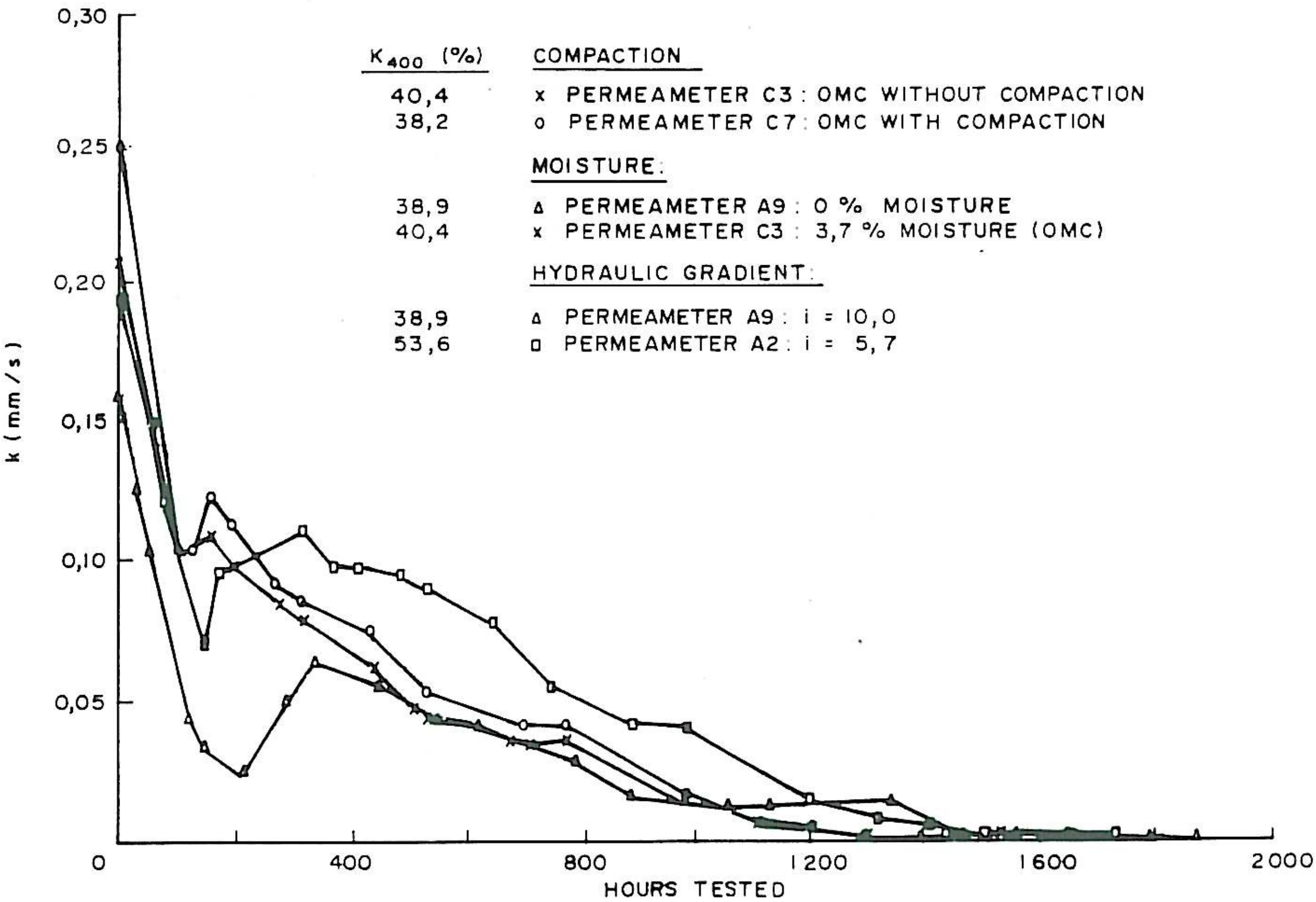


FIGURE 5.6 *LABORATORY FLOW TEST RESULTS: INFLUENCE OF MOISTURE, COMPACTION AND HYDRAULIC GRADIENT*

results. With all other soils tested, the water head was sufficient to compact the soil to at least 80 per cent Mod AASHTO. With the heavy clay, however, additional compaction was needed since the pressure exerted by the water resulted in only 60% Mod AASHTO. Flow tests with this low density soil sample resulted in a much lower rate of permeability reduction.

The influence of the hydraulic gradient was also investigated. The gradient was decreased by increasing the height of the soil sample (a larger soil sample). This resulted in a lower rate of permeability reduction (see Figure 5.6). For a hydraulic gradient i = 5,7, the resultant K_{400} was 53,6%, as opposed to 38,9% for i = 10,0.

5.2.3 Summary

From the investigation described above, it was concluded that

- The repeatability of the test method was good when all parameters were the same.

- Factors that influenced the results most significantly were hydraulic gradient and chemical clogging due to electrolytic reaction. The influence of other factors was relatively small.

Based on these findings and taking practical considerations into account, the following procedure was adopted as standard for subsequent tests:

Support mesh	:	3 mm brass mesh
Top mesh	:	None
Soil protection	:	None, but initial filling of permeameter was done with a small diameter pipe and spray nozzle to minimise disturbance of the surface.
Initial moisture	:	OMC

Compaction	:	None, unless compaction under water pressure was less than 80% Mod AASHTO, e.g. with heavy clays.
Soil sample size	:	1,0 kg dry mass.
Water head	:	1000 ± 25 mm above geotextile.

5.3 MATERIALS TESTED

Various soils, geotextiles and granular filters were used in the investigation. The materials used are described in this section.

5.3.1 Soil characteristics

A total of 19 soil types were used in the tests, of which 14 were gradings made up in the laboratory by mixing different proportions of grain sizes. A description of the soils is given in Table 5.1 and details of the gradings and other characteristics in Table 5.2.

Table 5.1 Soils used in the investigation

Abbreviation	Name, Origin	Description
W	Warmbad	Medium grained sand
S	Silverton test site	Red silty sand
B	Du Plessis building sand	Uniformly graded sand
R	Stocks river sand	Washed river sand
M	Montana, Pretoria	Red medium clay
S1-S5	Silverton	Gradings made up from Silverton soil
W1-W8	Warmbad	Gradings made up from Warmbad soil

The parameters given in Table 5.2 to define soil characteristics are as follows:

Table 5.2 Soil characteristics.

Soil:	W	S	B	R	M	S1	S2	S3	S4	S5	W1	W2	W3	W4	W5	W6	W7	W8
Grading:																		
% passing sieve (mm)																		
6.70	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
4.75	100.0	100.0	100.0	96.9	98.7	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0	100.0
2.00	100.0	99.9	99.7	78.5	97.0	99.8	99.8	99.9	99.9	99.9	99.9	100.0	100.0	100.0	100.0	100.0	100.0	100.0
0.850	99.8	99.2	96.3	45.9	95.7	98.9	98.5	99.4	99.4	99.4	99.8	99.8	99.9	99.8	99.7	99.6	99.7	99.7
0.425	89.8	86.8	77.2	24.3	90.4	82.6	74.6	90.9	90.5	90.8	91.9	91.8	93.8	87.8	86.8	80.8	82.8	86.8
0.250	47.7	63.8	49.0	15.2	82.1	55.4	39.4	72.0	71.2	51.8	51.9	51.7	55.7	43.7	41.7	74.7	68.7	56.7
0.150	19.1	53.8	27.7	9.4	73.8	41.3	17.3	66.1	64.9	45.8	25.4	25.1	31.1	13.1	25.1	37.1	33.1	25.1
0.075	6.4	48.6	14.8	0.0	66.6	32.0	0.0	65.0	44.9	44.6	14.8	4.4	2.4	8.4	9.4	15.4	13.4	9.4
0.060	2.6	42.9	14.1	0.0	49.9	28.3	0.0	57.4	39.6	39.4	6.0	1.8	1.0	3.4	3.8	6.3	5.4	3.8
0.020	0.6	36.2	6.9	0.0	47.1	23.9	0.0	48.4	33.4	33.2	1.4	0.4	0.3	0.8	0.9	1.5	1.2	0.9
0.006	0.2	32.9	4.7	0.0	44.3	21.7	0.0	44.0	30.4	30.2	0.5	0.1	0.1	0.3	0.3	0.5	0.4	0.3
0.002	0.0	28.7	3.2	0.0	41.1	18.9	0.0	38.4	26.5	26.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
<0.002	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Grain sizes:																		
D85	0.405	0.411	0.599	2.971	0.311	0.488	0.610	0.370	0.375	0.399	0.395	0.395	0.385	0.414	0.418	0.520	0.480	0.414
D60	0.301	0.212	0.318	1.347	0.069	0.280	0.352	0.065	0.132	0.287	0.285	0.286	0.270	0.315	0.321	0.211	0.22669	0.269
D50	0.260	0.095	0.256	0.995	0.060	0.212	0.303	0.027	0.094	0.220	0.243	0.244	0.227	0.275	0.282	0.184	0.197	0.229
D15	0.1258	0.0010	0.0762	0.2466	0.0007	0.0016	0.1400	0.0008	0.0011	0.0011	0.0764	0.1134	0.107	0.1562	0.1017	0.0743	0.0811	0.1017
D10	0.09630	0.00070	0.03722	0.16034	0.00049	0.00106	0.11835	0.00052	0.00075	0.00076	0.06682	0.09529	0.09486	0.10053	0.07787	0.06610	0.06862	0.0778
Grading parameters:																		
U	3.13	302.86	8.59	8.42	141.80	254.54	2.98	124.80	174.90	377.40	4.26	3.00	2.85	3.13	4.12	3.19	3.29	3.45
GM	1.038	0.647	1.083	1.972	0.460	0.854	1.254	0.441	0.646	0.646	0.933	1.038	1.038	1.038	1.038	1.038	1.038	1.038
FM	6.338	4.072	6.064	8.267	3.120	4.970	6.702	3.184	3.992	4.385	6.038	6.249	6.157	6.427	6.323	5.841	5.953	6.173
SD	1.139	3.143	2.020	1.511	3.104	2.908	1.060	3.115	2.922	3.143	1.367	1.089	1.036	1.180	1.305	1.457	1.402	1.276
Other parameters:																		
PI	non pl.	17.7	non pl.	non pl.	31.9													
OMC	3.7	13.4	8.0	9.4	10.5													
MDD	1762	1855	1986	1825	2004													

(i) Grading:

The values shown are the cumulative percentages passing.

(ii) Grain sizes:

D_{85} is the diameter of a soil particle where 85% of all other particles are smaller. The same for D_{60}, D_{50}, etc.

(iii) Uniformity coefficient (U):

$$U = \frac{D_{60}}{D_{10}}$$

where D_{60} and D_{10} are the particle diameters at 60% and 10% respectively of the particle size distribution.

The uniformity coefficient gives an indication of the particle size distribution and higher values indicate a distribution over a wider range of particle sizes.

(iv) Grading Modulus (GM):

Grading modulus is an indication of the particle size distribution for particle sizes of 2 mm and smaller and is given by

$$GM = \frac{R_{2,0} + R_{0,425} + R_{0,075}}{100}$$

where R_d = total percentage material retained on sieve with opening d mm.

(v) Fineness Modulus (FM) and Standard Deviation (SD):

These are parameters used in concrete mix design (Fulton, 1977). The fineness modulus is a measure of the average size of the particles and the standard deviation gives the distri–

bution of the particle sizes or the amount of deviation from the fineness modulus. A low SD indicates that a higher percentage of particles is close to the average size and a higher SD indicates a wider distribution.

The average size of the particles is measured in terms of sieve increment numbers and not in actual sieve sizes. The sieves are numbered, starting at 1 for the smallest sieve. It should be noted that this parameter can only be used for a particular set of sieves where the sieve sizes increase geometrically or to a fixed rule. Furthermore, the fineness modulus will differ from one set of sieves to another.

The parameters are defined as follows:

$$FM = \frac{\sum(P_{dd'} . n)}{100}$$

$$SD = \sqrt{\frac{\sum(P_{dd'} (n - FM)^2)}{100}}$$

Where $P_{dd'}$ = Percentage retained between two sieves with opening d and d'

n = incremental sieve number.

(vi) PI : Plasticity index.

(vii) OMC : Optimum moisture content at Mod AASHTO compaction.

(viii) MDD : Maximum dry density at Mod AASHTO compaction.

5.3.2 Geotextiles

Thirty different geotextiles were used in combination with the soils described above. In-isolation geotextile parameters that were evaluated were normal permeability and opening sizes, the values of which were given in Sections 4.2.1 and 4.2.2 respectively. A description of the geotextiles was given in Section 3.2.5.

5.3.3 Granular filters

In addition to the geotextiles, permeameter flow tests were also performed on a number of granular (natural) filters. These were designed according to the filter criteria which were discussed earlier (Section 3.3.1). The granular filters were made up in the laboratory, using different fractions of Pretoria quartzite crushed stone.

The following is an example of how one such a filter was designed.

The grading of the Warmbad soil given in Table 5.2 is shown on Figure 5.7. The Terzaghi criteria were applied as follows:

Piping criterion : $D_{15}f \leq 5 \times D_{85}S$
$\leq 5 \times 0{,}405$
$\leq 2{,}025$

Permeability criterion : $D_{15}f \geq 5 \times D_{15}S$
$\geq 5 \times 0{,}1258$
$\geq 0{,}629$

Compatibility criterion : $D_{50}f \leq 25 \times D_{50}S$
$\leq 25 \times 0{,}260$
$\leq 6{,}50$

The grading of a filter that conforms to these criteria is shown on Figure 5.7 (Filter 1).

The granular filters were placed in the bottom half of the permeameters. The outlet from the permeameter was too large

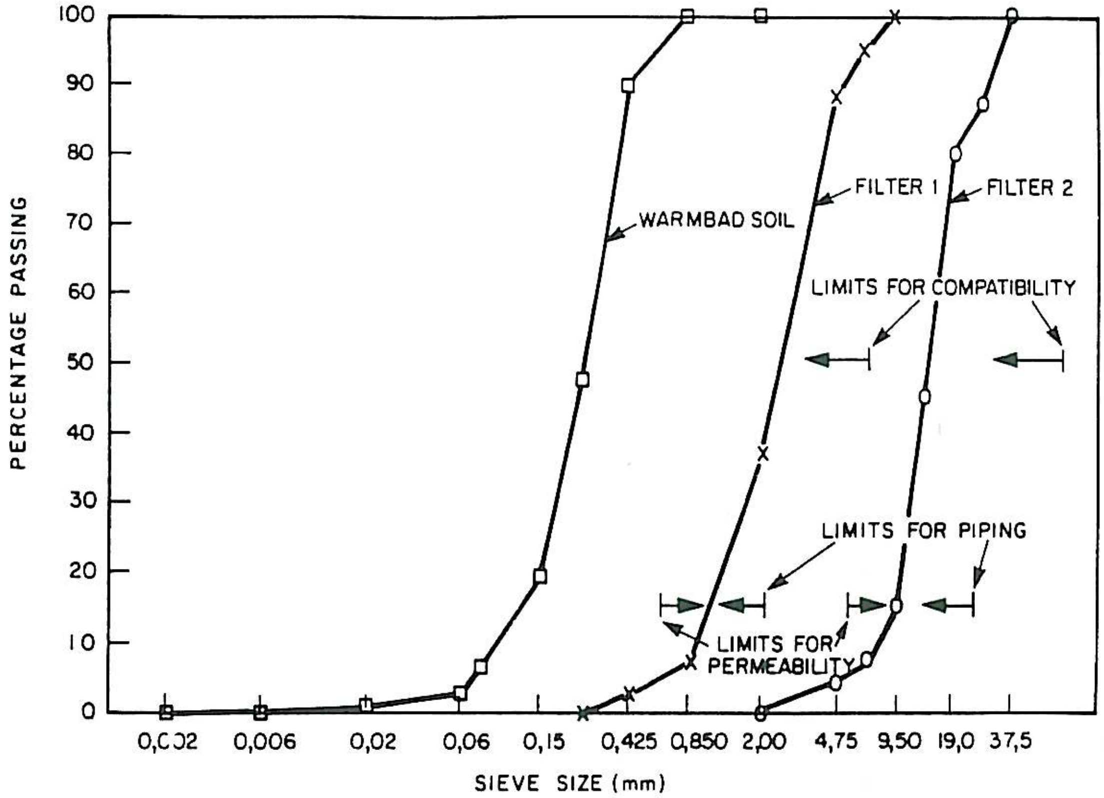


FIGURE 5.7

GRADINGS OF GRANULAR FILTERS : WARMBAD SOIL

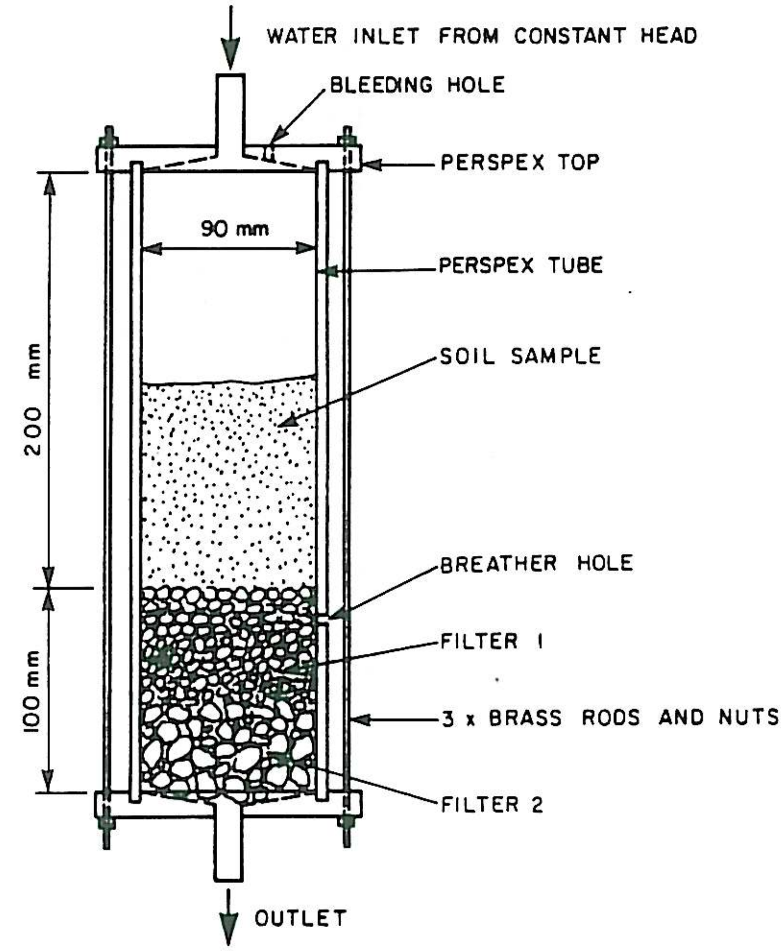


FIGURE 5.8

SOIL/GRANULAR FILTER FLOW TESTS

(10 mm) to hold the filter described above. A second filter was designed (Filter 2), using the same criteria, and assuming Filter 1 to be the base soil. Filter 2 was placed at the bottom of the permeameter, followed by Filter 1 and the soil on top of that (without any geotextile). This configuration is illustrated in Figure 5.8.

In addition to the Terzaghi filter criteria described above, the filters were also evaluated for internal stability. The evaluation of Filter 1 described above, together with its grading, is shown on Table 5.3.

Table 5.3 Filter 1 (Warmbad soil) - evaluation of internal stability.

Sieve (mm)	% passing	Y (%)	Dy = D_{85F} (mm)	Y + 15 (%)	$D_{(y+15)}$ = D_{15C} (mm)	I_R = D_{15C}/D_{85F}	x = Y/0,85
9,5	100	5	0,661	20	1,348	2,04	5,88
6,7	95	10	0,965	25	1,540	1,59	11,76
4,75	88	15	1,157	30	1,732	1,50	17,65
2,00	37	20	1,348	35	1,923	1,43	23,53
0,85	7,0	30	1,732	45	2,431	1,40	35,29
0,425	2,5	40	2,162	55	2,971	1,37	47,06
0,250	0	50	2,701	65	3,510	1,30	58,82
		60	3,240	75	4,049	1,25	70,59
		70	3,779	85	4,588	1,21	82,35
		80	4,319	95	6,700	1,55	94,12

The method is explained in more detail in Section 3.3.1.2. The whole grading is divided into small sections, each of which is divided into a course and a fine fraction and D_{15C}/D_{85F} is calculated for each section where :

$D_{15C} = D_{15}$ of the course fraction

$D_{85F} = D_{85}$ of the fine fraction.

In the example D_{85F} is calculated at each of the Y percentages (5, 10, 15, etc.) by interpolation, e.g. :

$$D(5) = D_{85F}(5) = \frac{5 - 2,5}{7,0 - 2,5} \times (0,85 - 0,425) + 0,425$$
$$= 0,661$$

Similarly D_{15C} is calculated at each of the Y + 15 percentages (20, 25, 30, etc.) as follows:

$$D(5 + 15) = D_{15C}\ (20) = \frac{(20 - 7,0)}{(37 - 7,0)} \times (2,00 - 0,85) + 0,85$$
$$= 1,348$$

The internal stability ratio (I_R) can then be obtained:

$$I_R = D_{15C}/D_{85F} = 2,04$$

The filter is internally stable if $I_R < 5$ throughout the grading, as is the case with Filter 1 shown on Table 5.3.

The grading of each of the granular filters used is shown on Table 5.4. Each filter was designed to meet all the criteria described above, i.e. piping, permeability, compatibility and internal stability.

Table 5.4 <u>Summary of granular filters used</u>

Sieve Size (mm)		Percentage Passing							
	Soil:	W		S		B		R	
	Filter:	F1	F2	F1	F2	F1	F2	F1	F2
37,5			100						Not required
26,5			87,0						
19,0			80,0				100	100	
13,2			45,0		100		60,0	85,0	
9,5		100	15,0		85,0		15,0	70,0	
6,7		95,0	7,0		75,0	100	0,0	50,0	
4,75		88,0	4,0		65,0	95,0		30,0	
2,00		37,0	0,0	100	30,0	60,0		0,0	
0,85		7,0		70,0	0,0	10,0			
0,425		2,5		50,0		0,0			
0,250		0,0		35,0					
0,150				25,0					
0,075				0,0					

5.4 FLOW TEST RESULTS

5.4.1 Results obtained from permeameters

An example of the results obtained from the flow tests is shown in Table 5.5. The values shown were obtained as follows:

(i) Hours tested: Total time elapsed from the start of the test.

(ii) Flow time: Time in minutes and seconds for a measured volume of water to flow through the system.

(iii) Sample height: Height of the soil sample in the permeameter (in mm).

(iv) Volume: Volume of water that flowed through the system in a measured time period. This volume was usually 500 ml, unless it took longer than 30 minutes, in which case the volume was measured after approximately 30 minutes.

Table 5.5 <u>Example of flow test results</u>
(Soil: W, geotextile 1)

HOURS TESTED	FLOW: MINS	FLOW: SECS	SAMPLE HEIGHT	VOLUME	HEAD DIF.	K (mm/s)	SLOPE (mm/s/h)	% OF ORIGINAL	LOG K	FLOW (ml/s)	TOTAL FLOW (m^3/m^2)
0.12	7	50	100	5000	9	0.1657	0.000000	100.000	-0.781	10.638	0.70
19.87	10	47	99	5000	6	0.1195	0.004156	72.131	-0.922	7.728	113.29
26.17	12	27	99	5000	1	0.1041	0.002458	62.787	-0.983	6.693	139.00
92.42	30	31	99	5000	11	0.0420	0.000936	25.362	-1.376	2.731	315.65
121.70	25	33	98	5000	-5	0.0505	-0.000289	30.468	-1.297	3.262	365.30
144.03	18	16	98	5000	-3	0.0705	-0.000895	42.531	-1.152	4.562	414.74
168.33	15	10	98	5000	0	0.0846	-0.000582	51.071	-1.072	5.495	483.88
192.37	17	0	99	5000	0	0.0763	0.000348	46.028	-1.118	4.902	554.58
266.72	39	20	99	5000	-2	0.0330	0.000582	19.933	-1.481	2.119	702.27
288.58	32	58	98	5000	0	0.0389	-0.000270	23.496	-1.410	2.528	731.02
359.13	31	57	98	5000	3	0.0401	-0.000016	24.171	-1.397	2.608	833.54
431.27	30	59	98	2205	8	0.0181	0.000304	10.937	-1.742	1.186	910.98
457.78	30	7	98	1800	6	0.0153	0.000108	9.204	-1.817	0.996	927.35
525.20	31	59	98	1500	9	0.0119	0.000049	7.201	-1.923	0.782	961.27
601.70	37	6	98	2040	11	0.0140	-0.000027	8.426	-1.855	0.916	998.02
668.55	30	5	98	1660	9	0.0140	-0.000001	8.472	-1.853	0.920	1032.75
697.92	30	51	98	1620	6	0.0134	0.000022	8.086	-1.873	0.875	1047.66
764.97	34	1	98	1480	11	0.0110	0.000035	6.667	-1.957	0.725	1078.02
815.22	31	27	98	1120	7	0.0091	0.000039	5.479	-2.042	0.594	1096.77
861.02	31	27	98	840	12	0.0068	0.000050	4.089	-2.169	0.445	1110.23
934.90	32	23	98	500	12	0.0039	0.000039	2.364	-2.407	0.257	1124.92
985.78	30	1	98	390	14	0.0033	0.000012	1.985	-2.483	0.217	1131.74
1028.82	34	24	98	290	18	0.0021	0.000027	1.283	-2.672	0.141	1136.09
1103.65	30	0	98	160	16	0.0013	0.000010	0.813	-2.870	0.089	1140.95
1148.72	30	2	98	140	22	0.0012	0.000004	0.707	-2.931	0.078	1143.07
1198.82	30	0	98	105	18	0.0009	0.000006	0.533	-3.054	0.058	1145.00
1269.42	33	12	98	85	14	0.0006	0.000003	0.391	-3.188	0.043	1147.02
1316.53	30	1	98	69	19	0.0006	0.000001	0.349	-3.237	0.038	1148.10
1604.58	30	0	98	35	14	0.0003	0.000001	0.178	-3.530	0.019	1152.80
1844.82	31	43	98	28	16	0.0002	0.000000	0.135	-3.652	0.015	1155.12
1942.77	32	34	98	28	25	0.0002	0.000000	0.130	-3.667	0.014	1155.93

(v) Head difference: Head difference, i.e. the difference between actual water head 1000 mm.

(vi) k: Permeability coefficient (in mm/s) of the soil-geotextile system calculated using Darcy's law of seepage, e.g. for the first value in Table 5.5:

$k = Q/iA$

where Q = volume of flow

= 5 000 ml in 7 min 50 sec

$= (5\ 000 \times 10^3)/470 \quad mm^3/S$

i = hydraulic gradient

$$= \frac{\text{water head (above bottom of geotextile)}}{\text{length of flow path (sample height)}}$$

$$= \frac{1\ 000 \text{ mm + head difference on stand pipe}}{100 \text{ mm}}$$

$= (1\ 000 + 9)/100$ (dimensionless)

A = Cross section area

$= (\pi \times d^2)/4$

$= \pi \times (90)^2/4 \quad mm^2$

By substituting these values in Darcy's equation then

$k = 0{,}1657$ mm/sec.

(vii) Slope: The slope (in mm/s/h) is the rate at which the permeability decreased (positive) or increased (negative) between two measurements, e.g. the second value in Table 5.5:

$$\text{Slope} = \frac{\text{Second value of k - first value of k}}{\text{Time difference between two measurements}}$$

$$= (0,1657 - 0,1863)/(3,80 - 0,12)$$

$$= 0,005588$$

(viii) % of original: The value of k expressed as a percentage of the first value of k that was determined, e.g. the second value in Table 5.5:

$$\% \text{ of original} = (0,1863)/(0,1657) \times 100$$

$$= 112,419 \%$$

(ix) Flow: Flow rate in ml/s at the time of measurement, e.g. the first value in Table 5.5:

Volume = 5000 ml
Time = 7 min 50 sec.

Flow = (5000)/470 ml/sec

= 10,638 ml/sec.

(x) Total flow: Total amount of water per unit area that flowed through the system from the start of the test.

A linear decrease (or increase) in flow is assumed, e.g. the second value in Table 5.5:

Average flow between 0,12 and 3,80 hours

$$= \frac{11,876 + 10,638}{2} = 11,257 \text{ ml/s}$$

Total flow between 0,12 and 3,80 hours

$$= 11,257 \times (3,80 - 0,12) \times 3600$$
$$= 149133 \text{ ml}$$
$$= 0,149133 \text{ m}^3$$
$$= 23,47 \text{ m}^3/\text{m}^2 \quad (A = 0,00636 \text{ m}^2)$$

Total flow from start of test $= 23,47 + 0,70$
$= 24,17 \text{ m}^3/\text{m}^2$.

Figure 5.9 shows, for the example in Table 5.5 (Soil W and geotextile 1), the change in permeability coefficient K, with time. On this figure, both time and K are on a linear scale. Figure 5.10 shows the same results with K on a logarithmic scale (log K from Table 5.5). It is practical, where results from various soil types are shown on the same figure, to use a logarithmic scale (e.g. Figure 3.12, Section 3.4.1.7). It can be misleading, however, as is evident from these results. On the linear scale (Figure 5.9), the flow seems to have stabilised after about 400 hours, whereas on the log scale (Figure 5.10) there is still a decrease after 1600 hours.

5.4.2 Parameters to quantify flow test results

A total of 85 sets of results, similar to those shown in Figure 5.9, were obtained, using various combinations of soils and geotextiles. To evaluate these and to obtain relationships between flow test results and in-isolation properties of soils and geotextiles, parameters had to be defined to quantify the flow tests results. The following possibilities existed:

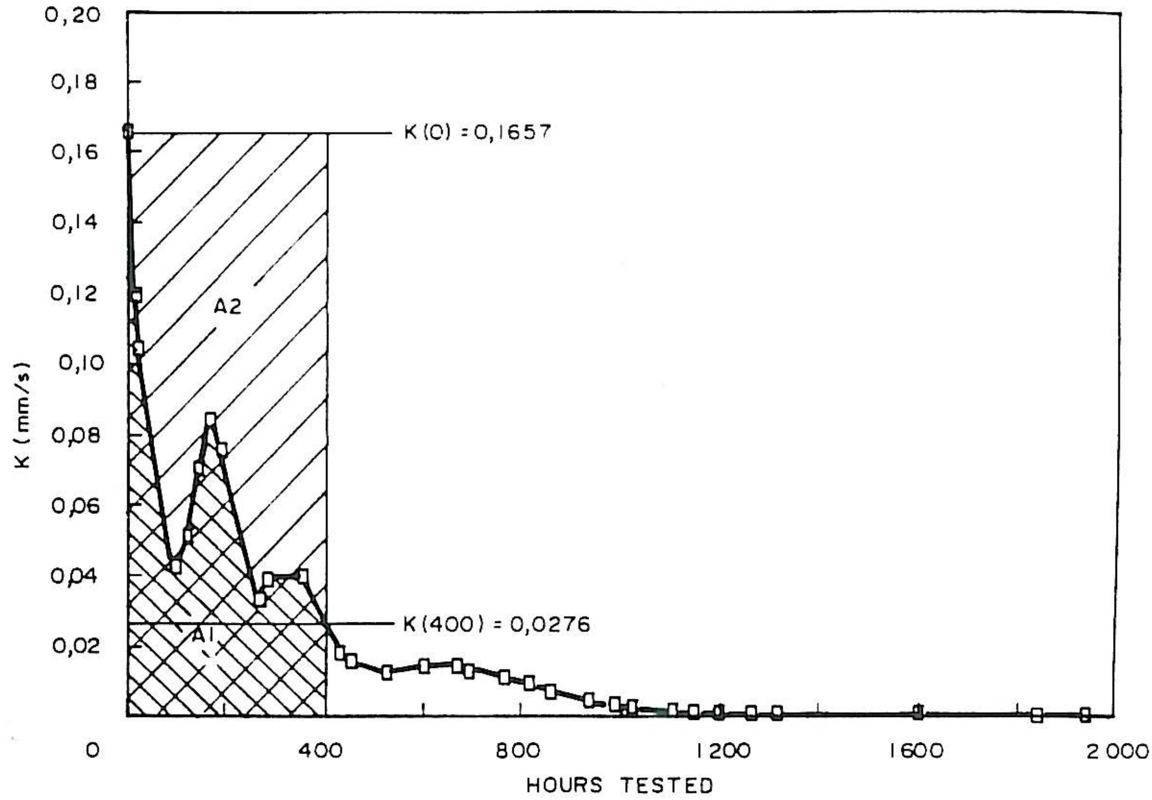


FIGURE 5.9

EXAMPLE OF FLOW TEST RESULTS (SOIL: W, GEOTEXTILE 1)

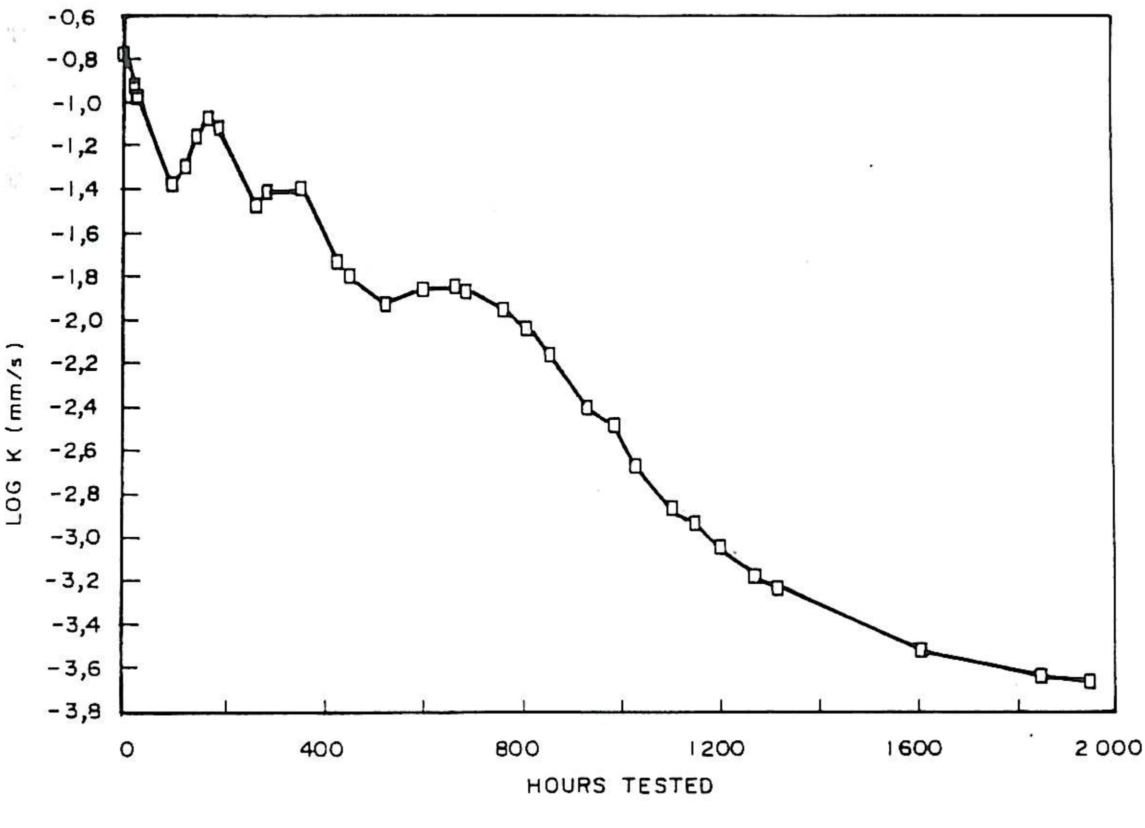


FIGURE 5.10

FLOW TEST RESULTS, LOG SCALE (SOIL: W, GEOTEXTILE 1)

- Total amount of water that flowed through the system after a certain length of time.
- The permeability coefficient of the soil/geotextile system after a certain length of time.
- The time taken to reach stability and the properties of the system at that point.

The magnitude of K and the value of water that flowed through the system was largely determined by the soil type. To evaluate the permeability reduction characteristics, the parameters used had to be expressed as a percentage of the original values at the start of the test to allow comparisons between different soil types.

The field investigation (Chapter 6) indicated that 400 hours of testing were equivalent to approximately 100 years of service in the field. On that basis, 400 hours was chosen as one position at which to evaluate the results. More critical field conditions with higher hydraulic gradients are possible, however, and for that reason the results were also evaluated at 1000 hours.

Based on these factors, the following parameters were defined:

(i) F_{400} = total amount of water that flowed through the system after 400 hours, expressed as a percentage of the total amount of water that would have flowed through the system after 400 hours if the permeability had remained constant at the original value.

$$= \frac{\text{Area under the curve (Area A1)} \times 100}{\text{Area A2}} \quad \text{(See Figure 5.9)}$$

F_{400} can be calculated for the example shown from values given in Table 5.5. The initial flow was 10,638 ml/s.

$$\text{Then A2} = \frac{10,638 \times 3600 \times 400}{1000 \times 1000}$$

$$= 15,3187 \text{ m}^3$$

$$= 2408,60 \text{ m}^3/\text{m}^2 \quad (A = 0,00636 \text{ m}^2)$$

The total flow of water after 400 hours (Area A1) can be obtained by linear interpolation from the last column in Table 5.5:

$A1 = 877,41 \text{ m}^3/\text{m}^2$.

$$\text{Then } F_{400} = \frac{877,41}{2408,60} \times 100$$

$$= 36,4\%$$

(ii) K_{400} = Permeability coefficient after 400 hours of testing

expressed as a percentage of the permeability coefficient at the start of the test.

$$= \frac{K(400) \times 100}{K(0)} \quad \text{(See Figure 5.9)}$$

For this example, by linear interpolation from Table 5.5 :

$$K_{400} = \frac{0,0276 \times 100}{0,1657}$$

$$= 16,66\%.$$

(iii) F_{1000} and K_{1000} : As for F_{400} and K_{400} defined above, but after 1000 hours testing.

(iv) S = Testing time in hours at which the slope became 0,0001 mm/s/h, and remained less than 0,0001 mm/s/h for the duration of the test.

For the example shown, from Table 5.5, by linear interpolation:

S = 466,9 hours.

(v) F_S and K_S : As for F_{400} and K_{400} defined above, but after S hours testing.

Each of the parameters defined above was calculated for the 85 tests. A summary of the results is shown in Table 5.6. Abbreviations and numbers are given for the soils and geotextiles used. Descriptions of the soils are given in Tables 5.1 and 5.2 and the geotextiles are described in Sections 3.2.5, 4.2.1 and 4.2.2. A letter "G" under geotextiles denotes a granular filter, details of which are given in Table 5.4.

5.5 EVALUATION OF PARAMETERS QUANTIFYING FLOW TEST RESULTS

The parameters defined in Section 5.4.2 above were evaluated to select a parameter (or parameters) for further evaluation of the data against in-isolation characteristics and, ultimately, to serve as a parameter for design criteria. The parameters were also evaluated to determine whether any relationship existed, e.g. between results after 400 hours and results after 1000 hours.

Table 5.6 Summary of flow test results

Soil	Geotextile	400 hours		1000 hours		Slope = 0,0001		
		F_{400} (%)	K_{400} (%)	F_{1000} (%)	K_{1000} (%)	S (hrs)	F_s (%)	K_s (%)
W	I	36,4	16,67	18.8	1,75	466,9	33,1	8,93
W	2	39.0	18.12	20.2	2.24	456.5	36.1	10.24
W	3	30.9	13.51	15.7	1.52	321.8	34.1	20.55
W	4	38.4	17.19	19.1	1.51	451.4	35.7	9.02
W	5	33.2	21.24	17.0	2.12	477.0	29.6	10.20
W	6	39.4	17.10	20.2	1.96	518.3	33.1	8.42
W	11	38.2	17.66	19.6	2.08	532.9	31.5	8.37
W	13	40.4	18.23	20.7	2.65	528.0	33.5	8.69
W	14	46.9	21.17	24.7	3.49	513.1	39.8	9.93
W	15	55.9	28.35	30.2	3.32	507.3	48.6	14.94
W	16	30.6	13.80	16.2	1.31	462.0	28.1	10.26
W	17	38.9	16.48	19.8	1.95	486.1	34.4	10.60
W	18	38.0	15.53	19.6	1.87	479.7	33.9	10.61
W	19	34.9	14.91	17.7	1.48	476.8	31.2	10.08
W	20	21.1	7.07	10.4	0.56	297.4	25.4	8.25
W	21	18.9	5.44	9.0	0.66	295.2	23.2	6.81
W	22	35.0	14.27	18.0	1.29	474.2	31.6	9.98
W	23	36.1	14.23	18.1	1.30	482.6	32.0	9.63
W	24	36.6	15.11	18.6	1.59	477.7	32.7	10.70
W	25	36.8	16.56	18.3	1.13	498.0	32.2	9.59
W	26	29.3	10.60	14.6	1.19	475.2	26.2	7.73
W	27	29.5	10.73	15.0	1.15	458.1	27.0	8.61
W	28	21.7	6.78	10.6	0.64	302.0	26.1	7.99
W	29	19.9	0.78	8.4	0.31	300.9	26.1	1.79
W	30	22.5	3.63	10.2	0.64	297.6	29.1	3.25
W	31	21.2	0.34	8.6	0.07	291.3	28.8	1.13
W	32	30.4	11.14	15.3	0.88	455.6	27.9	9.07
W	33	30.6	12.66	15.5	1.22	481.7	27.3	8.91
W	G	25.4	4.30	11.4	0.00	384.1	26.3	4.50
S	1	45.2	6.63	21.6	2.60	341.0	51.3	13.08
S	5	77.8	13.10	37.1	9.44	354.3	85.8	18.70
S	20	71.4	7.90	33.9	7.20	351.6	80.1	9.52
S	21	41.6	7.80	19.5	3.98	367.8	44.4	10.50
S	24	53.2	7.40	25.0	5.41	350.3	59.6	8.50
S	26	39.3	3.27	17.4	1.84	349.1	44.6	3.04
S	27	49.7	2.70	21.9	2.05	351.6	56.0	4.32
S	29	37.5	0.63	15.2	0.23	347.5	43.0	1.26
S	32	40.1	6.23	18.8	3.85	359.5	43.8	9.27
S	G	64.7	17.70	32.0	6.27	368.2	68.4	23.12

Table 5.6 Summary of flow test results, continued

Soil	Geotextile	400 hours		1000 hours		Slope = 0,0001		
		F_{400} (%)	K_{400} (%)	F_{1000} (%)	K_{1000} (%)	S (hrs)	F_s (%)	K_s (%)
B	1	60.8	5.30	25.7	1.65	342.0	69.9	9.69
B	5	51.8	5.60	22.4	2.10	339.7	59.8	9.79
B	20	75.2	7.65	32.2	2.47	348.7	84.8	13.48
B	21	55.2	3.52	23.3	1.73	338.9	64.3	6.64
B	24	51.2	8.55	23.3	3.67	340.0	58.3	15.74
B	26	60.3	7.36	26.2	2.58	346.0	68.2	14.33
B	27	48.4	1.74	20.0	0.95	322.6	59.1	7.43
B	29	64.0	5.78	27.2	1.93	347.2	72.5	13.08
B	32	46.4	5.82	20.3	2.29	345.5	52.5	10.56
B	G	52.3	4.80	22.2	1.75	349.9	59.0	8.77
R	1	76.5	46.15	36.2	1.66	853.6	42.1	2.12
R	5	62.5	32.56	29.4	1.07	887.9	33.0	1.01
R	20	37.1	7.01	16.5	0.68	848.6	19.3	0.83
R	21	55.6	28.12	25.8	1.62	818.3	31.1	1.25
R	24	69.1	40.98	32.2	1.60	837.8	38.2	1.84
R	26	63.6	29.20	29.1	1.64	779.8	36.8	2.10
R	27	48.9	20.70	22.7	1.73	750.4	29.7	2.88
R	32	65.8	40.36	32.2	1.94	940.0	34.1	2.32
R	G	61.8	33.19	28.6	1.15	828.6	34.2	1.83
S1	1	201.6	10.99	88.6	12.17	342.6	233.4	17.54
S2	1	93.9	76.00	59.9	7.39	813.3	70.9	13.45
S3	1	107.1	3.67	44.5	3.07	355.5	120.0	4.67
S4	5	46.7	2.96	20.1	2.53	283.6	64.3	7.18
S5	5	55.8	3.00	24.4	3.76	89.6	85.6	128.99
W1	6	55.2	29.27	29.6	1.64	144.7	79.8	42.95
W2	6	59.8	44.20	35.5	2.62	620.4	50.7	19.90
W3	6	54.3	35.21	31.8	2.14	609.7	45.7	18.63
W4	28	57.2	32.03	31.8	3.39	565.1	48.2	16.44
W5	28	72.6	34.29	39.5	3.58	152.9	107.6	56.98
W6	28	143.6	12.58	60.6	2.49	411.6	139.9	11.43
W7	28	130.1	29.70	59.6	3.05	393.5	131.8	31.31
W8	28	98.1	56.55	53.9	5.01	551.7	84.0	30.10
M	1	36.6	2.14	15.4	1.11	295.1	48.6	3.37
M	2	52.2	2.63	21.9	1.22	302.0	68.1	3.14
M	5	25.1	2.45	11.0	2.83	286.7	33.6	4.64
M	13	41.5	12.47	21.7	4.77	304.5	50.1	14.38
M	14	42.5	1.62	17.9	2.24	303.6	55.3	2.64
M	18	29.3	2.56	13.1	1.73	186.0	50.0	25.52
M	21	36.7	2.77	16.1	3.60	297.8	48.1	3.36
M	24	36.8	2.32	15.5	1.76	297.2	48.4	4.01
M	26	28.0	2.56	12.3	1.37	272.3	39.5	3.79
M	27	48.4	4.80	21.0	3.24	295.6	62.6	9.21
M	29	44.4	1.25	18.3	0.63	308.7	56.8	2.83
M	31	52.4	1.28	21.6	1.69	311.7	66.4	2.96
M	32	51.3	11.33	25.9	5.45	311.4	62.1	13.48
M	33	56.4	9.98	25.9	4.41	318.5	67.7	12.02

5.5.1 Variation in identical tests

Table 5.7 below shows the results from the two sets of identical tests performed to evaluate the repeatability of the test method, which was discussed earlier in Section 5.2.2.1.

Table 5.7 Results from repeatability tests

Permeameter	F_{400} (%)	K_{400} (%)	F_{1000} (%)	K_{1000} (%)	S (hours)	F_s (%)	K_s (%)
A1	19.7	24,3	13,4	1,26	904,1	14,7	2,28
B5	18,5	20,1	13,6	2,23	892,9	14,9	3,93
C8	21,2	25,1	16,4	3,54	671,6	20,8	11,32
Average	19,8	23,2	14,5	2,34	822,9	16,8	5,84
CoV (%)	6,8	11,7	11,6	48,8	15,9	20,6	82,40
B6	55,0	40,6	35,1	9,37	720,1	43,1	16,73
B7	57,4	43,0	37,2	9,75	1148,0	33,5	4,04
B10	54,8	39,2	31,6	5,97	743,4	39,8	8,07
Average	55,7	40,9	34,6	8,36	870,5	38,8	9,61
CoV (%)	2,6	4,7	8,2	24,9	27,6	12,6	67,4

For each parameter the average result from the three tests and the coefficient of variance (CoV) is shown, the latter giving an indication of the variation of the particular parameter for three tests performed in an identical manner.

The values of CoV in Table 5.7 indicate that a larger degree of variation occurred with the first set of tests (permeameters A1, B5, C8) than with the second set. The comparisons between values of CoV obtained with the various parameters described below were therefore made for each set individually.

It is evident that the smallest variation, with both sets of results, occurred with the 400 hour parameters, i.e. F_{400} and K_{400}. At 1000 hours the variation in flow (F_{1000}) was not large but the permeability (K_{1000}) resulted in a large variation. Although the values of K_{1000} did not differ much in magnitude (only 2 per cent difference between the smallest and largest values), these differences were large in relation to the actual values, resulting in a high degree of variation. The values obtained with K_{1000} are therefore not sensitive enough to quantify flow test results.

The time at which stability was reached (slope $\leq$ 0,0001 mm/s/h), i.e. S hours, and the values of F_s and K_s also resulted in high degrees of variation; the highest variation occurring with K_s. This is due to the fact that it was difficult to determine the exact time at which stability occurred. In many of the results the slope increased to above the maximum value (0,0001 mm/s/h) after maintaining a slope of less than that value for several hundred hours, resulting in a choice between two or more values of S.

To summarise: smaller variations were obtained with the 400 hour results than with the 1000 hour results, the flow results (F_{400} etc.) resulted in smaller variations than the permeability results (K_{400} etc.) and the parameters quantifying the point of stability resulted in the largest variations.

5.5.2 Flow versus permeability

The relationship between F_{400} and K_{400} for tests performed with one soil type (Warmbad) and different geotextiles is shown on Figure 5.11. Using linear regression, a correlation coefficient, r^2, of 0,875 was obtained, indicating a good correlation. Figure 5.12 shows the same relationship, but for all the test results, i.e. different soil types and various geotextiles. The correlation in this case was very poor (r^2 =

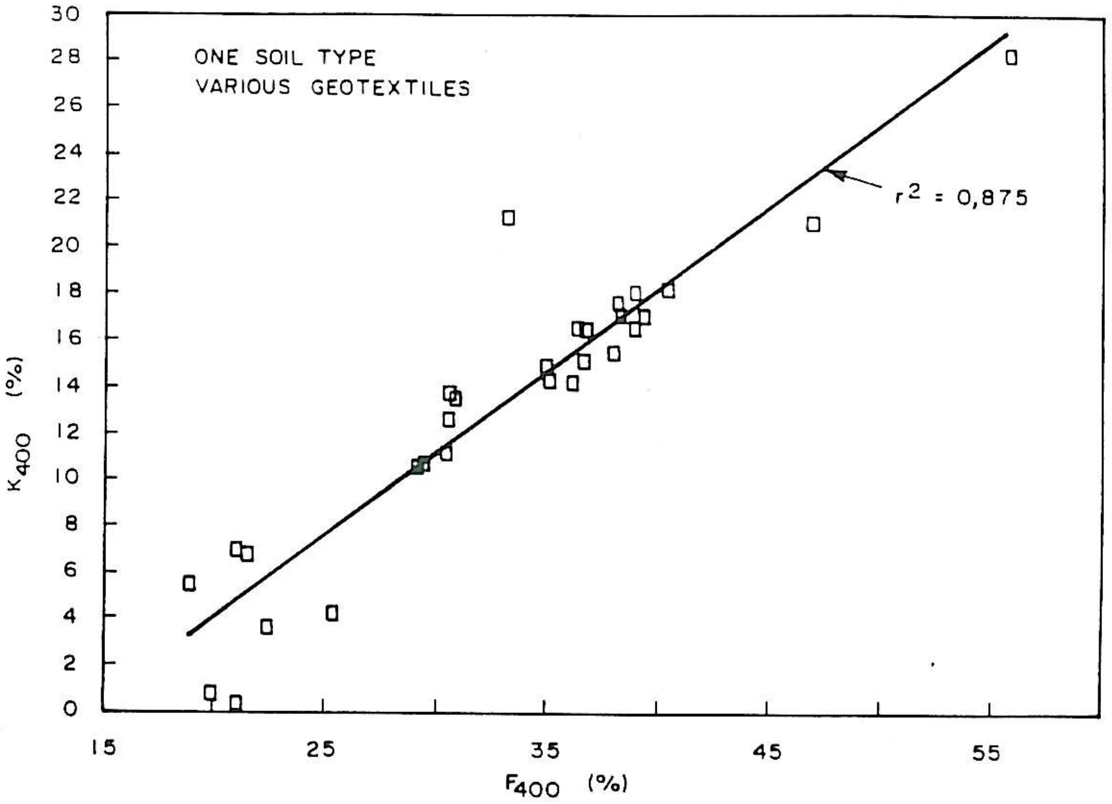


FIGURE 5.11

K_{400} VERSUS F_{400} (WARMBAD SOIL)

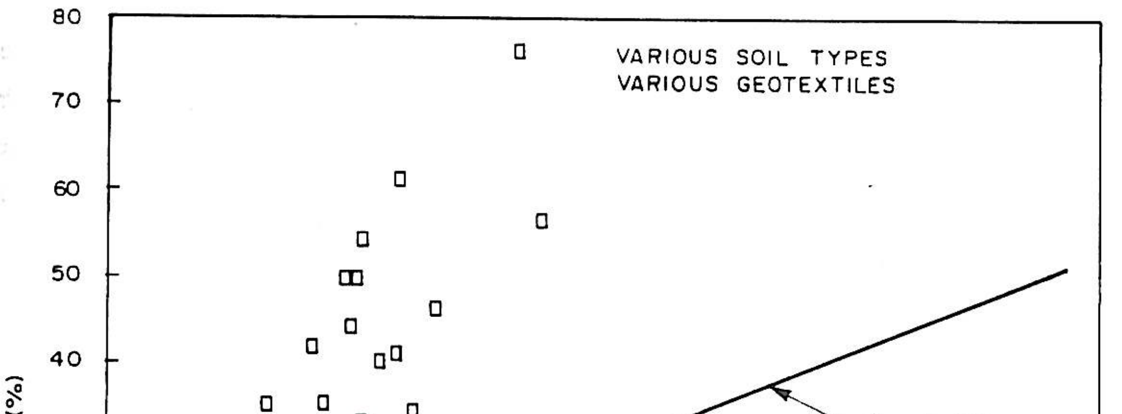


FIGURE 5.12

K_{400} VERSUS F_{400} (ALL TESTS)

0,135). Similar results were obtained for F_{1000} and K_{1000} (r^2 = 0,847 and 0,451 for Warmbad soil and all tests respectively).

These findings indicate that, for the same soil, irrespective of the geotextile used, the shape of the flow curves were similar and hence the good correlation between F_{400} and K_{400}. This was confirmed by a visual evaluation of the flow curves. Different soil types, however, resulted in different curve shapes and therefore the poor correlation shown in Figure 5.12.

An important conclusion that can be drawn from these results is that the soil type has a larger influence on the results obtained with flow tests than the geotextile used.

5.5.3 400 hours versus 1000 hours

A good correlation was obtained between K_{400} and K_{1000} for test results with Warmbad soil (r^2 = 0,839). For different soil types (all tests), however, there was virtually no correlation (r^2 = 0,033) between K_{400} and K_{1000}.

The correlations obtained between the total flow at 400 hours and the total flow at 1000 hours were very good. Figure 5.13 shows the relationship between F_{400} and F_{1000} for tests with Warmbad soil (r^2 =0,991) and Figure 5.14 shows the relationship for all tests (r^2 = 0,934).

The results show that, if K_{400} is determined for a certain soil/geotextile combination, K_{1000} can be calculated with reasonable accuracy for that particular combination if K_{1000} is known for other geotextiles in combination with the same soil. The relationship between F_{400} and F_{1000} (Figure 5.14), however, makes it possible to obtain F_{1000} for any soil/geotextile combination if F_{400} is known.

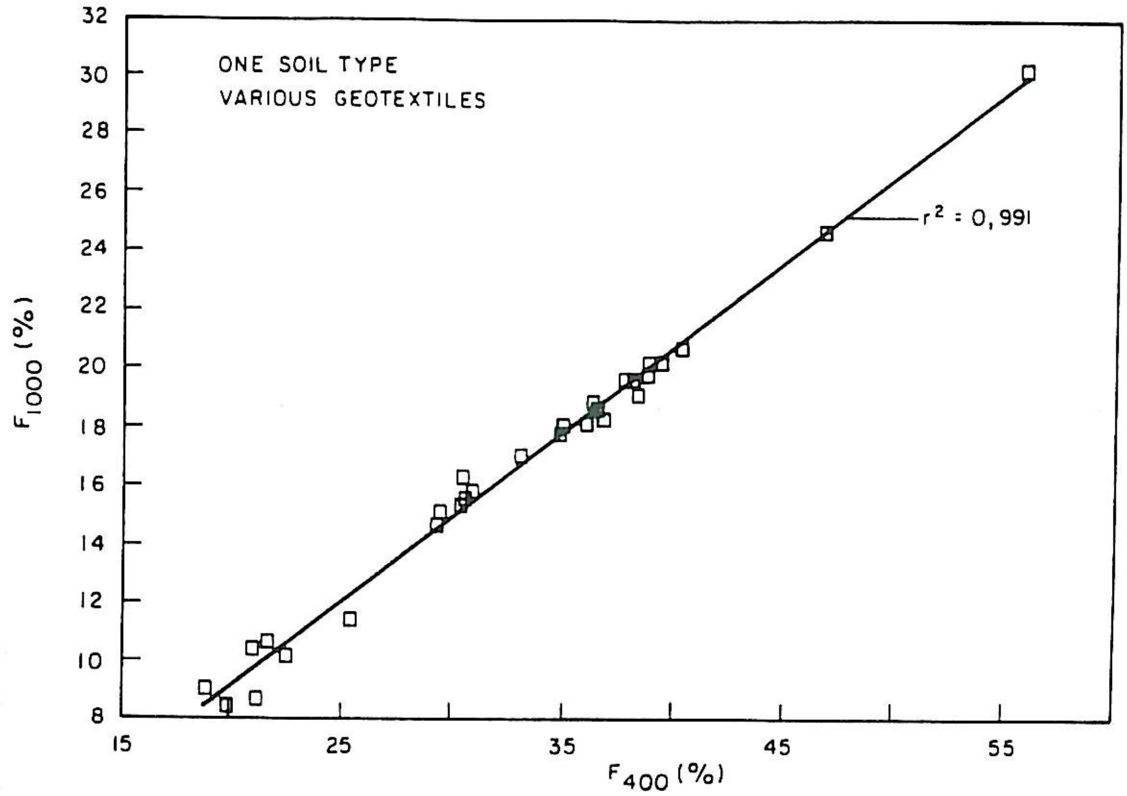


FIGURE 5.13

F_{1000} VERSUS F_{400} (WARMBAD SOIL)

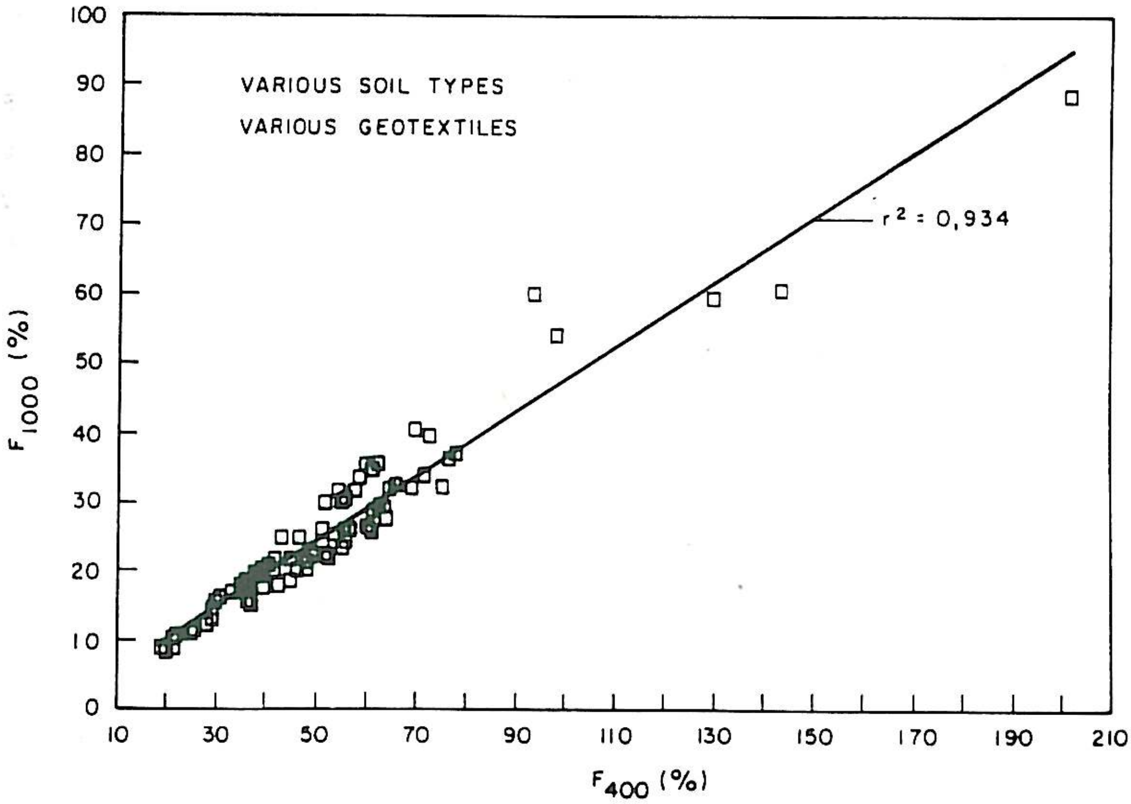


FIGURE 5.14

F_{1000} VERSUS F_{400} (ALL TESTS)

These findings are of importance. The 400 hours was based on a particular field installation. It is possible that sites may be encountered with larger hydraulic gradients resulting in higher flow rates, in which case a 400 hour laboratory test may not be adequate. In such cases, if F_{400} is known, the laboratory performance after 1000 hours can be predicted, using the results described above. Furthermore, it is less time consuming and therefore less expensive to do 400 hour tests than 1000 hour tests.

5.5.4 Stability (S, F_s, K_s)

The time after which the system stabilised, according to the definition in Section 5.4.2 (S hours), varied between 90 and 940 hours, with an average of 394,1 hours (Table 5.6). This is an average value for a fairly large number of soil/geotextile combinations and indicates that, on average, stability is reached around 400 hours. This fact is a further reason, although not the most important, why the 400 hour parameters (F_{400} and K_{400}) should be used for quantifying the flow test results.

No noteworthy correlations were obtained with parameters quantifying stability. A summary of correlation coefficients obtained is given below in Table 5.8.

Table 5.8 Summary of correlation coefficients (r^2) obtained with parameters relating to stability

Parameters correlated	Correlation coefficient, r^2	
	Tests with Warmbad soil	All tests
S, Ks	0,414	0,102
K_{400}, S	0,667	0,506
K_{400}, Ks	0,479	0,013
K_{400}, Es	0,638	0,0007
E_{400}, S	0,683	0,029

The best correlations obtained were those between K_{400} and S (r^2 = 0,667 and 0,506 for Warmbad soil and all tests respectively). The relationships for these are shown on Figures 5.15 and 5.16. It is evident from these figures that there is in fact no relationship between these parameters.

5.5.5 Conclusions

The following conclusions were drawn from the evaluation described above:

- The 400 hour parameters (F_{400} and K_{400}) are the most suitable for further evaluation of the data for three reasons, namely: these parameters had the smallest variation in identical tests, the average time for stability to be reached was approximately 400 hours and the field investigation indicated 400 hours to be a reasonable testing time.

- Good relationships were obtained between total flow and permeability (e.g. F_{400} and K_{400}) for a specific soil type, but not for different soil types. This indicated that soil type had a larger influence on flow test results than the type of geotextile used.

- Good correlations were obtained between F_{400} and F_{1000}, making it possible to predict performance after 1000 hours with 400 hour test results.

5.6 THE INFLUENCE OF SOIL CHARACTERISTICS ON FLOW TEST RESULTS

In an attempt to evaluate the influence of in-isolation parameters on flow test results, the soil characteristics were considered first without considering the geotextile parameters. In this section the influence of the soil parameters (described in Section 5.3.1) on the parameters quantifying flow test results (defined in Section 5.4.2) are described.

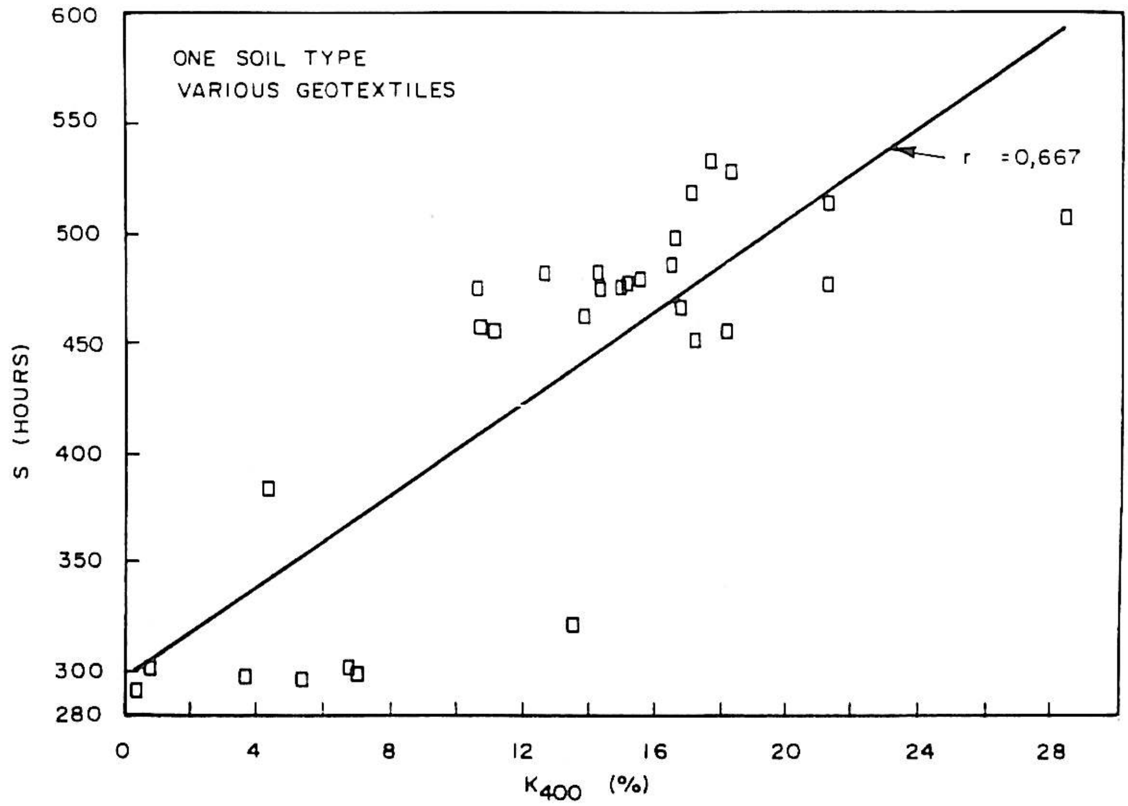


FIGURE 5.15

S VERSUS K_{400} (WARMBAD SOIL)

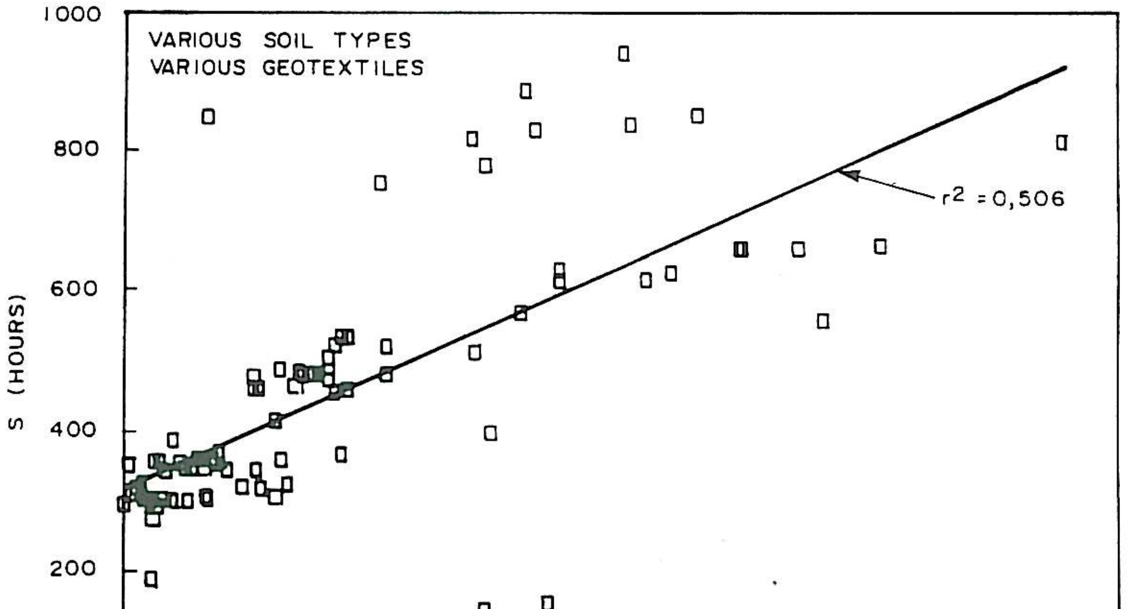


FIGURE 5.16

S VERSUS K_{400} (ALL TESTS)

5.6.1 K_{400}

The analysis in Section 5.5 above indicated K_{400} and F_{400} to be the most suitable parameters for quantifying flow test results. The influence of soil parameters on K_{400} is discussed here.

(i) Percentage passing

The cumulative percentage passing the indicated sieve sizes are given in Table 5.2 for each soil used in the investigation. Figure 5.17 shows the influence of percentage passing the 0,075 mm sieve on flow test results as quantified by K_{400}. The results shown on Figure 5.17 were obtained with different soils but only one geotextile (Geotextile No. 1). It is evident that a relationship exists, indicating that, for a specific geotextile, a high percentage passing the 0,075 mm sieve will result in a low K_{400}. Better performance can therefore be expected with soils containing a low percentage passing 0,075 mm. Similar results (but different curves) were obtained for other geotextiles analysed in the same way.

The same results are shown on Figure 5.18, but using the results from all the tests, i.e. various soil types and different geotextiles. Here a relationship is not evident and a curve cannot be fitted. However, there is a downward tendency, indicating lower permeabilities with higher percentage fines and an envelope of maximum values can be fitted. This envelope indicates the maximum value of K_{400} that can be expected to be obtained for a specific soil type with a certain percentage fines. The value that will in fact be obtained (values below the envelope) depends on the geotextile used.

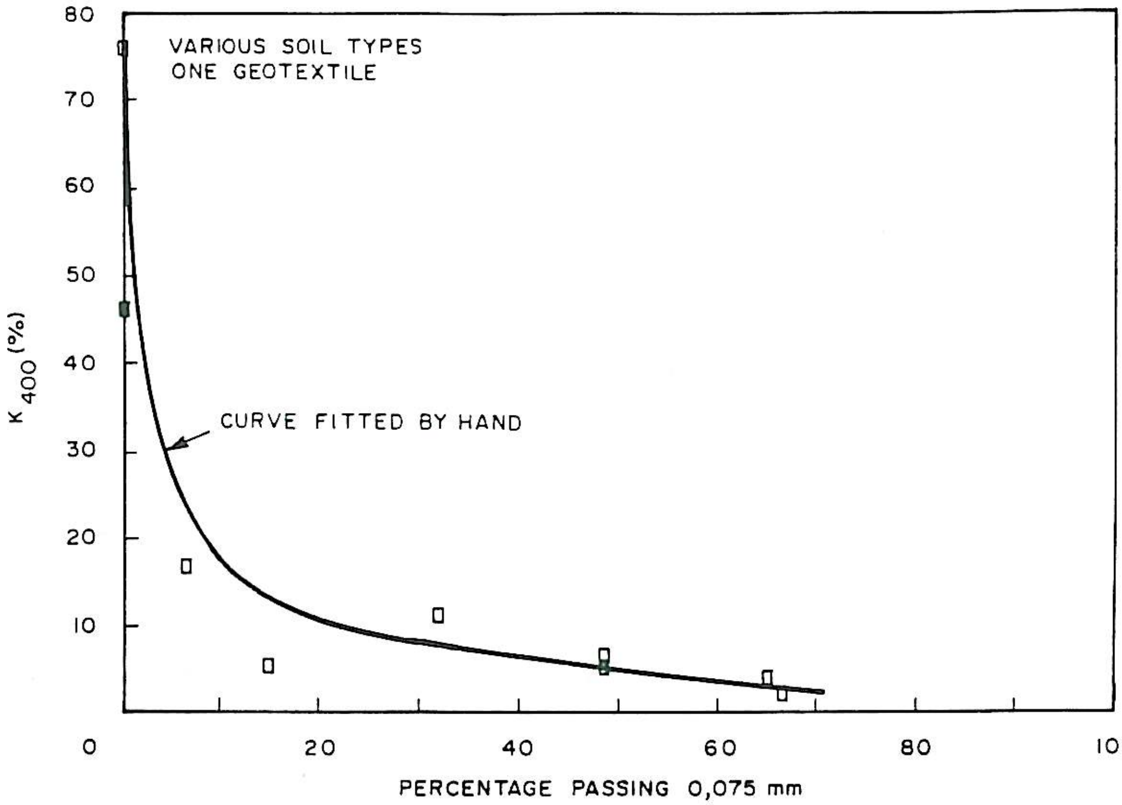


FIGURE 5.17

K_{400} VERSUS PERCENTAGE PASSING 0,075mm (GEOTEXTILE 1)

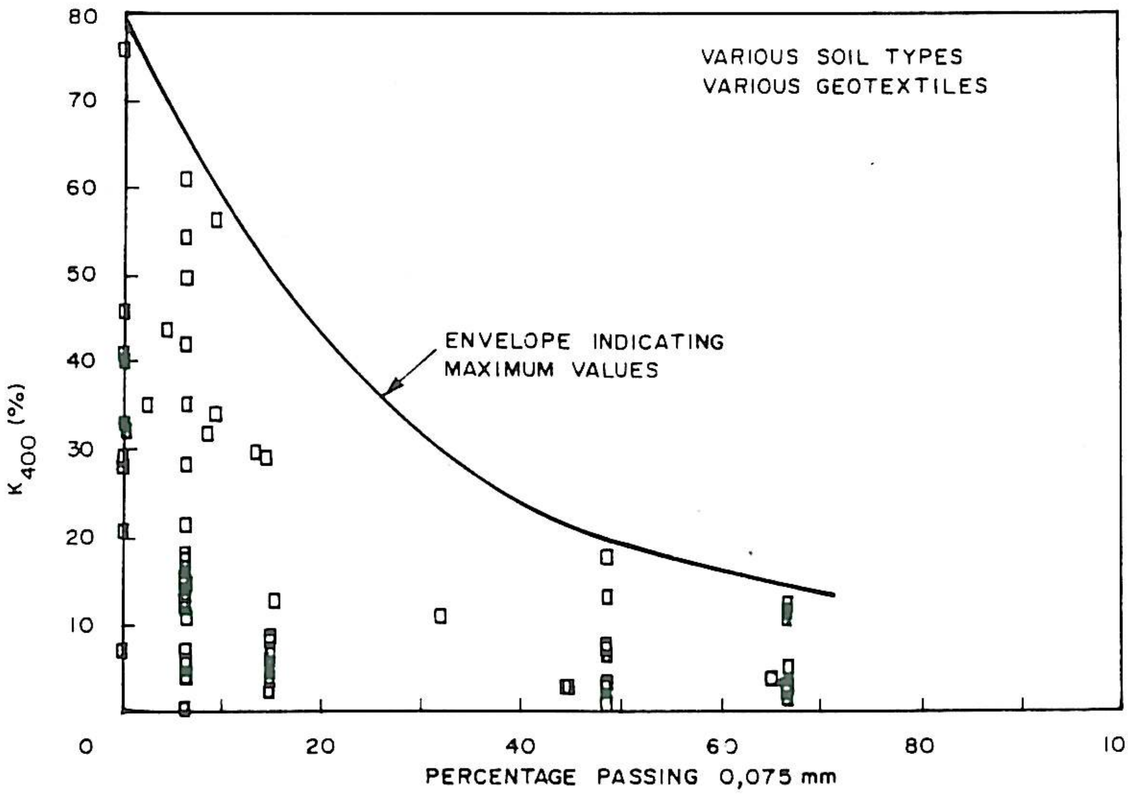


FIGURE 5.18

K_{400} VERSUS PERCENTAGE PASSING 0,075 mm (ALL TESTS)

Sieve sizes other than 0,075 mm gave similar results. Figure 5.19 shows the results from all the tests, using percentage passing 0,150 mm. Once again an envelope curve can be fitted. The position of this curve is to the right of the one in Figure 5.18. Figure 5.20 shows a family of curves, each being an envelope obtained with different sieve sizes. The envelope curves from Figures 5.18 and 5.19 are also shown (0,075 and 0,150 mm). The larger sieve sizes (e.g. 2,00 mm) slope in the opposite direction, indicting that a higher percentage passing that sieve will result in better performance (higher values of K_{400}). This is not surprising, since a higher percentage passing a large sieve indicates coarser soil. The intermediate sieves (0,250 and 0,425 mm) resulted in envelope curves in both directions. Only small differences were evident between the curves obtained with sieve sizes smaller than 0,075 mm, indicating less sensitivity. Of the curves shown on Figure 5.20, those obtained with 0,075 mm and 0,150 mm sieves give the most meaningful results in terms of sensitivity and in terms of covering a wide range of values. Furthermore, the percentage passing 0,075 mm (also known as percentage fines) is a commonly used parameter in quantifying soils and as such is more meaningful.

(ii) Particle sizes

The soil particle sizes shown in Table 5.2 (D_{10},D_{50}, etc.) are essentially a different way of expressing percentage passing. These parameters are used specifically in filter design and as such need to be examined. Figure 5.21 shows, for one geotextile and different soil types, the influence of D_{50} on the flow test results (K_{400}). The results are similar to those obtained with percentage passing (Figure 5.17), but with an upward curve since larger values of D_{50} indicate coarser material with less fines. The same results, but from all the tests using different geotextiles, are shown in Figure 5.22. Once again an envelope can be fitted, indicating maximum values, as described in Section 5.6.1 (i) above. Similar

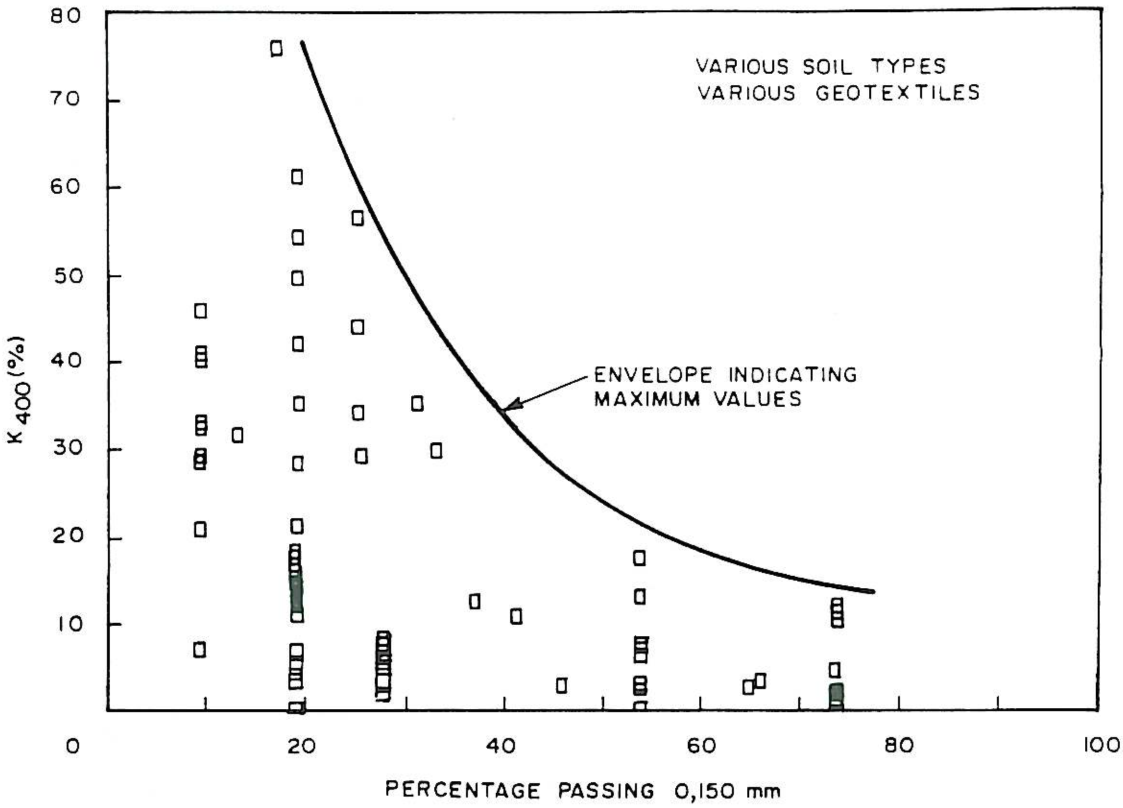


FIGURE 5.19

K_{400} VERSUS PERCENTAGE PASSING 0,150 mm (ALL TESTS)

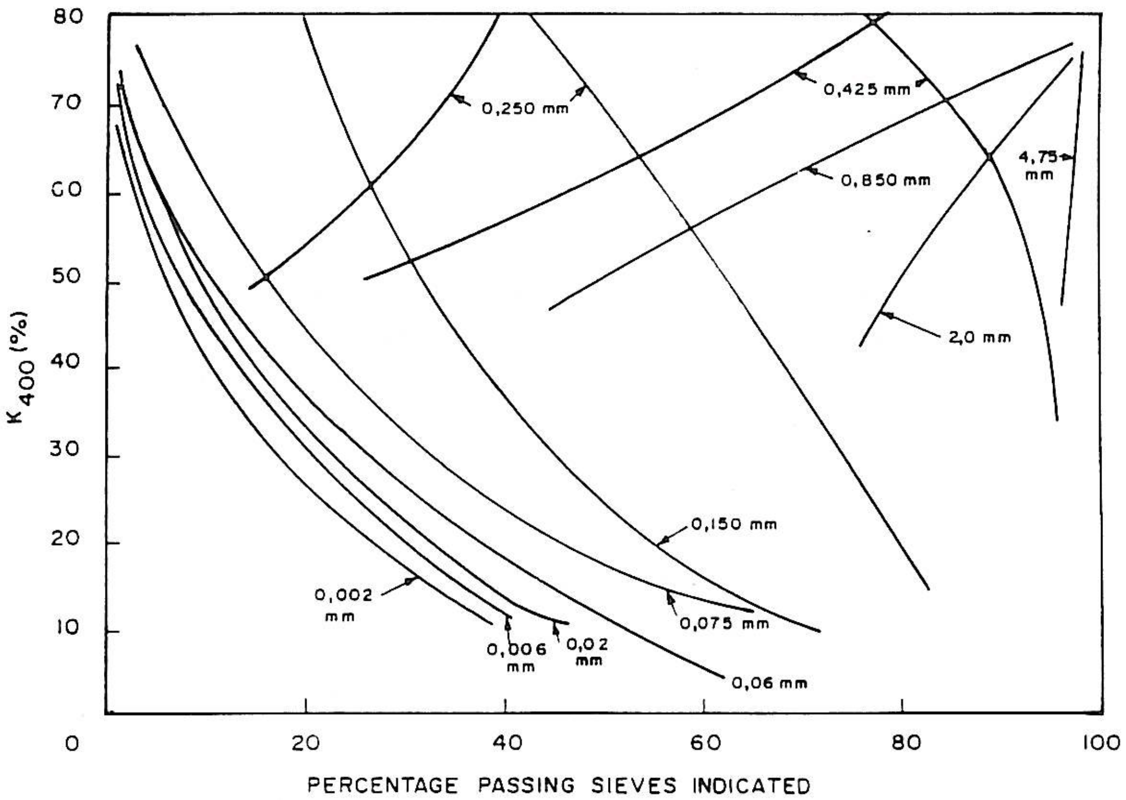


FIGURE 5.20

ENVELOPE CURVES FOR K_{400} VERSUS PERCENTAGE PASSING

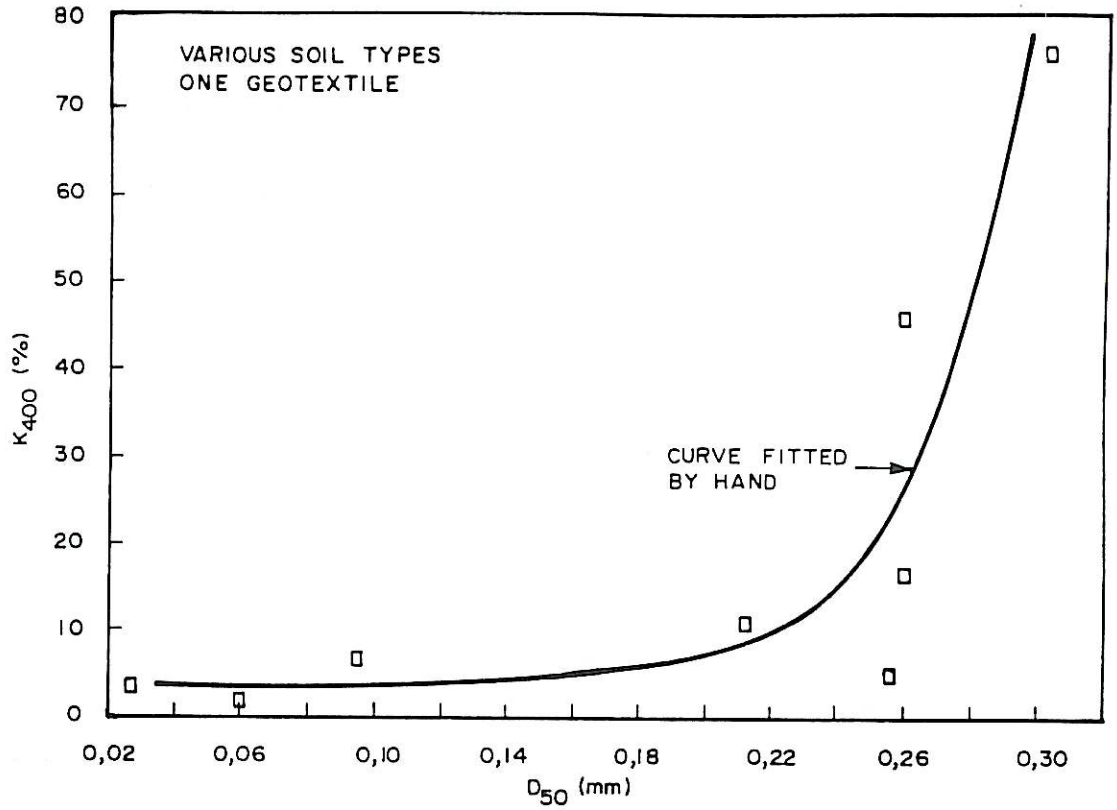


FIGURE 5.21

K_{400} VERSUS D_{50} (GEOTEXTILE I)

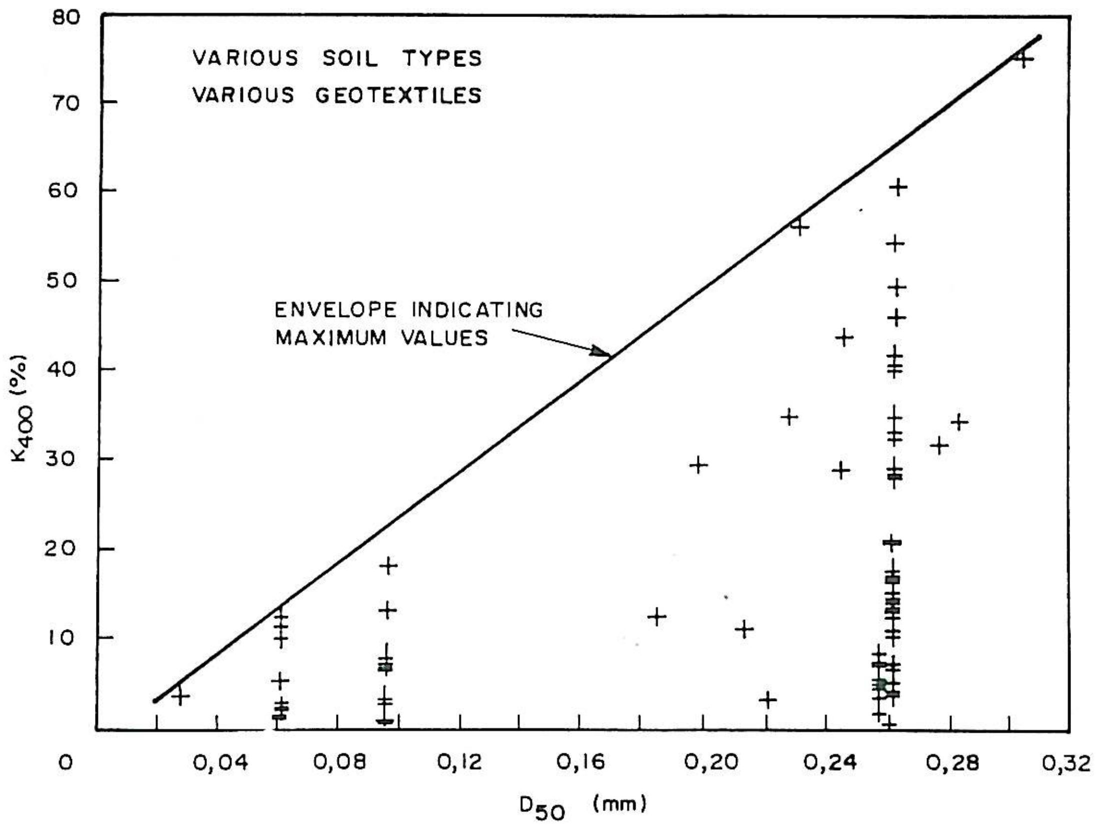


FIGURE 5.22

K_{400} VERSUS D_{50} (ALL TESTS)

results were obtained for D_{10}, D_{15} and D_{60} particle sizes, but with the envelope lines in different positions, as was the case with percentage passing described above.

(iii) Other grading parameters

The influence of other grading parameters on K_{400} are discussed here. These parameters include uniformity coefficient (U), grading modulus (GM), fineness modulus (FM) and standard deviation (SD). The parameters are described in Section 5.3.1 and their values for the various soils given in Table 5.2.

Figures 5.23 to 5.26 show the influence of these parameters on K_{400} for all test results, i.e. different soils and different geotextiles.

Uniformity coefficient (U), which is an indication of the distribution of the particle sizes, shows no discernible influence on K_{400} (Figure 5.23). The same is true for results using only one geotextile.

Grading modulus (GM) is an indication of the particle sizes retained between 0,075 mm and 2,00 mm. Figure 5.24 indicates that higher values of GM tend to result in higher values of K_{400}. A high grading modulus indicates a large percentage material in the range 0,0075 mm to 2,00 mm and therefore less material passing 0,075 mm, thus resulting in better flow test results. The relationship between GM and K_{400} for only one geotextile, however, was poorer than that between K_{400} and percentage passing 0,075 mm.

Fineness modulus (FM) is a measure of the average particle sizes over the whole grading range, a higher value indicating coarser material. Figure 5.25 indicates, as may be expected, that coarser material (higher values of FM) can be expected to result in higher values of K_{400}. Once again the relationship between K_{400} and FM for one geotextile showed a poorer correlation than was the case with percentage passing or particle size.

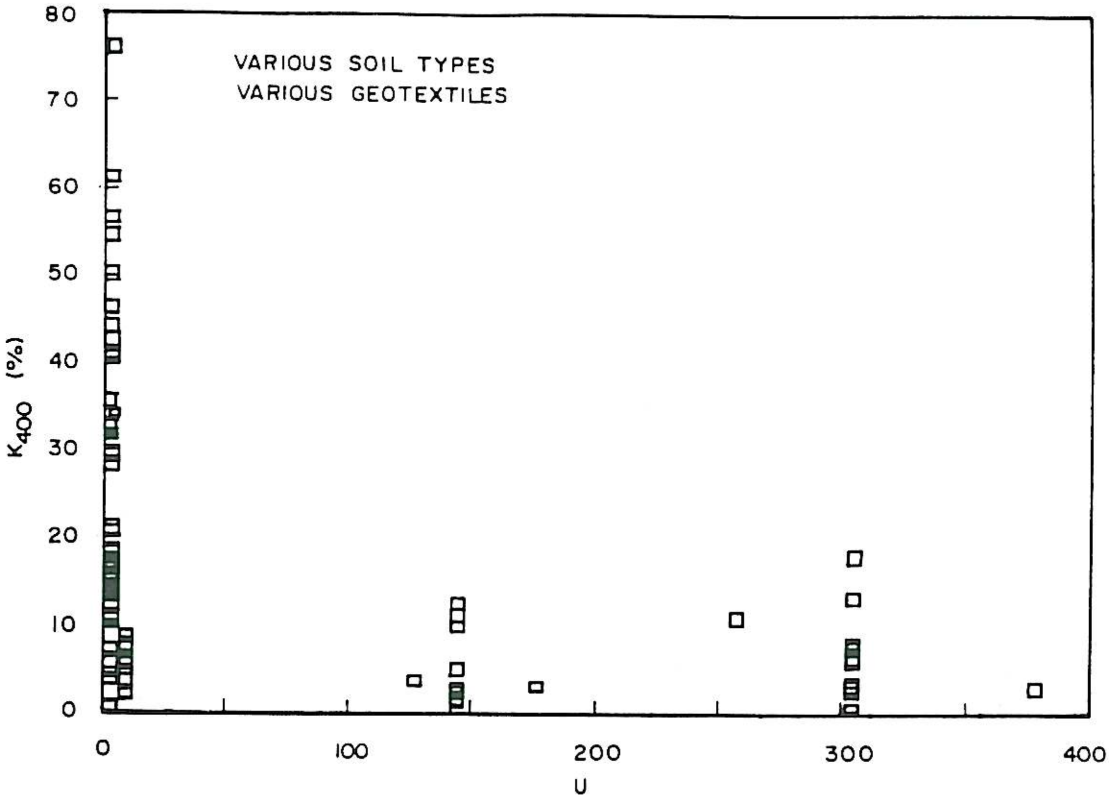


FIGURE 5.23

K_{400} VERSUS UNIFORMITY COEFFICIENT (U) (ALL TESTS)

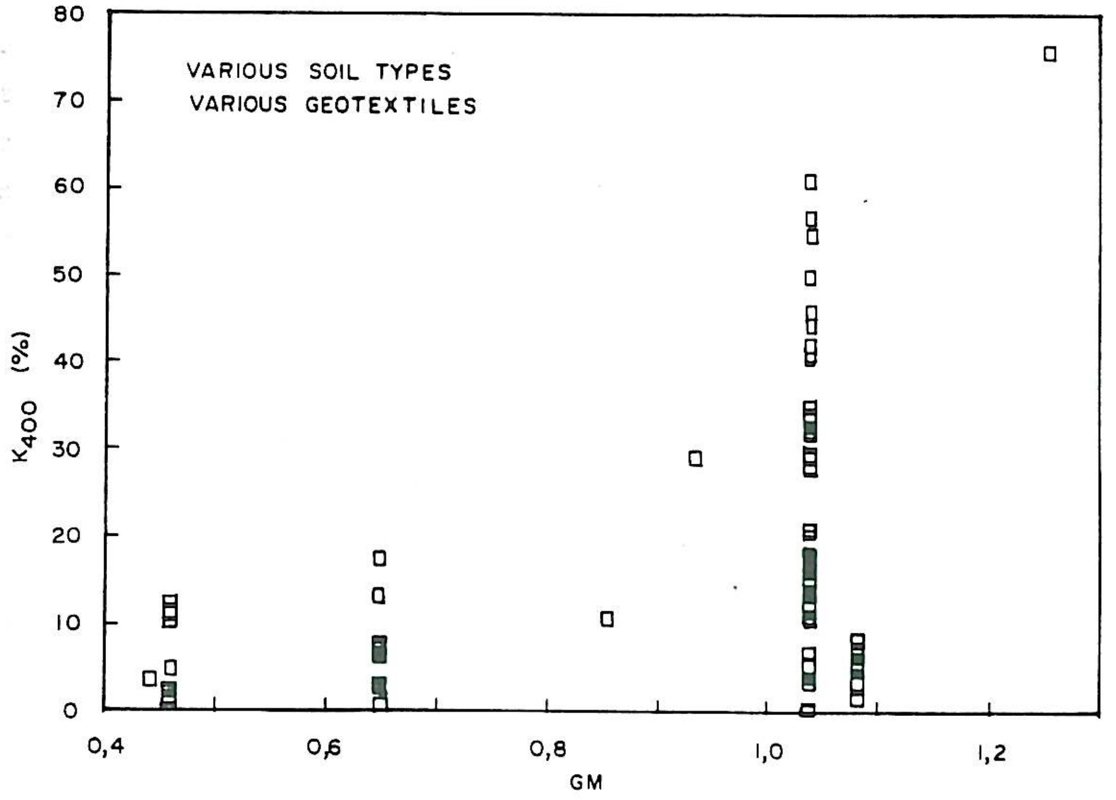


FIGURE 5.24

K_{400} VERSUS GRADING MODULUS (GM) (ALL TESTS)

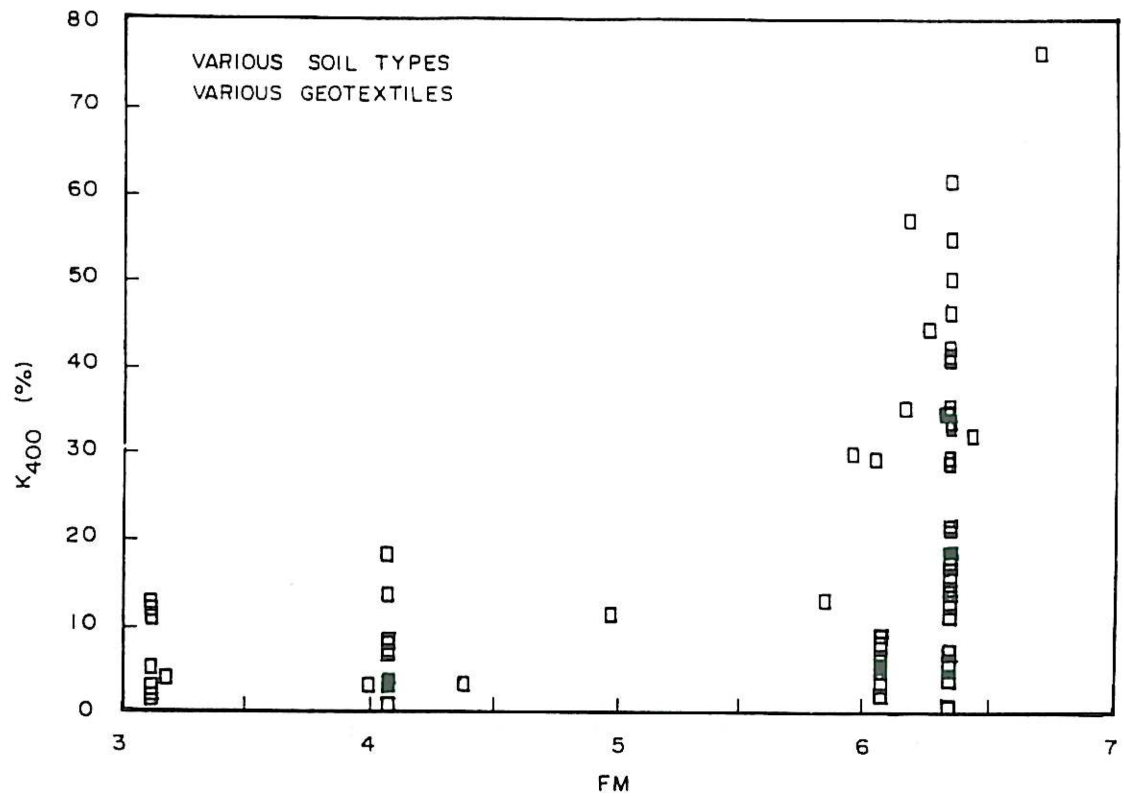


FIGURE 5.25

K_{400} VERSUS FINENESS MODULUS (FM) (ALL TESTS)

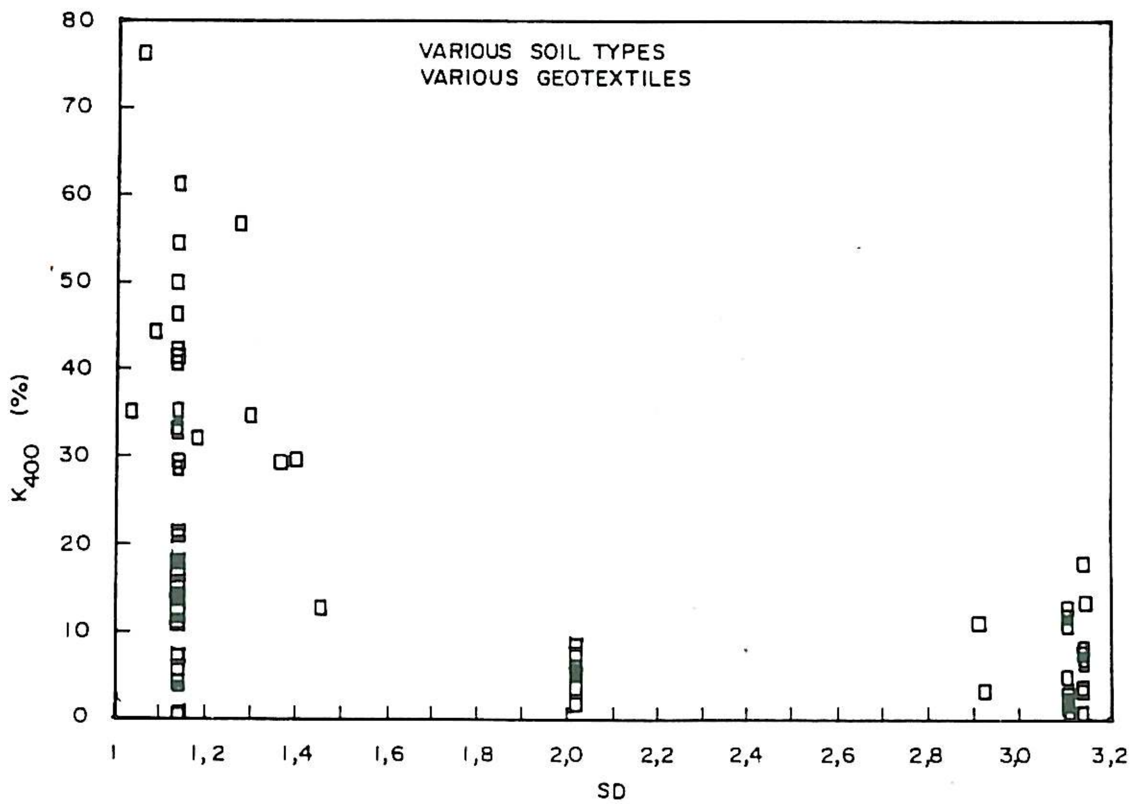


FIGURE 5.26

K_{400} VERSUS STANDARD DEVIATION (SD) (ALL TESTS)

Standard deviation (SD) is a measure of the particle distribution or the amount of deviation from the fineness modulus. For this parameter, a downward trend is evident (Figure 5.26), high values of SD tending to result in lower values of K_{400}. The trend is not as well defined, however, as was the case with other parameters described earlier.

5.6.2 Other parameters (e.g. F_{400})

Analyses similar to that described in Section 5.6.1 above, but using other parameters quantifying flow test results (e.g. F_{400}, K_{1000}, F_s, etc.) gave similar results to those obtained using K_{400}. In general, however, the relationships were somewhat poorer and the envelope curves less clearly defined than was the case using K_{400}. The results were similar, however, and the reasons for the various relationships were seen to be the same as described above.

5.6.3 Conclusions

Reasonably good relationships were obtained between flow tests results (K_{400}) and parameters quantifying coarseness (or fineness) of the soil (e.g. percentage passing 0,075 mm), for one geotextile and different soil types. With different geotextiles and different soils, an envelope curve could be fitted, indicating maximum values of K_{400} that can be expected for a specific soil.

The results indicate that poorer results (lower K_{400}) can be expected from the flow test with finer than with coarser soils. This can probably be attributed to a denser (less permeable) filter zone being formed by the finer particles in the soil, thus resulting in a larger reduction in the permeability of the whole system.

The difference in results obtained with different geotextiles but using the same soil (results below the envelope line) can probably be attributed to geotextile characteristics and the interaction between the soil and the geotextile. These aspects are evaluated in Section 5.7 below.

5.7 INFLUENCE OF GEOTEXTILE PARAMETERS ON FLOW TEST RESULTS

In Section 5.6 above, it was shown that the flow test results are influenced by both soil and geotextile characteristics. The influence of soil characteristics was analysed and described in Section 5.6 and in this section the influence of geotextile parameters is investigated. The in-isolation parameters of geotextiles that were measured are normal permeability and opening sizes. These parameters and the values obtained are described in Sections 4.2.1 and 4.2.2.

5.7.1 Normal permeability

With certain soil/geotextile combinations, the initial permeability coefficient obtained with flow tests was considerably less than that of the soil. In these cases the normal permeability of the geotextile was less than that of the soil and therefore not adequate. This is discussed in more detail in Chapter 7. The test results discussed in this chapter did not include these results. In all the tests of which the results are given here, the initial normal permeability of the geotextiles was more than that of the soil.

The influence of normal permeability on flow test results as quantified by K_{400} is shown on Figure 5.27 for one soil type (Warmbad soil) and using different geotextiles. It is evident that no relationship is discernible. An envelope can be fitted, however, giving minimum values of K_{400} that can be expected to be obtained with a certain geotextile permeability. Lower permeabilities, however, will not

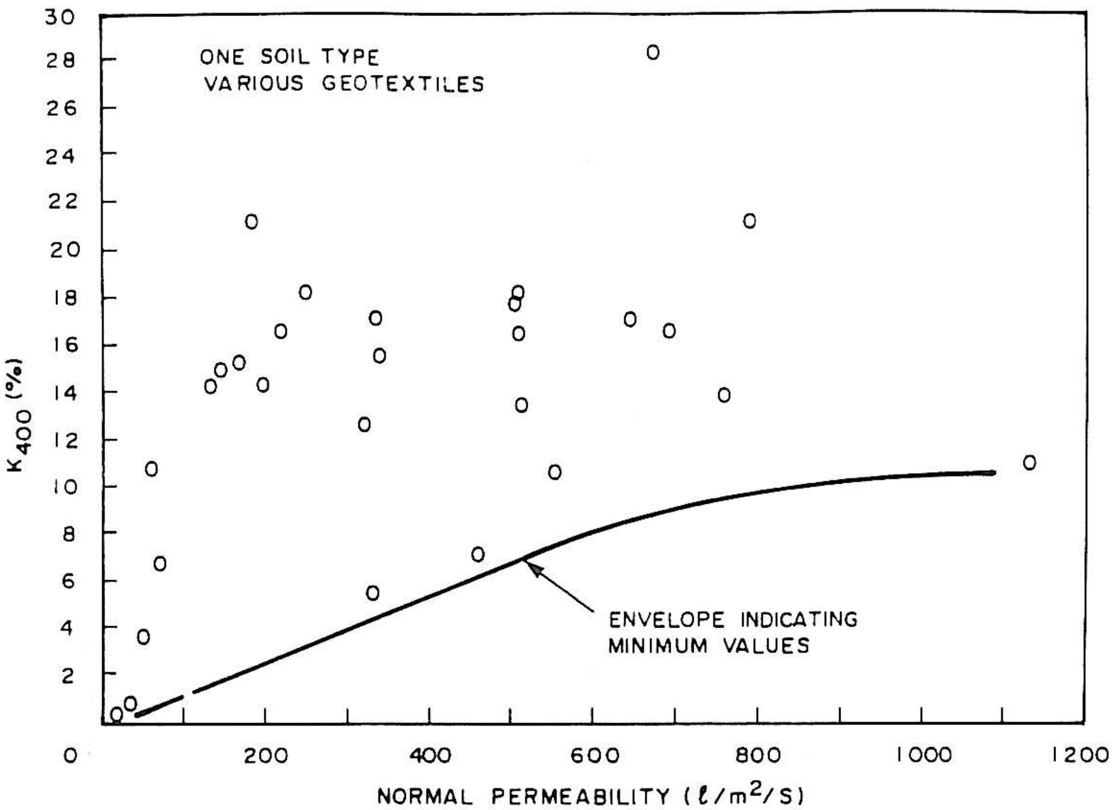


FIGURE 5.27

K_{400} VERSUS NORMAL PERMEABILITY (WARMBAD SOIL)

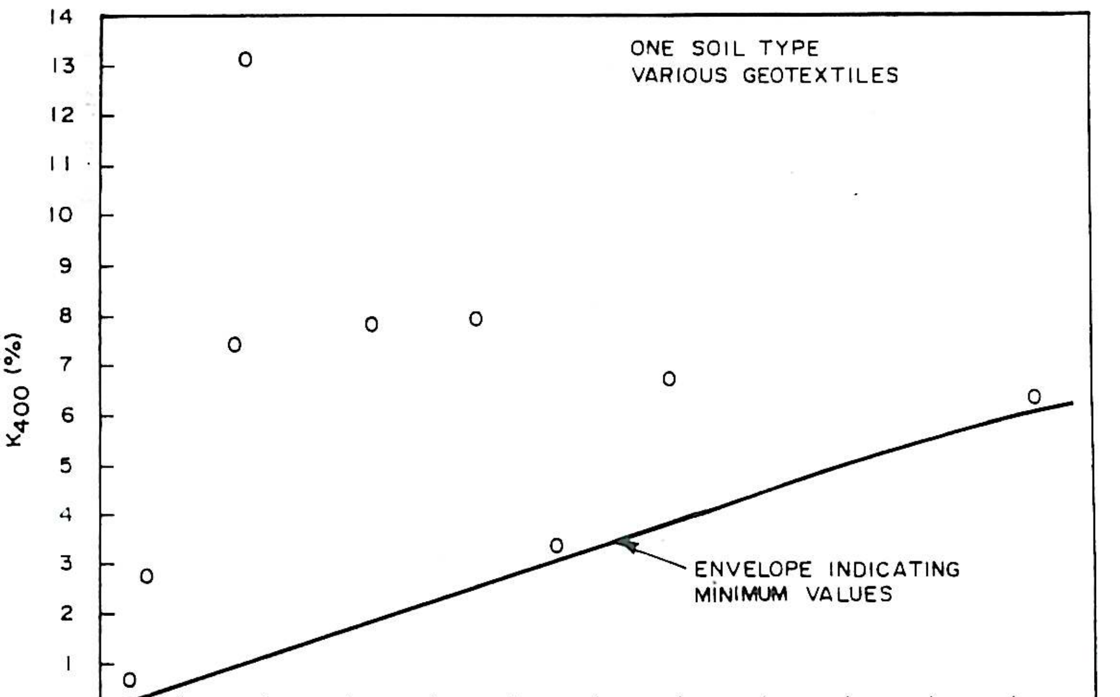


FIGURE 5.28

K_{400} VERSUS NORMAL PERMEABILITY (SILVERTON SOIL)

necessarily result in lower values of K_{400}. It should be noted that the envelope line is not very well defined, as was the case with the soil characteristics (e.g. Figure 5.18). Similar results were obtained with other soils, e.g. Silverton soil (Figure 5.28) and Du Plessis building sand (Figure 5.29). In each of these cases an envelope of minimum values could be fitted. This was not the case with all soil types, however, as is evident from Figure 5.30 (Montana clay). For this particular soil neither a relationship nor an envelope is discernible.

As may be expected, no relationships were evident when the results from all the tests were used (different soil types). This is illustrated in Figure 5.31.

Similar results to those described above were obtained with flow test parameters other than K_{400}. Figure 5.32 is an example of such a comparison, using F_{400} and results obtained with one soil (Warmbad). Once again, it is possible to fit an envelope of minimum values.

5.7.2 Pore sizes

The inverse sieving test (Section 4.2.1) allowed different pore sizes to be calculated for each geotextile, e.g. O_{90}, O_{50}, etc. The influence of these opening sizes on flow test results are described here.

Figure 5.33 shows the influence of O_{90} of the geotextiles on flow test results as quantified by K_{400}. These results are from tests using one soil type only (Warmbad soil). It is evident that no relationship or envelope line exists for these results. The same is true for other opening sizes, e.g. O_{50} (Figure 5.34) and O_{10} (Figure 5.35), as well as for other soils, e.g. Montana clay, O_{50} (Figure 5.36). Similar results were obtained using other flow test parameters (F_{400} etc.).

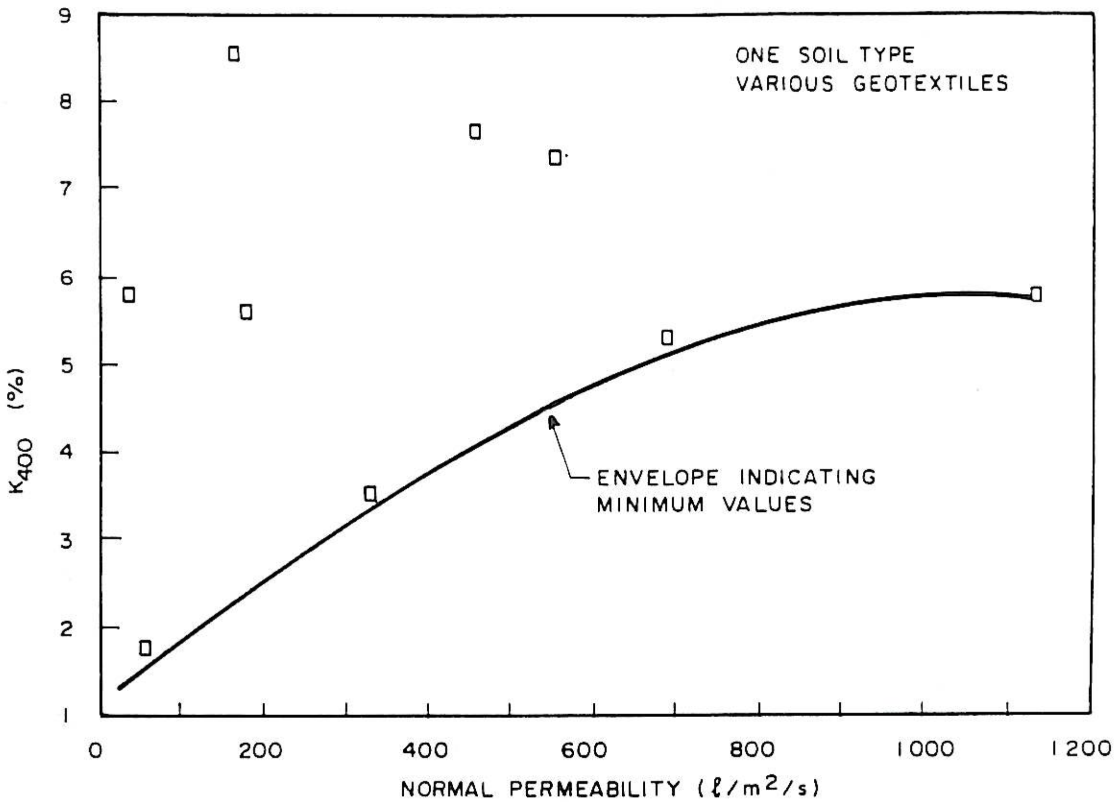


FIGURE 5.29

K_{400} VERSUS NORMAL PERMEABILITY (DU PLESSIS BUILDING SAND)

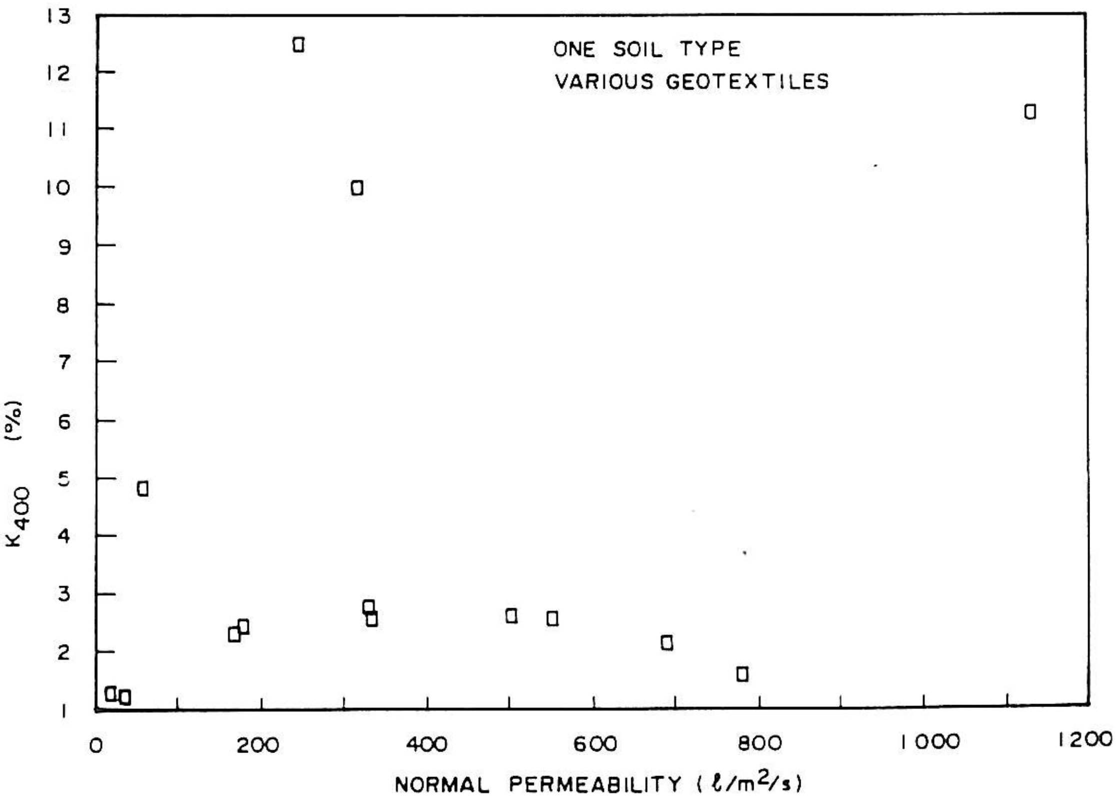


FIGURE 5.30

K_{400} VERSUS NORMAL PERMEABILITY (MONTANA CLAY)

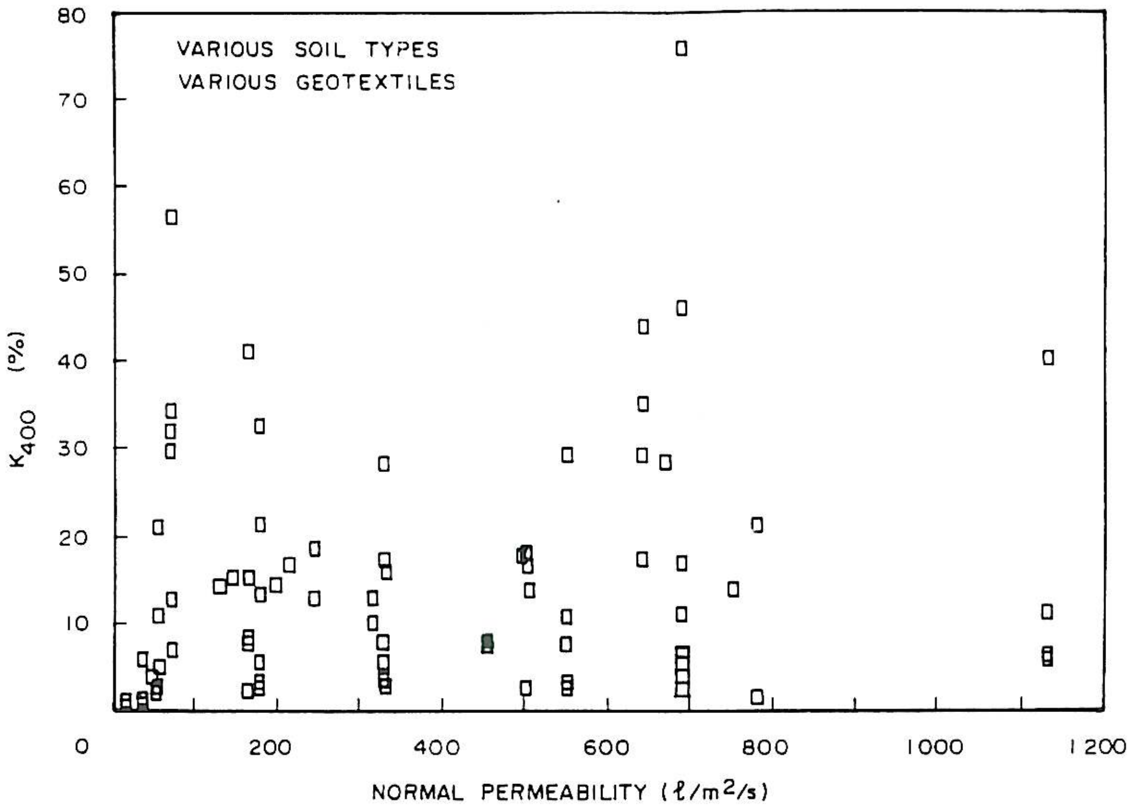


FIGURE 5.31

K_{400} VERSUS NORMAL PERMEABILITY (ALL TESTS)

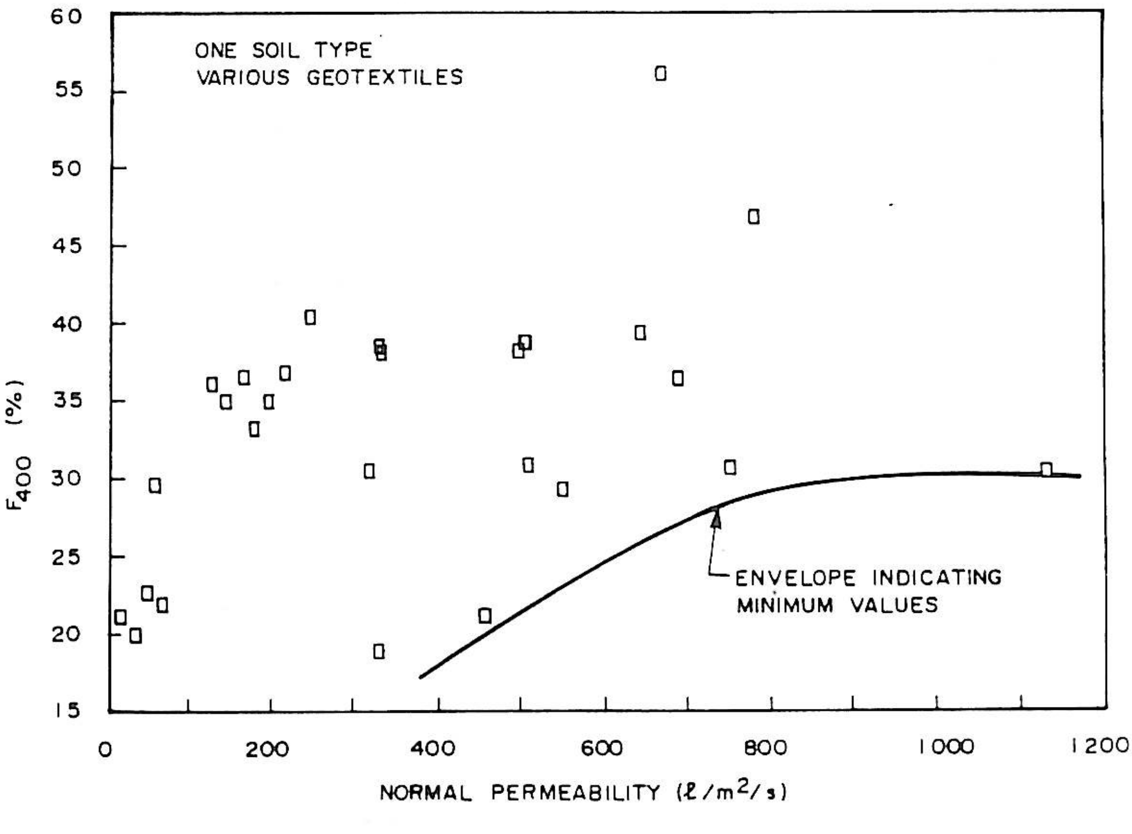


FIGURE 5.32

F_{400} VERSUS NORMAL PERMEABILITY (WARMBAD SOIL)

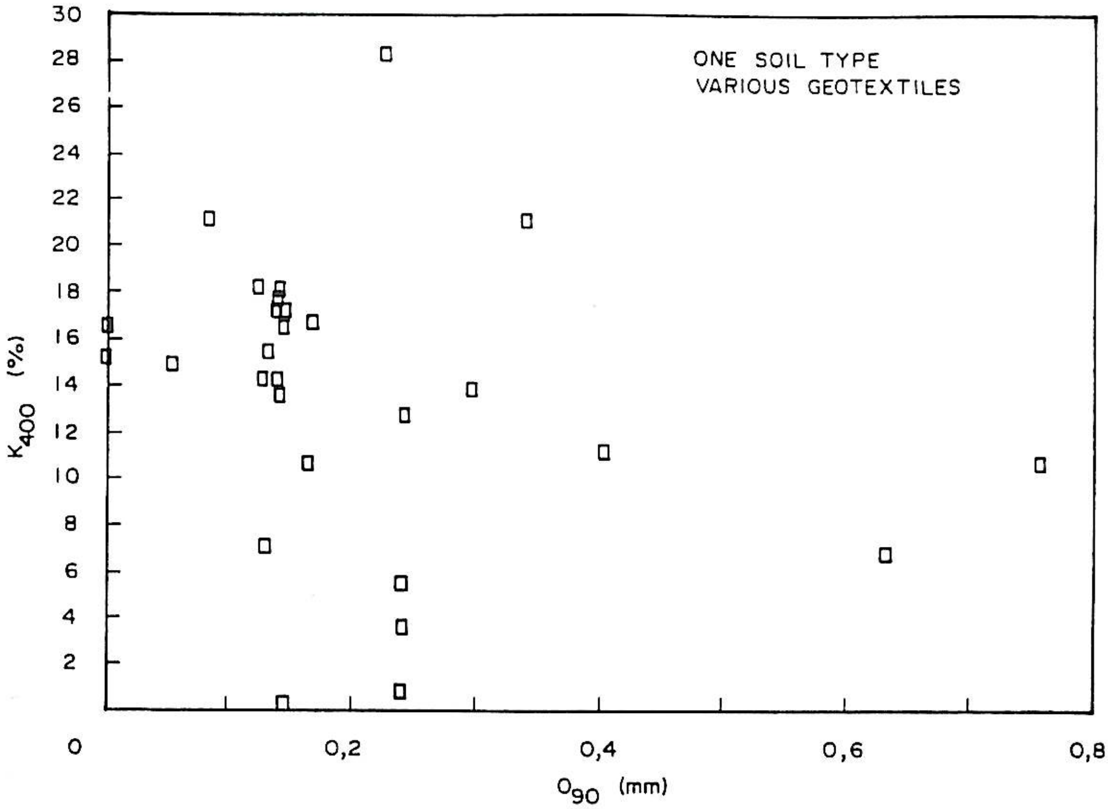


FIGURE 5.33

K_{400} VERSUS O_{90} (WARMBAD SOIL)

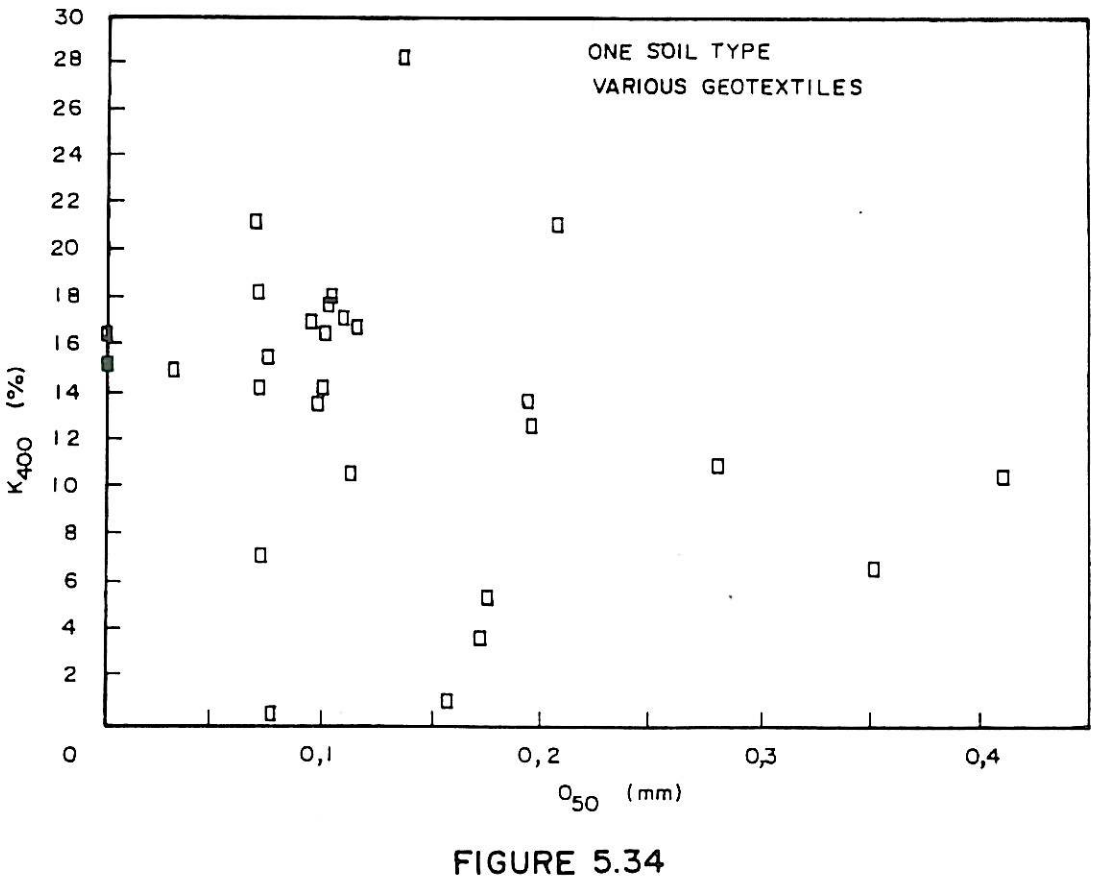


FIGURE 5.34

K_{400} VERSUS O_{50} (WARMBAD SOIL)

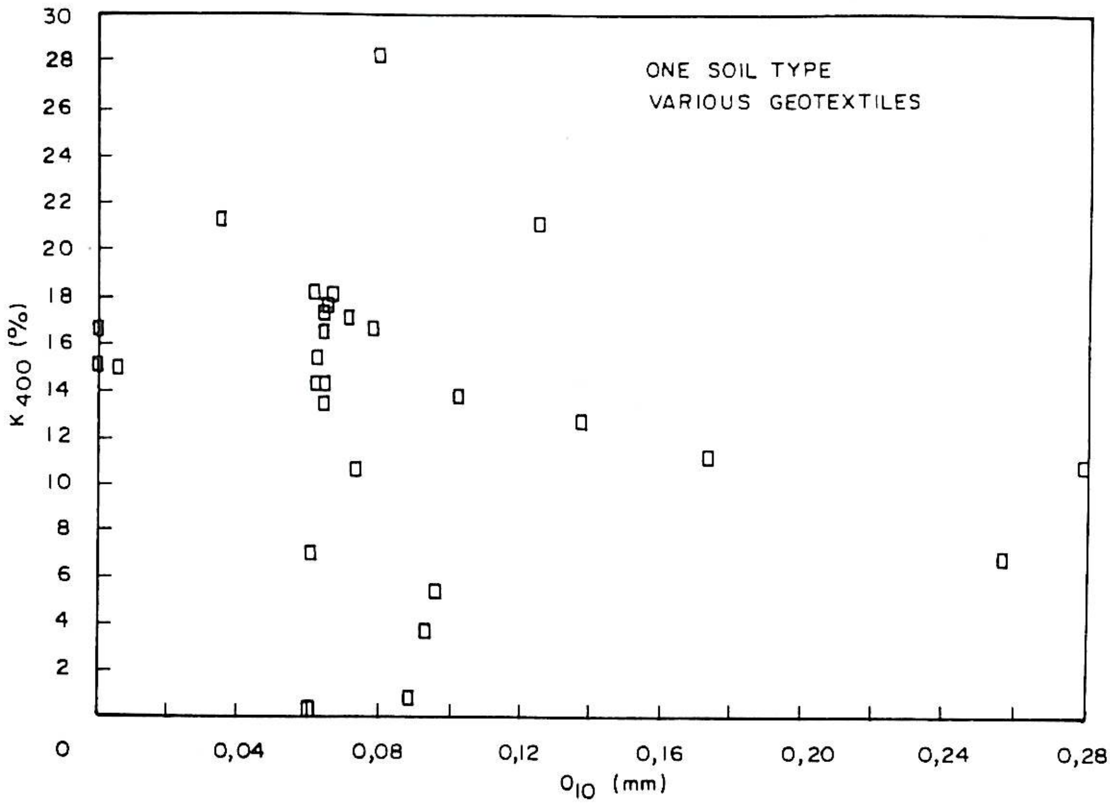


FIGURE 5.35

K_{400} VERSUS O_{10} (WARMBAD SOIL)

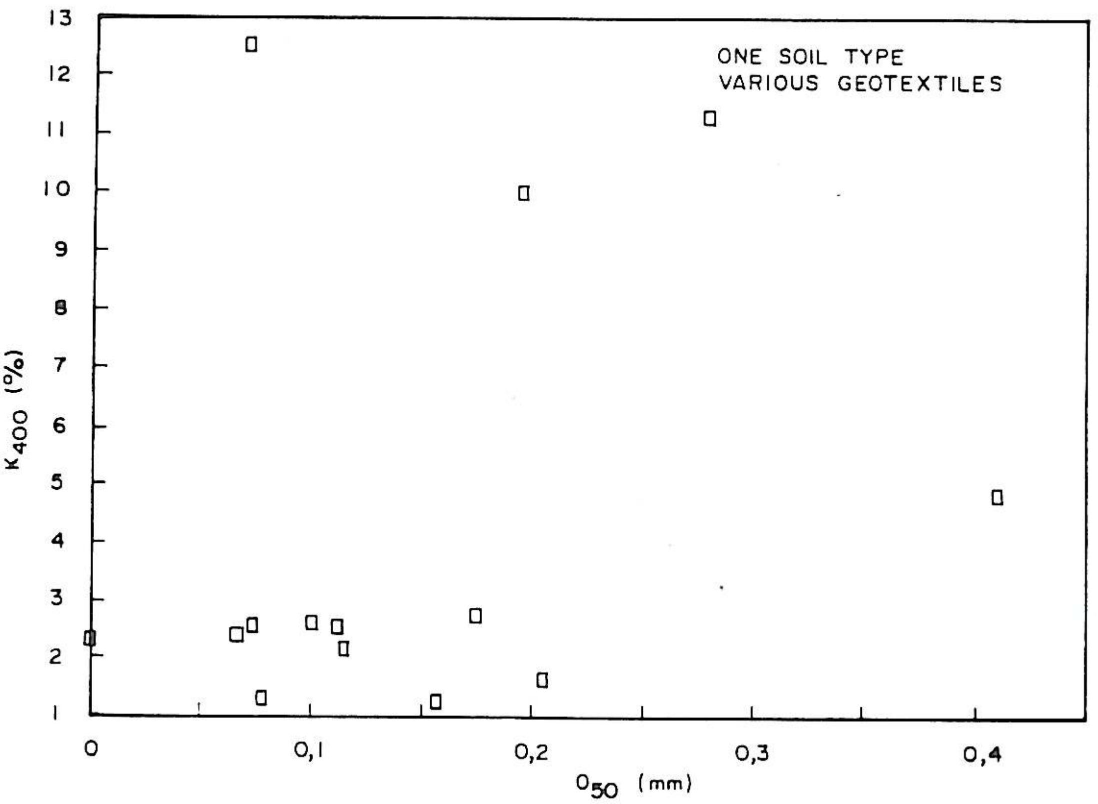


FIGURE 5.36

K_{400} VERSUS O_{50} (MONTANA CLAY)

5.7.3 Conclusions

Although in-isolation parameters of geotextiles are used widely for design criteria, it is evident from the results described above that no relationships exist between these and flow test results. Envelope lines could be fitted giving minimum values of K_{400} for given normal permeabilities, but these were not well defined and only applied with certain soils. In general, in-isolation characteristics of geotextiles could not predict the performance obtained with flow tests.

The analysis described in Section 5.6 indicated that geotextile characteristics do influence the results obtained with the flow tests. The analysis described in this section indicated that those geotextile characteristics that influence the results are not the known, measurable ones, i.e. normal permeability and opening sizes. The geotextile influence must therefore be ascribed to other characteristics. These may include the shape and distribution of openings (channels) in the geotextile. The sieving test does not measure or quantify these, but simply determines which size grains can be forced through the geotextile fibres.

It is clear that the soil/geotextile interaction is complex, and methods of quantifying geotextile characteristics that can predict their performance have not yet been established. The results show that the performance differs from one geotextile to another, which indicates a certain amount of clogging of the geotextiles in some cases. It follows then that, until geotextile characteristics can be adequately quantified, soil/geotextile compatibility tests, such as the flow test described in this report, must be performed as part of the geotextile filter design procedure.

5.8 INTERFACE FLOW CAPACITY (IFC) TESTS

IFC tests are soil/geotextile compatibility tests, as opposed to in-isolation tests, and were reported on by other researchers (Hoover, 1982 and Legge, 1986). The findings of these researchers were discussed earlier (Section 3.4.2).

5.8.1 Test method and results

The IFC tests by Legge (1986) were performed using a constant head permeameter. The tests described here were performed with the same falling head permeameter that was used for normal permeability. The apparatus was described in Section 4.2.2.1.

The test procedure was as follows: The geotextile was mounted in the permeameter and its normal permeability was determined by recording the time taken for the column of water to flow through the geotextile. The permeameter was again filled with water (using the same geotextile) and 5 g of single sized soil particles (e.g. soil retained on 0,425 mm sieve) was introduced into the water and allowed to settle on the geotextile surface. The normal permeability was then again determined. This was repeated until a total of 35 g of soil was introduced in 5 g intervals. Thereafter the geotextile was removed, another sample of the same geotextile was mounted and the test was repeated using another particle size of the same soil (e.g. soil retained on 0,250 mm sieve).

An example of the results obtained using Warmbad soil and a thin nonwoven geotextile (Geotextile No. 1) is shown on Figure 5.37. These results show the decrease in flow rate (normal permeability) with the introduction of more soil particles. The results obtained with two different soil particle sizes are shown, namely particles retained on the 0,425 mm and the 0,250 mm sieves. These particle sizes represent the D_{89} and the D_{48} of this soil. These values are close to the D_{85} and D_{50} values, and the latter values will be referred to in this discussion for clarity, since these were the values used by Legge and discussed earlier (Section 3.4.2).

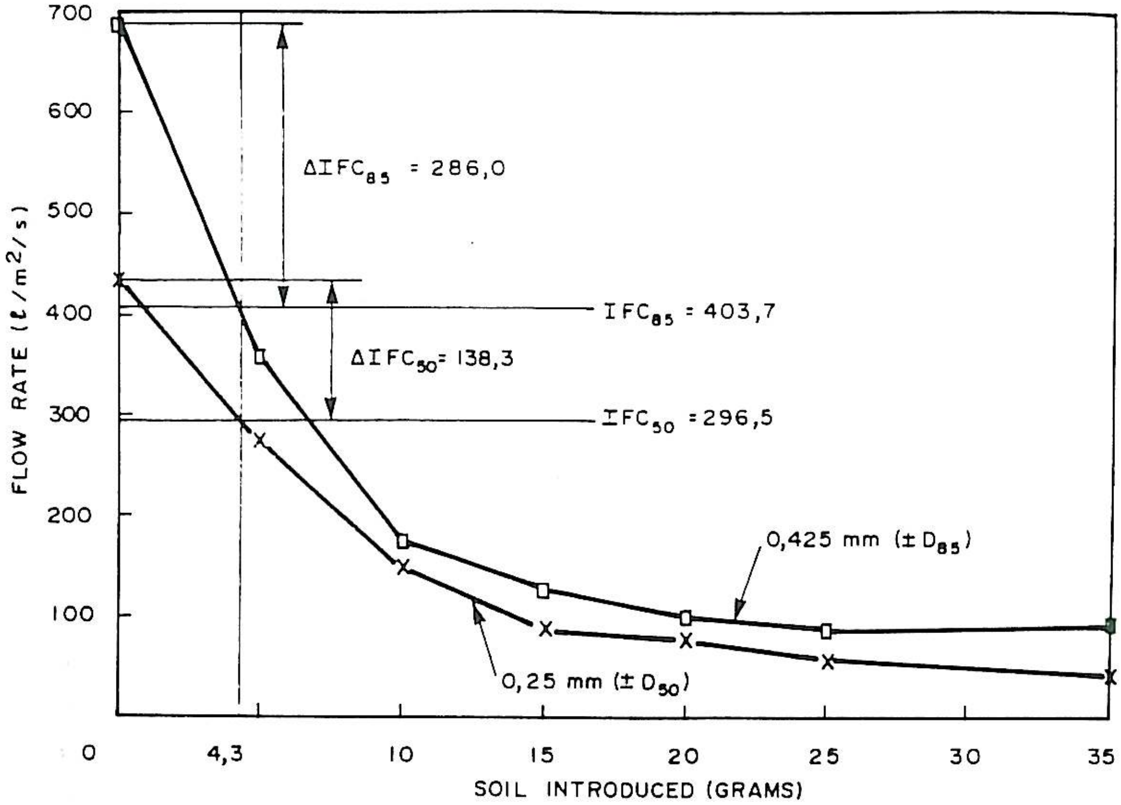


FIGURE 5.37

EXAMPLE OF IFC TEST RESULTS (SOIL: W, GEOTEXTILE 1)

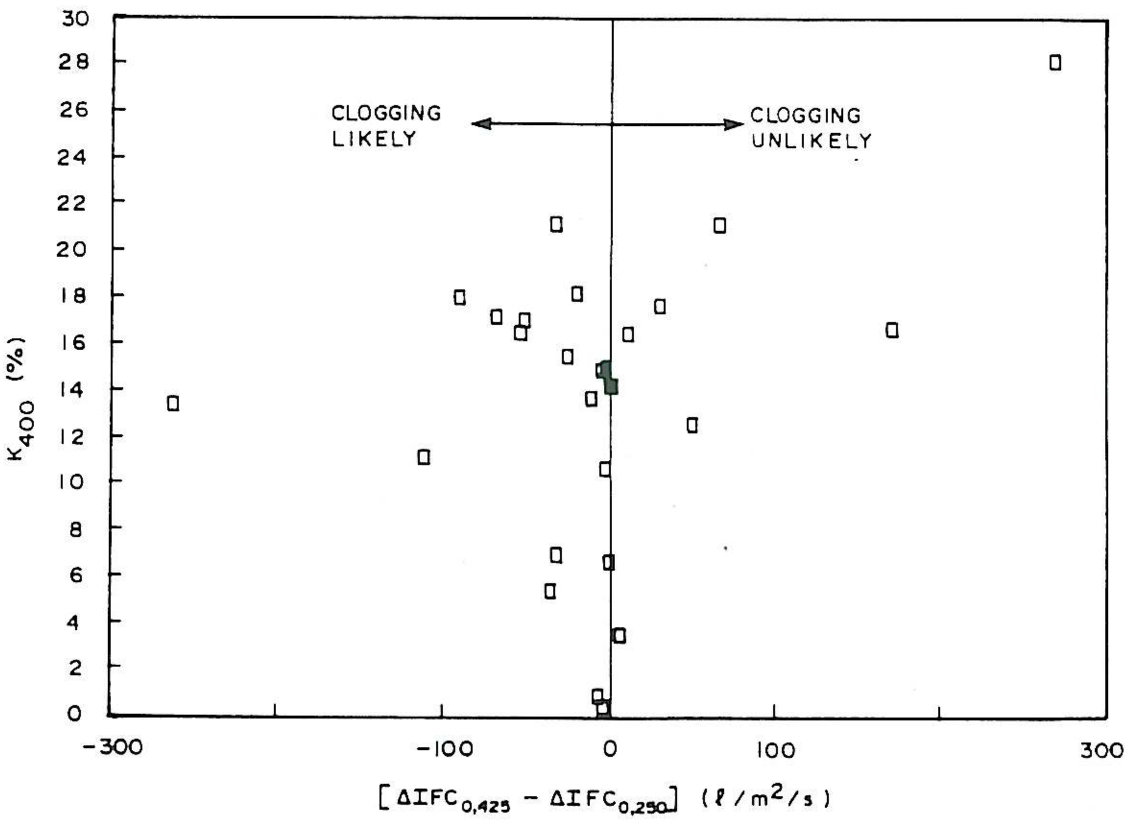


FIGURE 5.38

K_{400} VERSUS [$\Delta IFC_{0,425} - \Delta IFC_{0,250}$] (WARMBAD SOIL)

Interface flow capacity (IFC) is defined as the flow obtained when the amount of soil has been introduced that will cover the geotextile with a single layer of soil. It can be shown that, if spherical soil particles are assumed, this will occur with 4,3 grams of soil. The IFC values thus obtained for the D_{85} and D_{50} soil particles (IFC_{85} and IFC_{50} respectively) are shown on Figure 5.37.

The decrease in flow capacity (ΔIFC) is the difference between the flow obtained with no soil (normal permeability of the geotextile) and the flow obtained if the geotextile is covered with a single layer of soil (4.3 g in this example). These values are also shown on Figure 5.37 (ΔIFC_{85} and ΔIFC_{50} for the D_{85} and D_{50} values respectively).

The way in which these results can predict clogging, according to Hoover and Legge, is as follows: If the decrease in IFC for the D_{50} soil (ΔIFC_{50}) is larger than that for the D_{85} soil (ΔIFC_{85}) then the D_{50} particles will cause clogging of the geotextile. The reason being that, if both the D_{50} and the D_{85} particles merely rested on top of the geotextile, then ΔIFC would be the same in both cases. If ΔIFC_{50} is larger than ΔIFC_{80} it means that the D_{50} particles actually entered the pores or channels in the geotextile, thus clogging them and causing a reduction in flow. The theory also applies to other pairs of particle sizes, e.g. ΔIFC_{50} and ΔIFC_{10} obtained with D_{50} and D_{10} particle sizes.

Based on this theory, then, clogging is likely when

$$\Delta IFC_{50} > \Delta IFC_{85}$$

$$\text{or when } \Delta IFC_{50} - \Delta IFC_{85} > 0$$

$$\text{or when } \Delta IFC_{85} - \Delta IFC_{50} < 0$$

In the example shown on Figure 5.37:

$$[\Delta IFC_{85} - \Delta IFC_{50}] = 286,0 - 138,3$$

$$= 147,7$$

the value is greater than zero and this particular soil/geotextile system is therefore not likely to clog. From the flow tests described in Section 5.4 above, the value of K_{400} obtained with this particular soil/geotextile combination (Table 5.6) was 16,7%, which is reasonably high, indicating that it did in fact not clog substantially.

5.8.2 Analysis of results

IFC tests were performed with 27 different geotextiles, but using the same soil type (Warmbad soil). Each test was performed with five different particle sizes, namely, soil retained on the 0,425 mm; 0,250 mm; 0,150 mm; 0,075 mm sieves and the soil passing the 0,075 mm sieve.

In analysing the results, correlations were sought between the IFC results and K_{400} obtained from the flow test results described in Section 5.4. An example of such a comparison is shown on Figure 5.38, where K_{400} values are compared to $[\Delta IFC_{85} - \Delta IFC_{50}]$ or $[\Delta IFC_{0,425} - \Delta IFC_{0,250}]$ as described above. It is evident that no relationship is discernible. Most soil/geotextile combinations that gave $[\Delta IFC_{85} - \Delta IFC_{50}]$ values greater than zero did in fact also result in high values of K_{400}. The opposite, however, is not true, namely, that where $[\Delta IFC_{85} - \Delta IFC_{50}]$ was less than zero, K_{400} would be low.

Figure 5.39 shows the same comparison, but with other grain sizes, namely, 0,250 and 0,150 mm, as opposed to 0,425 and 0,250 mm in Figure 5.38. Once again, there was no relationship and with these grain sizes, the IFC test indicated that only four soil/geotextile combinations would not clog.

Yet another pair of grain sizes (those retained on 0,075 mm sieve and those passing 0,075 mm sieve resulted in the values

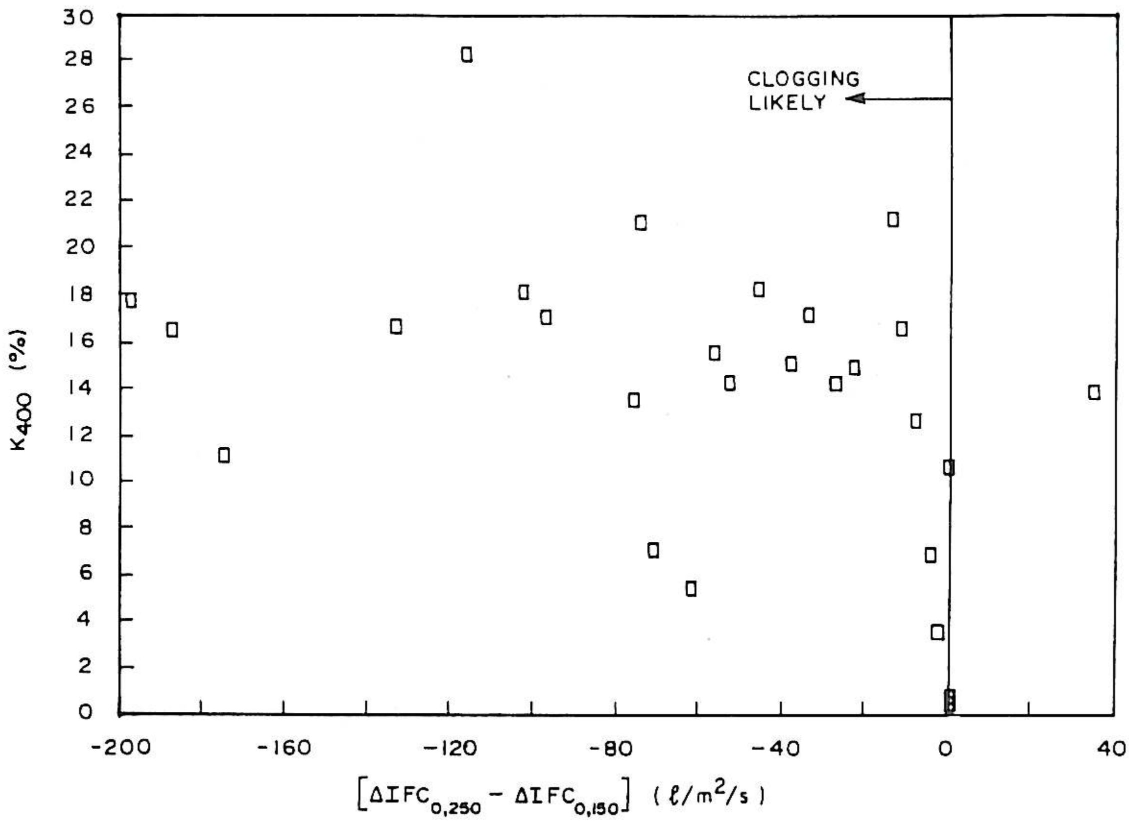


FIGURE 5.39

K_{400} VERSUS $[\Delta IFC_{0,250} - \Delta IFC_{0,150}]$ (WARMBAD SOIL)

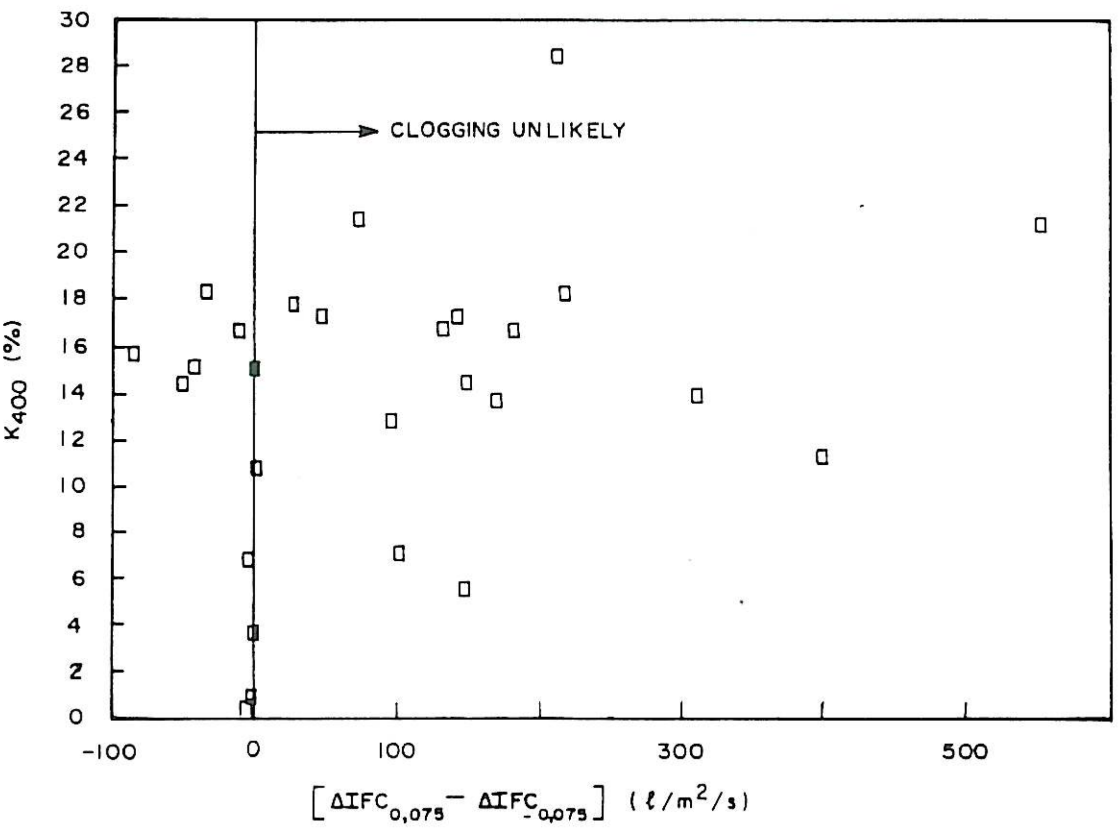


FIGURE 5.40

K_{400} VERSUS $[\Delta IFC_{0,075} - \Delta IFC_{-0,075}]$ (WARMBAD SOIL)

shown on Figure 5.40. These results indicated that very few of the combinations would clog.

The difference between ΔIFC values shown on the horizontal axes of Figures 5.38 to 5.40 can also be calculated for larger soil quantities introduced than that needed to form a single layer of soil on the geotextile surface. This means that a thicker layer of soil is formed and the permeability of the soil itself starts playing a role in the results. Figure 5.41 shows an example of a comparison between $[\Delta IFC_{85} - \Delta IFC_{50}]$ thus obtained and K_{400}.

The IFC values shown here were calculated for 20 g of soil introduced. The relationship (or lack thereof) on Figure 5.41 is similar to that on Figure 5.38, which was obtained with the same set of results, but at 4,3g of soil. This indicates that a reason for the poor relationships can probably be found in the fact that the results are influenced more by the soil permeability than by the soil/geotextile interaction. The flow tests are performed over a long period (400 hours) which allows the soil particles to settle into the pores of the geotextile and cause clogging. In the IFC tests, however, the water flows through the geotextile in a few seconds, which is probably not adequate for the soil/geotextile interaction to take place.

An attempt was also made to correlate K_{400} with the actual reduction in flow obtained, i.e. ΔIFC for one grain size, as opposed to the difference in ΔIFC between two grain sizes. An example of such a comparison is shown on Figure 5.42. Here an upward tendency can be observed, but with very poor correlation.

5.8.3 Conclusions

The results indicate that IFC tests cannot predict long term soil/geotextile compatibility accurately. The predictions

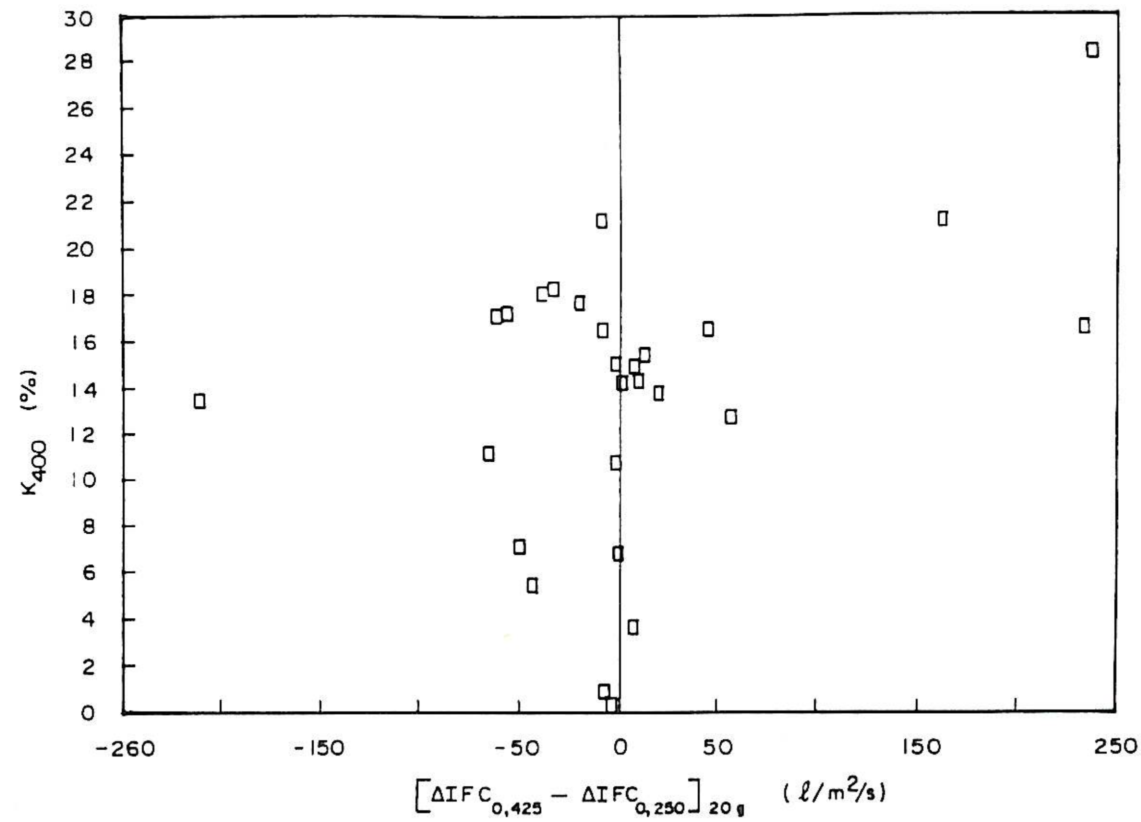


FIGURE 5.41

K_{400} VERSUS $[\Delta IFC_{0,425} - \Delta IFC_{0,250}]_{20g}$ (WARMBAD SOIL)

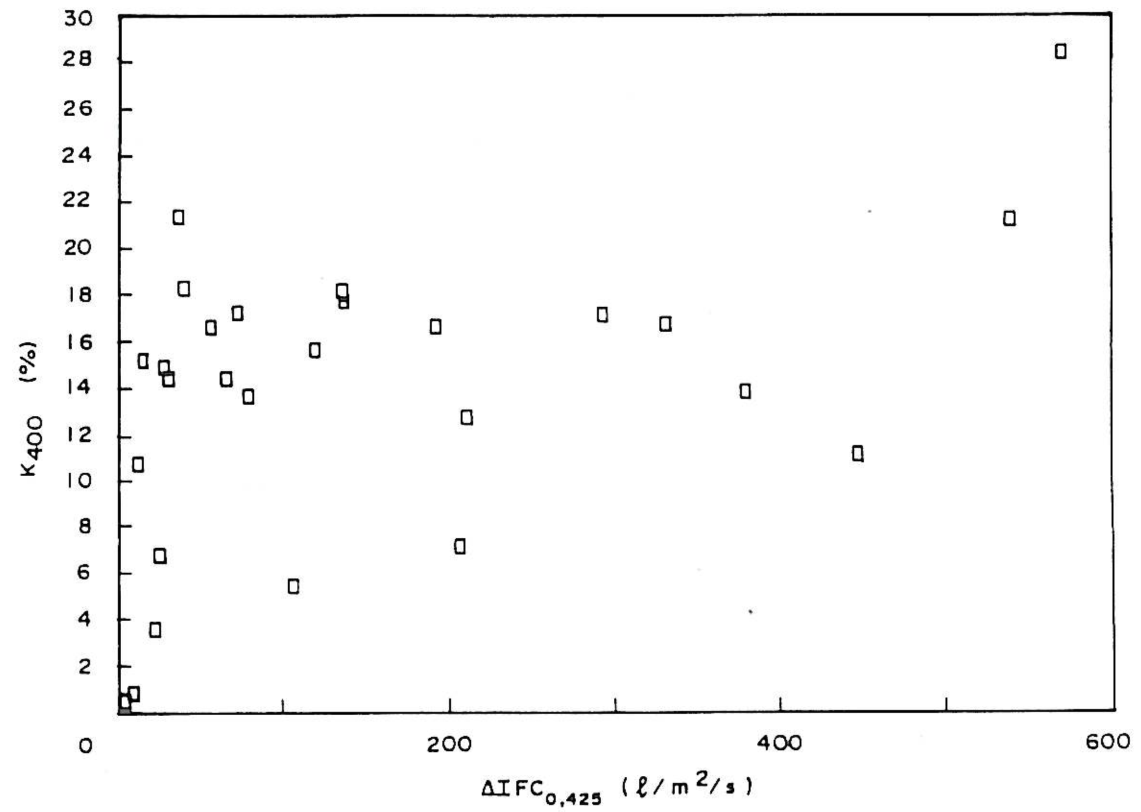


FIGURE 5.42

K_{400} VERSUS $\Delta IFC_{0,425}$ (WARMBAD SOIL)

from these tests would in fact not be any better than those obtained from in-isolation tests.

Reasons for the poor correlations can possibly be ascribed to the complexity of the soil/geotextile interaction, which is not adequately simulated by the IFC tests. Clogging of the geotextile is probably determined by the relationship between the pore matrix of the geotextile (shape, size and distribution of the pores) and the total grading of the soil, not only one or two particle sizes as is suggested in the IFC test theory. The time factor also contributes; the time in whcih the ICF tests are performed is not adequate to allow the soil/geotextile interaction to take place.

The IFC test is not recommended for the evaluation of soil/geotextile compatibility. Even if the test did predict long term performance adequately, the test is not a very practical one. Specialised, expensive equipment is needed and the sieving of soil particles into fractions is a tedious task. Although answers can be obtained much quicker than with the flow test, the cost of performing this test is higher than that of the flow tests.

5.9 CONCLUSIONS

The most important conclusions derived from the investigation reported in this chapter were as follows:

- Apparatus and a test method for the flow test were evaluated and standardised. The repeatability of the test was found to be good.

- The long term behaviour of soil/geotextile systems reported by previous researchers was confirmed. The permeability reduced initially and then stabilised after approximately 400 hours. The findings suppported the theory of a filter zone forming upstream of the filter, the permeability of which governed that of the system. The final permeability of the system (filter zone permeability) was less than that of the geotextile or the soil.

- Inadequate permeability was observed with some geotextiles. Continuous piping did not occur with any geotextile, but was induced with large opening meshes. Permeability and piping are discussed in more detail and are quantified in Chapter 7.

- The results obtained with granular filters were similar to those with geotextiles. These results are described in more detail in Chapter 7.

- Parameters to quantify flow test results were established. Based on results described in this chapter, as well as results from the field investigation (Chapter 6), flow test results after 400 hours were selected for quantifying flow tests. Although 1000 hour results could be predicted from F_{400}, better relationships were obtained between K_{400} and soil characteristics. K_{400} is also a less complex parameter and relatively easy to obtain and is recommended for quantifying flow test results.

- The results obtained with flow tests were more dependent on soil characteristics than on geotextile characteristics. Fine grained soils resulted in poorer performance (lower K_{400}) than coarse grained soils. Good relationships were obtained between flow test results and soil characteristics (e.g. percentage passing 0,0075 mm sieve) for individual geotextiles. Results obtained with different geotextiles, however, did not give relationships, indicating that these results were in fact influenced by the geotextile characteristics. These differences were attributed to a certain amount of clogging of some geotextiles.

- No relationships were discernible between flow test results and known in-isolation parameters of geotextiles. In-isolation tests are therefore not adequate for evaluating geotextiles and soil/geotextile compatibility tests should be incorporated in geotextile design criteria. The flow test described in this chapter is recommended for that purpose.

- IFC tests did not predict the long term performance of soil/geotextile combinations and are therefore not recommended as standard tests.

CHAPTER 6

SOIL/GEOTEXTILE COMPATIBILITY: FIELD EVALUATION

CHAPTER 6

CONTENTS PAGE

6.1 INTRODUCTION

The evaluation of soil/geotextile systems in the laboratory was described in Chapter 5. These results indicated that in-isolation tests of geotextiles are not adequate for geotextile selection and it was recommended that the flow test used for that investigation be incorporated in design criteria for geotextiles. In order to define a parameter with which to quantify flow test results, it was deemed necessary to evaluate a field subsurface drainage installation to obtain an indication of the relation between the laboratory tests and actual field conditions.

The field investigation was restricted to a single installation with only one soil/geotextile combination. The use of field investigations to evaluate various systems, such as described in Chapter 5, was considered, but was found to be impractical for a number of reasons:

- It would be extremely difficult to find an adequate number of sites, each with different soil/geotextile combinations but similar hydraulic conditions.

- It would not be practical, taking time and monetary restrictions into account, to investigate a large number of sites in different parts of the country.

- Soil and hydraulic conditions vary considerably over short distances in the field. For this reason, it would also be difficult to objectively compare different geotextiles in one installation.

For the reasons mentioned above, it was decided that the comparative study should be performed in the laboratory, under controlled conditions, and that the field investigation would only serve to put the laboratory tests into perspective.

It is envisaged, however, that more field installations will be investigated in future to gain more confidence with the laboratory test method and to refine the parameters that quantify flow test results. Field measuring equipment and techniques were developed with this in mind.

6.2 SITE LAYOUT AND DETAILS

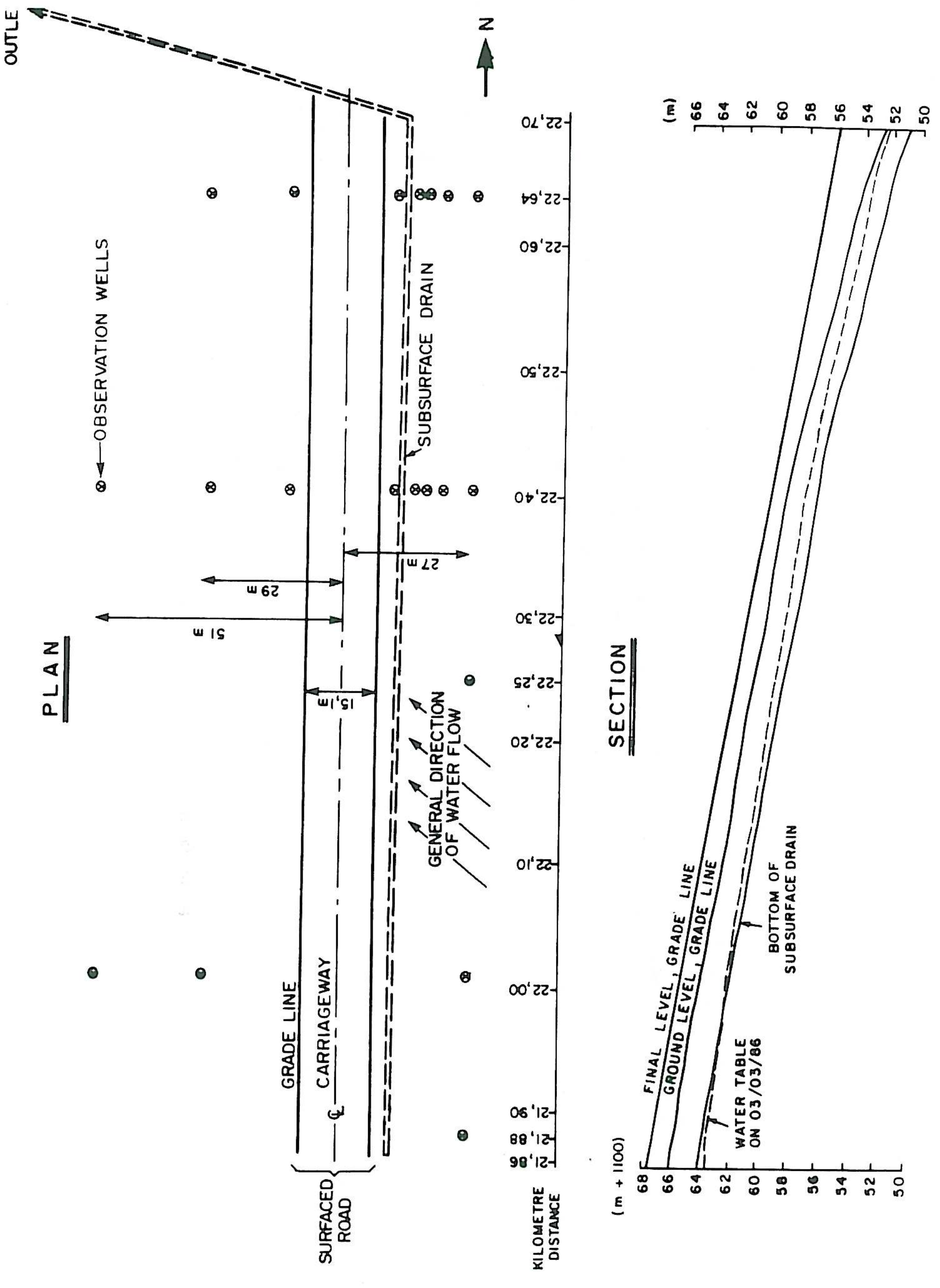
The site that was chosen to be monitored is on the N1/24 between Nylstroom and Middelfontein (North east of Warmbad) in the Northern Transvaal. The road was constructed in 1985/86 as part of the so-called Kranskop toll road.

A perched water table was encountered during the initial survey and during the earthworks phase of construction its extent was determined as being between km 21,86 to km 22,70 (840m). The general layout of the site is shown on Figure 6.1.

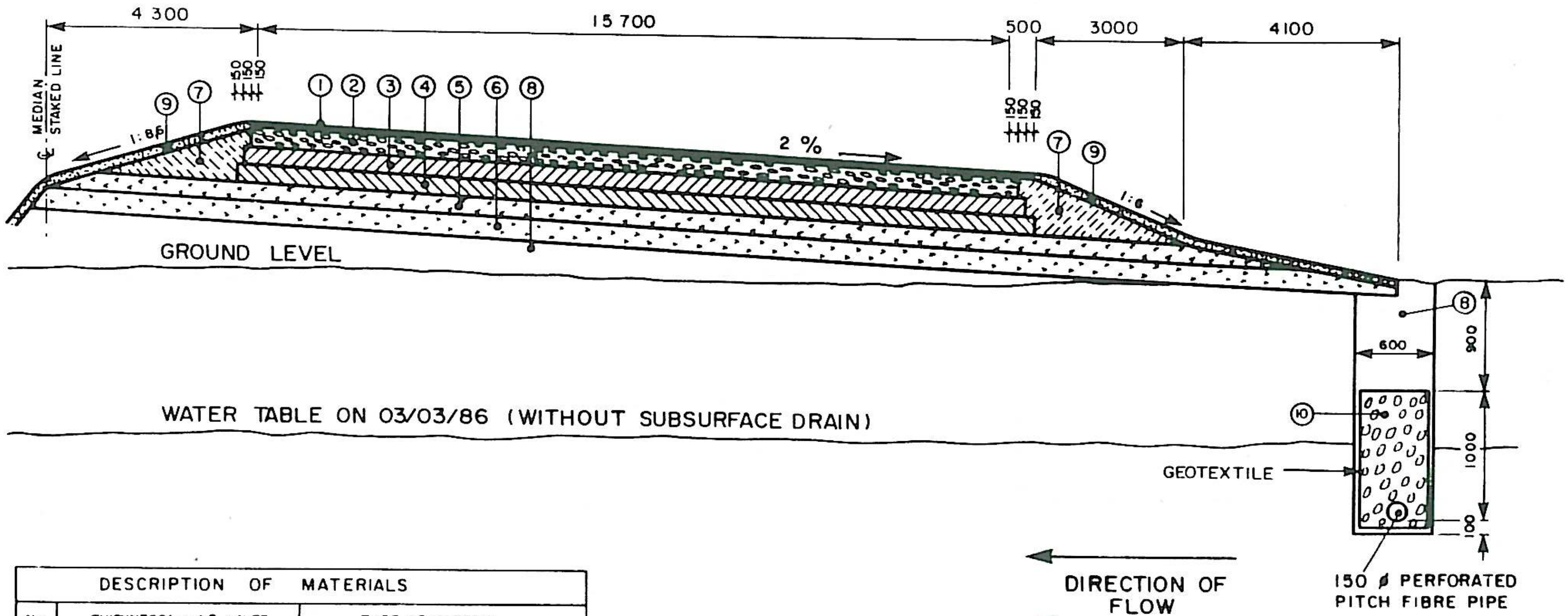
At the time of construction the water table was approximately 1m below the surface. A side drain was placed to a depth of 2m for the full length of the perched water table with an outlet at km 22,70. The side drain consists of a medium grade nonwoven geotextile (geotextile 3 as described in Section 3.2.5 and in Chapters 4 and 5) wrapped around 19mm aggregate with a 150mm pitch fibre pipe at the bottom. Details of the side drain are shown in Figure 6.2. The ground water flows in a general east to west direction and the side drain was placed on the upstream side of the road, as indicated on Figure 6.1.

This particular site was chosen for investigation for the following reasons:

- The type of subdrain used, consisting of a geotextile, natural aggregate and a perforated pipe, is widely used and specified by road authorities.



FICURE 6.1
LAYOUT OF SITE



DIRECTION OF FLOW

NOTE :

VERTICAL SCALE = 2,67 HORIZONTAL SCALE

DESCRIPTION OF MATERIALS		
No	THICKNESS (mm) & LAYER	TYPE OF MATERIAL
1	40 SURFACING (AS)	SEMI-GAP GRADED ASPHALT - 19mm NOM SIZE ROLLED IN PRECOATED CHIPS
2	150 BASECOURSE (G1)	CRUSHED STONE
3	150 UPPER SUBBASE (C3)	NATURAL GRAVEL
4	150 LOWER SUBBASE (C3)	NATURAL GRAVEL
5	150 UPPER SEL SUBGRADE (G7)	SAND OR NATURAL GRAVEL
6	150 LOWER SEL SUBGRADE (G9)	SAND OR NATURAL GRAVEL
7	SHOULDER	NATURAL GRAVEL
8	FILL	SAND OR NATURAL GRAVEL
9	TOP SOIL	TOP SOIL
10	FILTER MATERIAL	19 mm AGGREGATE

FIGURE 6,2

CROSS-SECTION AT km 22, 40

- The high water table resulted in water flowing through the system continuously. This was seen to be a fairly critical side drain application, placing high demands on the effectiveness of the geotextile. The average water table in South Africa is usually well below the surface (10m or more) and the vast majority of side drains are installed in cuts to remove ground water after a rain storm. Design criteria based on the field investigation described here, will therefore be conservative for road subsurface drainage.

- The system could be monitored from a very early stage, since the road was still under construction at the time when the investigation was started.

- The site is relatively close to Pretoria and frequent measurements could therefore be made.

6.3 MONITORING EQUIPMENT AND PROCEDURES

The aim of the investigation was to determine the amount of water that flowed through the geotextile compared to that in the laboratory and also to determine the long term performance (change in permeability with time) of the soil/geotextile system in the filed application. In order to obtain the required information, the height of the water table was monitored and the waterflow was measured at regular intervals.

Observation wells were placed along the upstream side of the side drain and at selected positions across the road, as indicated on Figure 6.1. Detail of an observation well is shown on Figure 6.3. The holes, 50mm diameter, were drilled and lined with perforated PVC pipes to support the surrounding material. Galvanized pipes at the surface were cast in concrete to ensure a permanent reference height. These were covered with lids to prevent material falling into the wells. Photograph 6.1 shows an observation well prior to casting the galvanized pipe in concrete.

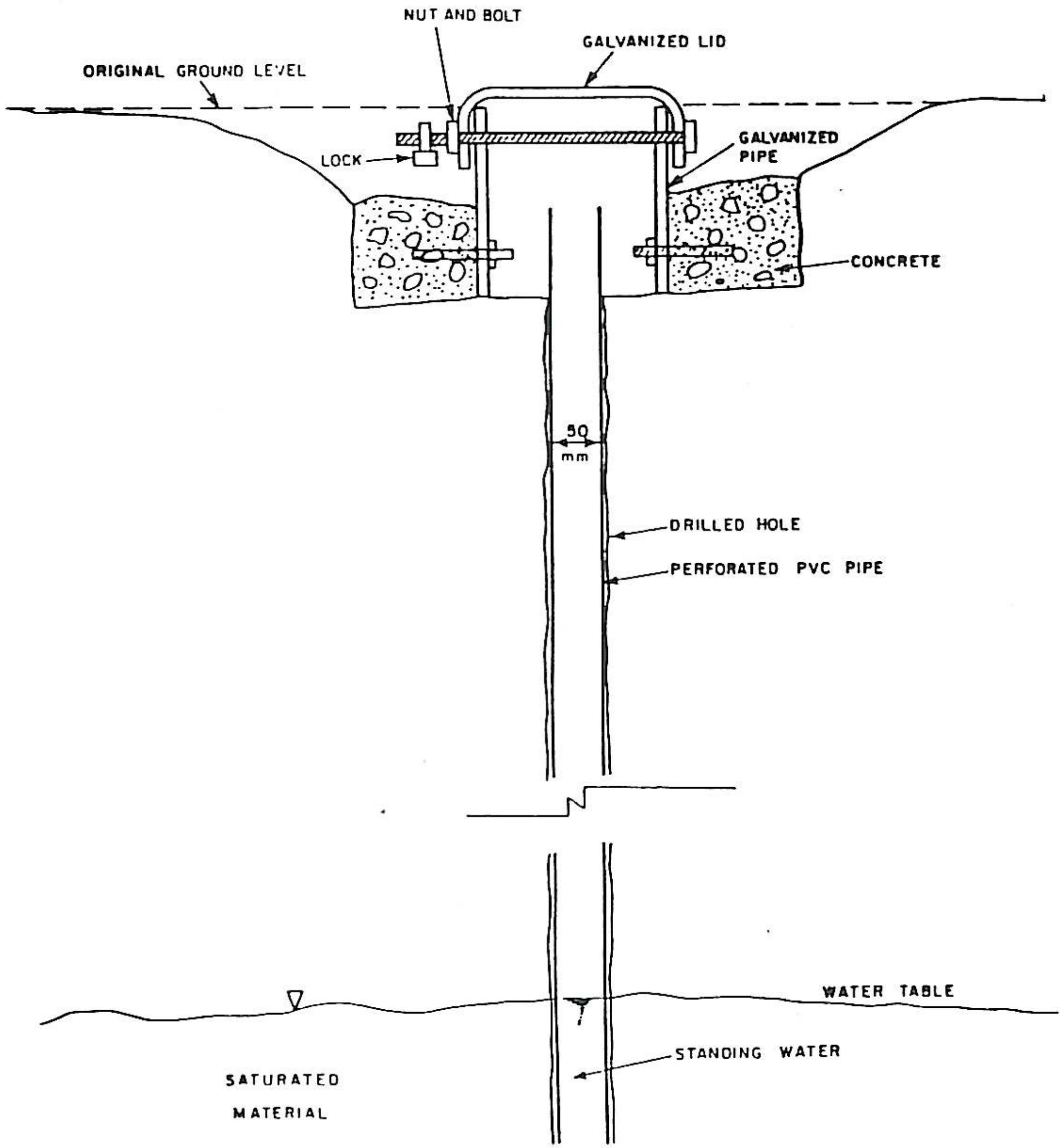


FIGURE 6.3
OBSERVATION WELL

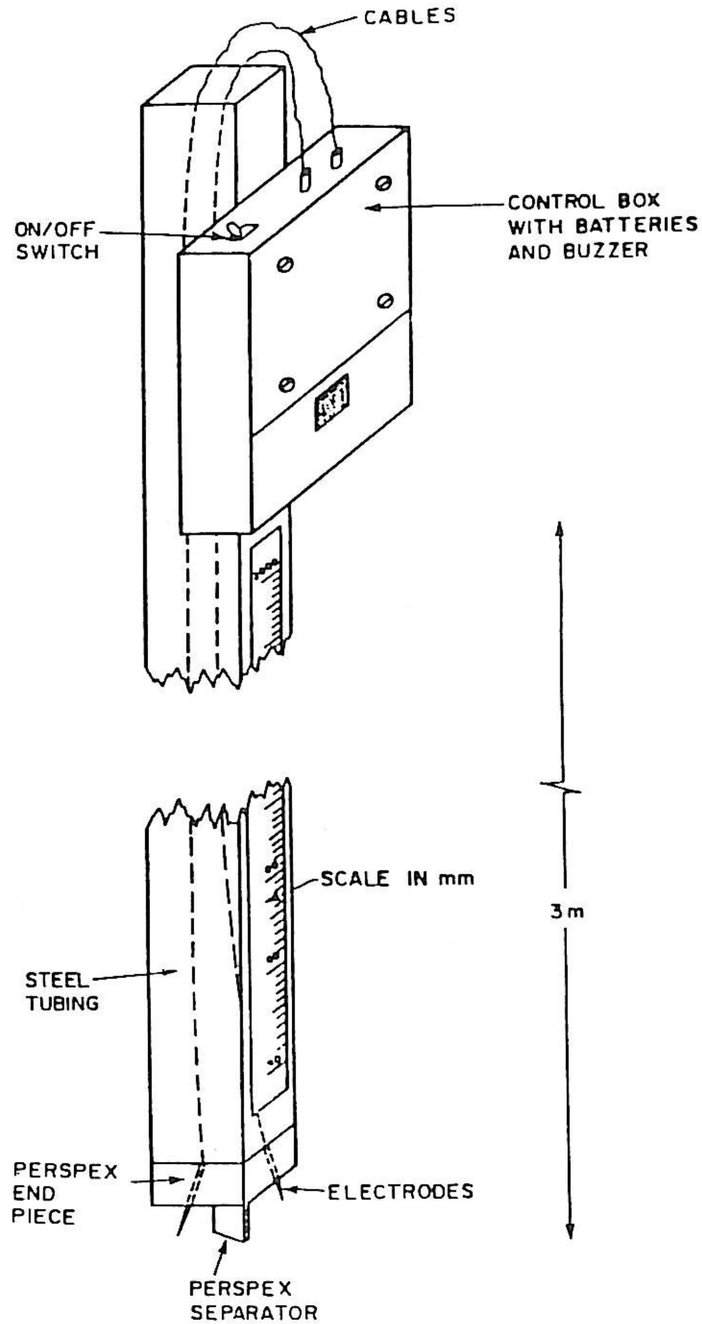
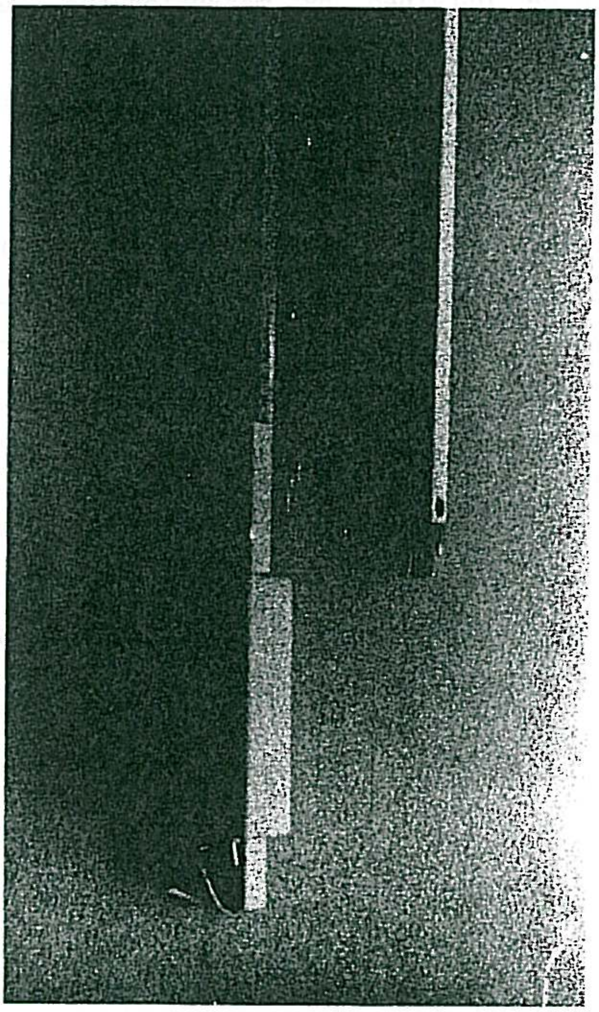


FIGURE 6.4
WATER DEPTH METER

PHOTOGRAPH 6.1
Observation well



PHOTOGRAPH 6.2
Water depth meter

A water depth meter was manufactured to measure the depth of the water table in the observation wells. Figure 6.4 and Photograph 6.2 show detail of this instrument. The meter is simply inserted in the observation well and when the electrodes touch free water, an electrical current flows through the water and a buzzer sounds in the control box. The depth is then read off the scale, using the top of the galvanized pipe (Figure 6.3) as reference.

The water flow was measured at the outlet for which a Vee-notch weir was manufactured. Details of this equipment is shown on Figure 6.5. The weir consists of a pitch fibre pipe with an end plate which, when placed horizontally as an extension of the existing outlet pipe, acts as a weir. The water then flows through the V-notch in the end plate. The height of the water level in the weir, H (See Figure 6.5) can then be obtained by measuring the height (h) through an opening placed back from the outlet. The water is drawn downwards at the outlet and accurate measurements cannot be made at that position.

Empirical formulae exist (Russel, 1941) with which the flow can be calculated for V-notch weirs. Different formulae exist for 30°, 60° and 90° V-notches. Smaller angles result in more accurate results for smaller volumes of water. In this investigation 30° and 60° notches were used. The formulae for a 30° notch is shown on Figure 6.5.

The Vee-notch weir was calibrated in the laboratory prior to using it in the field. The difference between actual flow and that obtained with the weir was found to be approximately 5 per cent. Photographs 6.3 and 6.4 show laboratory calibration of and field measurement with the Vee-notch weir respectively.

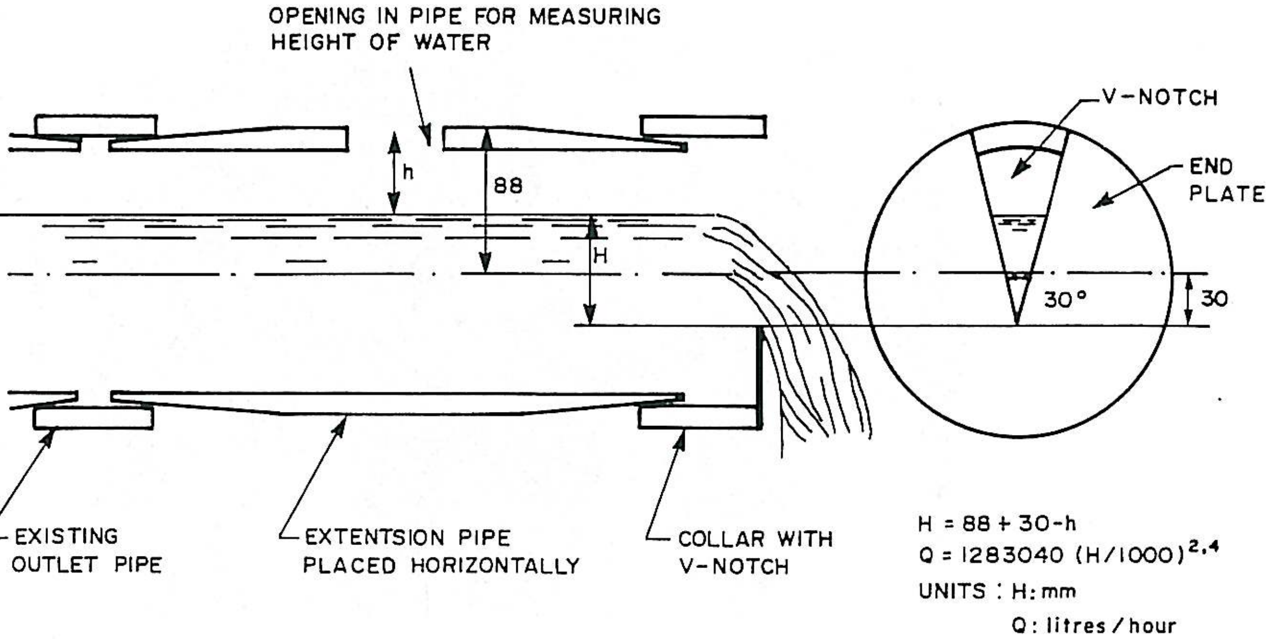


FIGURE 6.5

VEE-NOTCH WEIR

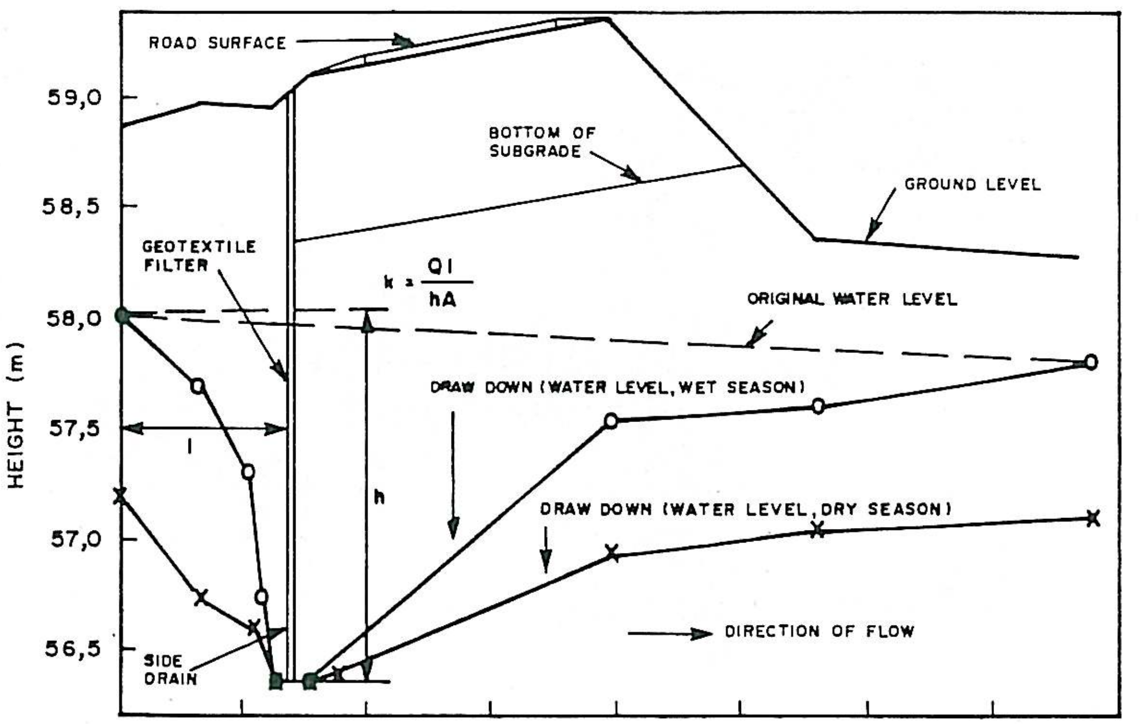
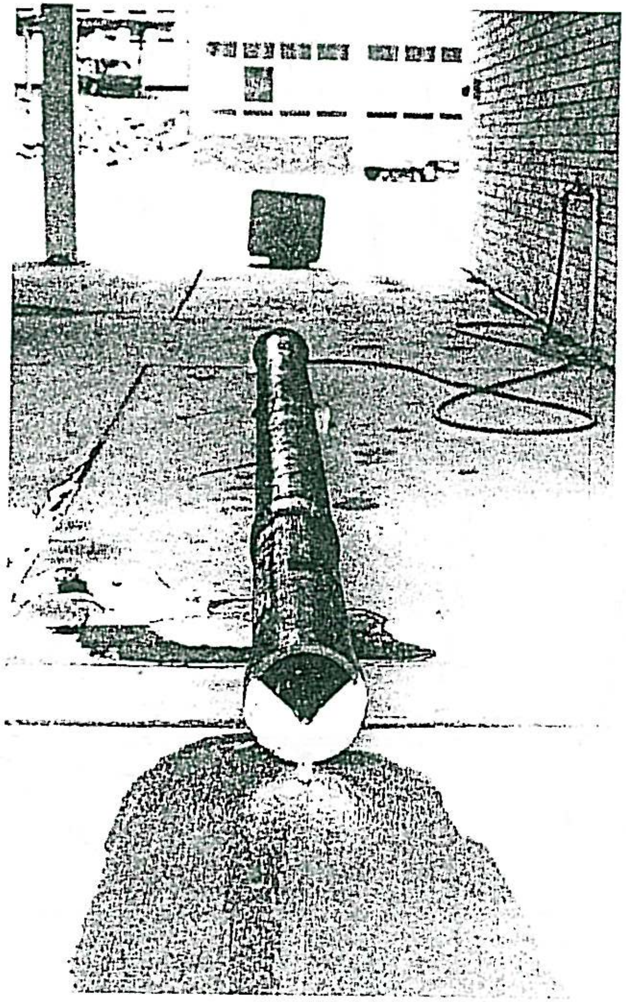


FIGURE 6.6

TYPICAL CROSS-SECTION THROUGH ROAD AND SIDE DRAIN



PHOTOGRAPH 6.3
Laboratory calibration of Vee-notch weir

PHOTOGRAPH 6.4
Outflow measurement with Vee-notch weir

6.4 RESULTS AND ANALYSIS

The observation wells across the road made it possible to determine the drawdown of the water table as a result of the side drain as indicated on Figure 6.6. With this information it was possible to calculate the permeability coefficient of the system, as follows:

$$K = \frac{Q}{iA}$$

where Q = total flow measured at the drain outlet
i = hydraulic gradient using h and ℓ , as indicated on Figure 6.6.
A = total area through which the water flowed (calculated for the full length of the side drain)

The total area, A, was calculated from the measurements obtained with the observation wells along the length of the side drain. These wells were all placed at a distance of 13,5m upstream of the side drain (See Figure 6.1). The corresponding water table height on Figure 6.6 is the first measurement indicated (distance 0m). This means an assumption that these observation wells are at a position where the water table is still at its original level, without being affected by the side drain. This is not strictly true, but observation wells could not be placed further upstream, since that would have placed them outside the road reserve in private property where they could not be maintained on a permanent basis. A preliminary investigation of the height of the water table indicated the error to be small, however, resulting in slightly more concervative results.

A summary of the results measured and calculated as described above, is shown in Table 6.1 below.

TABLE 6.1 Summary of field investigation results

Date	Total outflow Q (ℓ/hour)	Total area Submerged A (m^2)	Permeability coefficient k ($m/s \times 10^{-6}$)
19/6/1985	594	105	
10/12/1985	328	187	
15/1/1986	68	252	
23/1/1986	201	213	
31/1/1986	482	194	
6/2/1986	279	241	
13/2/1986	168	268	
3/3/1986	1 468	560	
26/8/1986	124	58	10,6
9/12/1986	2 477	962	5,0
6/1/1987	3 811	860	8,9
25/2/1987	1 100	462	7,5
8/5/1987	1 006	466	7,5
26/5/1987	301	245	5,9
24/7/1987	238	182	7,1
28/8/1987	99	138	4,0
7/10/1987	238	259	3,2
4/11/1987	372	290	4,3
5/1/1988	2 476	1 025	7,6

The permeability coefficient, k, could not be obtained before August 1986, since the road was still under construction at that time and the cross section observation wells could not be placed.

The change in permeability with time is shown on Figure 6.7. Although the results show a fair amount of scatter, it is evident that there is a downward trend in permeability. The permeability reduction curve for the same soil/geotextile combination (soil W, geotextile 1) obtained in the laboratory (as described in Chapter 5) is shown on Figure 6.8. The scatter in the results obtained from the field investigation is probably due to the way in which the total area, A, was calculated and the fact that this area and the water level fluctuate. In the laboratory the geotextile is submerged permanently and the filter zone forms in one position on a

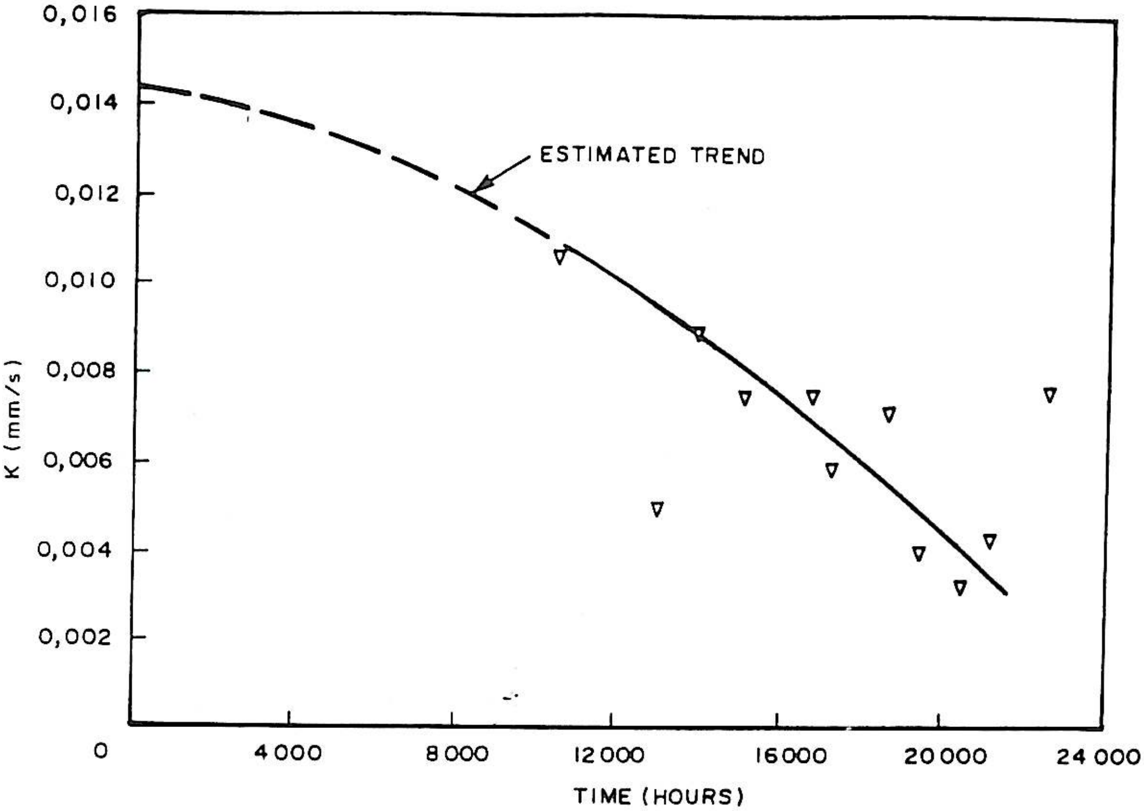


FIGURE 6.7

PERMEABILITY REDUCTION OF FIELD INSTALLATION

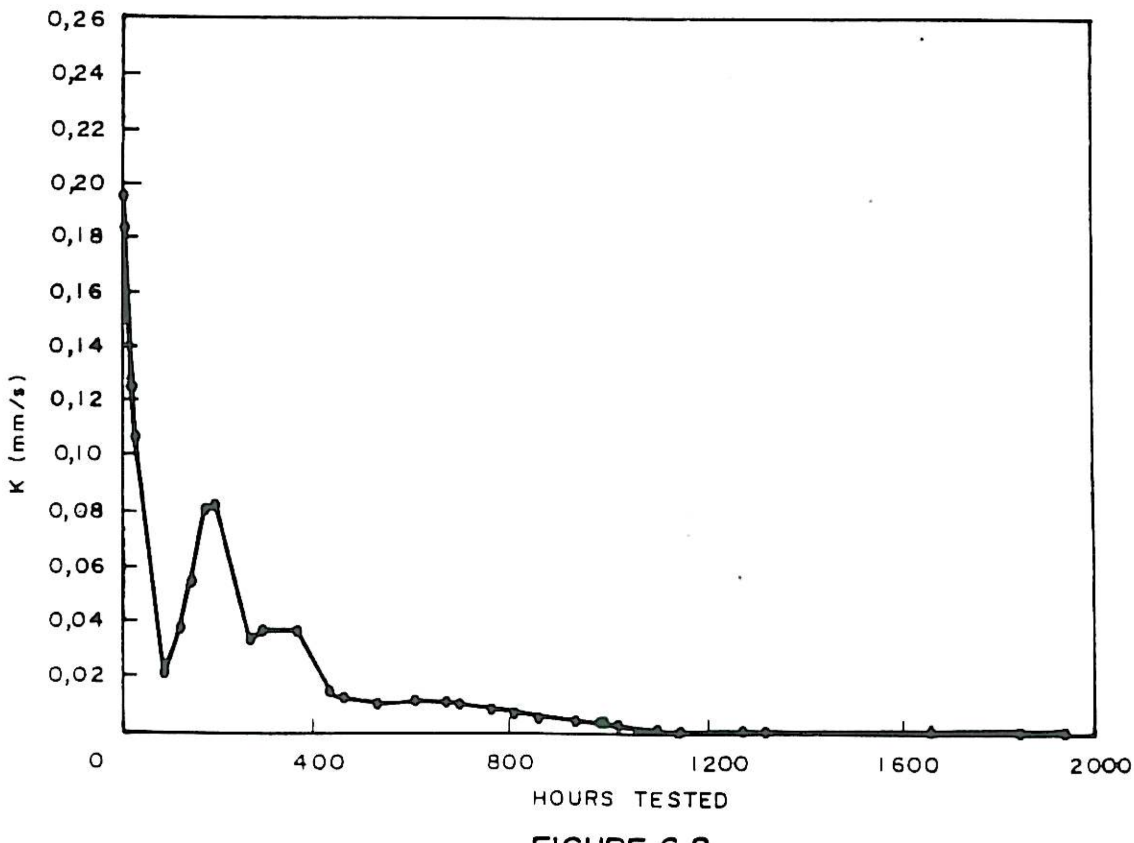


FIGURE 6.8

LABORATORY PERMEABILITY REDUCTION (SOIL: W, GEOTEXTILE 1) continuous basis. In the field, the area of the geotextile and adjacent soil that is subjected to flow, varies with the fluctuating water level and the filter zone in the soil is formed intermittently in different positions.

Despite these inaccuracies, it is evident that there is a reduction in permeability in the field similar to that obtained in the laboratory. It is not possible, however, to establish the relation between the field results (Figure 6.7) and the laboratory results (Figure 6.8) by merely comparing the two figures. In order to obtain such a relation, a comparison was made between the total amount of water that flowed through the geotextile per unit area in the field, and that which flowed through the geotextile in the laboratory.

The total amount of water that flowed through the system in the field over the measuring period of 22 500 hours was 24 000 m^3. This figure was obtained by calculating the area under the flow curve as described earlier for the laboratory flow tests (Section 5.4.2). The average area of geotextile that was submerged over the measuring period was 371m^2 The total amount of flow per unit area over the measuring period was therefore 24000/371 = 64,5m^3/m^2. In the laboratory, for the same soil/geotextile combination, that amount of water (64,5 m^3/m^2) flowed through the system after 10,7 hours of testing (obtained from Figure 6.8).

These results indicate that the laboratory tests were in fact accelerated tests, if compared to this particular field installation. The flow rate per unit area in the laboratory was 2100 times higher than the average rate in the field. The hydraulic gradient used in the laboratory was 100 times greater than that encountered in the field. Using the ratio of 2100 and extrapolating for the field condition, the following equivalent laboratory testing times and field service periods are obtained.

Field Service (years)	Laboratory testing (hours)
20	83
50	166
100	415
250	1 037

A road is usually designed for a life of 20 to 30 years. To counteract all the unknown factors, assumptions and varying field conditions, however, a large safety factor is desirable and on this basis a field service of 100 years was decided upon to use as a basis for quantifying the laboratory tests. This is equivalent to approximately 400 hours of laboratory testing.

6.5 ROAD PERFORMANCE

The ultimate test for the effectiveness of a road subsurface drainage installation is the performance of the section of road that is protected by that installation.

The performance of the section of road described in this chapter has been monitored by measuring the riding quality with the PCA Road meter. This instrument measures Present Serviceability Index (PSI) which is a function of the amount of movement between the rear axle and the body of the car in which the instrument is mounted. Riding quality is usually measured at a speed of 80km/h.

The PSI of a road usually decreases over time as the riding quality becomes poorer as a result of traffic. In order to determine the effectiveness of the subsurface drainage installation it was therefore necessary to compare the change in PSI of the section of road with the side drain to that of the road adjacent to the high water table area. If the drainage system is not effective, excess moisture will enter the road structure resulting in a more rapid decrease in riding quality of the drained section of road than that of the adjacent road sections.

Riding quality was measured in the slow lanes of both the north bound and south bound lanes. Each measurement consisted of four runs in each direction and the average value was used. A summary of the results is shown in Table 6.2 below.

TABLE 6.2 Summary of riding quality measurements

Layout:

km: 20,8 21,9 22,7 23,8

	→ North	
	South ←	

----
side drain

	"Dry" Sections (km20,8 - 21,9 and 22,7 - 23,8)		Drained Section (km21,9 - 22,7)		PSI (Drained) / PSI (Dry)	
Date	North (PSI)	South (PSI)	North (PSI)	South (PSI)	North	South
2/9/1986	3,928	3,860	4,005	4,024	1,020	1,042
4/11/1987	4,048	3,992	4,262	4,014	1,053	1,005
3/6/1988	4,016	3,915	4,319	4,056	1,075	1,036

The results under "Dry Sections" in Table 6.2 refer to the PSI results obtained on the sections of road adjacent to the high water table area ie 1100m north and 1100m south of the drained section (average of the two 1100m sections). To enable a comparison to be made between the drained section and the dry sections, the ratio PSI (Drained)/Psi (Dry) is also shown. If the drained section is deteriorating at a higher rate than the adjacent sections (Dry sections) then this ratio will decrease over time.

It is evident from the results in Table 6.2 that the changes in both riding quality (PSI) and the PSI ratio are small and that the riding quality of both the drained and the "dry" sections have not changed significantly over the two year period. The slight increase of the ratio between 1986 and 1987 for the southbound lane can therefore be attributed to differences in the calibration of the measuring device rather than a deterioration of the drained road section. This is supported by the fact that a slight increase was evident over the next few months and also by the fact that the riding quality increased slightly on all the sections over the two year period. An increase in riding quality is in fact almost impossible.

The results indicate therefore that neither the drained nor the "dry" sections have deteriorated, from which it can be concluded that the subsurface drainage installation is successful to date. The monitoring of this road wil be continued and should the side drain lose its effectiveness in future, the causes of failure will be further investigated and, if applicable, incorporated in the design criteria described in Chaper 7.

The performance evaluation described above gives an indication of the effectiveness of the drainage installation as a whole, and not of individual components eg. the soil/geotextile compatability. Should the installation therefore prove to be ineffective, it will have to be investigated further to determine the cause of failure. An indication of the cause of failure can be obtained, however, by monitoring the road performance on both slow lanes, as was done for the section described above. If the drain becomes blocked due to soil/geotextile incompatability or any other reason, water will enter the pavement from the upstream side and the deterioration will most probably be more pronounced on the southbound slow lane for this particular road. If, on the other hand, the drawdown of the water table is not sufficient resulting from a too shallow side drain, the downstream side ie. the northbound slow lane will more than likely be the most seriously affected by moisture ingress.

It should be noted, however, that the effectiveness of the geotextile in this particular installatin can probably be monitored more effectively by measuring the position of the water table at regular intervals, as described in Section 6.3 and 6.4 above. The method of monitoring the road performance can be applied to road subsurface drainage installatins not instrumented with observation wells.

6.6 CONCLUSIONS

- For practical reasons it was not possible to use field investigations to evaluate the performance of different soil/geotextile combinations. A single field installation was monitored to obtain an indication of the relation between laboratory tests and field applications.

- The site that was chosen for this investigations was a fairly extreme example with continous flow and it is expected that the criteria based on the results obtained will therefore be conservative.

- A decrease in permeability of the soil/geotextile system, similar to that obtained in the laboratory, was observed in the results obtained from the field installation.

- The flow rate per unit area of geotextile in the field application that was monitored was 2100 times less than that of the laboratory flow tests. A laboratory test of 400 hours is therefore equivalent to approximately 100 years of service in the field. On this basis a testing time of 400 hours was adapted for the laboratory tests.

- Riding quality measurements indicated that the subsurface drainage installation has performed efffectively for the past two years.

- It is recommended that the monitoring of the subsurface drainage installation described in this chapter be continued and that similar installations elsewhere also be monitored with the equipment and procedures that have been developed.

CHAPTER 7

GEOTEXTILE FILTER CRITERIA

CHAPTER 7

CONTENTS PAGE

7.1 INTRODUCTION

The main aim of the geotextile investigation described in this document, was to establish practical filter criteria for road subsurface drainage. The shortcomings of existing filter criteria for geotextiles were discussed in Chapter 3 and illustrated in Chapter 5. These include the fact that the criteria are based on in-isolation characteristics and do not address the long term performance of soil/geotextile systems, the permeability reduction that occurs in such systems and the potential for clogging.

In this chapter, filter criteria are recommended which are based on the soil/geotextile compatibility tests that were performed as described in Chapter 5. In addition to piping and permeability criteria, a third criterion is recommended namely the permeability reduction criterion. The latter is aimed at preventing the use of soil/geotextile combinations that will result in clogging and is probably the most important of the three criteria.

Each of the three criteria can be evaluated by performing a laboratory flow test using a permeameter as described in Chapter 5, and as such this test is recommended as the best method for selecting geotextiles. However, an attempt has been made in Sections 7.2 and 7.3 below, to establish piping and permeability criteria based on in-isolation characteristics.

The criteria based on in-isolation characteristics should be viewed as tentative guidelines, however, and the final selection should be based on soil/geotextile compatibility tests, since in-isolation characteristics cannot predict clogging potential and also due to the difficulties associated with measuring in-isolation characteristics.

7.2 PIPING

The piping criterion is aimed at preventing the soil from passing through the geotextile continously. For the filter zone to form in the soil, it is inevitable that a certain amount of piping will occur if there is fine material present in the soil. The piping criterion should limit the amount of soil passing through the geotextile to acceptable quantities.

7.2.1 Analysis

Continuous piping was not evident with any of the soil/geotextile combinations tested. Since piping could not be induced using geotextiles, flow tests were performed with open meshes of varying sizes in combination with the soil types described in Section 5.3.1. These were steel meshes with single opening sizes that are used for laboratory soil sieves.

Piping was gauntified by visual observation of the water flowing out of the permeameters and when there was no discolouration of the water, piping was assumed to have stopped. With the majority of soil/geotextile combinations this occured within the first 30 minutes, but with the finer soils, notably Silverton soil, piping continued for up to 4 hours with some geotextiles. Using the open meshes, piping was evident for up to 8 hours with the largest size meshes that did not result in continuous piping. With those meshes that were defined as resulting in continuous piping, piping occured until a cavity had formed in the soil or until the whole soil sample had washed through the mesh.

As mentioned above, piping should be restricted to an acceptable limit, which, until quantified by field observations, is a subjective concept. The limitation in field applications is that the amount of soil passing through the filter should not be sufficient to block the downstream elements of the drainage system eg. the drainage pipes. Comprehensive field investigations were not possible, as discussed in Section 6.1, and the piping limits given below may therefore be altered if future field observations prove the necessity. Visual observations of the laboratory tests did indicate, however, that the amount of soil that was deposited in the outlet pipes was minimal with all the geotextiles as well as with those meshes that did not result in continuous piping (ie. less than 8 hours of discoloured water). Based on these observations it is therefore recommended that the piping criterion for the permeameter tests should be that the visual discolouration of the water should not continue for longer than 8 hours.

The opening sizes of the meshes that were used to induce piping were not measured since they are standard laboratory sieve meshes of which the opening sizes are controlled and certified by the manufacturers. They could be measured, however, by the image projection technique described in Section 3.3.2. The meshes have a single opening size which could be termed O_{100}, but this implies that there are other, smaller opening sizes. Calhoun (1972) used the term effective opening size (EOS) for meshes with single opening size (See Section 3.3.4.1). The term "effective" can be misleading, however, and Rankilor (1981) interpreted EOS to be equivalent to O_{50}. Since it is the largest (maximum) opening size that should be restricted to avoid piping, the author prefers to use the term maximum opening size (MOS). A summary of the results obtained with the open meshes is given in Table 7.1 below, showing the MOS of the largest mesh that did not result in continuous piping for each soil type.

TABLE 7.1 : SUMMARY OF PIPING TESTS WITH OPEN MESHES

SOIL		MESH MOS (mm)	$MOS/D_{85}S$
TYPE	$D_{85}S$ (mm)		
M	0,311	0,850	2,73
W	0,405	0,850	2,10
S	0,411	1,180	2,87
B	0,599	2,00	3,34
R	2,971	3,00	1,01

The parameter to quantify the soil, namely $D_{85}S$, is the same that is used in the granular filter criteria. The criterion to restrict piping for granular filter is:

$$\frac{D_{15}f}{D_{85}S} \leq 5$$

The results in Table 7.1 show that, for coarse grained soils ($D_{85}S$ = 3mm), the ratio $MOS/D_{85}S$ should be equal to or less than approximately 1,0. For the finer grained soils ($D_{85}S$ < 0,6mm), a larger maximum opening size can be allowed with $MOS/D_{85}S$ between 2 and 3. Soils with $D_{85}S$ between 0,6 and 3,0mm were not tested and are assumed to require a $MOS/D_{85}S$ ratio of less than 1,0 (a conservative assumption since the actual value is probably between 1,0 and 3,0). Based on these values, piping criteria using in-isolation characteristics were derived and are given in Section 7.2.2 below.

It should be noted that the opening size (MOS) was (or could have been) determined with the image projection method and not by inverse sieving. The latter is not recommended for reasons mentioned previously. The results obtained in this investigation indicate that if the opening sizes of a geotextile are too small to be measured with the image projection method, which will in fact be the case with most available geotextiles, then continuous piping is not likely to occur and the opening sizes need not be determined.

7.2.2 Recommended piping criteria

The criteria based on in-isolation characteristics ((a) below) should only be used as a first indication of possible continuous piping. The final geotextile selection should be based on the flow test criterion. ((b) below).

(a) In-isolation parameters:

If $D_{85}S \leq 0{,}6mm$ then:

$$MOS \leq 2 \times D_{85}S$$

If $D_{85}S > 0{,}6mm$ then:

$$MOS \leq D_{85}S$$

(b) Flow test:

Discolouration of the water should not be visually discernable after 8 hours of testing.

7.2.3 Evaluation of existing piping criteria

The piping criteria recommended by the various researchers as described in Section 3.3.4 were evaluated against the criteria given above and the results obtained with the flow tests.

The criteria given by Calhoun (1972) can be compared directly with the recommended criteria since that researcher used EOS obtained with the image projection method which is basically the same as MOS. Calhoun's piping criteria are:

If $D_{50}S \geq 0,075mm$
then $EOS \leq D_{85}S$
If $D_{50}S < 0,075mm$
then $EOS \leq 0,212mm$

Four of the five soils tested fall into Calhoun's first category ($D_{50}S < 0,075mm$). Of these, one soil is in the authors second category ($D_{85}S > 0,6mm$) and the author's and Calhoun's criteria are identical for that particular soil. For the other three soils, the criterion given by Calhoun is more conservative. One of the five soils tested falls into Calhoun's second category and for that particular soil, Calhoun's criterion is more conservative.

The piping criteria given by Calhoun are therefore very similar to those recommended in Section 7.2.2 above, with Calhoun's criteria being slighly more conservative.

The piping criteria given by all other researchers as described in Section 3.3.4 are based on opening sizes determined with the inverse sieving method. For each soil/geotextile combination tested (as summarized in Table 5.6) those combinations that should have resulted in piping according to the various criteria were identified, using the opening sizes measured with the inverse sieving test described in Section 4.2.1 (results given in Table 4.3). The in-isolation characteristics of soils and geotextiles used in this investigation did not always fall within the ranges given by the various criteria and not all tests could therefore be evaluated. In some cases none of the materials fell within the specified ranges. A summary of the evaluation is shown in Table 7.2 below. Shown are the number of tests that could be evaluated and the number of tests (soil/geotextile combinations) that should have resulted in piping according to the various criteria.

TABLE 7.2: EVALUATION OF EXISTING PIPING CRITERIA

CRITERIA	NUMBER OF TESTS THAT COULD BE EVALUATED	NUMBER OF TESTS THAT SHOULD HAVE RESULTED IN PIPING ACCORDING TO THE CRITERION
Zitscher (1975)	56	0
Ogink (1975)	85	10
Giroud (1982)	85	24
Heerten (1982)	85	10
Ragutzki (1973)	85	49

As was mentioned earlier, none of the tests using geotextiles did in fact result in continuous piping. The number of tests predicted to result in piping by the various criteria also differed widely, ranging from 0 to 49. This evaluation confirms the fact that criteria based on in-isolation characteristics and particularly those based on the inverse sieving test should not be used for geotextile selection.

7.3 PERMEABILITY

The permeability criterion ensures that the permeability of the filter (geotextile) is adequate to avoid the build up of excess pore water pressures. The limitation of excess pore water pressure or pressure build up only needs to be applied if the permeability of the filter is less than that of the soil.

7.3.1 Analysis

The results from two flow tests are shown on Figure 7.1. The two tests were performed with Warmbad soil and geotextiles 3 and 35. It is evident from the figure that the initial

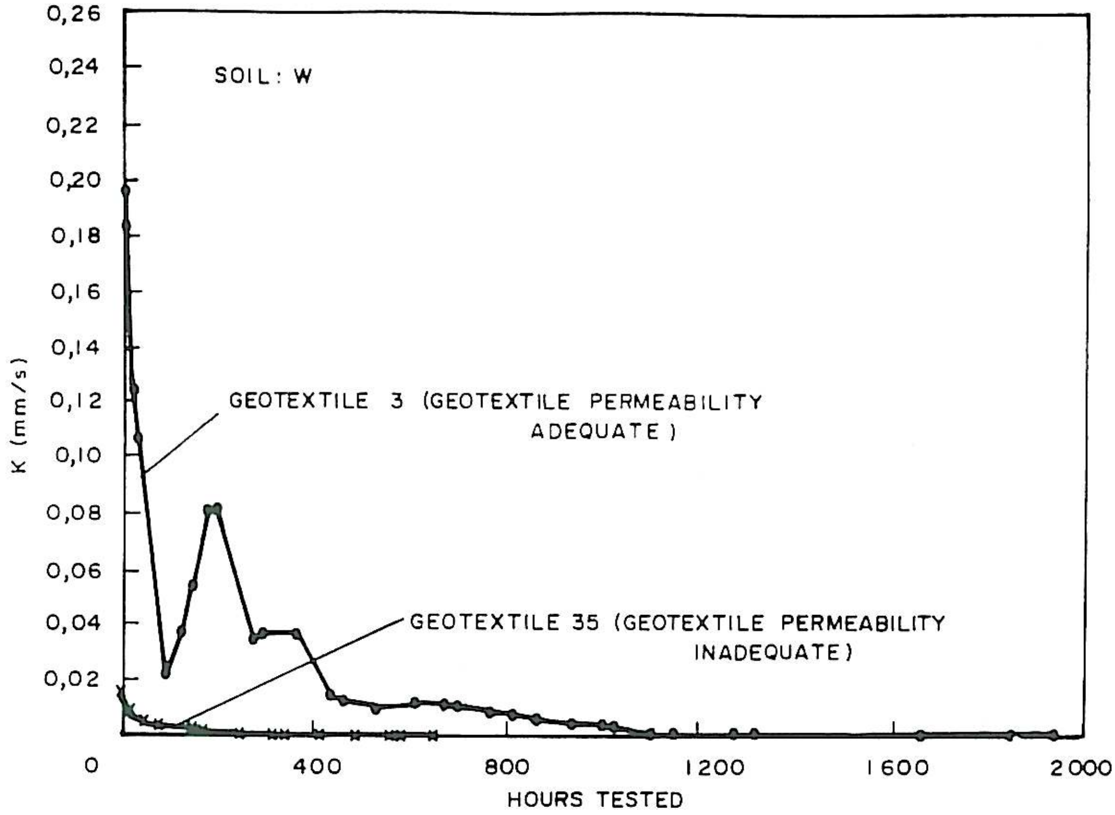


FIGURE 7.1

FLOW TEST RESULTS INDICATING GEOTEXTILE WITH INADEQUATE PERMEABILITY

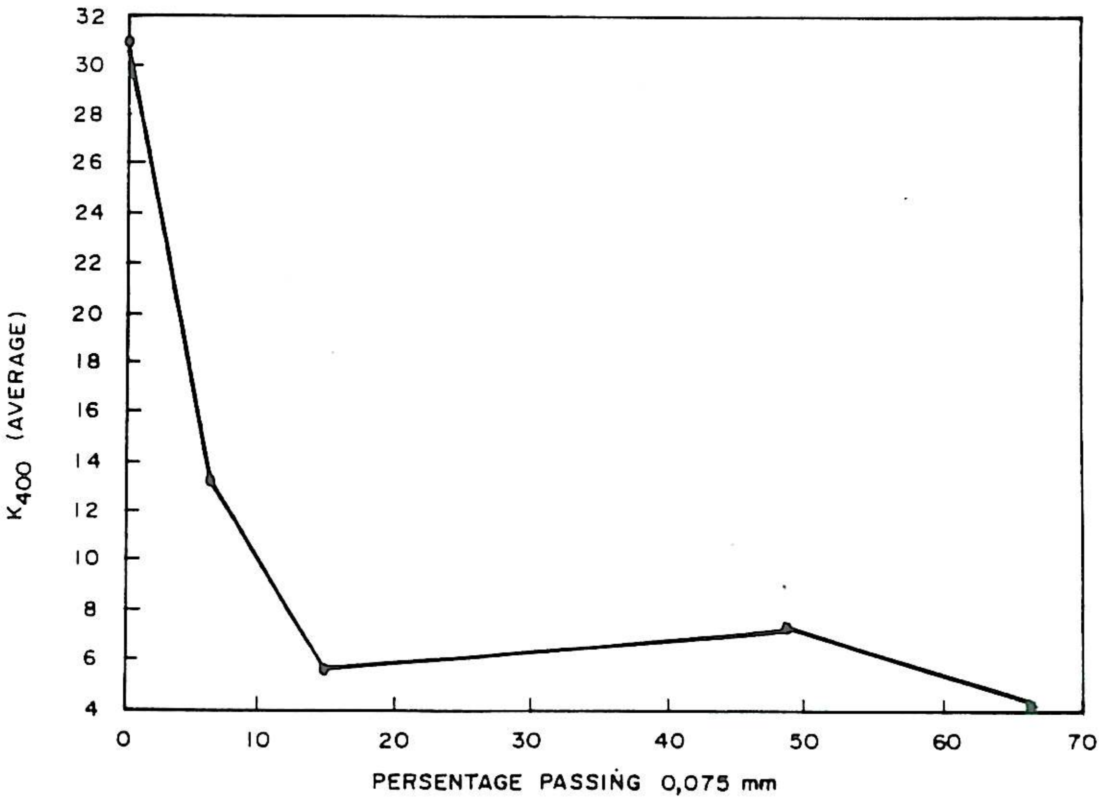


FIGURE 7.2

AVERAGE VALUES OF K_{400} OBTAINED WITH DIFFERENT SOILS permeability of the system with geotextile 3 was higher than that of geotextile 35. The initial permeability obtained with geotextile 3 was in fact that of the soil. The goetextile permeability was higher than that of the soil and did therefore not affect the initial permeability or result in any pore pressure build up. With geotextile 35, however, the permeability of the system was that of the geotextile, which was considerably lower than that of the soil. The permeability of this system continued to be less than that with geotextile 3 for the duration of the test.

The results obtained from all the tests performed, as described in Chapter 5, indicated that the highest permeability was obtained at the start of the test and that the permeability decreased thereafter, as a result of the forming of a filter zone in the soil. It follows then that, if the permeability of the goetextile is equal to or larger than that of the soil, then the initial permeability of the system will be that of the soil and the permeability of the geotextile is therefore adequate.

For this criterion to be applied, the permeability of the soil must be known. This is easily determined, however, by performing a flow test with a geotextile or a porous disk which is known to have a permeability higher than that of the soil.

It should be noted that very few geotextiles tested did not have adequate permeability. The permeability of geotextiles 34 and 35 was less than that of all the soils tested. These geotextiles are thick woven geotextiles that are normally not considered for any drainage applications and are mainly used for reinforcement. Geotextile 29 was found to have a lower permeability than the washed river sand. Although this geotextile is often used in subsurface drainage, the river sand was not a natural soil and the fine material was purposely washed out to obtain a high permeability soil.

Table 7.3 shows a summary of the soil and geotextile permeabilities for the two cases where the geotextile permeability was too low.

TABLE 7.3: EVALUATION OF GEOTEXTILE PERMEABILITIES

	GEOTEXTILE					SOIL
Permeability status	No	γm ($\ell/m^2/s$)	γs ($\ell/m^2/s$)	Kc (mm/s)	Kp (mm/s)	Ks (mm/s)
(a) Washed River Sand						R:0,754
Adequate	27	56	24,8	0,348		
Inadequate	29	36	19,4	0,274	0,006	
(b) Other Soils						W:0,182
Adequate	31	17	8,4	0,288		S:0,061
Inadequate	35	15	5,8	0,181	0,0149	M:0,053
Inadequate	34	6	2,3	0,038	0,0006	B:0,022

The permeabilities of geotextiles shown that were "adequate" (geotextiles 27 and 31) are of those geotextiles with the lowest permeabilities that were sufficiently high to be more than that of the soil.

Other values shown on Table 7.3 are as follows:

γm: Geotextile normal permeability as measured with the falling head permeameter (Section 4.2.2, Table 4.5).

γs: Geotextile normal permeability with SABS constant head permeameter (Section 4.2.2, Table 4.5).

Kc: Permeability coefficient of geotextile calculated using γm and geotextile thickness (geotextile thickness as described in section 4.3.2, Table 4.7).

Kp: Permeability coefficient of geotextile as obtained from those flow tests where the permeability of the soil/geotextile system was less than that of the soil.

Ks: Permeability coefficient of the soil as obtained from the flow tests (initial permeability, average value of all tests where geotextile permeability was adequate).

It is evident from the results shown that the calculation of the permeability coefficient, K, of the geotextiles gives varying results. The values calculed from the falling head permeameter results were considerably higher than those obtained from the flow test results.

The permeability criteria using in-isolation parameters given below in Section 7.3.2 were obtained from the values shown in Table 7.3. The lowest geotextile permeabilities that were high enough to be more than that of the soil, were adopted as the minimum requirements. The values were rounded off to the nearest 5 $\ell/m^2/s$ and are slightly more conservative than the actual values shown on Table 7.3.

7.3.2 Recommended permeability criteria

The criteria based on in-isolation characteristics ((a) below) should only be used as a first indication of adequate geotextile permeability. The final geotextile selection should be based on the flow test criterion ((b) below).

(a) In-isolation parameters:

SOIL	Minimum geotextile permeability, γm ($\ell/m^2/s$)	
	Falling head permeameter	Constant head permeameter
Clean sands, gravels	60	25
Soils with K < 0,2mm/s	20	10

(b) Flow Test

The permeability of the goetextile should be equal to or more than that of the soil ie. the initial permeability obtained with the flow test should not be less than that of the soil.

7.3.3 Evaluation of existing permeability criteria

Of the criteria described in Section 3.3.4, only two permeability criteria could be evaluated. The others either did not give specific permeability criteria or the soils and geotextiles tested fell outside the ranges specified for the criteria.

The permeability criteria of ICI (1978) are based on minimum values of O_{50}. According to these criteria, 38 of the tests were performed with geotextiles that had inadequate permeability. The total number of tests where the geotextile permeability was in fact inadequate, was 5.

Giroud (1982) recommends that the permeability coefficient, k, of the goetextile should be more than one tenth times that of the soil. It is not really possible to evaluate this criterion, due to the difficulties association with calculating the permeability coefficient of a geotextile, as was illustrated in Table 7.3.

7.4 PERMEABILITY REDUCTION

The permeability criterion described in Section 7.3 above ensures that the water is drained effectively immediately after installation, but does not address the permeability reduction with time or the potential for clogging of the goetextile. For that purpose a permeability reduction criterion is required.

The permeability criterion, as evaluated with the flow test, is simply that the permeability of the geotextile should be more than that of the soil. The setting of limits for permeability reduction, however, should be based on comprehensive field investigations undertaken over long periods. As was discussed earlier, such a comprehensive investigation was not performed as part of this investigation.

Without limits for permeability reduction, the designer of subsurface drainage installations can only rely on comparisons between the alternatives available and select that filter which will result in the lowest permeability reduction, irrespective of what that reduction may be. It is the author's opinion that, even when acceptable limits have been established by field observations, the method of "choosing the best alternative" will still be applied in selecting geotextiles for subsurface drainage. This is due to the fact that, unlike materials where strength is required, the purchasing price of a geotextile is not directly related to the potential permeability reduction. The results obtained from the flow tests (Table 5.6) indicated that the best results, ie. the highest values of K_{400}, were often obtained with the cheaper, lighter grades of geotextiles.

Despite the above, limits have to be set to give some indication of acceptable performance. It is foreseen that the limits given below will be adjusted when more information is available from field observations.

7.4.1 Analysis

The limits set for permeability reduction were based on comparisons between the results obtained with different geotextiles. All the geotextiles that were available at the time were evaluated and the results are therefore a reasonable indication of the performance that can be obtained. A comparison was also made between the results obtained with the goetextiles and that obtained with the only other alternative filter material namely natural (granular) filters. These were designed according to accepted criteria, as discussed in Section 5.3.3.

Table 7.4 shows the average values of K_{400} that were obtained with the flow tests using the various soil types.

TABLE 7.4: AVERAGE RESULTS OBTAINED WITH FLOW TESTS

SOIL	R	W	B	S	M
Percentage passing 0,075mm	0,0	6,4	14,8	48,6	66,6
D_{10}	0,16034	0,09630	0,03722	0,00070	0,00049
K_{400} (average)	30,9	13,2	5,6	7,3	4,3

It is evident that the average value obtained differed for each soil type. This is not surprising, since it was determined earlier (Section 5.6) that the flow test results were influenced by the soil type and that reasonable correlations existed between K_{400} and percentage passing 0,075mm. Figure 7.2, obtained from Table 7.4 above, shows the average values of K_{400} against percentage passing 0,075mm. This analysis of average values indicates that soils with smaller percentages passing 0,075mm (coarser soils) will generally result in higher values of K_{400} and higher limits should therefore be set for these soils.

Table 7.5 below gives a summary of all the results obtained from the flow tests. The results are given in descending order with the best result obtained for each soil at the top of the list. Shown on Table 7.5 are the values of K_{400}, K_{400} expressed as a percentage of the average value of K_{400} obtained for that particular soil and K_{400} expressed as a percentage of the value obtained using a granular filter with

TABLE 7.5: COMPARISON OF FLOW TEST RESULTS

GEOTEXTILE	SOIL	K_{400}	K_{400} as percentage of average	K_{400} as percentage of granualr filter
1	R	46.15	149.26	139.05
24	R	40.9	132.54	123.47
32	R	40.36	130.54	121.60
G	R	33.19	107.35	100.00
5	R	32.56	105.31	98.10
26	R	29.20	94.44	87.98
21	R	28.12	90.95	84.72
27	R	20.70	66.95	62.37
20	R	7.01	22.67	21.12
15	W	28.35	214.32	659.30
5	W	21.24	160.57	493.95
14	W	21.17	160.04	492.33
13	W	18.23	137.82	423.95
2	W	18.12	136.99	421.40
11	W	17.66	133.51	410.70
4	W	17.19	129.96	399.77
6	W	17.10	129.28	397.67
1	W	16.67	126.02	387.67
25	W	16.56	125.19	385.12
17	W	16.48	124.59	383.26
18	W	15.53	117.41	361.16
24	W	15.11	114.23	351.40
19	W	14.91	112.72	346.74
22	W	14.27	107.88	331.86
23	W	14.23	107.58	330.93
16	W	13.80	104.33	320.93
3	W	13.51	102.14	314.19
33	W	12.66	95.71	294.42
32	W	11.14	84.22	259.07
27	W	10.73	81.12	249.53
26	W	10.60	80.14	246.51
20	W	7.07	53.45	164.42
28	W	6.78	51.26	157.67
21	W	5.44	41.13	126.51
G	W	4.30	32.51	100.00
30	W	3.63	27.44	84.42
29	W	0.78	5.90	18.14
31	W	0.34	2.57	7.91

TABLE 7.5: COMPARISON OF FLOW TEST RESULTS (continued)

GEOTEXTILE	SOIL	K_{400}	K_{400} as percentage of average	K_{400} as percentage of granualr filter
24	B	8.55	152.35	178.13
20	B	7.65	136.32	159.38
26	B	7.36	131.15	153.33
32	B	5.82	103.71	121.25
29	B	5.78	102.99	120.42
5	B	5.60	99.79	116.67
1	B	5.30	94.44	110.42
G	B	4.80	85.53	100.00
21	B	3.52	62.72	73.33
27	B	1.74	31.00	36.25
G	S	17.70	241.28	100.00
5	S	13.10	178.57	74.01
20	S	7.90	107.69	44.63
21	S	7.80	106.32	44.07
24	S	7.40	100.87	41.81
1	S	6.63	90.38	37.46
32	S	6.23	84.92	35.20
26	S	3.27	44.57	18.47
27	S	2.70	36.80	15.25
29	S	0.63	8.59	3.56
13	M	12.47	290.19	
32	M	11.33	263.66	
33	M	9.98	232.25	
27	M	4.80	111.70	
21	M	2.77	64.46	
2	M	2.63	61.20	
18	M	2.56	59.57	
26	M	2.56	59.57	
5	M	2.45	57.01	
24	M	2.32	53.99	
1	M	2.14	49.80	
14	M	1.62	37.70	
31	M	1.28	29.79	
29	M	1.25	29.09	

the particular soil. For example, the soil/geotextile combination that was evaluated in the field (soil W, geotextile 3) resulted in a K_{400} value of 13,51, which was slightly above average (102,1 per cent of average) and more than three times higher than the granular filter (314,2 per cent of K_{400} for the granular filter). Note that the granular filter is indicated as "G" in the geotextile column.

The results obtained with the granular filters relative to those with the geotextiles are noteworthy. With the river sand the performance of the granular filter was just above average. With the Warmbad soil it was well below average with only 3 geotextiles resulting in lower values. The granular filter gave the highest value of K_{400} with the Silverton soil and the lowest value with the building sand. This indicates that, when granular filters are used, flow tests should also be performed to evaluate the permeability reduction. This confirms a point of concern that is often expressed by geotextile manufacturers, namely that there is suspicion among engineers as regards the clogging potential of geotextiles but the same does not apply to natural filters.

The permeability reduction criteria given below were obtained from the values shown in Table 7.4 above. The limits for high and low performance were arbitrarily set at 50 and 150 per cent respectively of the average value obtained for the various soil types. As was mentioned earlier, these limits will probably be adjusted when more information is available from field observations. Based on these criteria, the permeability reduction of 11 soil/geotextiles combinations and 1 soil/granular filter combination were in the low performance range.

7.4.2 Recommended permeability reduction criteria

Flow tests should be performed on geotextiles considered for use as filters and that geotextile resulting in the highest value of K_{400} should be selected. The limits given below should be used as a guideline to evaluate the results obtained from the flow tests. Geotextiles selected should yield results in the high or medium ranges.

SOIL	P (%)	$P \leq 1,0$	$1,0<P\leq7,0$	$7<P\leq50$	$P>50$
	D_{10} (mm)	$D_{10}\geq0,150$	$0,095\leq D_{10}<0,150$	$0,0007\leq D_{10}<0,095$	$D_{10}<0,0007$
K_{400} (%)	High	$K_{400}\geq46,0$	$K_{400}\geq19,0$	$K_{400}\geq9,0$	$K_{400}\geq6,0$
	Medium	$15,0\leq K_{400}<46,0$	$6,0\leq K_{400}<19,0$	$3,0\leq K_{400}<9,0$	$2,0\leq K_{400}<6,0$
	Low	$K_{400}<15,0$	$K_{400}<6,0$	$K_{400}<3,0$	$K_{400}<2,0$

P = Percentage passing 0.075 mm

7.5 CONCLUSIONS

- Filter criteria for geotextiles in road subsurface drainage applications were derived and are summarized in Sections 7.2.2, 7.3.2 and 7.4.2 above. These criteria control piping, permeability and permeability reduction respectively, and were obtained from long term flow tests.

- Laboratory flow tests, using soil/geotextile combinations, can be used to evaluate each of the three criteria and are recommended for that purpose. The criteria based on in-isolation parameters should only serve as guidelines and should not be used for geotextile selection.

- An evaluation of existing filter criteria showed that those criteria gave widely differing results and could not predict the results obtained in this investigation.

- It is envisaged that information obtained from future field observations will result in certain adjustments to the recommended limits, notably those relating to permeability reduction.

- Flow tests using granular filters often resulted in higher rates of permeability reduction than geotextiles and these filters should therefore also be subjected to permeability reduction tests if they are being considered as alternatives to geotextiles.

CHAPTER 8

DRAINAGE LAYERS

CHAPTER 8

CONTENTS

8.1 INTRODUCTION

Subsurface drainage methods that are aimed at intercepting water on the sides and below the pavement structure, ie seepage from high ground, capillary forces etc, need only be applied in those areas where ground water problems are expected eg high water table areas, cuts, embankments etc. Drainage methods (both surface and subsurface) aimed at controlling water entering the pavement from the surface, however, are applicable in varying degrees to all roads in South Africa. If the road surface is impermeable for any reason and rainfall occurs, then water will enter the pavement structure through the surface. Drainage methods that can be applied to counteract this source of water include the following:

- Adequate surface (stormwater) drainage. This includes horizontal and vertical geometric design as well as pipes and channels to remove the rain water in the shortest possible time thus minimizing the amount of water that can enter the pavement structure.

- The provision and maintenance of an impermeable surfacing.

- Horizontal subsurface drainage, which is the inclusion of a drainage layer in the pavement structure. The purpose of such a layer is to remove any water that enters that layer from the surface without allowing the water to enter the underlying pavement layers.

- Vertical subsurface drainage. Here the water that enters the pavement is simply allowed to drain vertically through the pavement structure under gravitational forces.

In South Africa the Road Authorities and design engineers in general favour the first two methods which are aimed at keeping the water out of the pavement structure. In certain other countries, notably the USA and Australia, the use of drainage layers in the pavement structure are often included in their subsurface drainage design documents (eg FHWA, 1972; Victoria Road Construction Authority, 1982; Genke, 1979; Transportation Research Board, 1982). Drainage layers are probably not used widely in those countries but certain drainage engineers (eg Cedergren, 1974) strongly recommend the use of such layers.

Those who favour the use of drainage layers argue that no pavement surface is impermeable and that water ingress from that source is inevitable and must be controlled. The counter argument is that the surface should be kept as impermeable as possible and that the stormwater drainage should be effective so that the amount of water entering the pavement is minimal, thus eliminating the need for drainage layers which, due to their high permeabilities, are structurally inadequate.

In this chapter the evaluation of a drainage layer is described. The layer was evaluated both structurally and hydraulically. The overall purpose of including a drainage layer is to prolong the life of the pavement. To do this, the layer must meet certain structural and hydraulic requirements. Structurally the layer must carry the design traffic in both wet and dry conditions. The layer itself must therefore not be water susceptible. Hydraulically the layer should have adequate permeability to remove any water that enters the pavement structure before this water can enter the other pavement layers, thus resulting in traffic-induced damage. These are clearly two opposing requirements since permeability is dependent on low density and strength is dependent on high density.

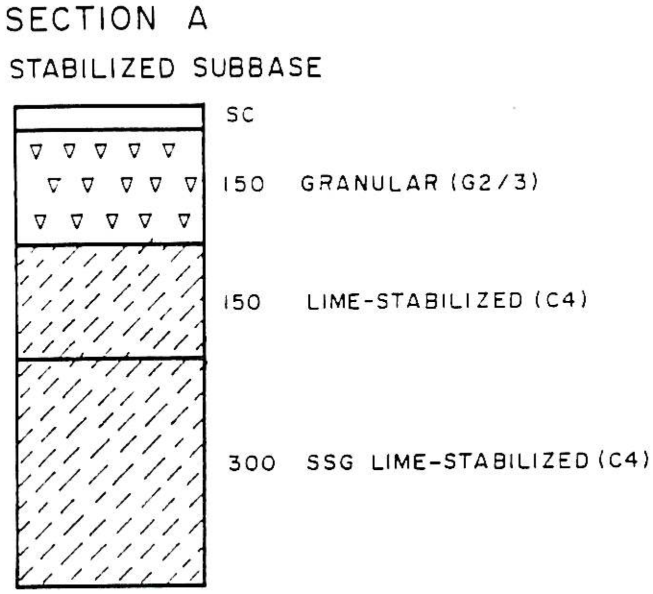
8.2 PAVEMENT STRUCTURE AND MATERIALS

The drainage layer that was evaluated was a subbase in a pavement structure that formed part of a set of experimental sections that were built on Trunk Road TR/86 near East London. These experimental sections were included in a program for testing with the Heavy Vehicle Simulator (HVS) of the Division of Roads and Transport Technology (DRTT).

The pavement structure with the drainage layer (drainable subbase) is shown in Figure 8.1 (Section C). The pavement structure of Section A (Figure 8.1) was similar to that of Section C, but it incorporated a stabilized subbase. Both structures were evaluated and a comparison could therefore be made between the behaviour of the two types of subbases. The abbreviations for material types in Figure 8.1 (eg G2, C4) are those given by the DRTT (1985). The material of the drainable subbase was estimated to be equivalent to a G5 material.

Various laboratory and field tests were carried out during the construction of the experimental sections, details of which are documented elsewhere (Van der Merwe, 1985). A summary of the results is given in Table 8.1.

The selected subgrades (decomposed dolerite) of Sections A and C differed somewhat in the amount of lime added. For Section A the total depth of 300 mm was modified only for plasticity index (PI) with an addition of 1,5 % lime. The same treatment was applied to the bottom half of Section C's selected subgrade, but the top half was stabilized with 4 % lime. However, DCP measurements on the completed layer indicated very little difference in the strengths, and hence both layers were assumed to be lime-stabilized (C4). The densities measured on these layers were well in excess of the required 93 % Mod. AASHTO.



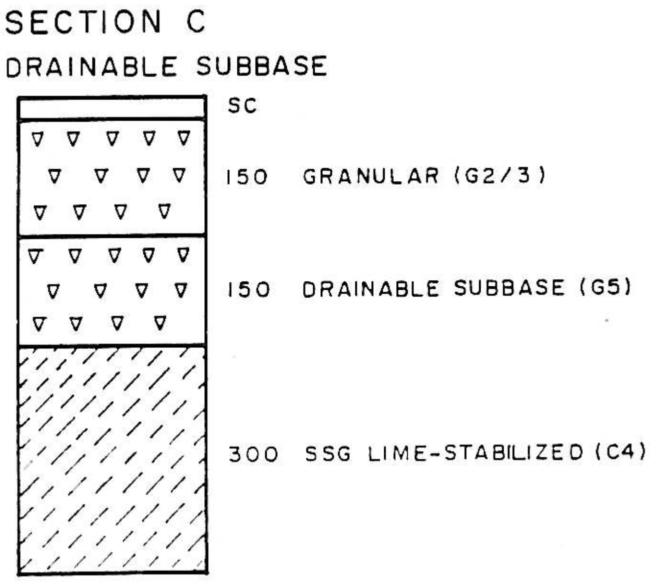


FIGURE 8.1

PAVEMENT STRUCTURES OF EXPERIMENTAL SECTIONS

TABLE 8.1 Summary of material test results (laboratory and field) for drainage layer

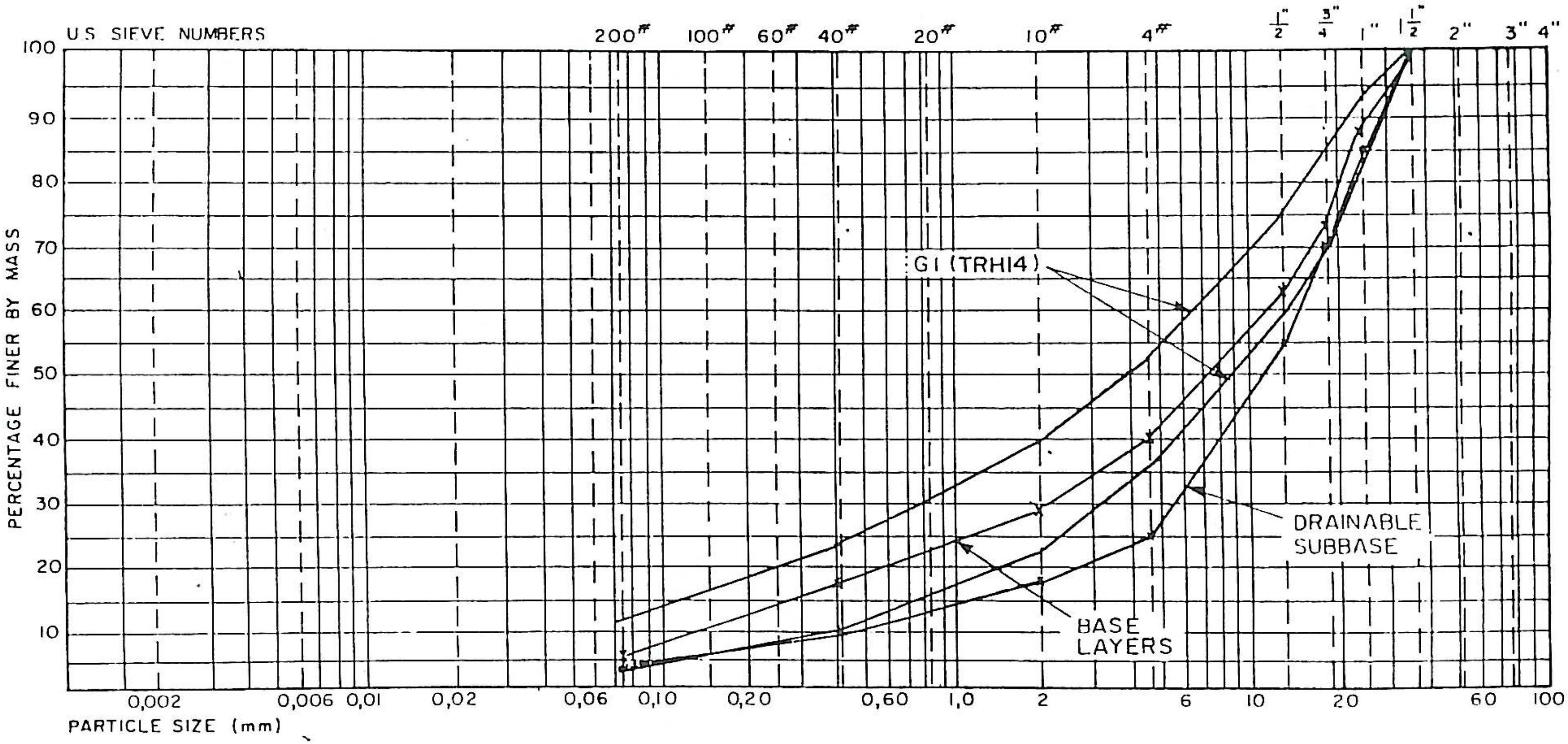
	Section A				Section C			
	Base	Subbase	SSG1*	SSG2	Base	Subbase	SSG1	SSG2
Maximum Mod. AASHTO density (kg/m^3)	2198	2134	2171	2292	2198	2027	2165	2278
Optimum moisture	7,7	10,6	10,1	7,5	7,7	9,9	10,4	8,0
Apparent density (kg/m^3)	2658				2658	2643		
Grading % passing (mm)								
53	100	100	100	100	100	100	100	100
37,5	100	96,0	98,3	93,3	100	100	97,0	93,0
26,5	91,0	89,0	96,3	91,0	85,3	85,0	94,3	89,3
19,0	75,8	86,0	94,7	87,7	69,0	70,0	91,0	86,3
13,2	63,3	83,0	92,3	84,3	56,0	55,7	88,7	82,3
4,75	41,5	69,0	78,7	73,0	35,0	29,7	75,3	71,0
2,00	30,8	47,0	56,3	50,3	25,5	18,7	51,0	48,7
0,425	19,5	19,0	22,7	19,3	16,5	10,0	20,0	17,7
0,075	6,5	8,0	9,7	8,7	5,3	5,0	8,3	7,7
Plasticity Index	SP	SP			SP	SP		
Field density								
% Mod. AASHTO	98,5	97,8	99,3	97,7	97,8	99,9	99,7	98,4
% App. density	81,4				80,8	76,6		
Field moisture %	2,9	8,8	7,5	7,0	4,4	2,8	8,1	7,1
DCP (mm/blow)			5,2				4,9	

* SSG1 = Top 150 mm of selected subgrade
SSG2 = Bottom 150 mm of selected subgrade

The subbase of Section A was very similar to the selected subgrades and was stabilized with 4 % lime. The densities were in excess of the required 95 % Mod. AASHTO.

The base layers were constructed with freshly crushed rock to which fines were added. The grading of the material was well within the normal envelope for a crushed-stone layer (Figure 8.2) and the fines were semiplastic. The densities achieved were below 100 % Mod. AASHTO and the material was classified as G2 to G3 quality.

940-4-4910/6 B.D.



CLAY FRAC-TION	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	COBBLE FRACTION
	SILT FRACTION			SAND FRACTION			GRAVEL FRACTION			

FIGURE 8.2

GRADING OF CRUSHED STONE USED IN BASE LAYERS AND DRAINABLE SURBASE

The drainable subbase was constructed with the same crushed stone used for the bases, but without the added fines. The grading of this material is also shown in Table 8.1 and in Figure 8.2. The densities achieved on this layer were low in terms of percentage of apparent density (76,6 %). This was expected since the layer was required to have a high permeability.

Constant-head permeability tests were carried out on compacted samples in the laboratory. The results of these tests as well as permeability measurements on the base material are shown in Figure 8.3. The permeability coefficient, k, of the drainable subbase at field density was approximately 10×10^{-6} m/s.

8.3 STRUCTURAL EVALUATION

8.3.1 Mechanistic analysis

A mechanistic analysis was carried out to predict the performance of the pavement structures, details of which are documented separately (Van der Merwe et al, 1986). The material classification given in Section 8.2 above and shown in Figure 8.1 was used for this purpose. The effective elastic moduli used were those suggested by Freeme (1983).

The pavement structures were analysed in three phases relating to the pre-cracked, post-cracked and wet states (Freeme, 1983). The calculated structural lives under the most severe conditions indicated that both Sections A and C would reach the E2 traffic class (0,8 to 3×10^{6} E80s).

8.3.2 Heavy Vehicle Simulator (HVS) tests

To evaluate the actual performance of the drainage layer under traffic, HVS tests were carried out on both Sections A and C.

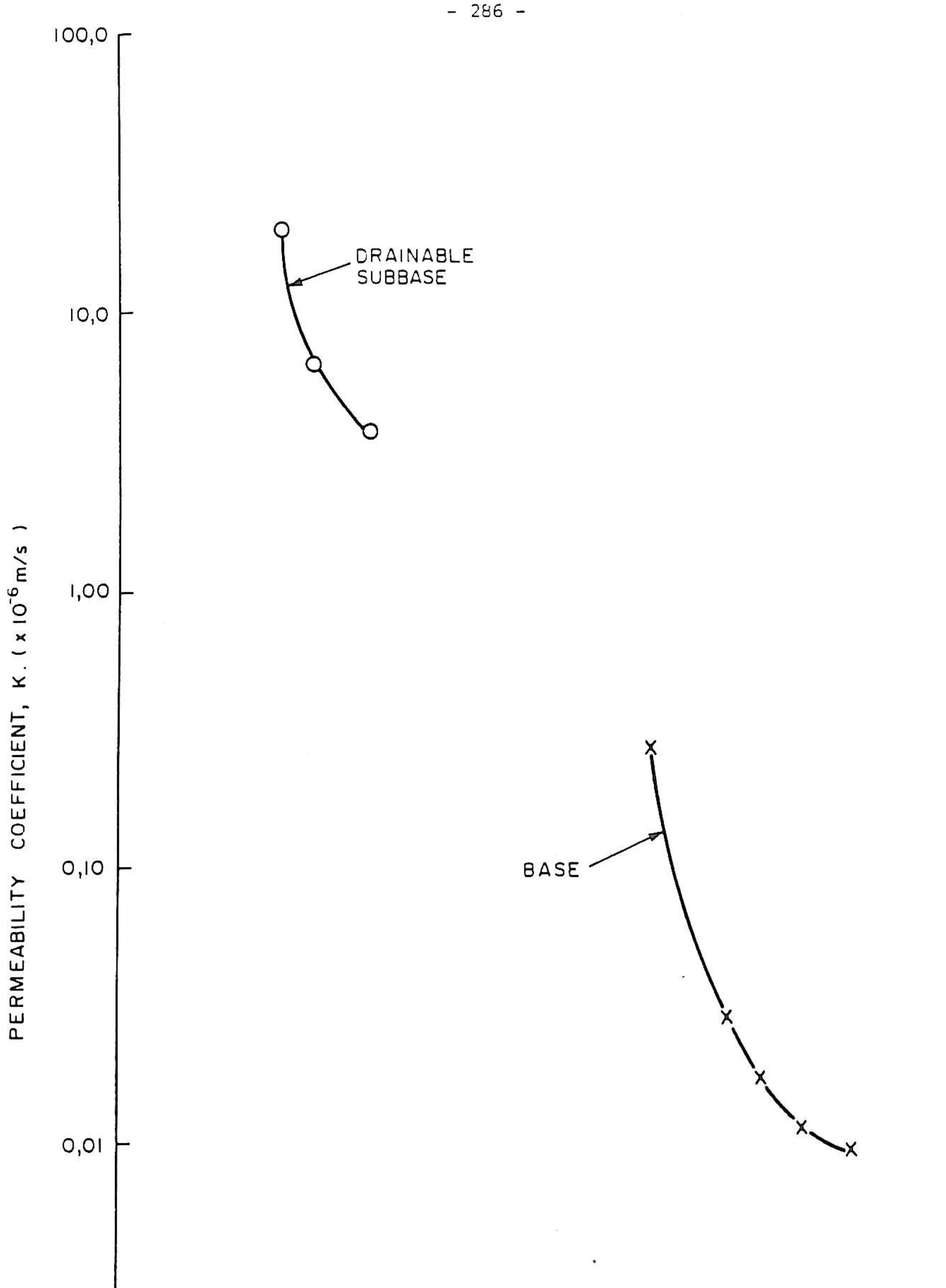


FIGURE 8.3
LABORATORY PERMEABILITY TESTS

Various tests (two on Section C and three on Section A) were carried out to evaluate various aspects of the experimental sections. However, only one test on each section was used to evaluate the drainage layer, and these tests are discussed here. The conditions of the tests, ie wheel- load and moisture conditions, were similar for both sections and therefore comparisons can be made.

One of the major forms of distress of a granular-base pavement is the permanent deformation measured at the surface (Maree et al, 1982). Figure 8.4 shows the increase in permanent deformation with traffic on both Sections A and C. The traffic shown in Figure 8.4 is expressed in equivalent 80 kN axles (E80s). To calculate the E80s, the exponent, n, in the equivalency factor formula, was assumed to be 3 (Maree et al, 1982). Permanent deformation was measured at various positions on the trafficked section, and the average value is shown in Figure 8.4.

On both sections a high wheel load (100 kN) was used, initially on a dry pavement. The wheel load was then decreased to 40 kN, and water was introduced by means of a system of perforated pipes through which the water flowed. These pipes were inserted to a depth of 600 mm.

After completion of each test, a trench was opened across the trafficked section to study the profile of the pavement. Densities were measured in the base and subbase of both sections inside and outside of the trafficked area. These densities can be interpreted as densities before and after trafficking. The results of these tests are shown in Figure 8.5.

The behaviour of Section A was typical of a granular-base pavement with a cemented subbase. The permanent deformation (Figure 8.4) initially increased at a high rate (the so-called settling-in period). This is usually the result of initial densification of the base layer. Continued dry testing

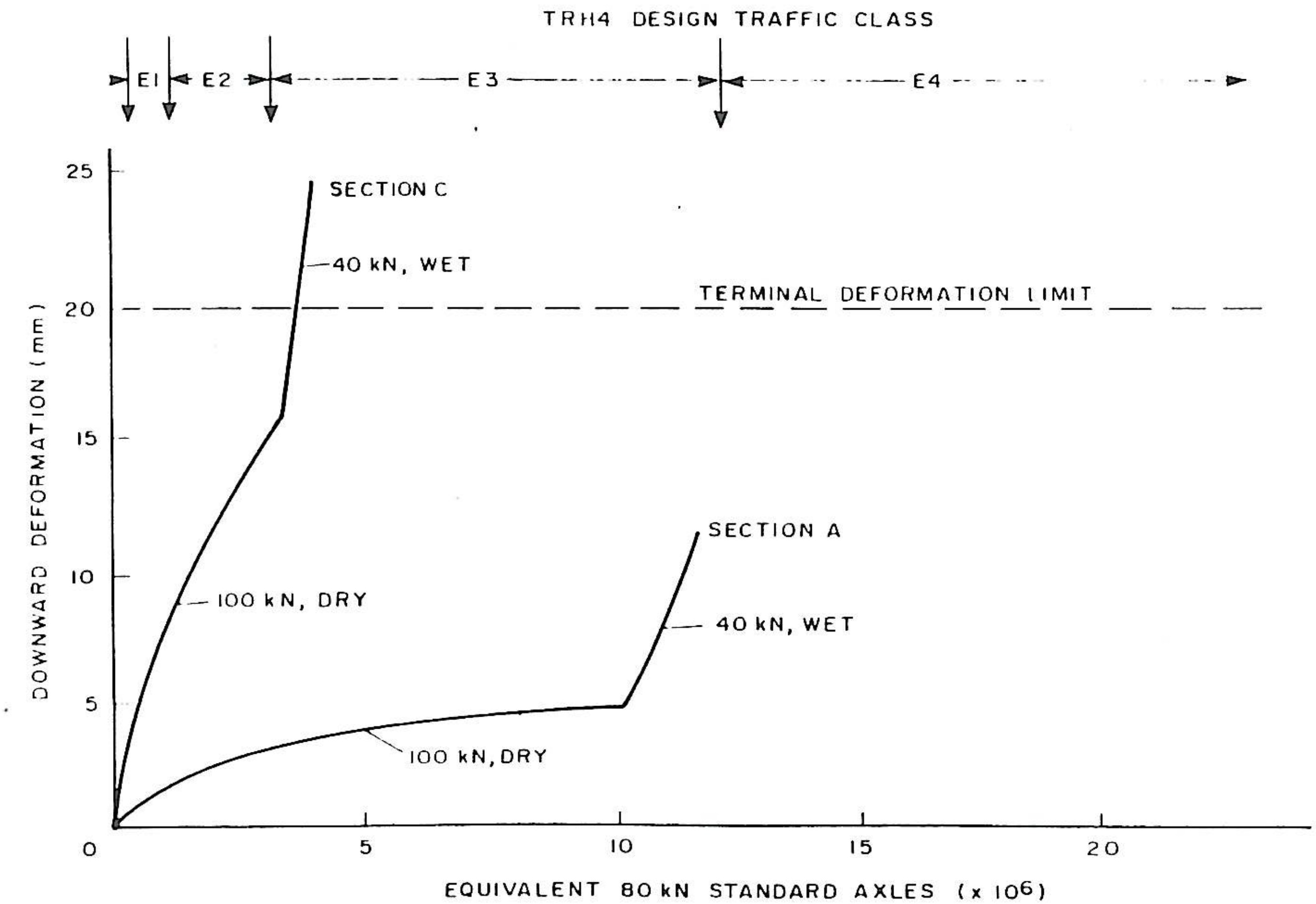


FIGURE 8.4

HVS TESTS: PERMANENT SURFACE DEFORMATION

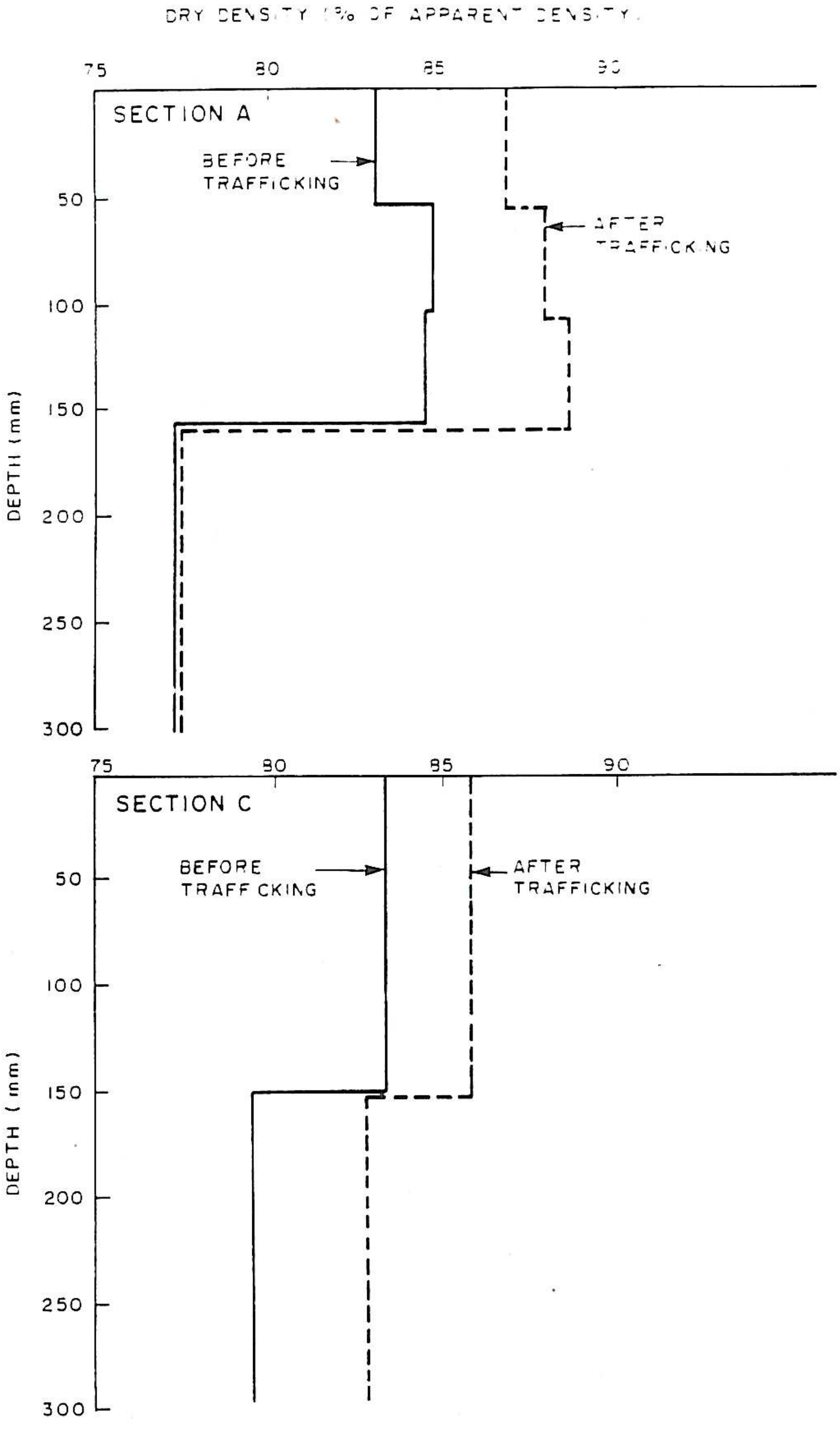


FIGURE 8.5

CHANGE IN DENSITY DUE TO TRAFFICKING

resulted in a flattening of the curve and it is evident from Figure 8.4 that, in dry conditions, this pavement structure would reach E4 traffic class. With the introduction of water, however, the rate of deformation increased considerably. The shape of both the dry and the wet portions of the curve is typical of G2 or G3 base, which the base was classified as, prior to HVS testing.

The profile measurements on Section A after completion of the test indicated that all the permanent deformations originated in the base in the form of densification of the material. There was no sign of shear failure. The density measurements (Figure 8.5) before and after trafficking confirmed this.

In Figure 8.4 the permanent deformation of Section C (with the drainage layer) is shown to be similar to that of Section A, but with a much higher rate of deformation, especially in the dry state. At the point where water was introduced, the rate of deformation was decreasing and it can probably be said that the total deformation measured up to that point was part of the initial "settling-in" period. Although the deformation was high, the pavement structure still carried E2 traffic, as was predicted with the mechanistic analysis. The classification of the drainable subbase as a G5 material was therefore fairly accurate.

The profile and density measurements (Figure 8.5) showed that the permanent deformation originated from densification of both the base and the subbase. Once again there was no indication of shear failure. The higher rate of permanent deformation of Section C was therefore due to the densification of the drainage layer.

The problem mentioned earlier was confirmed, ie that the high permeability required for a drainage layer results in reduced structural performance. In this case the material was densified under traffic, resulting in permanent surface deformation. To overcome this problem, a material is needed that will not densify under traffic and yet has adequate permeability. One possible solution is to stabilize an open-graded material with lime or cement in order to produce a product similar to a no-fines concrete. Another possibility is to use a waterbound macadam as a drainage layer. Recent work on this material (Horak et al, 1986) showed that a waterbound layer, especially with a uniformly graded sand as filler, has a very high permeability. Owing to the interparticle contact of the large stones in a waterbound layer, the material is very stable and has a high structural strength. (See Chapter 2, Section 2.2.5)

During the wet testing phase on Section A, pumping of fines was observed, resulting from excess porewater pressures in the base. On Section C, no pumping was evident and the pore pressures were probably dissipated by the drainage layer. The fact that this did not improve the performance of the pavement can be attributed to the method by which water was introduced during HVS testing. The drainage layer was designed to remove excess water entering the pavement layers during rain (see Section 8.4 below). In HVS testing the water flows into the pavement continuously, thus not giving the drainage layer the opportunity to perform the function that it was designed for, namely to remove the water in the shortest possible time and thus to ensure a "dry" state. It is possible, then, that under naturally wet conditions the performance of Section C would be better than that of Section A.

One other problem was encountered with this type of drainage layer during the construction phase. The layer did not give adequate support for the compaction of the base layer and considerable extra effort was needed to obtain the required density in the base layer. In this particular case, a similar density was obtained on the bases of both Sections A and C. However, with a more compactable crushed stone, it would probably have been possible to obtain a G1-quality base on Section A but not on Section C. In those circumstances, the difference in performance between the two sections would have been even more pronounced.

8.4 HYDRAULIC EVALUATION

The basic function of the drainage layer is to remove water that enters through the surface of the pavement before this water can enter the lower layers. The permeability of this layer must therefore be high enough to remove water that enters through a permeable surface and passes through the base layer. The permeability of the underlying stabilized layer must be low in relation to the drainage layer in order to prevent water from entering that layer.

In all the calculations given below, Darcy's law of seepage was used, which assumes saturated flow:

$$Q = k\ i\ A$$

where Q = flow (m^3/s)
k = saturated permeability coefficient (m/s)
i = hydraulic gradient (dimensionless)
A = area (m^2)

8.4.1 Permeability of drainage layer

The permeability of the material was determined in the laboratory, as discussed in Section 8.2 above, and is illustrated in Figure 8.3. The value obtained at field density was approximately 20×10^{-6} m/s. As a first approach, recommendations given in overseas literature were considered.

Cedergren (1974) recommends that the permeability of any drainage layer should not be less than 1,0 cm/s ($10{,}000 \times 10^{-6}$ m/s). According to this criterion the permeability of the drainage layer of Section C is 500 times too low.

The Federal Highway Administration (FHWA, 1972) recommends that the one-hour/one-year frequency precipitation rate (the maximum rainfall in one hour that can be expected to occur on the average of one time each year) be used as a design parameter. The FHWA further recommends that this value be multiplied by a coefficient of between 0,33 and 0,50 to determine the design infiltration rate for an asphalt pavement. This means that between one-third and one-half of all rainfall is expected to infiltrate the pavement.

The one-hour/one-year frequency was obtained for the East London area (Weather Bureau, 1983). For a return period of 25 years the expected value is 35,9 mm. To calculate the infiltration, a section 1 m in length and half the width of the road (8 m) is considered, as shown in Figure 8.6.

Infiltration rate = C x f x A

where C = coefficient = 0,33

f = one-hour/one-year frequency

= 35,9 mm/hour

A = area = 8 m^2

Infiltration rate = 0,095 m^3/hour

The rate at which the drainage layer can remove the water is calculated with Darcy's equation. Under saturated-flow conditions the hydraulic gradient will be equal to the effective gradient (2,1 %) as indicated in Figure 8.6. The area is the cross-sectional area of the drainage layer.

$$\text{Rate of removal} = 20 \times 10^{-6} \times 0{,}02 \times 1 \times 0{,}15$$
$$= 2{,}16 \times 10^{-4} m^3/\text{hour}$$

Therefore, according to the FHWA criterion, the permeability of this particular layer is approximately 400 times too low.

It seems very unlikely, however, that one-third of all rainfall will infiltrate the pavement. Following a different approach, an attempt was made to calculate the maximum rate of infiltration when the road surface is severely cracked.

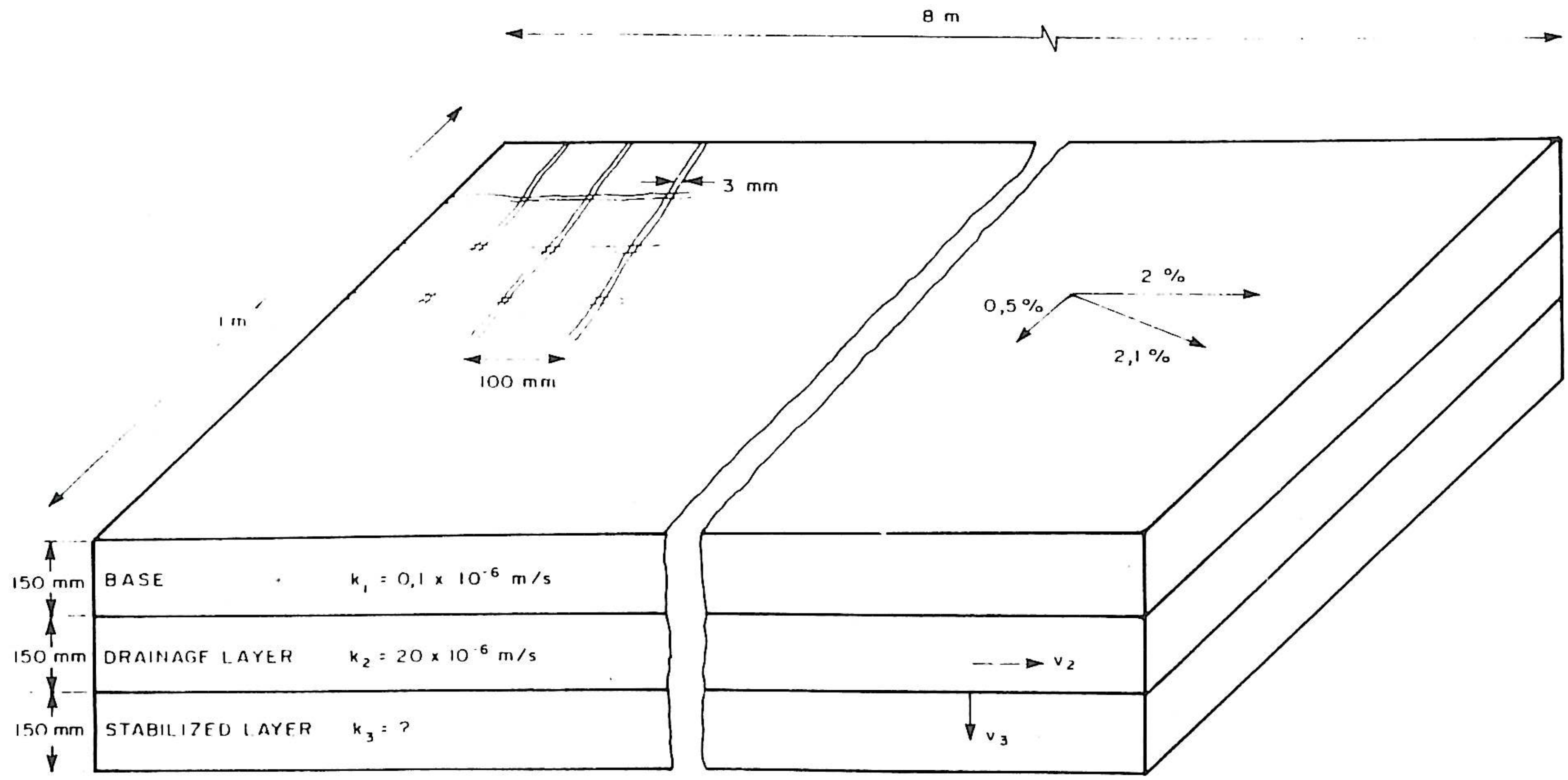


FIGURE 8.6

HYDRAULIC EVALUATION OF DRAINAGE LAYER

A severe condition has been observed on a road with a granular base and surface treatment (Jordaan, 1987) with 3 mm-wide crocodile cracks spaced at approximately 100 mm intervals, as illustrated schematically in Figure 8.6. If it is assumed that water flows freely into the cracks during rain and that only the surfacing is cracked, then the downward flow of water is governed by the permeability of the base material, irrespective of the amount or duration of the rainfall. The permeability of the base layer was determined in the laboratory (see Figure 8.3) as $0,1 \times 10^{-6}$ m/s. The area of cracks on a 1 m length of road is 0,16 m^2. The hydraulic gradient is 1,0. Using Darcy's formula, the infiltration rate is then $5,76 \times 10^{-5} m^3$/hour, approximately four times less than the rate of removal. Under the traffic the hydraulic head and therefore the hydraulic gradient are increased. A tyre pressure of 720 kPa is equivalent to a 73,5 m head of water and a hydraulic gradient of 490 (layer thickness is 0,15 m). Assuming an average tyre contact width of 200 mm per wheel and that the road is under traffic one-tenth of the duration of a rainstorm, the total infiltration will be $1,98 \times 10^{-4} m^3$/hour, slightly less than the rate of removal.

Although the infiltration rates calculated above are based largely on assumptions, these assumptions are conservative. It is felt that the criteria given by Cedergren (1974) and the FHWA (1972) are unrealistic, and that the permeability of the layer was in fact adequate.

8.4.2 Permeability of stabilized underlying layer

To remove water before it enters the stabilized layer, the velocity of flow in the drainage layer (v_2 in Figure 8.6) must be larger than the downward velocity in the stabilized layer (v_3, Figure 8.6).

Darcy's equation can be written as:

$$v = ki$$

where v = velocity of flow

To meet the said requirement:

$$v_2 > v_3$$

or $$k_2 i_2 > k_3 i_3$$

$$i_2 = 0{,}021 \text{ and } i_3 = 1{,}0$$

then $k_2 > 48k_3$

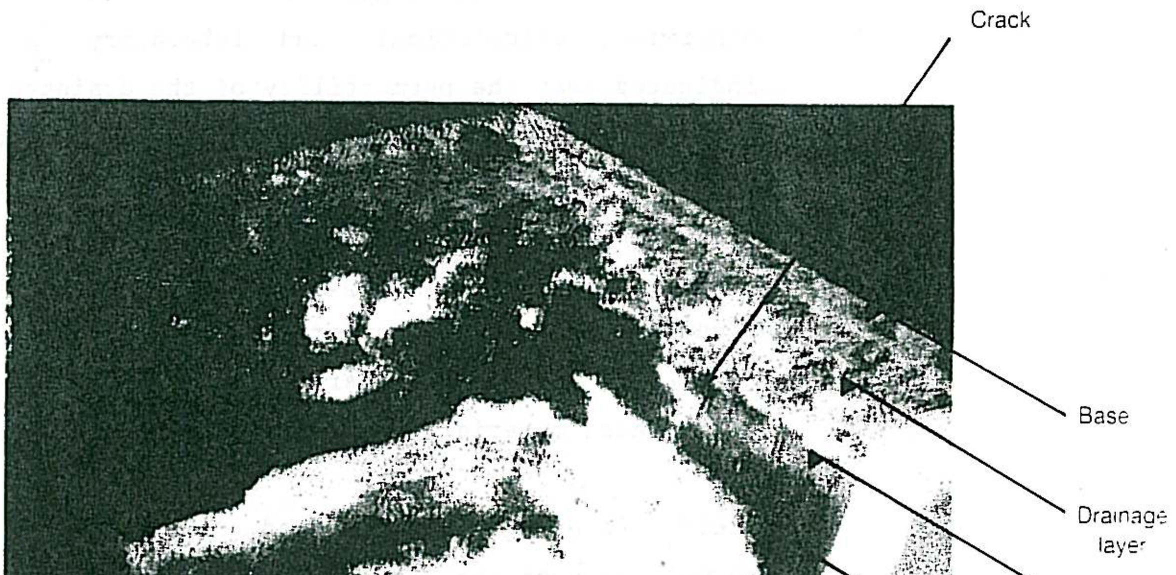
The permeability coefficient of the underlying layer was not measured in the laboratory, but an indication of its magnitude was obtained during field tests. A method was developed to study the flow patterns of water in the pavement structure. The method (described in more detail elsewhere: Van der Merwe, 1988) utilizes water which contains a water soluble fluorescent dye. When this solution flows through the pavement structure the fluorescent dye leaves a deposit in the material which is visible under ultra-violet light, even after the material has dried out completely. A method was also developed to photograph the flow patterns thus obtained.

The water with fluorescent dye was allowed to flow into the pavement structure through induced "cracks" that were sawed through the surfacing. A profile trench was then opened and the flow patterns could be studied under ultra-violet light.

Photograph 8.1 shows the pavement structure in the profile trench as seen under ordinary white light and Photograph 8.2 shows the same profile under ultra-violet light. It is clear

PHOTOGRAPH 8.1

Pavement with drainage layer (white light)



PHOTOGRAPH 8.2

Pavement with drainage layer (ultra-violet light) from this photograph that, although the permeability of the underlying layer was sufficiently low in some areas not to allow the ingress of water, in other areas the water did in fact enter that layer and where a crack occurred, the water flowed through the crack and into the subgrade. These results stress an important point regarding permeability measurement namely that the value obtained on a small sample in the laboratory may in fact be very misleading. A large variation in permeability occurs in the field and the effective permeability of a layer is often determined by cracks in that layer.

Based on the fluorescent dye investigation, it was concluded that the effective permeability of the underlying layer was too high.

8.5 CONCLUSIONS AND GENERAL REMARKS ON DRAINAGE LAYERS

- The investigation showed that the structural strength of the drainage layer was inadequate due to densification under traffic. Although overseas literature suggested otherwise, calculations and laboratory measurements indicated that the permeability of the drainage layer was adequate to remove water infiltrating through the pavement surface. For a drainage layer to succeed both structurally and hydraulically a material must be obtained that has high permeability and which is not prone to densification under traffic. Possibilities that could be investigated are lime / cement stabilized open-graded materials and waterbound macadams.

- Field investigations showed that the effective permeability of the layer below the drainage layer was too high to prevent water from entering the subgrade. This was mainly due to cracks that occurred in that layer. A possible solution to prevent water from entering the layer directly below the drainage layer would be to place an impermeable membrane such as a polyethylene sheet between the layers. This would pose a structural problem, however, since it would not be possible to obtain a bond between the respective layers and the membrane.

- The field tests were conducted on a small section of road. For a drainage layer to function on a large scale, certain other practical aspects need to be considered as regards discharge and gradients. The drainage layer must daylight on the side of the road, or edge drains must be provided, for the water to be discharged. The discharge zone must be well maintained both during construction and during the life of the pavement. Failure to do so will result in a reservoir in the pavement structure. If the vertical gradient of the road is larger than the cross-fall, the water in the drainage layer will be carried to the lowest point of the road, and unless provision is made at that point to remove large quantities of water, failures will result.

- Based on the investigation described in this chapter, the use of drainage layers is not recommended for heavily trafficked rural roads. For urban roads which are "boxed" in by kerbs and actually act as canals for stormwater drainage, the use of drainage layers may be a viable solution. In the field of stormwater management, the use of drainage layers is seen as an available stormwater management technique (eg Field et al, 1982). Porous pavements and drainage layers can be utilized to reduce the rate of runoff and decrease the flow velocities thereby minimizing peak floods, erosion and sedimentation.

CHAPTER 9

OTHER ASPECTS OF ROAD SUBSURFACE DRAINAGE

CHAPTER 9

CONTENTS

9.1 INTRODUCTION

In Chapter 1 the various sources of water and subsurface drainage methods were described. The research work on which this thesis is based, as described in preceding chapters, covered only certain aspects mentioned in Chapter 1. Certain other aspects were encountered by the author during the course of the research work and were briefly investigated. These are described in this chapter.

9.2 FIN DRAINS

Before geotextiles were available, side drains typically consisted of granular filters with a porous pipe or a perforated pipe as illustrated in Figure 9.1(a). The granular filters were designed according to the Terzaghi criteria as described in Sections 3.3.1 and 5.3.3. Depending on the grading of the base soil and the granular materials available, it was often necessary to use two or more filters in stages, as was the case with the soil/granular filter compatibility tests described in Chapter 5 (Section 5.3.3). Geotextiles eliminated this problem since these materials could be used as filters with the granular material acting only as a water carrier, as shown on Figure 9.1(b). Side drains constructed in this way are the most widely used in South Africa at present and are specified by most road authorities. The granular material is usually single sized (typically 19mm) and is cheaper than a graded granular filter. The major shortcoming of these side drains has been the design of the geotextile filter, which is dealt with in detail in Chapters 3 to 7 of this thesis. Where the side drains transport water over dry areas, the geotextile is replaced with an impermeable membrane (Figure 9.1(c)).

In the geosynthetic industry, a variety of synthetic water carriers have been developed over the past ten years. These materials, termed geonets; geospacers; flawnets; waffles etc, are aimed at replacing the granular water carriers in the conventional side drains. The geonet (with thickness varying from 5 to 100mm) is wrapped in a geotextile filter to form what

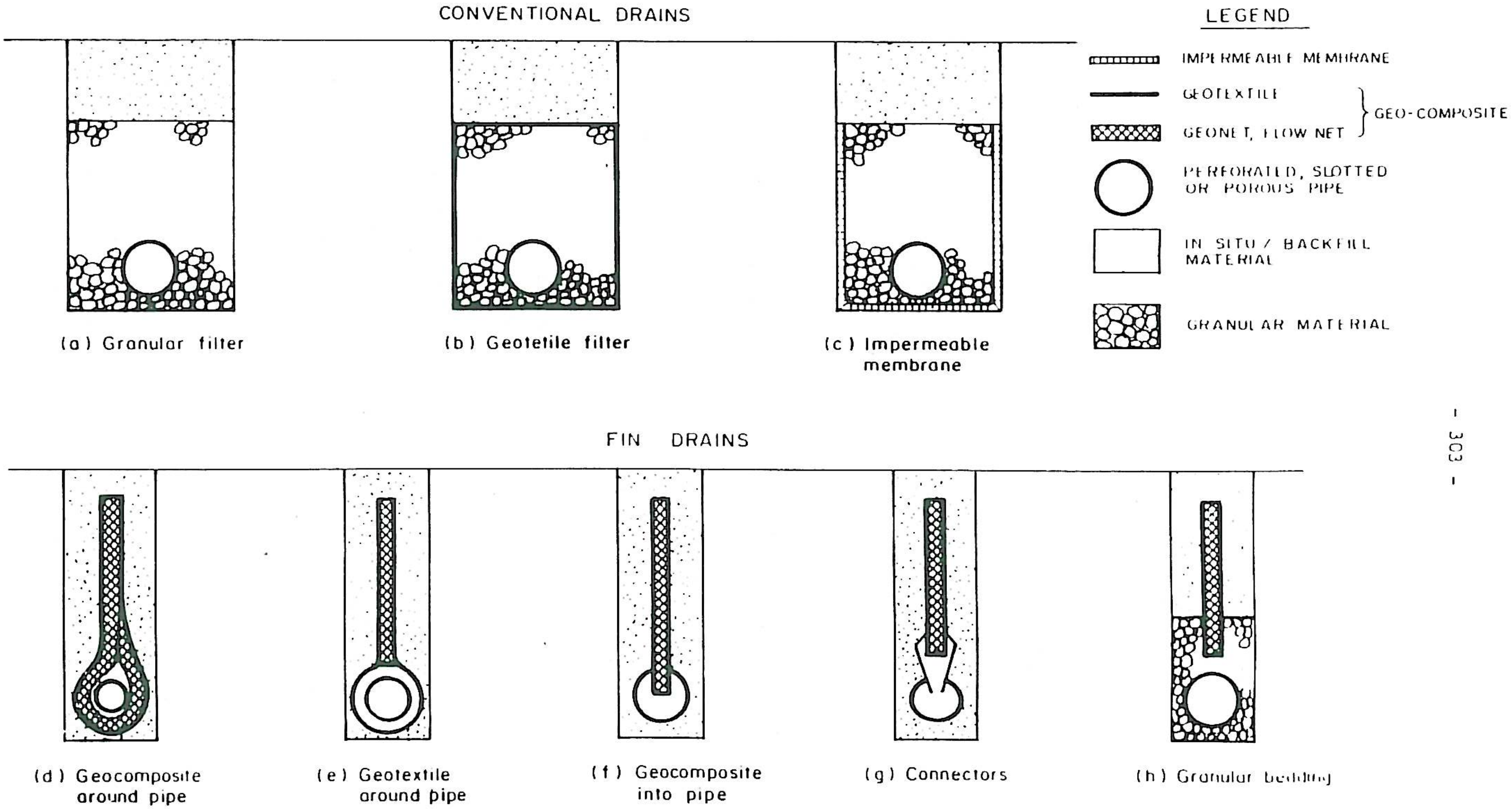


FIGURE 9.1 *TYPICAL SIDE DRAINS*

is often referred to as a geocomposite. The geocomposite in combination with a drainage pipe forms what is generally known as a fin drain, examples of which are shown in Figure 9.1(d) to (h).

Various flawnets, geotextiles, prefabricated geocomposites, synthetic pipes and connecting systems between the geocomposite and the pipe exist, resulting in a large variety of available fin drains. The fin drains are usually pre-assembled, either in the factory or at the construction site, before being installed in a trench. The dimensions of these drains make it possible to use considerably narrower trenches than with conventional side drains (usually 200mm as opposed to 600mm).

Although road authorities and designers are hesitant to use fin drains on a large scale, they have been installed in limited quantities and certain advantages as well as disadvantages have been identified. Advantages include lower cost, ease of construction and versatility. The main disadvantage is, as was the case with geotextiles, the lack of design methods and selection criteria.

9.2.1 Cost

The cost benefit of fin drains are derived from the narrower trench and lower material cost. In Chapter 2 (Section 2.6.2) the cost of a typical conventional drain (that is evaluated in Chapter 6; 600mm wide, 2m deep with 1,1m actual drain and 900mm backfill) was calculated. A comparison is given below in Table 9.1, between the cost of that drain and a fin drain with similar dimensions (1,1m actual drain) as well as the case where the fin drain is placed for the full 2m depth.

Table 9.1 Cost comparison between conventional drain and fin drains. (R/m)

	Conventional	Fin Drain	Fin Drain
Trench dimensions	0,6 x 2,0m	0,2 x 2,0m	0,2 x 2,0m
Drain depth	1,1m	1,1m	2,0m
Excavation	7,37	2,46	2,46
Backfill	4,31	3,19	3,19
Aggregate	25,66	0,00	0,00
Pipe	9,35	9,35	9,35
Geotextile	6,38	5,02	9,12
Flawnet	0,00	8,25	15,00
Total Cost: R/m	53,07	28,27	39,12

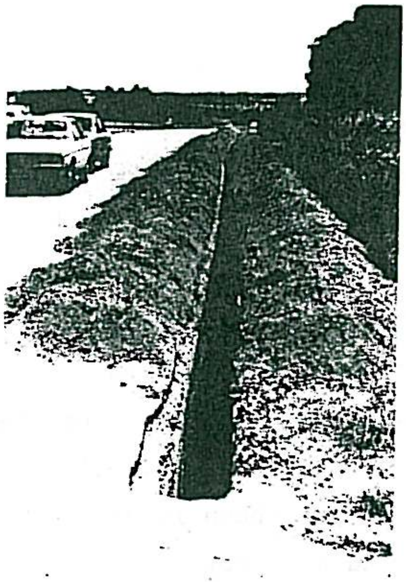
It is evident from the comparison in Table 9.1 that, even when the fin drain is placed for the full 2m depth of the trench, the cost is considerably lower than that of a conventional side drain. The biggest cost saving is obtained by eliminating the aggregate, which is the most expensive item in the conventional drain. The flawnet of which the cost is shown in Table 9.1 (7,50/mí) is a flawnet that has been used to date, also in the examples described in Section 9.2.2 below.

9.2.2 Construction

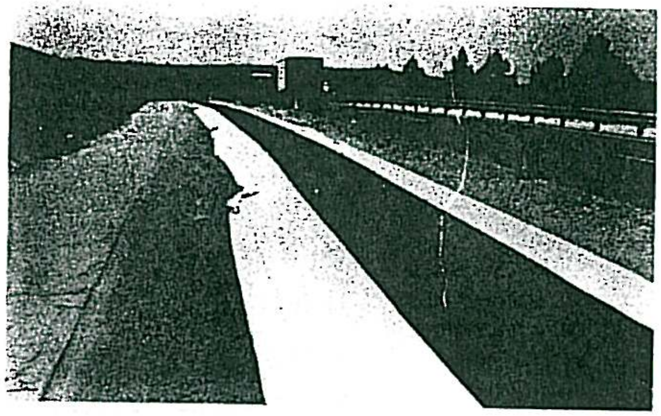
The relative ease of construction of fin drains is as a result of the narrow trenches and the fact that all the components can be assembled prior to placing the completed product in the trench, as opposed to the individual installation of each item in the case of conventional drains.

Photograph 9.1 shows a series of photographs taken during the installation of a fin drain. The trench, being only 200mm wide, was excavated mechanically (Photograph 9.1(a)). Photographs 9.1(b) to (d) show the different components, assembling process and the end product respectively. This particular fin drain was similar to that shown in Figure 9.1(d) with the geocomposite wrapped around a perforated pitch fibre pipe. The geotextile and flownet were merely stapled together, since the highest stresses that are exerted on the fin drain are during installation. Once the end product has been installed, the various components are held in place by the backfill material. Photograph 9.1(e) shows the installation procedure, whereby the completed product was merely lowered into the trench by hand. The geocomposite was then stapled to the side of the trench and the trench was backfilled with the original in-situ material. Certain geocomposites, that are manufactured as a unit, are still and need not be fastened to the trench sides. The literature supplied by the various manufacturers of fin drain materials indicate that the geocomposite is placed in the centre of the excavated trench, as shown on Figure 9.1(d) to (h). It is more practical, however, to place the geocomposite on the side of the trench, thus allowing more space for backfilling and compaction of the in-situ material.

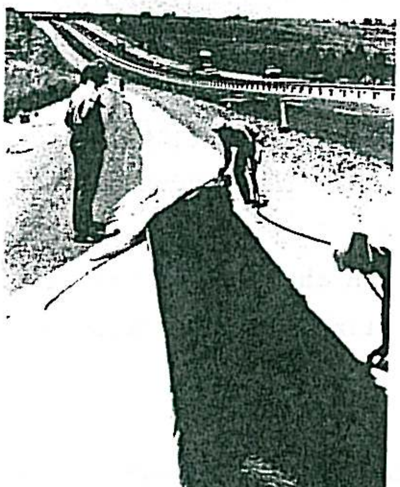
The compaction of the backfill material in the narrow trench proved to be a major problem. Compaction is required to ensure stability of the adjoining material which forms the road shoulder. Equipment exists which is claimed to be capable of compacting to a high density in a narrow trench, but the effectiveness of such equipment was not established. Research work is needed in this area to determine what density is in fact required and whether that density can be obtained in a narrow trench.



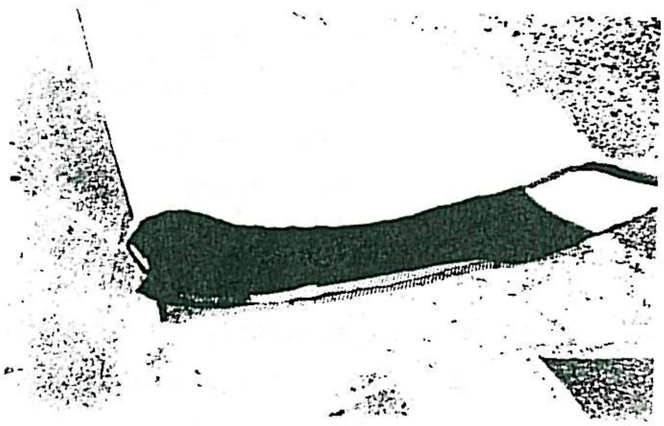
(a) Trench



(b) Components



(c) Assembly



(d) End product

(e) Installation

(f) Impermeable side

PHOTOGRAPH 9.1

Fin drains

The fact that fin drains can be assembled prior to installation and the availability of a large variety of materials, make fin drains versatile products that can be modified with relative ease for specific problems, site conditions and special applications. An example of a modified fin drain is shown in Photograph 9.1(f), where the geotextile on the road side of the trench was sprayed with a bituminous binder to form an impermeable membrane, thus eliminating the movement of moisture between the subgrade and the verge, a problem described in Chapter 1 (Section 1.1(4)). The road side geotextile could also have been replaced with an impermeable geomembrane (polyethylene sheet etc).

9.2.3 Tests and design parameters

The main disadvantage of fin drains is that no standardized test methods or design parameters exist at present which can be used to select the most suitable product for each particular application. Hunt (1982) identified three tests that need to be performed to evaluate the effectiveness of fin drains namely: the flow capacity of the core material (flow net), the filtration characteristics of the filter (geotextile) and the compression resistance of the flow net.

The characteristics of the geotextile filter have been dealt with in detail in this thesis and apply equally to fin drains and conventional side drains.

Dempsey (1988) performed "core flow capacity" tests on six fin drain materials, using a 8m long laboratory channel and concluded that the flow capacity of these materials were comparable or better than conventional materials. It is recommended that similar apparatus be manufactured, test methods standardized and design criteria established for use in South Africa.

The compression resistance of the core material must be adequate to ensure that the flow capacity is retained under stresses imposed on the installed fin drain. Standardized test methods and design criteria are also required for this aspect and could be incorporated in the core flow capacity test.

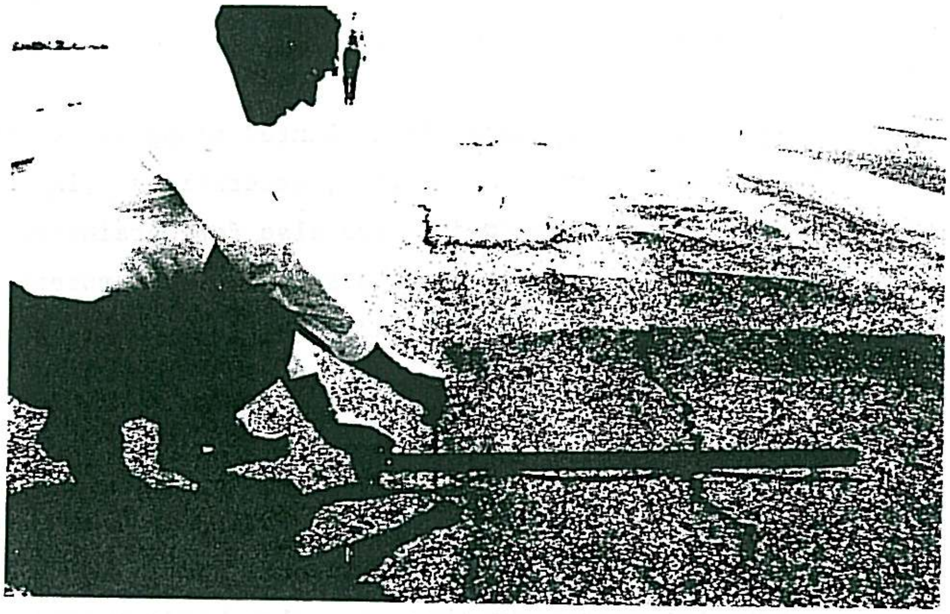
9.3 DRAINS IN THE PAVEMENT STRUCTURE

An alternative to a drainage layer below the subgrade (see Chapter 1, Figure 1.1(b)) is to place a system of subsurface drains below the pavement structure (in the subgrade) to remove water from the subgrade. The so-called "herringbone" system is well known where subsurfacing drains are installed in a herringbone pattern in high water table areas.

The runway pavement of the JG Strydom Airport outside Windhoek in South West Africa showed signs of distress due to excess moisture. It was decided to install subsurface drains in the subgrade to alleviate the problem. Photograph 9.2 shows major failures in the form of cracking and uplifting that became evident after the installation of these drains. These cracks (up to 40mm wide) and lifting of the surfacing (by up to 35mm) only occurred directly above the installed drains.

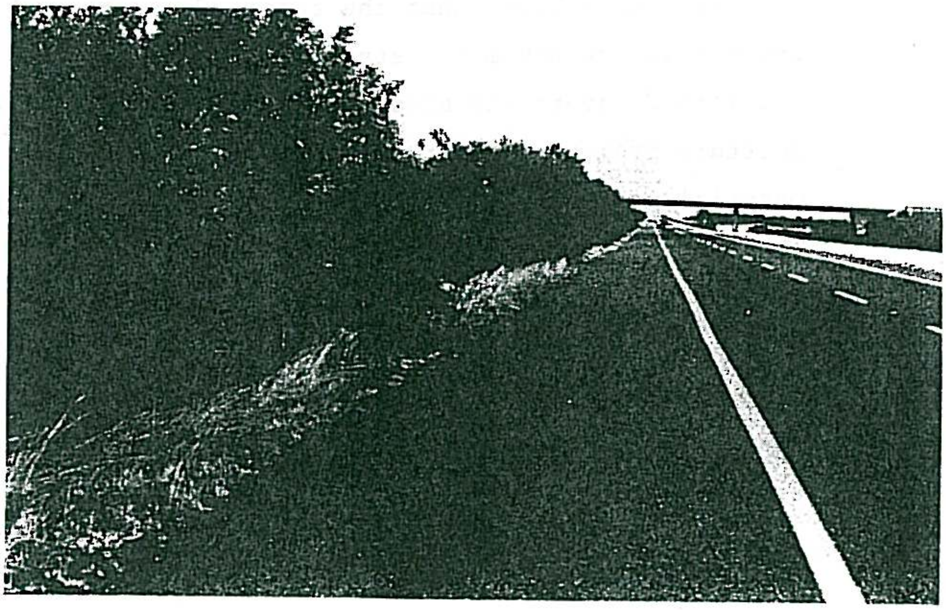
The pavement structure of the runway consists of a premix base on an unstabilized subbase. The drains (typical conventional side drains with granular material and geotextile filters) were covered with a stabilized layer to form the subbase in those areas. Thus the subbase, unstabilized elsewhere, is stabilized in those areas where drains were installed. These materials have different shrinkage characteristics and moisture and temperature changes (temperature changes are fairly severe in Windhoek) probably resulted in differential shrinkage resulting in cracking and uplifting of the premix layer.

It is concluded from this investigation that subsurface drains in a pavement structure should be installed below the pavement layers so as not to disturb the uniformity of the layers which can result in differential shrinkage and failure.



PHOTOGRAPH 9.2

Pavement distress caused by subsurface drains
(J G Strydom Airport, Windhoek)



PHOTOGRAPH 9.3

Pavement distress caused by vegetation
(N1, Pienaarsrivier)

9.4 VEGETATION

Trees and shrubs are often planted alongside a road for various reasons including aesthetics, separations (in the medium of a dual carriageway road) and also for drainage. Trees in the vicinity of drainage outlets serve the purpose of removing excess water that may accumulate should the outlet become partially blocked.

Karee trees (Rhus lancea) were planted alongside the N1 near Pienaarsrivier for removing excess moisture. These trees are known for their ability to remove large quantities of moisture from the ground but they are also hardy indigenous trees that can survive dry periods and cold weather.

The subgrade of the N1 in that area is a heavy clay which is prone to swell and shrinkage. Photograph 9.3 shows a typical situation that has arisen from these factors. Cracks have formed in the shoulder in the vicinity of the Karree trees. An investigation showed that the roots of the trees have actually grown into the pavement structure, forcing themselves between stabilized layers and also into the subgrade. The removal of moisture from the subgrade has resulted in shrinkage of that material, subsidence of the pavement and cracking of the surfacing.

This experience illustrates that vegetation should be kept at a safe distance from the pavement structure, especially in the case of clayey subgrades where a constant moisture content is essential.

9.5 CONSTRUCTION AND MAINTENANCE

During the course of the research described in this thesis, discussions were held with various consulting engineers, contractors and materials engineers from road authorities to determine those areas of subsurface drainage where problems exist and where research is required. It is noteworthy that no failed subsurface drainage systems that were reported were as a result of design inadequacies. Although many engineers expressed their hesitancy to use geotextiles as filters, failures as a result of these materials not functioning adequately could not be identified.

No existing failures could be located to be investigated by the author, but several past failures that had been repaired or replaced were described by those involved. For each case that was described, the reason for failure could be ascribed to either poor construction techniques or inadequate maintenance procedures.

In one side drain that was replaced, the pipes were totally blocked. Investigations revealed that rain had occurred during the initial installation after the pipes had been placed in the trench. Large quantities of soil were deposited in the pipes by the rain water and no attempt was made to clean the pipes before placing the granular material and the backfill. Subsequent water in the subsurface drain carried the soil to the pipes at the dawnstream end of the system, resulting in those pipes becoming completely blocked.

Maintenance of the drain outlets seems to be almost totally neglected in many areas. Outlets should be clearly marked both in the field and on plans and a routine maintenance schedule should be followed to ensure that the outlets are not blocked by vegetation or for any other reason.

Emphasis has been placed on the maintenance of subsurface drains in recent times, which has led to the inclusion of rodding eyes which allow the pipes to be cleaned. It is the author's opinion, however, that the emphasis on maintenance should be focused on the drain outlets. If the filter has been selected to avoid excessive piping and the drain has been constructed with care and under proper supervision, the need for rodding eyes does not really exist. The drain outlets, on the other hand, can become blocked and will become overgrown with natural vegetation, thus needing regular maintenance.

CHAPTER 10

CONCLUSIONS AND RECOMMENDATIONS

CHAPTER 10

CONTENTS | PAGE

10.1 CONCLUSIONS

Relevant conclusions were given in each chapter of this thesis. It is not intended to repeat all the conclusions here, but merely to give the most important conclusions in a summarized form. The relevant chapters are indicated.

10.1.1 Water sources (Chapter 1)

Water can enter a pavement structure from various sources and in different ways. If excess water is encountered in a road structure or if a threat is suspected, the source needs to be identified and investigated before deciding on applicable subsurface drainage methods.

10.1.2 The need for drainage (Chapter 2)

It was illustrated that water affects different pavement types in different ways and that excess pore water pressures play a significant role in water related deterioration.

Severity factors, which quantify the increase in the rate of pavement deterioration as a result of excess water, were obtained and found to vary between 1,5 and 820 for different pavement types. Although the method of water introduction in HVS tests is severe, a single field study indicated that the severity factors obtained from HVS tests may in fact be conservative.

An economic evaluation indicated that the installation of subsurface drainage systems will generally be justified if a water threat exists.

10.1.3 Geotextiles (Chapters 3, 4, 5, 6 and 7)

A variety of synthetic materials have been developed during the past 30 years, including geotextiles. These materials are versatile and can be used in a vast number of applications.

There is, however, a lack of standardized test methods, proven design criteria and field experience to aid the designer in selecting appropriate materials.

Existing filter criteria for geotextiles are largely based on theoretical analyses and in-isolation, non-standardized tests. A number of criteria exist, each of which gives a different answer to a specific problem. The criteria address piping and permeability and do not consider long term performance and clogging potential.

Soil/geotextile compatibility tests that have been performed by various researchers show similar tendencies and indicate a reduction in permeability over time, resulting from a filter zone forming in the soil structure. Filter criteria cannot be obtained from these results, however, mainly as a result of non-standardized test methods and apparatus.

Tests that were performed on available geotextiles as part of the work reported on in this thesis include in-isolation tests (hydraulic and mechanical), soil/geotextile compatibility tests and a field evaluation. Test procedures and apparatus were developed, evaluated and standardized and selection parameters were defined. Tests that were identified as suitable for use in the road industry are:

Tests recommended for geotextile selection:

- Penetration load (CBR) test
- Long term flow test (permeameter).

Other useful tests, not recommended for geotextile selection:

- Mass per unit area
- Constant head normal permeability
- Maximum opening size (MOS) by image projection.

Filter criteria were developed and are recommended for use in road subsurface drainage applications. These criteria control:

- piping
- permeability
- permeability reduction.

It is recommended that these criteria and the applicable test methods also be used for natural (granular) filters when these are being considered as alternatives to geotextiles.

Existing filter criteria were evaluated and found to give widely differing results. No correlations could be obtained between in-isolation characteristics, on which existing criteria are based, and long term performance.

10.1.4 Drainage layers (Chapter 8)

The use of a drainage layer in the pavement structure is not recommended as a method of subsurface drainage for highly trafficked rural roads. Such layers may be a viable solution in urban areas, but various aspects need to be evaluated including structural capacity, permeability of the drainage layer and the effective permeability of the underlying layer.

10.1.5 Fin drains (Chapter 9)

Fin drains, which contain synthetic water carriers or cores, have certain advantages over conventional side drains where granular materials are used as water carriers. The most significant advantages are lower cost and ease of construction. Test methods and design criteria need to be developed, however, for evaluating fin drains. These tests should include core flow capacity and compression resistance.

10.1.6 Other aspects (Chapter 9)

Poor construction techniques and inadequate maintenance are more often than not the main reasons for subsurface drainage systems not functioning effectively. Particular attention should be given to supervision during construction and the effective maintenance of drain outlets.

Subsurface drains in the pavement structure should be placed at subgrade level to avoid possible pavement distress.

Vegetation alongside the road can lead to pavement distress, especially in the case of clayey subgrades.

10.2 RECOMMENDATIONS FOR FUTURE RESEARCH

Certain areas were identified where further research is required. In a number of cases apparatus and test methods have been developed which can be used in future research. The areas that were identified include:

- Investigation of pore pressures in different pavement types, under different conditions and using different methods of water introduction with HVS tests and also in roads where natural wet conditions exist. Such an investigation will provide a clearer understanding of failure mechanisms and also give an indication of the relative severity of HVS-type water introduction. A pore pressure apparatus has been developed and tested under HVS testing.

- Field studies of pavement performance under wet and dry conditions to verify severity factors obtained from HVS tests.

- Durability of geotextiles against chemical, biological and ultra-violet attack. Test methods need to be selected and standardized and criteria need to be established.

- Field investigations of subsurface drainage systems to obtain acceptable limits for geotextile strength, piping and permeability reduction criteria. Methods and apparatus for such investigations have been developed and applied in one investigation.

- Evaluation of alternative materials for drainage layers, eg waterbound macadams and open-graded cemented materials. Methods and apparatus described in this thesis, including the pore pressure apparatus and the fluorescent dye technique, can be used in such investigations.

- Fin drains, including the development of stardardized test methods and selection criteria and an investigation into aspects related to compaction in narrow trenches.

REFERENCES

AGERSCHAU, H A. 1961. Synthetic material filters in coastal protection. Journal of American Society of Civil Engineers (Waterways and Harbours Division), 87, no. WW1, 111-124
(as referred to by Rankilor, 1981).

ALFHIEM, S L and SORLIE, A. 1977. Testing and classification of fabrics for application in road constructions. Proceedings of the International Conference on the use of Fabrics in Geotechnics, Paris, 333 - 338

ATTERBERG, A. 1908. Studies in the field of soil science. Sandu Versuchsanstalt, 69
(as referred to by Rankilor, 1981)

BAUDONNEL, J, GIROUD, J P and GOURC, J P. Experimental and theoretical study of tensile behaviour of nonwoven geotextiles. Proceedings of the Second International Conference on Geotextiles, Vol III, 823 - 828

BECKHAM, W K. 1935. Cotton-fabric-reinforced roads. Engineering News Record, 115,14, 453-455
(as referred to by Giroud, 1986).

BERTRAM, G E. 1940. An experimental investigation of protective filters. Graduate School of Engineering, Harvard University, Publication No 267.
(as referred to by Cedergren, 1967).

CALHOUN, C C. 1972. Development of design criteria and acceptance specifications for plastic filter cloths. Vicksburg, Mississippi: Army Engineer Waterways Experiment Station, Technical Report S-72-7, 98p.

CEDERGREN, H R. 1967. Seepage, drainage and flow nets. New York, John Wiley, 534 pp.

CEDERGREN, H R. 1974. Drainage of highway and airfield pavements. New York, Wiley, 285 pp.

CLEAVER, C H and JONKER, P R 1985. Geotextile appreciation and application - a resume. Municipal Engineer, August 1985, 49-51.

COETZEE, C H. 1988 Accelerated testing of thin jointed concrete pavements. Pretoria, DRTT, CSIR, Research Report DPVT-8.

DAVIES, P L. 1986. Geotextiles in South Africa - History and materials available. Filters Symposium, Johannesburg: South African Institution of Civil Engineers (Geotechnical Division), 12p.

DE BEER, M. 1985. Behaviour of cementitious subbase layers in bitumen base road structures. Pretoria: University of Pretoria, M Eng dissertation.

DE BEER, M. 1986(a). The development of a DCP classification system for pavements with lightly cemented materials (in Afrikaans). Pretoria, DRTT unpublished report, TP/53/86.

DE BEER, M. 1986(b). Mechanistic analysis: HVS Section 289A4 (Rooiwal). (in Afrikaans) Pretoria, DRTT unpublished report, TP/91/86.

DE BEER, M. 1987. Mechanistic analysis: HVS Section 294A4 (Rooiwal), (in Afrikaans). Pretoria, DRTT unpublished report, TN/43/87.

DE BEER, M and HORAK, E, 1987. The effect of poor drainage on pavement structures studied under HVS testing. Proceedings of the Annual Transportation Convention (ATC 1987), Vol 5B, Pretoria. (DRTT Reprint RR 506).

DE BEER, M. 1988. Mechanistic analysis. HVS Section 306A4 (Road 2212, Bultfontein). (in Afrikaans). Pretoria, DRTT unpublished report I/FP/16/88.

DE GARIDEL, R and MOREL, G. 1986. New soil strengthening techniques by textile elements for low-volume roads. Proceedings of the Third International Conference on Geotextiles, Vol I, Vienna, 153-158.

DE MELLO, V F B. 1975. Some lessons from unsuspected, real and fictitious problems in earth dam engineering in Brazil. 6th African Regional Conference on Soil Mechanics and Foundation Engineering, Durban, South Africa, Vol II, 285 - 305.

DE MELLO, V F B. 1977. Reflections on design decisions of practical significance to embankment dams, London, Geotechnique 27, No 3, 279 - 355.

DEMPSEY, B J. 1982. Laboratory and field studies of channelling and pumping. Washington, DC. Transportation Research Record No 849,1-12.

DEMPSEY, B J. 1988. Core flow capacity requirements of geocomposite fin-drain materials utilized in pavement drainage, University of Illinois, Paper prepared for 67th TRB Meeting, Washington, DC.

DIVISION OF ROADS AND TRANSPORT TECHNOLOGY (DRTT), 1984(a). Bituminous pavement rehabilitation design. Pretoria, CSIR, Technical Recommendations for Highways, Draft TRH12, 72 pp.

DIVISION OF ROADS AND TRANSPORT TECHNOLOGY (DRTT). 1984(b). Subsurface drainage for roads. Pretoria, CSIR, Technical Recommendations for Highways, Draft TRH15, 64pp.

DIVISION OF ROADS AND TRANSPORT TECHNOLOGY (DRTT), 1985(a). Structural design of interurban and rural road pavements. Pretoria, CSIR, Technical Recommendations for Highways, TRH4, 70 pp.

DIVISION OF ROADS AND TRANSPORT TECHNOLOGY (DRTT), 1985(b). Guidelines for road construction materials. Pretoria, CSIR, Technical Recommendations for Highways, TRH14, 57 pp.

FEDERAL HIGHWAY ADMINISTRATION (FHWA). 1972. Guidelines for the design of subsurface drainage systems for highway structural sections. Washington, DC. Report No. FHWA-72-30.

FIELD, R, MASTERS, H and SINGER M. 1982. An overview of porous pavement research. American Water Resources Association, Water Research Bulletin, Vol 18, No 2.

FREEME, C R. 1983. Evaluation of pavement behaviour for major rehabilitation of roads. Pretoria, NITRR, Technical Report RP/19/83.

FREEME, C R and COETZEE, C, 1985. Reference guide to information on Heavy Vehicle Simulator testing - March 1984 to February 1985. Pretoria, DRTT unpublished report, TP/57/85.

FREEME, C R, SERVAS, V P and WALKER R N, 1986. Pavement behaviour as determined from Heavy Vehicle Simulator testing. Proceedings of the 1986 International Conference on Bearing Capacity of Roads and Airfields, Plymouth, England. (DRTT Reprint RR 436).

FULTON, F S. 1977. Concrete Technology. Johannesburg, The Portland Cement Institute.

GERKE, R J. 1979. Subsurface drainage progress report September 1979. Australia Road Research Board. Report No AIR 317 - 1.

GIROUD, J P. 1982. Filter criteria for geotextiles. Proceedings of the Second International Conference on Geotextiles, Vol I, Las Vegas, 103-108

GIROUD, J P. 1986. From geotextiles to geosynthetics: a revolution in geotechnical engineering. Proceedings of the Third International Conference on Geotextiles, Vol I, Vienna, 1-18.

GRAY, D H and Al-REFEAI, T. 1986. Behaviour of fabric-versus fiber-reinforced sand. Journal of Geotechnical Engineering, Vol 112, no 8, 804-820.

GREENWAY, D R, HO, K W and BRAND, E W. 1986. Filtration and degradation behaviour of some non-woven geotextiles in Hong Kong. Proceedings of the Third International Conference on Geotextiles. Vol. II, Vienna, 605-609.

HALIBURTON, T A and WOOD, P D 1982. Evaluation of the U S Army Corps of Engineer gradient ratio tests for geotextile Proceedings of the Second International Conference on Geotextiles, Vol I, Las Vegas, 97-101

HEERTEN, G, 1982. Dimensioning the filtration properties of geotextiles considering long-term conditions. Proceedings of the Second International Conference on Geotextiles. Vol I, Las Vegas, 115-120.

HOEKSTRA, S E and BERKHOUT, H C. 1986. Geotextile/"geospacer" composites for environmental projects. Proceedings of the Third International Conference on Geotextiles, Vol II, Vienna, 343-347.

HOOVER, T P (1982). Laboratory testing of geotextile fabric filers. Proceedings of the Second International Conference on Geotextiles. Vol. III, Las Vegas, 839-843.

HORAK, E and MAREE, J H, 1982. The behaviour of a slag base pavement structure at P39/1 Erasmia under Heavy Vehicle Simulator loading. Pretoria, DRTT Technical Report RP/5/82.

HORAK, E and TRIEBEL R H H, 1986. Waterbound macadam as a base and as a drainage layer. Washington DC, Transportation Research Record No 1055. (DRTT Reprint RR446).

HUNT, J A. 1982. The development of fin drains for structure drainage. Proceedings of the Second International Conference on Geotextiles, Vol I, Las Vegas, 25 - 30.

ICI Fibres. 1978. Designing with Terram. Pontypool, Gwent, Great Britain: ICI Fibres, Terram Section
(as referred to by Rankilor, 1981).

JORDAAN, G J 1985. A practical approach to the economic appraisal of pavement rehabilitation options. Pretoria, DRTT, CSIR, Technical Report RP/10.

JORDAAN, G J. 1987. Personal communication. Pretoria: NITRR.

KARPOFF, K P. 1955. The use of laboratory tests to develop design criteria for protective filters. ASTM, Vol 55, 1183.

KLEYN, E G, FREEME, C R and TERBLANCE, L J, 1985. The impact of Heavy Vehicle Simulator testing in the Transvaal. Proceedings of the 1985 Annual Transportation Convention (ATC 1985), Pretoria.

KORRNER, R M and KO, F K. 1982. Laboratory studies on long term drainage capability of geotextiles. Proceedings of the Second International Conference on Geotextiles, Vol I, Las Vegas, 91-95.

LAWSON, C. 1986. Geotextile filter criteria for tropical residual soils. Proceedings of the Third International Conference on Geotextiles, Vol II, Vienna, 557-562.

LEGGE, K R. 1986. A new approach to geotextile selection. Filters Symposium, Johannesburg. South African Institution of Civil Engineers (Geotechnical Division), 21p

LEGGE, K R. 1987. Personal communication.

LEGGE, K R and VAN DER MERWE, C J, 1987. The testing of geotextiles. Proceedings of the Annual Transportation Convention, Vol. 5B, Pretoria.

MACLEAN, M D and MACMASTER, B. 1984. The use of geotextiles in improving the drainage of pavement structures. Transportation Forum, Vol 1-2, 49-57.

MAIN ROADS DEPARTMENT. 1983. Evaluation of geotextiles. Brisbane, Report R1324, 52p.

MAREE, J H, 1978. Design parameters for crushed stone in pavements, (in Afrikaans). Pretoria. University of Pretoria, M Eng dissertation.

MAREE, J H, 1982. Aspects of design and behaviour of pavements with granular bases, (in Afrikaans). Pretoria. University of Pretoria, D Eng Thesis.

MAREE, J H, FREEME, C R, VAN ZYL N J and SAVAGE, P F, 1982. The permanent deformation of pavements with untreated crushed-stone bases as measured in Heavy Vehicle Simulator tests. Proceedings of the 11th Australian Road Research Board Conference, Melbourne, p16-28 (DRTT Reprint RR 355).

MARKS, B D. 1975. The behaviour of aggregate and fabric filters in sub-drainage applications, Tennessee, University of Tennessee Report (as referred to by Rankilor, 1981).

MCGOWN, A and SWEETLAND, D B. 1973. Fabric screen research and development, University of Strathclyde Report, Dept of Civil Engineering (as referred to by Rankilor, 1981).

MCGOWN, A and KABIR, M H. 1982. Compressibility and hydraulic conductivity of geotextiles. Proceedings of the Second International Conference on Geotextiles. Vol I, Las Vegas, 167-172

MURRAY, R T and MCGOWN, A. 1982. The selection and testing procedures for the specification of geotextiles. Proceedings of the Second International Conference on Geotextiles, Vol II, Las Vegas, 291-296.

MYLES, B and CARSWELL, I G. 1986. Tensile testing of geotextiles. Proceedings of the Third International Conference on Geotextiles, Vol III, Vienna, 713 - 718.

OGINK, H J M. 1977. Investigations on the hydraulic characteristics of synthetic fabrics. Delft: Delft Hydraulics Laboratory, Publication No 146.
(as referred to by Rankilor, 1981).

POLYCROFT, D and DENNIS JONES, P. 1982. Geotextile performance and current filter criteria. Proceedings of the Second International Conference on Geotextiles, Vol I, Las Vegas, 67-72.

RAGUTZKI, G. 1973. Beitrag zur ermitlung der filterwirtsamkeit durchlassiger kunsstoffe. Forschungstelle fur Insel und Kusstenshutz. Sonderdruck aus Jahrebericht. Band 25.
(as referred to by Wates, 1986).

RANKILOR, P R. 1981. Membranes in ground engineering. Chichester: Wiley, 377p.

RAUMAN, G. 1982. Geotextiles: Construction materials in evolution. Proceedings of the Second International Conference on Geotextiles, Vol IV, Las Vegas, 10-15.

RIDEGEWAY, H H 1982. Pavement subsurface drainage systems. Washington, DC, Transportation Research Board, NCHRP Synthesis Report 96, 38 pp.

RUSSEL, G.E. Hydraulics. New York, Henry Holt & Co, 1941

SARAF, C, CHOU, C and MCCULLOUGH, B F, 1987. Effect of Rainfall on the performance of continuously reinforced concrete pavements in Texas. Washington DC, Transportation Research Record No 1121, 45-49.

SCULLION, T, LUX, S M and HOLLAND, C N. 1984. Testing procedures, specifications and applications for geofabrics in highway pavements. College Station, Texas: Texas Transportation Institute, Research Report 414-1F, 100p.

SCHEURENBERG, R J. 1986. Internal stability of granular filters. Filters Symposium, Johannesburg, South African Institution of Civil Engineers (Geotechnical Division), p13.

SCHOBER, W and TEINDL, H. 1979. Filter criteria for geotextiles. Proceedings of the Ninth European Conference on Soil Mechanics and Foundation Engineering, Brighton, England
(as referred to by Wates, 1986).

SHERARD, J L, RICHARD J W, et al. 1963. Earth and earth rock dams. New York, Wiley, p84.

SHERARD, J L. 1984. Basic properties of sand and gravel filters. Journal of Geotechnical Engineering, ASCE, Vol 110, No 6.

SOUTH AFRICAN BUREAU OF STANDARDS (SABS). 1987. Draft Specification for Geotextiles. Unpublished, Pretoria.

TERZAGHI, K. 1922. The failure of dams and its prevention, Die Wasserkraft, 178, 445 - 449.
(as referred to by Terzaghi and Peck, 1948).

TERZAGHI, K and PECK, R S. 1948. Soil Mechanics in engineering practice. New York. John Wiley & Sons, Inc. 729p.

TRANSPORT AND ROAD RESEARCH LABORATORY. 1952. Soil mechanics for road engineers. London, HMSO, 541p.

TRANSPORTATION RESEARCH BOARD. 1982. Pavement subsurface drainage systems. Washington, DC. NCHRP Synthesis document 96.

TRANSVAAL PROVINCIAL ADMINISTRATION (TPA), Roads Department 1988. Contract unit prices: roads and bridgework, (in Afrikaans). Pretoria, TPA internal document K20.

US ARMY CORPS OF ENGINEERS. 1941. Investigation of filter requirements for underdrains. Technical Memorandum No 183 - 1.
(as referred to by Cedergren, 1967).

US ARMY CORPS OF ENGINEERS. 1977. Plastic Filter Cloths. Washington, DC: Office, Chief of Engineers, Civil Works Construction Guide Specification No CW-02215
(as referred to by Haliburton, et al, 1982)

VAN DER MERWE, C J. 1985. The construction of experimental sections on Trunk Road 86 near East London. Pretoria, DRTT unpublished report TPC/5/85.

VAN DER MERWE, C J and HORAK E. East London HVS experimental sections. Objectives and preliminary investigation. Pretoria, DRTT unpublished report TPC/1/86.

VAN DER MERWE, C J. 1987(a). Geotextile evaluation: normal permeability. Pretoria, DRTT unpublished report TC/30/87.

VAN DER MERWE, C J. 1987(b). Apparatus for long term soil (geotextile compatibility tests; description and evaluation. Pretoria, DRTT unpublished report TC/20/87.

VAN WYK, A J and LOVELL C W, 1984. Importance of drainage to rigid pavement performance. Ohio River Valley Soils Seminar XV (Drainage), 25 pp.

VAN ZYL, N J W and MAREE, J H, 1983. The behaviour of a high-standard crushed-stone base pavement during a heavy vehicle simulator test. Johannesburg, The Civil Engineer in South Africa, July 1983. (DRTT Reprint RR 358).

VICTORIA ROAD CONSTRUCTION AUTHORITY. 1984. Drainage of subsurface water from roads. Victoria. Technical Bulletin No 32.

VILJOEN, A W, 1987. Recent development in concrete pavements. Paper delivered at Roads Information Forum, April 1987, Pretoria, DRTT.

WATES, J A. 1986(a). Filter criteria for natural filters. Filters Symposium, Johannesburg, South African Institution of Civil Engineers (Geotechnical Division), 19p.

WATES, J A. 1986(b). Filter criteria for synthetic materials. Filters Symposium, Johannesburg; South African Institution of Civil Engineers (Geotechnical Division), 16p

WATES, J A and WITTSTOCK, G H. 1986. Selection of geofabrics for filtration applications with particular reference to gold tailings. The Civil Engineer, March 1986, 83-89.

WEATHER BUREAU. 1983. Extreme values of rainfall, temperature and wind for selected return periods. Pretoria, Department of Transport, Report WB 36.

ZITSCHER, F F. 1975. Recommendations for the use of plastics in soil and hydraulic engineering. Die Bautechnik, 52, 12, 397-402
(as referred to by Rankilor, 1981)

APPENDIX B

RECOMMENDED TESTS AND TENTATIVE CRITERIA FOR GEOTEXTILES IN ROAD SUBSURFACE DRAINAGE APPLICATIONS

RESEARCH REPORT DPVT 92

RECOMMENDED TESTS AND TENTATIVE CRITERIA FOR GEOTEXTILES IN ROAD SUBSURFACE DRAINAGE APPLICATIONS

C J van der Merwe

Pavement Engineering Technology : E Horak
Reviewed by: A J Laatz

November 1989 ISBN 0 7988 4862 6 Research Report DPVT 92

Roads and Transport Technology
Pad- en Vervoer-tegnologie
CSIR
WNNR

DOCUMENT RETRIEVAL

Report No: DPVT 92
ISBN 0-7988-4862-6

Date of issue
November 1989

Pages
(ii) + 23

Title

Recommended tests and tentative criteria for geotextiles in subsurface drainage applications.

Author(s)

C J van der Merwe

Issuing organisation

Flexible Pavements Programme
Division of Roads and Transport Technology
CSIR, P O Box 395, Pretoria, 0001, Republic of South Africa

Keywords

Subsurface drainage, geotextiles, geosynthetics, test methods

Synopsis

The designer of road subsurface drainage installations is usually faced with a problem when having to select the most suitable geotextile. This is due to a lack of long term performance records, standard test methods and proven design criteria on which to base the selection. In this report test methods are described and criteria are given that are seen to be applicable to road subsurface drainage applications. These are based on research at the Division and include in-isolation tests as well as soil/geotextile compatibility tests.

Sinopsis

Dit is dikwels moeilik om die mens geskikte geotekstiel te bepaal vir gebruik in ondergrondse dreinering van paaie. Dit is as gevolg van die feit dat daar min inligting bestaan oor die langtermyngedrag van geotekstiele en ook die gebrek aan standaard toetsmetodes en ontwerpriglyne. Hierdie verslag is gebasseer op navorsing by die Divisie en bevat beskywings van aanbevole toetsmetodes asook voorgestelde ontwerpriglyne. Toetsmetodes sluit in toetse op geotekstiele alleen asook grond-geotekstiel versoenbaarheidstoetse.

CONTENTS

1. INTRODUCTION

Geotextiles are used fairly widely in road subsurface drainage applications. These materials have many advantages. They are durable, light weight, readily available and their characteristics are controlled under factory conditions. The most important disadvantage has been the lack of long term performance records, standard tests and design criteria on which to base geotextile selection and subsurface drainage design.

Two basic types of geotextile tests exist namely in-isolation tests, where tests are performed on geotextiles only and soil/geotextile compatibility type tests. Although in-isolation tests provide useful information regarding geotextile characteristics, grade and consistency, it is not recommended that geotextile selection and drainage design be based purely on such tests. The use of compatibility tests, such as that described in Section 4.1 of this report, should be incorporated where possible.

Research by the Division of Roads and Transport Technology in this area[1,2,3,4] has included the development and evaluation of various tests on geotextiles. Based on this research the most applicable tests for geotextiles in road subsurface drainage have been selected and are described in this document. Tentative criteria are also given. It should be noted that these criteria are based mainly on laboratory research and limited field experience. Feedback from industry may well indicate the need to adjust these values.

The South African Bureau of Standards has issued a code of practice on geotextile testing[5] which includes the description of certain in-isolation geotextile tests. Those SABS tests that have been identified as applicable for road subsurface drainage are recommended and described without any deviations under In-isolation Tests (Section 3) of this report. The standard atmosphere for testing given in Section 2.1.1, however, deviates from that recommended by the SABS which recommends a humidity of between 45 and 55 per cent and a temperature of between 21 and 25 °C. Experience gained at the Division suggests that these limits are probably too stringent for road subsurface drainage geotextile testing.

It is the aim of this document to provide, as a first attempt, standardized geotextile tests and criteria for road subsurface drainage applications. Section 2 below describes the recommended procedure for the preparation of geotextile tests. In Sections 3 and 4 the actual test methods are described in detail for in-isolation and compatibility tests respectively. Tentative criteria are given in Section 5. It is recommended that these tests and criteria be applied in practice and evaluated. Any comments will be welcomed and can be sent to the Division of Roads and Transport Technology, PO Box 395, PRETORIA 0001.

2. PREPARATION OF GEOTEXTILE SPECIMENS

2.1 Conditioning of geotextile samples

2.1.1 Standard atmosphere for testing

An atmosphere having a relative humidity of not more than 80 % and a temperature of between 15 °C and 35 °C.

2.1.2 Conditioning

Before a geotextile specimen is cut from the sample, condition the sample for a period of 24 hours in the standard atmosphere for testing, ensuring that there is a free flow of conditioned air through the sample.

2.2 Apparatus

(a) Stencils. Means for marking the position of specimens on the conditioned sample, including, where necessary, the accurate positioning of any holes for the passage of bolts or locating pins. Where holes are required in a specimen, the diameter of the locating hole in the template should exceed that of the bolt or pin by 3 mm.

(b) Cutting devices. Means for cutting a specimen to the accuracy required for the particular test.
NOTE: Hot-wire and rotating cutting devices are liable to fuse the fibres on the cut edges and such devices should not be used on specimens for the determination of mass per unit area and tensile strength.

(c) Punch. Means for punching out holes in the specimen that are of diameter 1 mm greater than that of the bolt or pin.

2.3 Preparation of specimens

(a) Lay the conditioned sample on the cutting table, ensuring that the sample is free from tension and is lying flat in the standard atmosphere for testing.

(b) Using a felt-tipped pen and a stencil or other suitable device, mark the position of the specimens on the sample. So position the specimens that:

(i) the distance between adjacent specimens is at least 50 mm;
(ii) the distance between a specimen and the edge or cut end of the sample is at least 150 mm;
(iii) damaged or creased areas are avoided; and
(iv) the specimens represent the geotextile as fully as possible.

(c) Where holes for bolts or pins have to be provided, use the felt-tipped pen to mark their position on the sample, ensuring that a full circle is marked.

2.4 **Cutting of specimens**

(a) Using a suitable cutting device, cut the specimen out of the sample to the accuracy required for the particular test.

(b) Where holes for bolts or pins have to be provided, carefully centre the punch over the marked position of the hole and, with the punch at right angles to the surface of the sample, punch out the hole with a single blow of a suitable hammer.

(c) Where specimens are to be tested in a specific direction, clearly mark on the specimen the machine direction of the geotextile.

2.5 **Storage of specimen**

Lay all specimens out flat and singly and maintain them in the standard atmosphere until testing is commenced.

3. IN-ISOLATION GEOTEXTILE TESTS

3.1 Mass per unit area

3.1.1 Apparatus

(a) Cutting device. Means for cutting out a square specimen of size 250 mm x 250 mm to an accuracy of 1 % (or better).
NOTE: Care must be exercised during the cutting and handling of specimens to prevent loss of fibres.

(b) Balance. A balance having a sensitivity of 0,001 g (or better).

3.1.2 Procedure

(a) Conduct the test on at least 10 specimens in the standard atmosphere for testing.

(b) Weight the specimen and record its mass to the nearest 0,01 g.

3.1.3 Calculation

Calculate the arithmetic mean of the 10 results. From this result calculate the mass per unit area of the geotextile, using the following formula:

Mass per unit area, $g/m^2 = 16\ M$

where M = arithmetic mean of mass of the specimens, g

Round off the result correct to 1 g/m^2.

3.2 Penetration load (CBR mould)

3.2.1 Apparatus

(a) Testing machine. A suitable compression testing machine of sufficient capacity and having:

(i) an accuracy and a repeatability that comply with the requirements for Grade A machines of BS 1610;

(ii) suitable bearing platens to provide for the attachment of the plunger to the upper platen and the accurate centring of the supporting sleeve/clamping ring assembly on the lower platen;

(iii) a rate of movement of the plunger of 60 ± 10 mm/min; and

(iv) an autographic recording device providing values of applied load and plunger travel distance accurate to 1 % (or better).

(b) Stencil. A suitable stencil as described in 2.2(a).

(c) Cutting device. Means for cutting out a circular specimen of diameter at least 250 mm.

(d) Punches. Means for punching out holes in the specimen that are of diameter 1 mm greater than those of the bolts and locating pins.

NOTE: The positions of the holes are marked with the felt-tipped pen, closely following the circumference of the hole in the stencil. The punch is then placed centrally over the marked position. If the positioning of holes is not accurate, the specimen will not lie flat on the mounting device.

(e) Clamping device. Means of clamping and positioning the specimen, and consisting of

(i) a mounting platen/supporting sleeve assembly (CBR mould or equivalent) (See Figure 1);

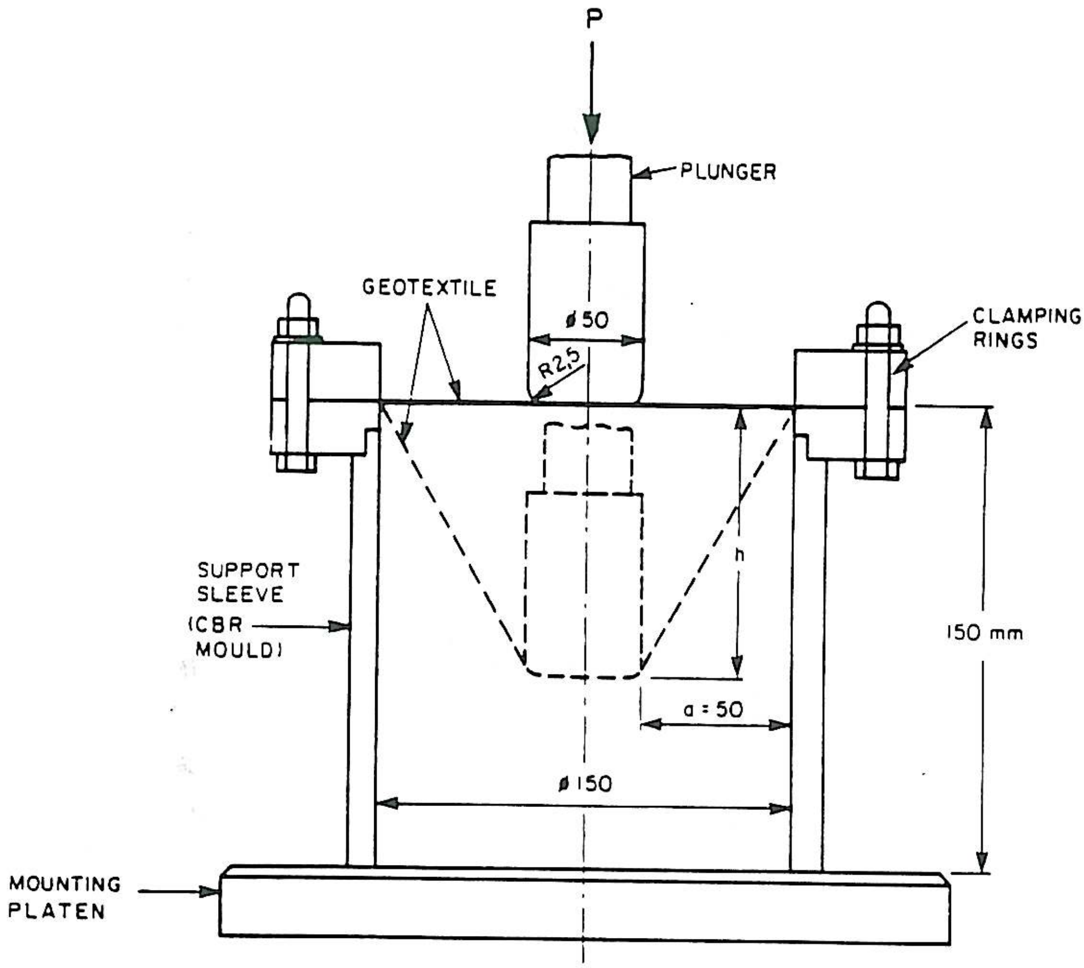
(ii) upper and lower clamping rings provided with securing bolts, locating pins and concentric matching "V" grooves for securing the specimen (see Figure 2).

(f) Mounting device (see Figure 3). Means to ensure that the specimen is located in the clamping device in a true plane and free from folds and stress, and consisting of

(i) a metal base;

(ii) a centring device secured to the baseplate and rebated to receive the lower clamping ring; and

(iii) two sockets welded to the baseplate to receive the heads of two diametrically opposite bolts to prevent rotation of the clamping rings during the initial tightening of bolts.

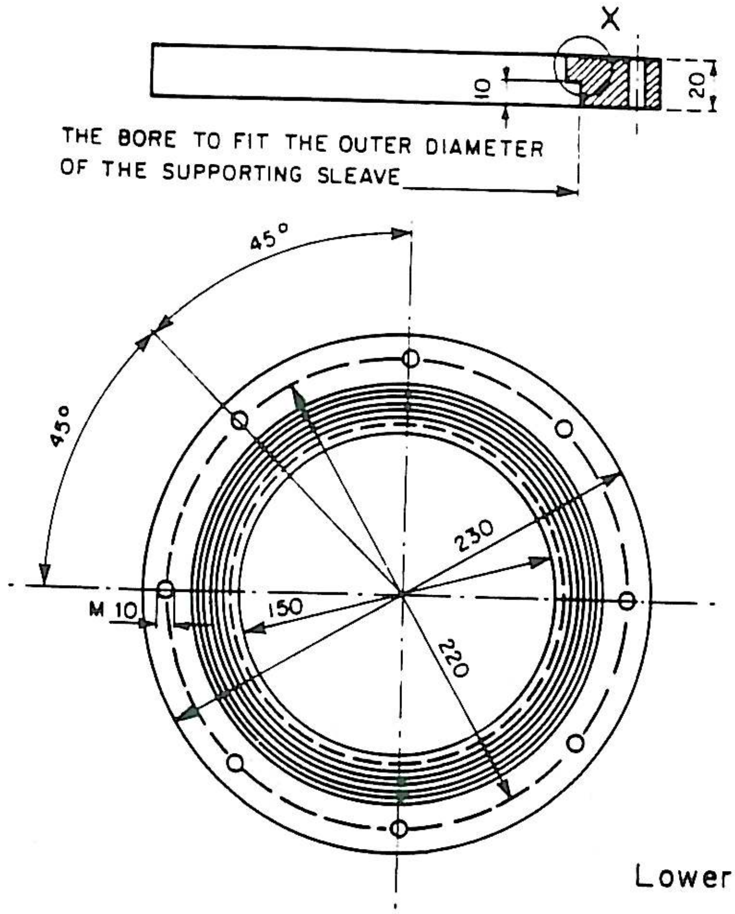


PENETRATION LOAD $= P$

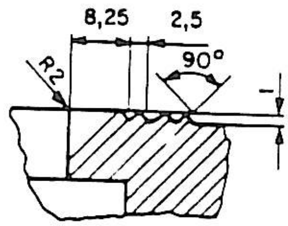
ELONGATION $= \dfrac{\sqrt{a^2 + h^2} - a}{a} \times 100$

FIGURE 1

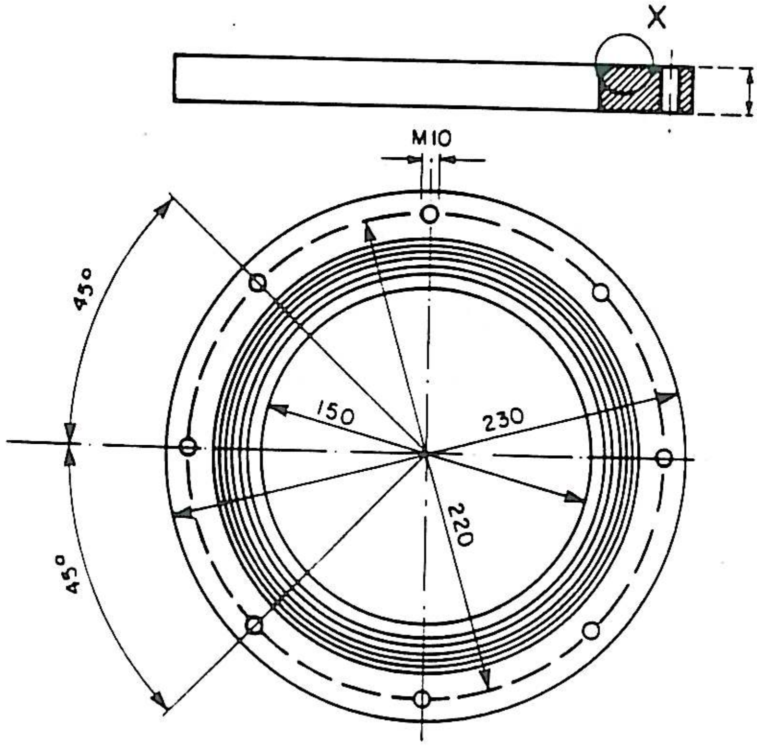
PENETRATION LOAD (CBR) TEST ARRANGEMENT



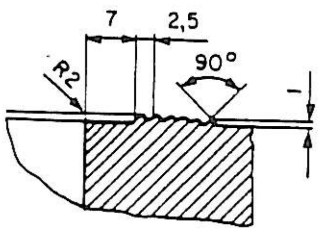
Detail X



Lower clamping ring



Detail X



Upper clamping ring

FIGURE 2

DETAILS OF CLAMPING RINGS

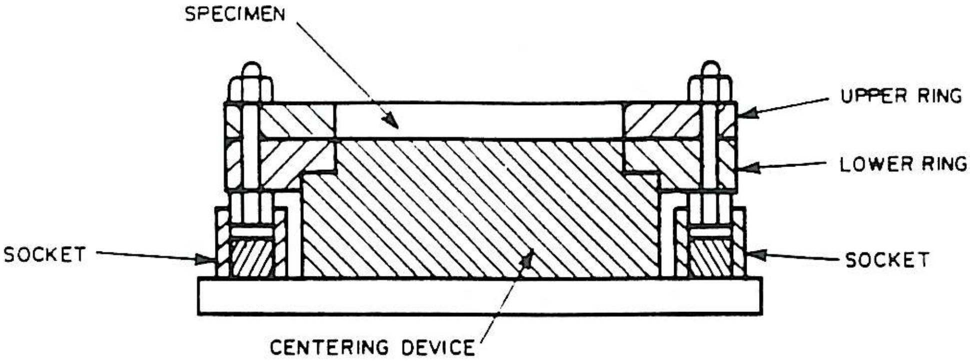


FIGURE 3

MOUNTING DEVICE

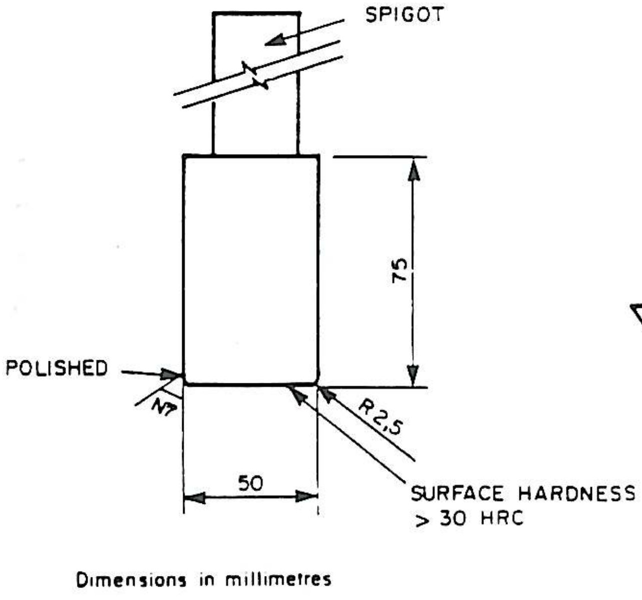
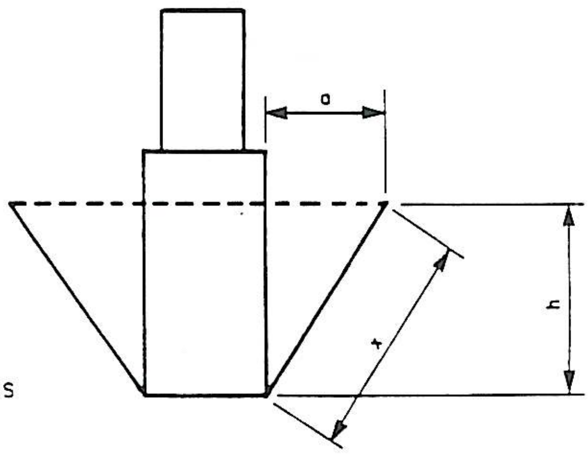


FIGURE 4

PLUNGER



$$x = \sqrt{a^2 + h^2}$$

FIGURE 5

SPECIMEN ELONGATION

(g) Plunger (see Figure 4). Means to apply the load to the specimen, and consisting of a cylindrical plunger provided with a suitable spigot for securing to the upper platen. The base and sides of the plunger are polished to prevent snagging of the specimen during application of load. (A CBR plunger with a rounded edge is a suitable plunger.)

In order to protect the polished surfaces from damage during storage and positioning or removal of the clamping ring assembly, a removable protective covering should be provided.

(h) Torque wrench. An adjustable torque wrench to ensure even tightening of bolts.

(i) Felt-tipped pen. A 3 mm wide felt-tipped pen to mark the positions of holes and an outline of the specimen to be cut out of the sample.

3.2.2 Specimens

Conduct the test on at least 10 specimens in the standard atmosphere for testing.

3.2.3 Procedure

(a) Assembly

(i) Place the lower clamping ring, with bolts in position, on the mounting device and carefully position the specimen over the bolts and locating pins, ensuring that it is free from tension and is lying flat.

(ii) Position the upper clamping ring over the bolts and pins and carefully lower it onto the specimen. Secure the ring in position by first tightening the two bolts held by the two sockets on the mounting device and then tightening the remainder of the bolts, tightening all nuts to a torque of 10 N.m.

(iii) With the felt-tipped pen, draw a reference line on the face of the specimen where contact is made with the inner face of the upper clamping ring, to determine whether slippage of the specimen occurs during the test. If slippage occurs, discard the specimen and increase the torque with the next specimen for all bolts. Report any change in the value of the torque.

NOTE: Excessive torque may damage the specimen.

(b) Testing. Place the clamping ring assembly on the supporting cylinder and remove the protective cover from the plunger. Set the autographic recorder in motion and operate the machine until the specimen ruptures.

3.2.4 **Calculation**

(a) Penetration load at rupture. Calculate the arithmetic mean of the loads at rupture obtained from the recorder for each specimen. Round off the result correct to 1 N.

(b) Elongation at rupture. Determine from the autographic recorder the travel distance of the plunger at rupture and calculate the arithmetic mean.
Calculate the elongation at rupture, using the following formula (see Figures 1 and 5):

$$V = \frac{\sqrt{(a^2 + h^2} - a}{a} \quad 100$$

where a = distance from inner edge of clamping ring to face of plunger = 50 mm
h = arithmetic mean of travel distances at rupture for each specimen, mm
V = elongation at rupture, %

Round off the result correct to 1 %.

(c) Load/elongation curves. Where it is desired to obtain curves of load versus elongation, the calculations given in (a) and (b) above can be employed, using either

(i) the loads for predetermined travel distance of the plunger; or
(ii) the travel distance of the plunger for predetermined loads, the elongation being calculated from the formula given in (b) above.

3.3 Tensile strength

3.3.1 **Apparatus**

(a) Testing machine. A suitable tensile strength testing machine of sufficient capacity and having

(i) an accuracy and a repeatability that comply with the requirements for Grade A machines of BS 1610;

(ii) two sets of jaws (one for woven specimens and one for non-woven specimens) of width greater than that of the ends of the test specimen, of a type that will not cut or damage the specimen and with one or both jaws free to swivel;

(iii) a constant rate of traverse of the pulling jaw of 60 ± 10 mm/min; and

(iv) an autographic recording device providing values of applied load and plunger travel distance accurate to 1 % (or better).

(b) Cutting device. Means of cutting out rectangular specimens of width at least 100 mm in the case of woven geotextiles, and at least 250 mm in the case of non-woven geotextiles, accurate to 1 % (or better);
NOTE: The length of the specimens must be sufficient to allow for a gauge length of 80 mm between the edges of the jaws.

3.3.2 Specimens

(a) General. In all cases conduct the test on at least 5 specimens in the machine direction and 5 specimens in the cross direction in the standard atmosphere for testing. Cut specimens of the appropriate dimensions given in 3.3.1(b).

(b) Preparation of woven specimens. Reduce the width of the specimen to 80 mm by the removal of an equal number of threads from each edge, taking care not to distort the remaining threads.

3.3.3 Procedure

Place the specimen symmetrically in the jaws of the machine with the axis of the specimen parallel to and coinciding with the axis of application of the force. Lightly clamp the specimen in position so that the distance between the jaws is 80 mm. Draw two reference lines on the face of the specimen where contact is made with each jaw, to determine whether slippage of the specimen occurs during the test. Apply a mounting tension to the specimen equivalent to approximately 0,25 % of the probable maximum load and clamp the specimen.

Set the autographic recorder in motion and operate the machine until the specimen breaks.

Record the breaking load and breaking extension except where

(a) the break occurs in two or more stages, in which case record as the breaking load and breaking extension those indicated in the course of breaking at the first stage; and

(b) the break starts within 5 mm of the line of contact at either of the jaws at a load substantially less than the average for normal breaks or if slippage occurs in one or both jaws. In such a case, record the fact and discard the results of breaking load and breaking extension for that specimen and repeat the test on an additional specimen.

NOTE: There are three causes of breaks at or near the jaws, namely:

(i) Randomly distributed weak places;

(ii) concentration of stress in the neighbourhood of the jaws (because the jaws prevent the specimen from contracting in width); and

(iii) damage to the specimen by the jaws.

The problem is how to distinguish between damage by jaws and the other two causes of jaw breaks. In practice this is rarely possible, so the most that can be done is to reject low values. There are statistical criteria for picking out abnormal values, but in the routine testing of geotextiles they are hardly applicable. When the use to which a geotextile is put makes low values of special interest, the purchasing specification for the geotextile should state precisely what is to be done with the result when a specimen breaks at or near the jaws.

3.3.4 Calculation

(a) Breaking strength. Calculate the arithmetic mean of the loads at break obtained from the autographic recorder for each direction of testing. Convert this load into a load per metre width by multiplying by

(i) 12,5 in the case of a woven geotextile; and

(ii) 4 in the case of a non-woven geotextile.

Round off the result correct to 1 N/m.

(b) Breaking elongation. Calculate the arithmetic mean of the elongations at break obtained from the autographic recorder for each direction of testing. Express this elongation as a percentage of the initial gauge length of the specimen.

Round off the result correct to 1 %.

3.4 **Normal permeability**

3.4.1 **Apparatus**

(a) Water supply. A supply of water, through a suitable pressure reducing valve or from an overhead tank, at approximately constant pressure.

(b) Permeameter (see Figure 6). A suitable flow cylinder, mounted on a horizontal base, and which includes

(i) control valves at each end of the cylinder;
(ii) a means of so securing the specimen that no leakage occurs between the specimen and the faces of the cylinder;
(iii) an inlet manometer and an outlet manometer fixed to the cylinder (one at each side of the specimen), each of length at least 300 mm and diameter at least 6 mm, the inlet manometer having markings at 197 mm and 203 mm and the outlet manometer having markings at 97 mm and 103 mm from the centre-line of the specimen;
(iv) a system capable of applying a hydraulic head across the specimen and maintaining the head at a constant value for the duration of one test; and
(v) a measuring device to measure the volume of water discharged.

(c) Cutting device. Means for cutting out a circular specimen of diameter at least 100 mm.

(d) Stop-watch.

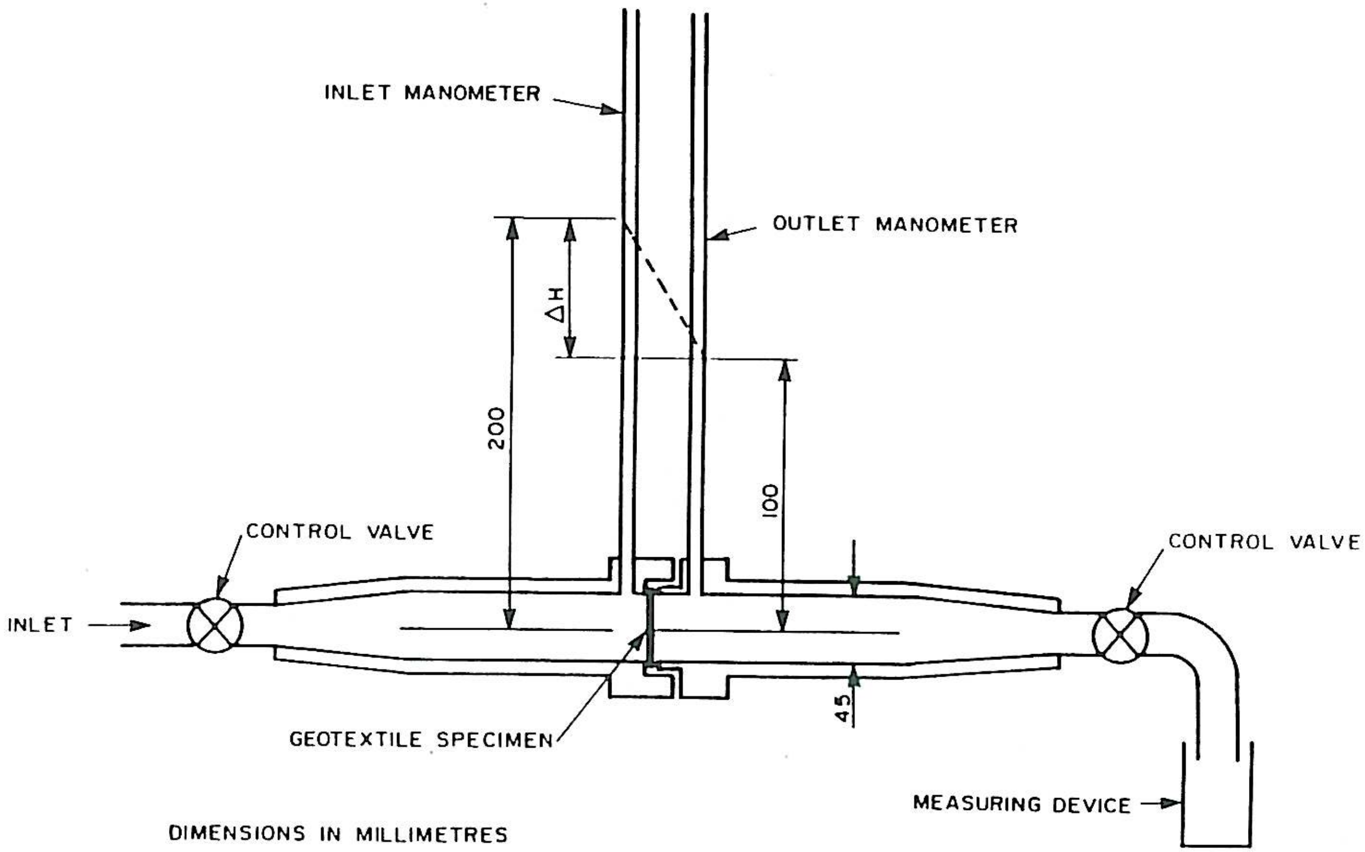


FIGURE 6

PERMEAMETER FOR NORMAL PERMEABILITY

3.4.2 Specimens

(a) Number of specimens. Conduct the test on at least 10 specimens in the standard atmosphere for testing.

(b) Preparation of specimens. Mark on the sample, in a row across the width, the positions of the required specimens. Cut out the specimens and maintain them in the standard atmosphere until testing is commenced.

3.4.3 Procedure

(a) Assembly. Carefully position the specimen in the flow cylinder and tighten the securing bolts.

(b) Testing

- (i) Conduct the test in the standard atmosphere for testing.
- (ii) With the control valves open, fill the apparatus with water, ensuring that all trapped air is eliminated. Allow the water to run for a minimum of 30 s.
- (iii) Maintain an inlet head of 200 ± 3 mm and a difference in head between the inlet and outlet manometer markings of 100 mm.
- (iv) Ensure the specified difference in head by so controlling the inlet and outlet valves that the level of the water head at each manometer is within the specified markings, thus giving a constant head across the specimen of 100 mm.
- (v) Record the volume of water discharged into the measuring device and the time taken for that volume to be discharged.

3.4.4 Calculations

(a) Depending on the expected permeability, record for each specimen

- (i) the average volume of water collected in the measuring device during a fixed time interval; or
- (ii) the average time taken to collect a fixed volume of water.

(b) Calculate and record the water percolation rate for each specimen from the following formula:

$$P = \frac{V}{AT}$$

where P = percolation rate of specimen, $l/m^2/s$

V = Volume of water collected, litres

A = surface area of the specimen exposed to water flow, m^2

T = time interval to collect volume V of water, s

(c) Calculate and record the average percolation rate for the sample. Round off the result to the nearest 1 $l/m^2/s$.

4. SOIL/GEOTEXTILE TESTS

4.1 Permeability reduction (Flow test)

4.1.1 Apparatus

(a) Water supply. A supply of water from an overhead tank with a constant head of 1000 ± 25 mm above the geotextile sample.

(b) Permeameter (see Figure 7) with a suitable means of mounting to ensure that the permeameter remains in a vertical position throughout the test. The permeameter consists of

(i) Two perspex cylinders, 90 mm inner diameter. The bottom cylinder has a recess for the support mesh and a breather hole.

(ii) Two perspex end caps with inlet/outlet nozzles. The end caps are machined as shown in Figure 7 to fit over the cylinders and to allow for air bubbles to escape through the inlet nozzle. A bleeding hole is also provided.

(iii) Three brass rods and nuts (wingnuts).

(iv) Brass or stainless steel mesh with 2,67 mm openings (standard soil sieve).

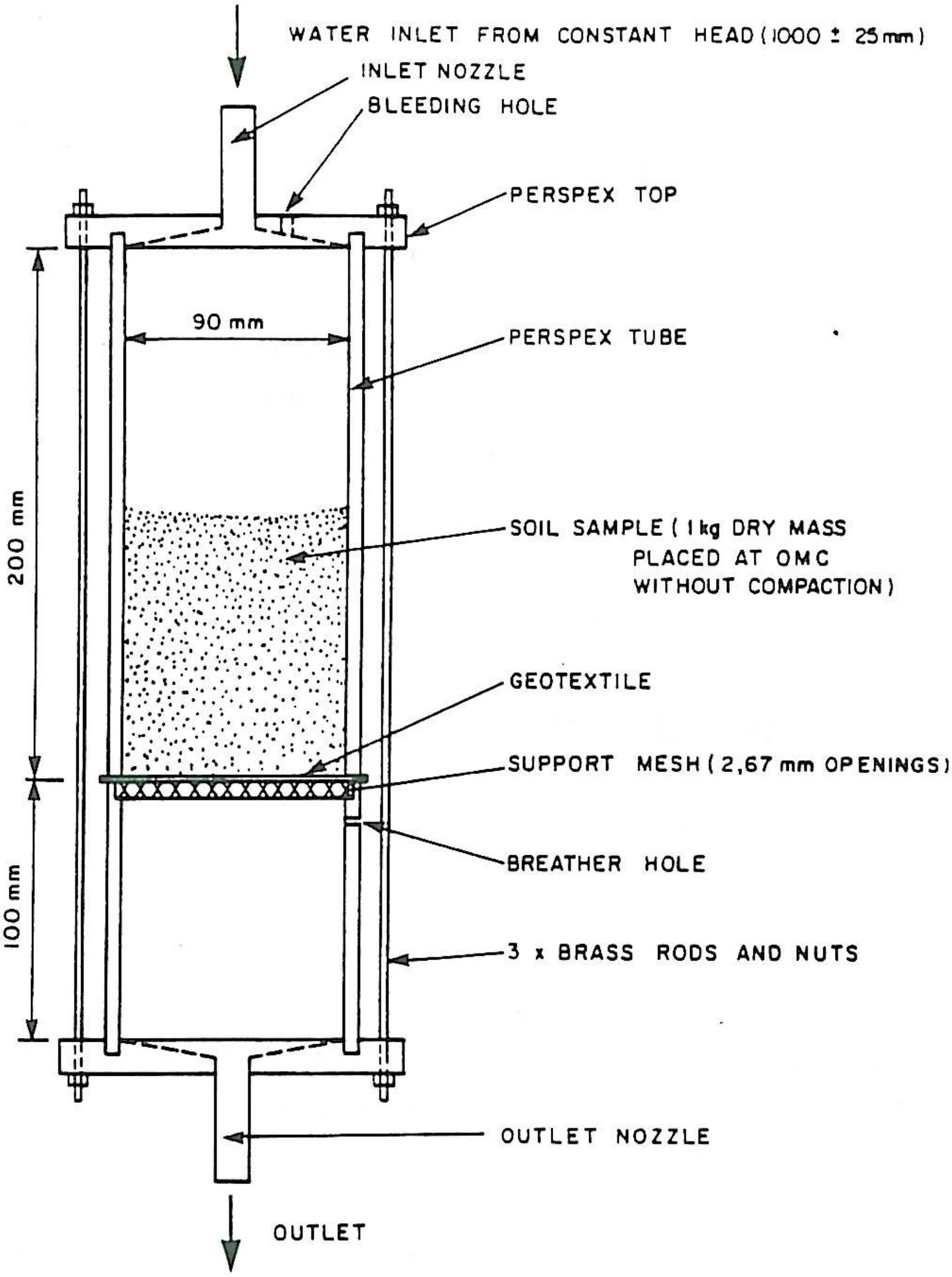


FIGURE 7

PERMEAMETER FOR PERMEABILITY REDUCTION TESTS (FLOW TESTS)

940 - 4 - 5874 BS

(c) Cutting device. Means of cutting out a circular specimen of diameter at least 110 mm.

(d) Containers with capacity of at least 5 litres.

(e) A riffler with 25,0 mm openings.

(f) A soil test sieve with 13,2 mm openings.

(g) A balance to weigh up to 1 kg, accurate to 1 g or better.

(h) Stopwatch, silicone grease, pans.

4.1.2 Specimens

(a) Number of specimens. Conduct one test for each soil/geotextile combination.

(b) Geotextile specimens. Mark and cut the required number of specimens and maintain in the standard atmosphere until tested.

(c) Soil samples. Sieve the oven-dried soil through a 13,2 mm sieve and discard the material larger than 13,2 mm. Riffle the remaining soil to obtain specimens of 1000 ± 5 g. Place in sample bags and label.

4.1.3 Procedure

(a) Place the geotextile specimen between the two cylinders on top of the mesh. Apply silicone grease to ensure that water does not leak out between the two cylinders.

(b) Mix the amount of water required to bring the soil sample to Mod AASHTO optimum moisture content (OMC). With dispersive soils, friable mudstones and heavy clays remoulding often results in an impermeable soil mass. In such cases this step is omitted and the soil sample is placed dry.

(c) Place the soil sample on top of the geotextile. Smooth the surface without compacting the soil.

(d) Place the perspex end caps in position and fasten the rods and nuts. Place the assembled permeameter in its mounting.

(e) Close the outlet and the breather hole in the bottom cylinder and slowly fill the whole cylinder with water from the top. Care must be taken not to disturb the surface of the soil sample. This can be done by using a small diameter pipe with a spray nozzle which is inserted through the top inlet to a height just above the soil surface. Fill the permeameter to the top of the inlet nozzle, remove the small diameter pipe and connect the hose from the constant head tank. Remove any entrapped air bubbles through the inlet nozzle and the bleeding hole.

(f) Open the outlet nozzle and breather hole in the bottom cylinder and record the time as the beginning of the test.

(g) The first outflow measurement should be taken between 1 minute and 5 minutes after the beginning of the test. Outflow is measured using a container and stopwatch. The container is placed under the outflow and the time recorded that it takes to fill the 5 litre container, or the amount of flow that occurs in 30 minutes, whichever occurs first. It is not important that exactly 5 litres or 30 minutes be used, but the time and volume must be recorded accurately. Record the following with each outflow measurement:

- Date and time (hours and minutes)
- Flow volume ($1000\ mm^3 = 1\ ml$) Calculate
- Flow time (minutes and seconds) flow mm^3/s
- Height of the sample (soil + geotextile) in mm
- Height of the water head above bottom of geotextile in mm (ensure this height remains at 1000 ± 25 mm)
- Whether the water at the outlet is discoloured.

(h) Outflow measurements should be taken at the beginning of the test (1 to 5 minutes after opening the outlet) and thereafter at least once a day. The test should be continued for at least 400 hours (17 days).

4.1.4 Calculations

(a) Permeability coefficient, k Calculate permeability coefficient for each outflow measurement as follows:

$$k = \frac{Q}{iA} \quad \text{(mm/sec)}$$

where Q = outflow (mm^3/s)

i = hydraulic gradient

$= \frac{\text{water head above geotextile (mm)}}{\text{sample height (mm)}}$

A = cross section area (mm^2)

= 6362 mm^2 for 90 mm diameter

(b) Permeability reduction factor, K400. K400 is determined at the end of the test, as follows:

$$K400 = \frac{\text{K at 400 hours}}{\text{K at beginning of test}} \times 100$$

If an outflow measurement was not taken at exactly 400 hours, K at 400 hours may be determined by linear interpolation.

5. TENTATIVE CRITERIA

The criteria given below are based on results from laboratory tests, limited field results and discussions with some users and suppliers of geotextiles. It is expected that, as the tests described in this document are applied and related to practical experience, these criteria will be adjusted. They should therefore not be treated as standard specifications on the basis of which geotextiles are approved or rejected, but merely as tentative guidelines. Furthermore, the values shown relate to road subsurface drainage applications (typically side drains) and not to river protection or reinforcement where higher strengths will most probably be required.

5.1 In-Isolation tests

Test	Parameter	Limits
Penetration load	Penetration load at rupture	Not less than 1200 N
	Elongation at rupture	Not more than 80 %
Tensile strength	Breaking strength	Not less than 9000 N/m
	Breaking elongation	Not more than 140 %
Normal permeability	Normal permeability	Not less than 10 $l/m^2/s$
	Normal permeability where soil to be drained is a clean sand or gravel with permeability coefficient, k, of 0,7 mm/s or more	Not less than 25 $/m^2/s$

5.2 Permeability reduction

The normal filter criteria namely piping and permeability as well as permeability reduction can be controlled with the permeability reduction test.

(a) Piping. Discolouration of the water should not be visually discernable for longer than 8 hours after the start of the test.

(b) Initial permeability, K. The permeability of the geotextile should be equal to or more than that of the soil. This means that the initial permeability obtained with the flow test should not be less than the permeability of the soil obtained with a porous disk or geotextile of which the permeability is known to be high.

(c) Permeability reduction, K400. The geotextile resulting in the highest value of K400 for a particular soil should be selected. Figure 8 gives ranges of K400 values for different soils (based on percentage passing 0,075 mm sieve) and can be used as a guide. Geotextiles selected should yield results in the high or medium ranges.

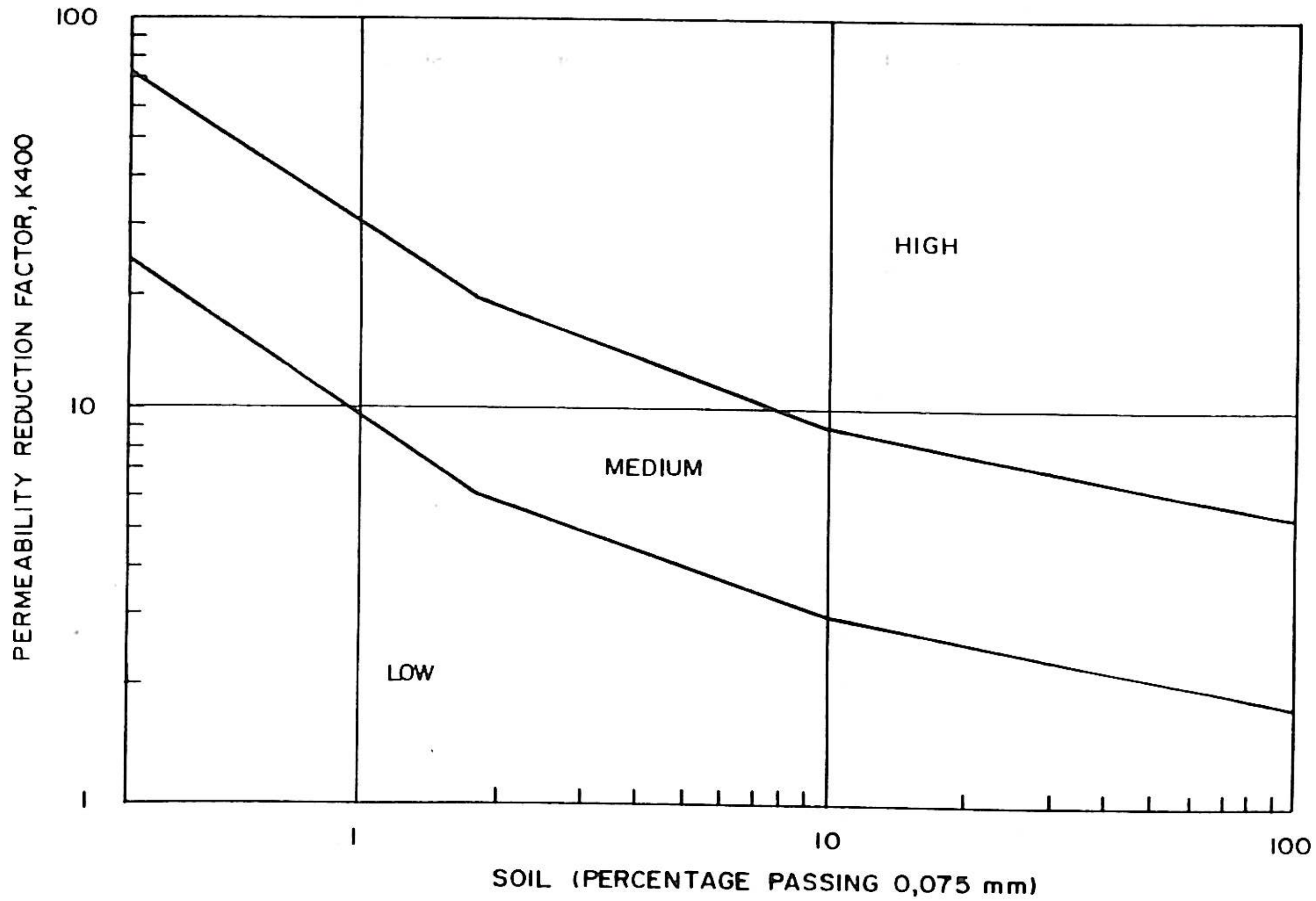


FIGURE 8

PERMEABILITY REDUCTION CRITERIA

6. REFERENCES

1. VAN DER MERWE, CJ and HORAK, E. Evaluation of soil/geotextile compatibility. Proc 12th Int Conf SMFE, Rio de Janeiro, August 1989.

2. VAN DER MERWE, CJ and JORDAAN, GJ. Road building materials: the geosynthetic alternative. Proceedings of the 1989 Annual Transportation Convention (ATC), Volume 5A, Pretoria, 1989.

3. VAN DER MERWE, CJ. A laboratory evaluation of soil/geotextile compatibility for road subsurface drainage systems. Roads and Transport Technology Research Report DPVT 93, CSIR, Pretoria, November 1989.

4. VAN DER MERWE, CJ. Geotextile evaluation mechanical tests. Draft research report 1/FP/1/88 (unpublished), Transportek, CSIR, March 1988.

5. SOUTH AFRICAN BUREAU OF STANDARDS (SABS), Code of Practice for the testing of geotextiles. SABS 0221-1988, Pretoria, 1988.

APPENDIX C

FIN DRAINS: A STATE OF THE ART REVIEW

TECHNICAL NOTE I/FP/29/89

FIN DRAINS: A STATE OF THE ART REVIEW

C J van der Merwe

Programme: Pavement Engineering Technology
Programme Manager: Dr E. Horak

November 1989 I/FP/29/89

DIVISION OF ROADS AND TRANSPORT TECHNOLOGY

DIVISION OF ROADS AND TRANSPORT TECHNOLOGY

CSIR

PRETORIA

Fin drains: A state of the art review

CONTENTS

1 Introduction

Before geotextiles were available, side drains typically consisted of granular filters with a porous pipe or a perforated pipe as illustrated in Figure 1(a). The granular filters were designed according to the Terzaghi criteria. Depending on the grading of the base soil and the granular materials available, it was often necessary to use two or more filters in stages. Geotextiles eliminated this problem since these materials could be used as filters with the granular material acting only as a water carrier, as shown on Figure 1(b). Side drains constructed in this way are the most widely used in South Africa at present and are specified by most road authorities. The granular material is usually single sized (typically 19mm) and is cheaper than a graded granular filter. The major shortcoming of these side drains has been the design of the geotextile filter, which has been dealt with in previous research (Van der Merwe, 1989). Where the side drains transport water over dry areas, the geotextile is replaced with an impermeable membrane (Figure 1(c)).

In the geosynthetic industry, a variety of synthetic water carriers have been developed over the past ten years. These materials, termed geonets; geospacers; flownets; waffles etc, are aimed at replacing the granular water carriers in the conventional side drains. The geonet (with thickness varying from 5 to 100mm) is wrapped in a geotextile filter to form what

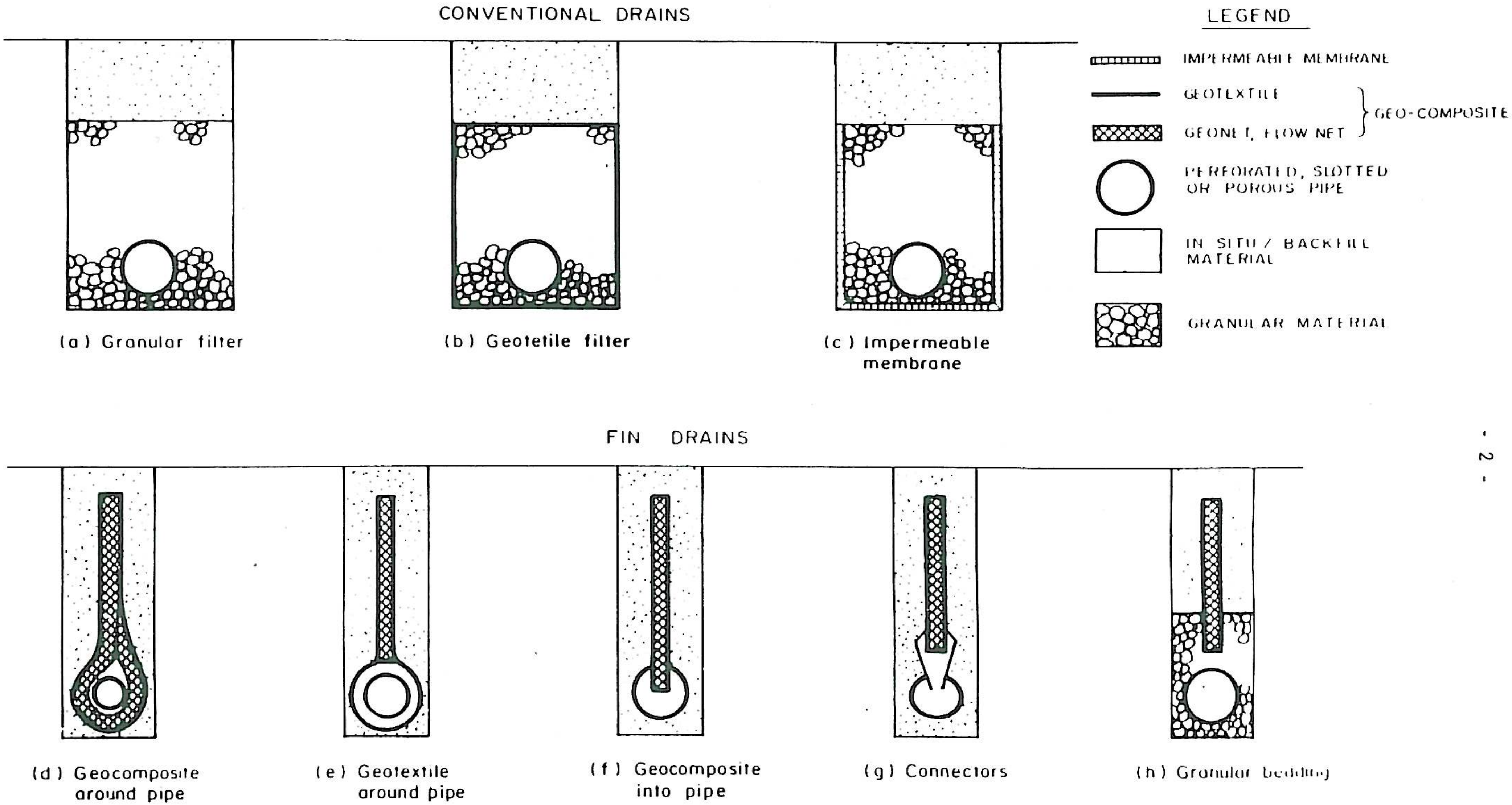


FIGURE I *TYPICAL SIDE DRAINS*

is often referred to as a geocomposite. The geocomposite in combination with a drainage pipe forms what is generally known as a fin drain, examples of which are shown in Figure 1(d) to (h).

Various flownets, geotextiles, prefabricated geocomposites, synthetic pipes and connecting systems between the geocomposite and the pipe exist, resulting in a large variety of available fin drains. The fin drains are usually pre-assembled, either in the factory or at the construction site, before being installed in a trench. The dimensions of these drains make it possible to use considerably narrower trenches than with conventional side drains (usually 200mm as opposed to 600mm).

Although road authorities and designers are hesitant to use fin drains on a large scale, they have been installed in limited quantities and certain advantages as well as disadvantages have been identified. Advantages include lower cost, ease of construction and versatility. The main disadvantage is the lack of design methods and selection criteria.

2 Cost

The cost benefit of fin drains are derived from the narrower trench and lower material cost. A comparison is given below in Table 1 between the cost of a conventional side drain (600mm wide, 2m deep with 1,1m actual drain and 900mm backfill) and a fin drain with similar dimensions (1,1m actual drain) as well as the case where the fin drain is placed for the full 2m depth.

Table 1 Cost comparison between conventional drain and fin drains. (R/m)

	Conventional	Fin Drain	Fin Drain
Trench dimensions	0,6 x 2,0m	0,2 x 2,0m	0,2 x 2,0m
Drain depth	1,1m	1,1m	2,0m
Excavation	7,37	2,46	2,46
Backfill	4,31	3,19	3,19
Aggregate	25,66	0,00	0,00
Pipe	9,35	9,35	9,35
Geotextile	6,38	5,02	9,12
Flownet	0,00	8,25	15,00
Total Cost: R/m	53,07	28,27	39,12

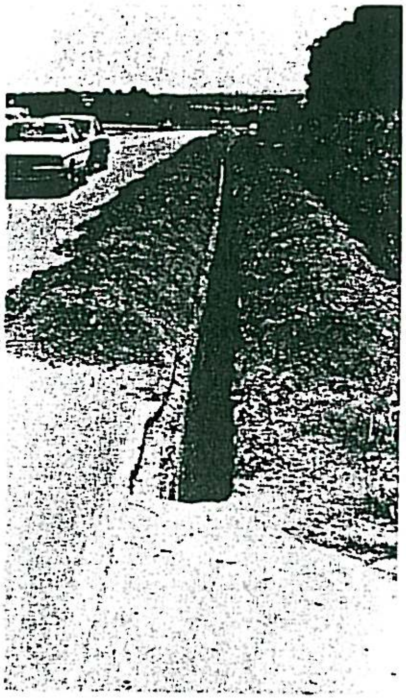
It is evident from the comparison in Table 1 that, even when the fin drain is placed for the full 2m depth of the trench, the cost is considerably lower than that of a conventional side drain. The biggest cost saving is obtained by eliminating the aggregate, which is the most expensive item in the conventional drain. The flownet of which the cost is shown in Table 1 (7,50/mi) is a flownet that has been used to date, also in the examples described in Section 3 below.

3 Construction

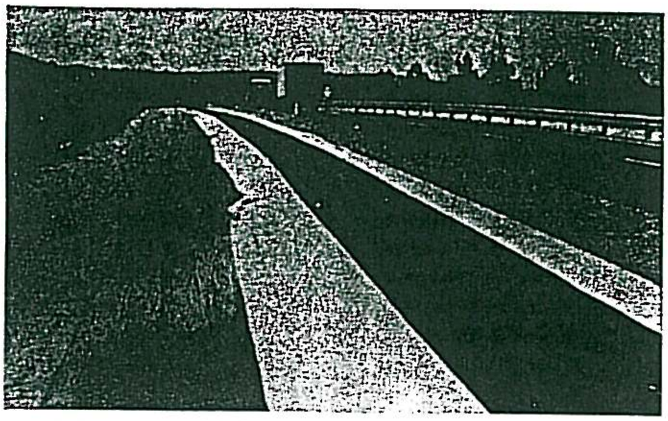
The relative ease of construction of fin drains is as a result of the narrow trenches and the fact that all the components can be assembled prior to placing the completed product in the trench, as opposed to the individual installation of each item in the case of conventional drains.

Photograph 1 shows a series of photographs taken during the installation of a fin drain. The trench, being only 200mm wide, was excavated mechanically (Photograph 1(a)). Photographs 1(b) to (d) show the different components, assembling process and the end product respectively. This particular fin drain was similar to that shown in Figure 1(d) with the geocomposite wrapped around a perforated pitch fibre pipe. The geotextile and flownet were merely stapled together, since the highest stresses that are exerted on the fin drain are during installation. Once the end product has been installed, the various components are held in place by the backfill material. Photograph 1(e) shows the installation procedure, whereby the completed product was merely lowered into the trench by hand. The geocomposite was then stapled to the side of the trench and the trench was backfilled with the original in-situ material. Certain geocomposites, that are manufactured as a unit, are stiff and need not be fastened to the trench sides. The literature supplied by the various manufacturers of fin drain materials indicate that the geocomposite is placed in the centre of the excavated trench, as shown on Figure 1(d) to (h). It is more practical, however, to place the geocomposite on the side of the trench, thus allowing more space for backfilling and compaction of the in-situ material.

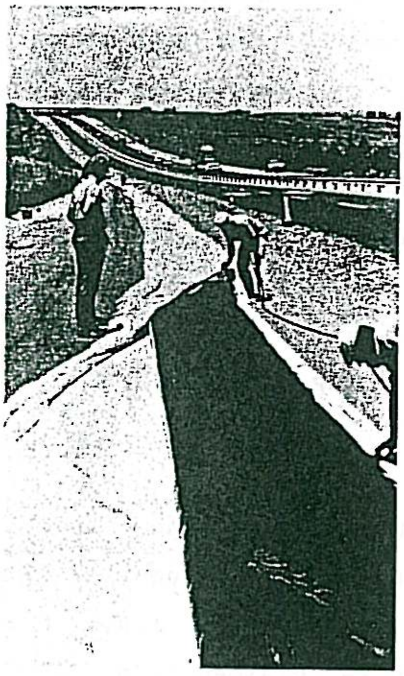
The compaction of the backfill material in the narrow trench proved to be a major problem. Compaction is required to ensure stability of the adjoining material which forms the road shoulder. Equipment exists which is claimed to be capable of compacting to a high density in a narrow trench, but the effectiveness of such equipment was not established. Research work is needed in this area to determine what density is in fact required and whether that density can be obtained in a narrow trench.



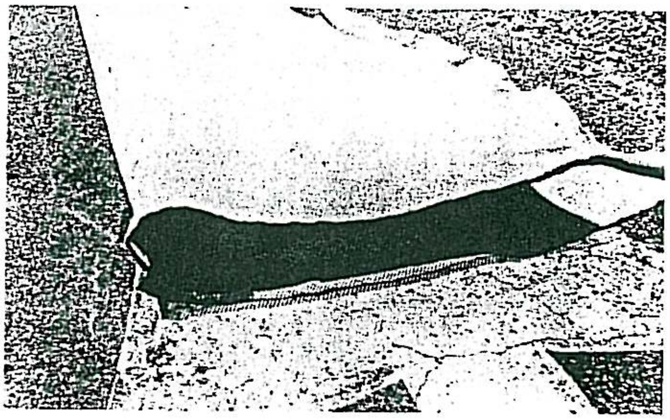
(a) Trench



(b) Components

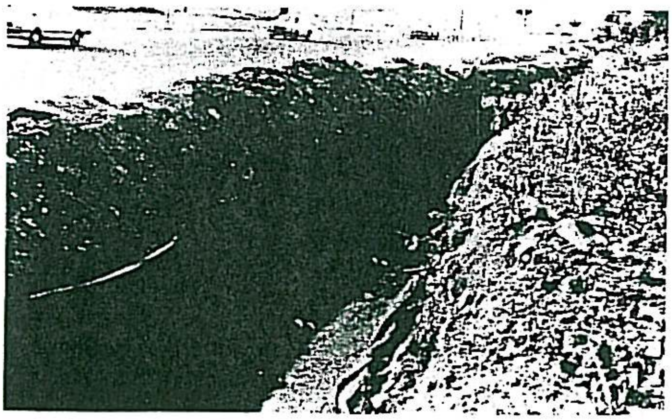


(c) Assembly



(d) End product

(e) Installation



(f) Impermeable side

PHOTOGRAPH 1

Fin drains

The fact that fin drains can be assembled prior to installation and the availability of a large variety of materials, make fin drains versatile products that can be modified with relative ease for specific problems, site conditions and special applications. An example of a modified fin drain is shown in Photograph 1(f), where the geotextile on the road side of the trench was sprayed with a bituminous binder to form an impermeable membrane, thus eliminating the movement of moisture between the subgrade and the verge. The road side geotextile could also have been replaced with an impermeable geomembrane (polyethylene sheet etc).

4 Tests and design parameters

The main disadvantage of fin drains is that no standardized test methods or design parameters exist at present which can be used to select the most suitable product for each particular application. Hunt (1982) identified three tests that need to be performed to evaluate the effectiveness of fin drains namely: the flow capacity of the core material (flow net), the filtration characteristics of the filter (geotextile) and the compression resistance of the flow net.

The characteristics of the geotextile filter have been dealt with in detail in earlier research and apply equally to fin drains and conventional side drains.

Dempsey (1988) performed "core flow capacity" tests on six fin drain materials, using a 8m long laboratory channel and concluded that the flow capacity of these materials were comparable or better than conventional materials. It is recommended that similar apparatus be manufactured, test methods standardized and design criteria established for use in South Africa.

The compression resistance of the core material must be adequate to ensure that the flow capacity is retained under stresses imposed on the installed fin drain. Standardized test methods and design criteria are also required for this aspect and should be incorporated in the core flow capacity test.

The layout of a core flow capacity test is shown on Figure 2 (a), (Murray, 1984). The method by which the pressures are applied are of great importance. If a rigid plate is used, as indicated on Figure 2(a), a bridging effect occurs which allows the central core to remain open without the geotextile affecting the flow capacity. This is not realistic, however, since the pressures in an installed drain are transferred by the soil which causes the geotextile to be pushed into the core thus reducing the flow capacity. The extent to which this occurs depends on the type of core as well as the type of soil. This should be simulated in the laboratory by using soil or a soft rubber membrane or bag, as illustrated in Figure 2(b).

The flow in a fin drain is neither saturated nor laminar and Darcy's formula can therefore not be used in a core flow capacity test. Flow rates must therefore be compared directly (Koerner et al, 1985).

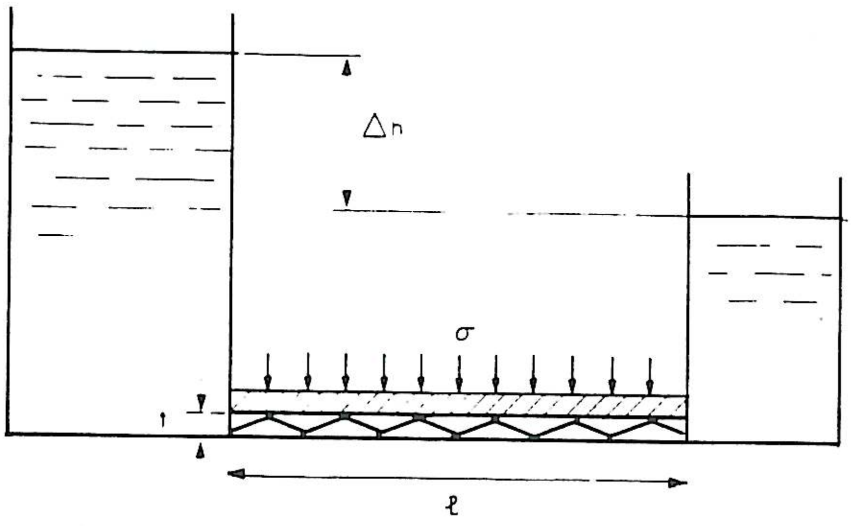
Rankilor(1985) recommends an hydraulic gradient of 1.0 and a maximum pressure of 100 kN/mí for core flow capacity test.

5 References.

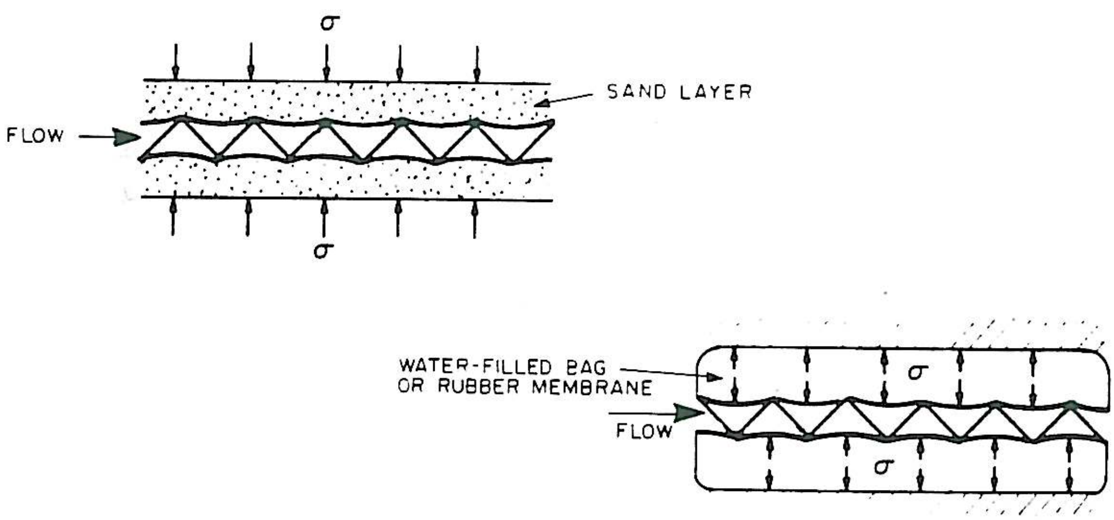
DEMPSEY, B J. 1988. Core flow capacity requirements of geocomposite fin drain materials utilized in pavement drainage. University of Illinois, Paper presented for 67th TRB Meeting, Washington, DC.

HUNT, J A. 1982. The development of fin drains for structure drainage. Proceedings of the Second International Conference on Geotextiles, Vol I, Las Vegas.

KOERNER, R M and BOVE, J A. 1985. Lateral drainage desings using geotextiles and geocomposites. Geotextile Testing and the



(a) LAYOUT OF TEST



(b) POSSIBLE LOADING MEMBRANES

FIGURE 2

FIN DRAIN CORE FLOW CAPACITY TESTS

Design Engineer. ASTM Special Technical Publication STP 952.

MURRAY, H R. 1984. Composite geotextiles for soil and structure drainage. Proceedings of the International Man-Made Fibres Congress, Dornbirn, Austria.

RANKILOR, P R. 1985. The specification and use of geotextile fin drains. Geotextiles and Geomembranes Vol 2, Elsevier Publications.

VAN DER MERWE, C J. 1989. Recommended tests and tentative criteria for geotextiles in road subsurface drainage applications. DRTT research report DPVT 92, CSIR, Pretoria.

APPENDIX D

DEVELOPMENT OF TRANSMISSIVITY TESTS TO DETERMINE FLOW RATES THROUGH FIN DRAINS UNDER LATERAL COMPRESSION.

TECHNICAL NOTE I/FP/13/90

DEVELOPMENT OF TRANSMISSIVITY TESTER TO DETERMINE FLOW RATES THROUGH FIN DRAINS UNDER LATERAL COMPRESSION

H L Theyse
C J v d Merwe

Program: Pavement Engineering Technology (Dr E Horak)

March 1990 **I/FP/13/90**

CONTENTS PAGE

1. INTRODUCTION

Current research in the subsurface drainage project is aimed at determining the flow capacity of fin drains under lateral earth pressure. The research is seen as part of a process that will lead to a better understanding of drainage systems utilizing geotextiles and geocomposites as filters and drains.

Previous research (Van Der Merwe, 1988) dealt with design criteria and standardized test methods for geotextile filters. This report briefly describes the development of an apparatus capable of testing the flow capacity of fin drain specimens over ranges of different lateral pressure, inlet pressure head and outlet pressure head.

The apparatus consists of two basic units, the transmissivity tester itself and a stilling well with a 10° V-notch weir for determining flow rates. Both units were manufactured from Perspex. Lateral pressure is applied to the specimen in the transmissivity tester by means of a confined pneumatic bag.

2. SIMULATION OF FIELD CONDITIONS.

Figure 1 depicts the field conditions that are simulated by the transmissivity tester. The basic diagram of the tester indicates how this is accomplished. The two basic field conditions taken into consideration are:

a. Varying inlet and outlet pressure

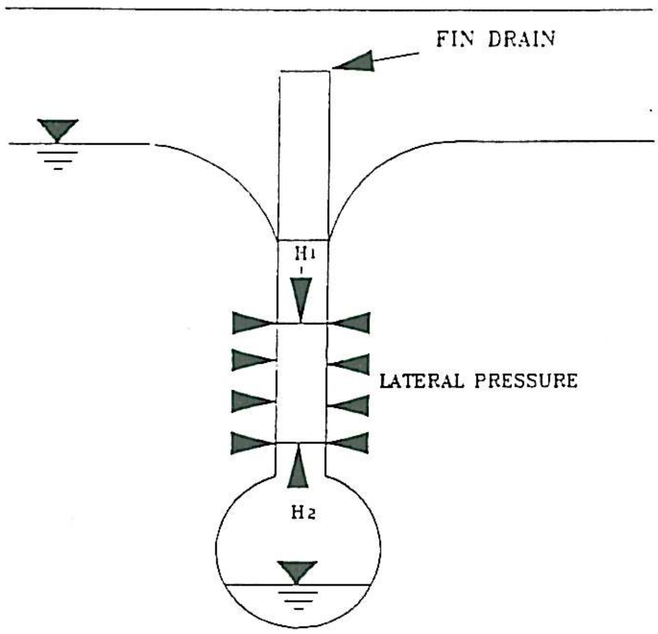
Depending on the position within the fin drain at which the simulation is done, the inlet and outlet pressure for the specimen will differ. At the bottom of the fin drain where the geocomposite enters into the drainpipe, the backpressure should be zero for well designed drainage systems.

In the transmissivity tester the above situation is accomplished by mounting spillways of different height on the two ends of the apparatus.

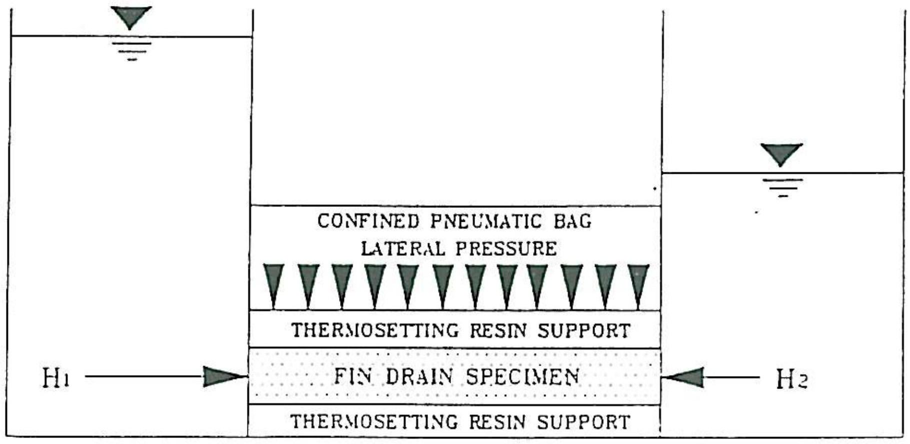
b. Reduction in flow capacity by lateral pressure.

The flow capacity of the geocomposite is expected to be reduced in two ways by lateral earth pressure:

(i) Firstly, the thickness of the geocomposite is reduced through the compression of the core material, especially for thicker drains. Research



FIELD CONDITIONS



BASIC DIAGRAM OF APPARATUS

FIGURE 1: SIMULATION OF FIELD CONDITIONS

done by Murray indicated a reduction in the region of 60% for lateral pressure of 20 kPa and a reduction in the region of 75% at lateral pressure of 50 kPa for three geocomposite cores thicker than 10 mm (Murray, 1984). He also found that the geocomposite showed a further decrease in thickness due to creep for extended periods of compression.

(ii) Secondly, the soil on the sides of the drain and the geofabric may be pushed into the voids of the core material, thereby further reducing the flow capacity.

In the transmissivity tester the specimen is confined between two supports. Pressure is applied to the top support by means of a confined, inflated pneumatic bag as indicated in Figure 1. The supports may be two rigid plates, soft thermosetting resin pads or geotextile bags containing soil.

3. DESIGN OF APPARATUS.

The upper limit of lateral stress has been set as follows:

Horizontal geostatic stress=(K)z(τ)

where K=Coefficient of lateral stress at rest.
z=Depth below ground level.
τ=Unit weight of soil.

The value of K varies as follows:
(Gravel) 0,3<K<0,7 (Clay) (Spangler and Handy, 1982)

The unit weight has been assumed to be 25 kN/m which is the maximum value expected for soil. Thus the maximum horizontal stress expected was taken at a depth of 5m and is 87,5 kPa. Therefore, the apparatus was designed to test at lateral pressures up to 100 kPa.

Two possible means of applying the lateral pressure has been investigated. It could have been accomplished by either stacking weights on to a loading plate or by inflating a confined pneumatic bag directly above the top support.

After investigation it has been decided to opt for the pneumatic bag, as it:

(i) Reduces the weight of the apparatus
(ii) applies a much more uniform load and
(iii) it is easy to reach pressures of up to 100 kPa with air pressure in a laboratory.

Figure 2 contains a vertical section through the apparatus. The specimen together with the two supports are inserted from the top of the apparatus. Two spacers are placed at the sides of the top support to allow for different thicknesses of specimen to be tested. The pneumatic bag is placed on the upper support and slightly inflated. The top plate is then lowered on to the pneumatic bag and the dowels are inserted above the top plate. The pneumatic bag is then inflated to the desired pressure and the test can commence. The specimen is removed by deflating the pneumatic bag and removing the dowels. The specimen, upper support, pneumatic bag and top plate can now be lifted from the apparatus.

A stilling well with a 10° V-notch weir has been designed to measure the flow through the drain specimen. Expected flow rates were taken from results obtained by Dempsey (Dempsey, 1988) and varied from 9 l/min to 126 l/min for different fin drain thicknesses at a channel slope of 0% and an entrance head of 460 mm. The water supply in the laboratory was found to be sufficient for the indicated flow rates.

Both the transmissivity tester and stilling well have been manufactured from Polymethyl Methacrylate (Perspex).

4. PROGRESS.

The main body of the test apparatus has been completed. The resin for the specimen supports has been ordered from the USA and the pneumatic bag is being manufactured.
The first phase of testing has not yet commenced but will evaluate the influence of different supports, ie soil, rigid plates and soft resin pads.

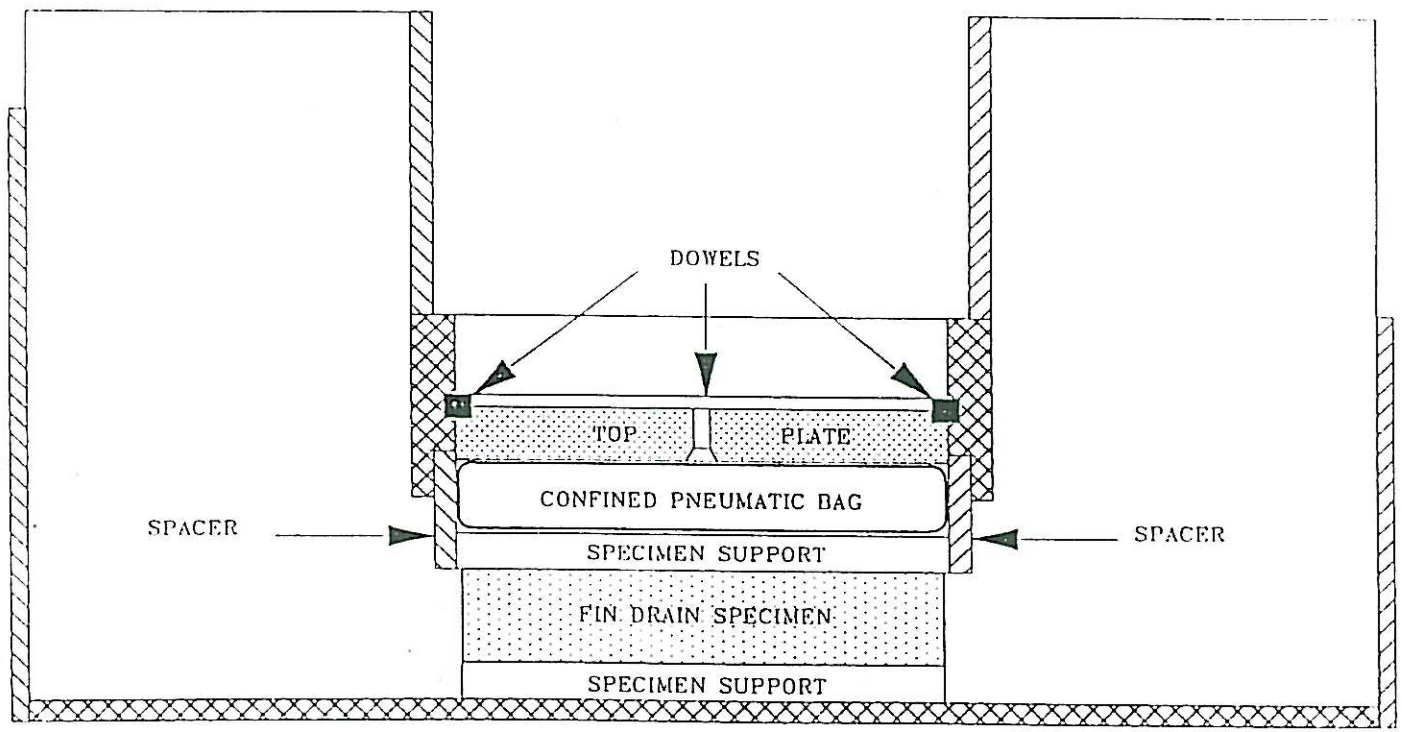


FIGURE 2: SECTION THROUGH TRANSMISSIVITY TESTER.

REFERENCES.

Van der Merwe C J (1988). Some Aspects of Road Subsurface Drainage in South Africa. Ph.D Dissertation, Department of Civil Engineering, Faculty of Engineering, University of Pretoria, 1988.

Murray H R (1984). Composite Geotextiles for Soil and Structure Drainage. International Man-made Fibres Congress, Dornbirn Austria, September 1984.

Spangler M G, Handy R L (1982). Soil Engineering. Harper and Row Publishers, New York, 1982, p 413.

Dempsey B J (1988). Core Flow Capacity Requirements of Geocomposite Fin Drain Materials utilized in Pavement Subdrainage. Paper for publication at 67th annual Transportation Research Board meeting, Washington D.C, March 1988.

APPENDIX E

FINDREIN VLOEIKAPASITEIT AS FUNKSIE VAN LATERALE DRUK EN DRUKGRADIËNT

TUSSENTYDSE VERSLAG IR 88/29/1

TUSSENTYDSE VERSLAG IR 88/29/1

SUID-AFRIKAANSE PADRAAD
NAVORSINGS- EN ONTWIKKELINGSADVIESKOMITEE

Findrein vloeikapasiteit as funksie van laterale druk en drukgradiënt

NAVORSING UITGEVOER
VIR EN NAMENS DIE
SUID-AFRIKAANSE PADRAAD DEUR
DIVISIE VIR PAD- EN VERVOERTEGNOLOGIE
WNNR
POSBUS 395
PRETORIA
Maart 1991

TITEL/TITLE FINDREIN VLOEIKAPASITEIT AS FUNKSIE VAN LATERALE DRUK EN DRUKGRADIENT	

VERSLAG NR: REPORT NO:	**ISBN:**	**DATUM: DATE:**	**VERSLAGSTATUS: REPORT STATUS:**
IR 88/29/1		Maart 1991	Interim Report

NOAK NR/RDAC NO: IR 88/29/1

GEDOEN DEUR:
CARRIED OUT BY:
Division of Roads and Transport
Technology, CSIR
P O Box 395
Pretoria
0001

OPDRAGGEWER:
COMMISSIONED BY:
Chief Director : National Roads
P O Box 415
PRETORIA
0001

OUTEUR(S):
AUTHOR(S):
H L THEYSE

NAVRAE:
ENQUIRIES:
Navplan
Private Bag X5
ALKANTRANT
0005

SINOPSIS:

Die verslag raporteer oor die ontwikkelling van apparaat om findrein vloeikapasiteit te bepaal. Die vloeikapasiteit kan teen 'n reeks van laterale druk en waterdrukgradiënte bepaal word.

Drie findrein tipes wat huidiglik in Suid-Afrika beskikbaar is, is getoets. Regressie modelle vir findreinkapasiteit is vir twee van die findreine opgestel met beide rubber en sandsakondersteunings. Rubber kussings sal in die toekoms as ondersteuning in die apparaat gebruik word.

Die laterale druk waarteen die verskillende findreine toetdruk, is ook bepaal.

Voorstelle aangaande die toekomstige ontwikkeling van'n teoretiese model vir die ontleding van vloei-en dreineringstelsels is gemaak. 'n Numeriese oplossing vir die model word aanbeveel, aangesien onversagtigde, ongestadigde vloei hanteer moet word.

SYNOPSIS:

This report describes the development of a fin drain transmissivity tester. It is possible to determapacity of a fin drain as a function of lateral pressure and pressure gradient with the apparatus.

Three fin drain types currently available in South Africa have been tested. Regression models for fin drain capacity have been derived for two fin drain types with both rubber and sandbag supports. It is recommended that the rubber supports be used in future.

The lateral pressure at which the different fin drains collapse has been determined.

Recommendations concerning the development of a theoretical model for the analyses of low- and drainage systems have been made. A numerical solution is suggested for this model as it have to solve unsteady, unsaturated flow conditions.

TREFWOORDE:
KEYWORDS: Findrein, geonet findrein, Eierkasfindrein, laterale gronddruk, drukgradiënt, findrein vloeikapasiteit, dreineringstelsels, vloeistelsels.

KOPIEREG COPYRIGHT	**VERSLAGKOSTE REPORT COST**

INHOUDSOPGAWE

1. INLEIDING

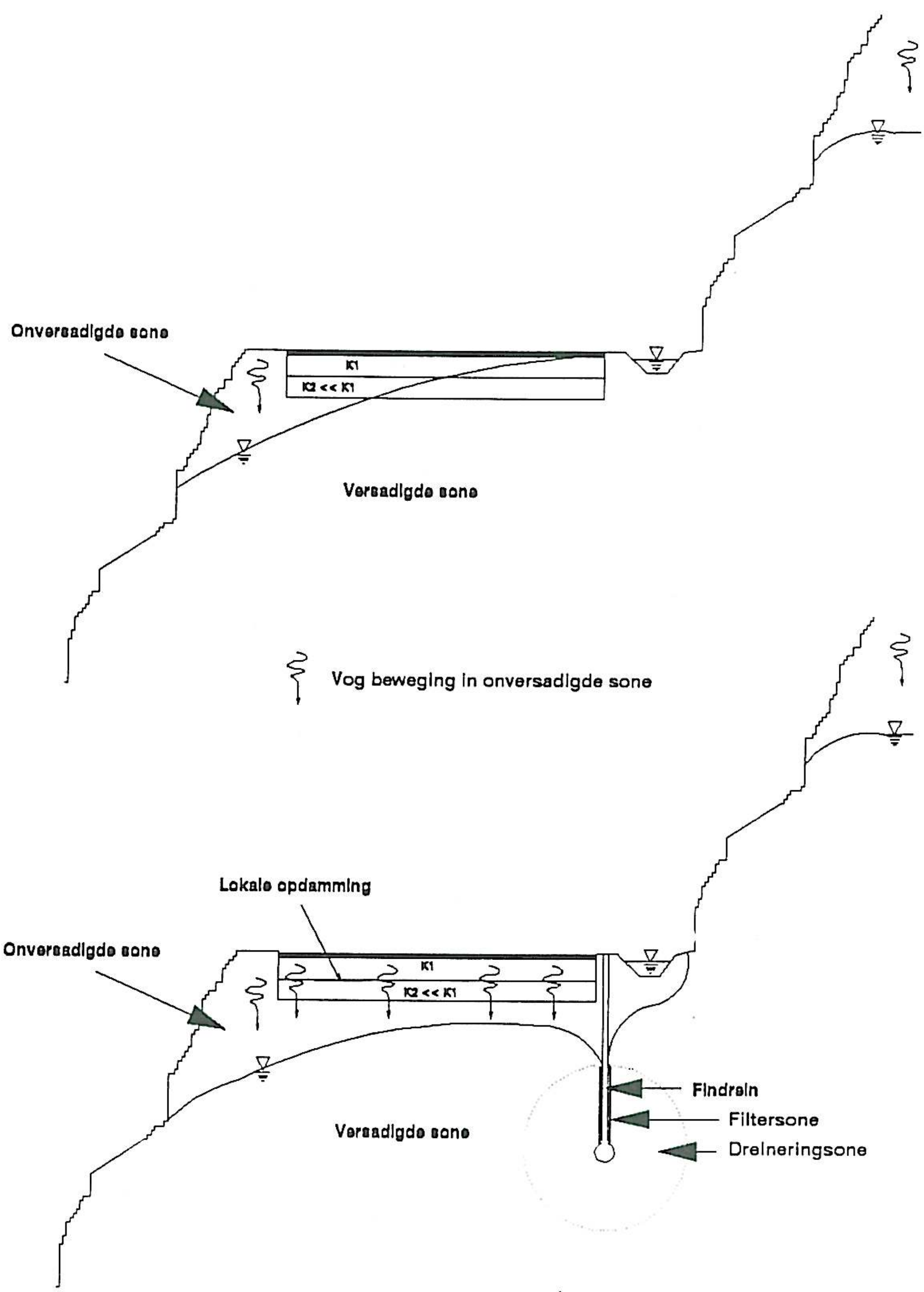
Ondergrondse water dra by tot die ontydige vermoeïng van paaie deur twee meganismes. Eerstens dra die aanwesigheid van grondwater in onversadigde padlae by tot 'n verlaging van materiaalsterktes. In die algemeen word laer Elastiese modulus waardes, kohesie konstantes en interne wrywingshoeke vir nat granulêre of ekwivalente granulêre plaveiselmateriale voorgeskryf in die Meganistiese Evaluasie Metode (Freeme, 1983). Van der Merwe ,(1988) het die gedrag van verskillende plaveiseltipes onder die invloed van oormatige vog in diepte bespreek.

Tweedens kan die pad beskadig word deur die meganiese werking van water in die versadigde sone, erosie van materiaal vind plaas en ruimtedruk wat onder belasting opbou, beïnvloed skuifsterkte deurdat die intergranulêre wrywing verlore gaan weens 'n verlies aan effektiewe spanning.

Vogtoestande is dus van belang in plaveiselontwerp en evaluering. Die plaveisel Ingenieur moet alle gebeurtenisse in die vloeiregime kan voorspel om sodoende daarvoor voorsiening te maak. Figuur 1 toon 'n paar komplekse vloeisituasies aan wat in 'n plaveisel opstelling verwag kan word. Die moontlike effek van 'n findrein wat in die vloeistelsel geïnkorporeer word, is ook getoon.

Die vloeiregime is saamgestel uit 'n klomp onderafdelings elk met sy eie unieke eienskappe. In die versadigde sone sal vloei beheer word deur die drukgradiënt en kan wiskundig gemodeleer word met die Laplace vergelyking. In die onversadigde sone word die vloei aangedryf deur gravitasie en kapilêre kragte. Lae in die plaveisel met 'n groot verskil in deurlaatbaarheid kan 'n lokale opbou van water tot gevolg hê en sodoende erosie in die plaveisellae veroorsaak.

e



Figuur 1: Komplekse vloeistelsels met versadigde en onversadigde sones.

Die vloei in die omgewing van die filter word weer beïnvloed deur die vorming van 'n filtersone met 'n lae relatiewe deurlaatbaarheid in vergelyking met die materiaal in die naasliggende omgewing. (Van der Merwe, 1988)

'n Verskeidenheid van toestande kan dus heers en dit is noodsaaklik om al hierdie toestande te verstaan om die korrekte geheelbeeld te vorm. Huidige navorsing in hierdie projek is beperk tot die dreineringsone om die filter en in hierdie verslag word die effektiwiteit, duursaamheid en kapasiteit van sintetiese findreine bespreek. Vrae wat be-antwoord moet word sluit ondermeer die volgende in. Wat is die kapasiteit van die findrein self onder verskillende laterale druk en waterdrukgradiënte? Gaan die kapasiteit van die stesel deur die findrein self, die naasliggende dreineringsone of miskien die deurlaatbaarheid van die grond en plaveisellae beïnvloed word?

Van der Merwe, (1988) het reeds ontwerpkriteria opgestel vir filters wat van geotekstiele gebruik maak en in detail ondersoek ingestel na die vorming van 'n filtersone wat langs die geotekstiel wand van die findrein ontstaan.

Die doel van hierdie verslag is om te rapporteer oor die onwikkelling van 'n aparaat om die kapasiteit van 'n findrein self te bepaal onder variërende laterale en waterdruk.

Die aparaat is gebruik om die kapasiteit van die findrein tipes wat op die oomblik algemeen in Suid-Afrika gebruik word, te bepaal en sluit in:

- i) 'n 5 mm Geonet findrein
- ii) 'n 10 mm Eierkas findrein en
- iii) 'n 37 mm Eierkas findrein.

Die apparaat kan oor 'n reeks van drukgradiënte verskillende vloeitempos meet. Laterale gronddruk word deur middel van lugdruk gesimuleer. Daar is goeie regressie modelle verkry vir die vloeikapasiteit as 'n funksie van die drukgradiënt en laterale druk vir twee van die findrein tipes.

Die logiese volgende stappe in die analise en sintese van ondergrondse vloeistelsels in plaveiselstrukture, wat nie in hierdie verslag aangespreek word nie, is om die vloei eienskappe van versadigde en onversadigde padbou- en verwante materiale te bepaal. 'n Wiskudige model kan dan ontwikkel word om versadigde, onversadigde en ongestadigde vloei te modeleer. Hierdie model kan gebruik word om vloeistesels te ontleed en die invloed van 'n dreineringstelsel in 'n vloeistelsel te voorspel. Op grond daarvan kan beter ontwerpe dan gedoen word.

Die model moet gebiede waar 'n groot verskil in relatiewe deurlaatbaarheid die lokale opdamming van water in die plaveisel tot gevolg sal hê, kan identifiseer. Sodoende kan erosie en 'n drastiese verlies van skuifsterkte as gevolg van ruimtedruk verhoed word, deur die ondergrondse dreineringstelselontwerp te verander. Verder moet die effek van effektiewe dreineringstelsels op die verlenging van plaveisellleeftyd in die veld gekwantifiseer word.

2. ONTWIKKELING VAN APPARAAT

Die basiese uitleg van die apparaat wat ontwikkel is, is in Figuur 2 getoon.

Die apparaat simuleer die laterale gronddruk wat neig om die Findrein toe te druk, met behulp van lugdruk in 'n rubber lugkussing. Die reeks van lugdrukke waaroor getoets word, is bepaal uit die volgende oorwegings. Die basiese vergelyking vir geostatiese spanning word gegee deur,

$$\sigma_h = K_0 z \tau$$

waar K_0= Koëffisiënt van spanning, (Granulêre materiaal) $0.3 < K_0 > 0.7$ (Klei)

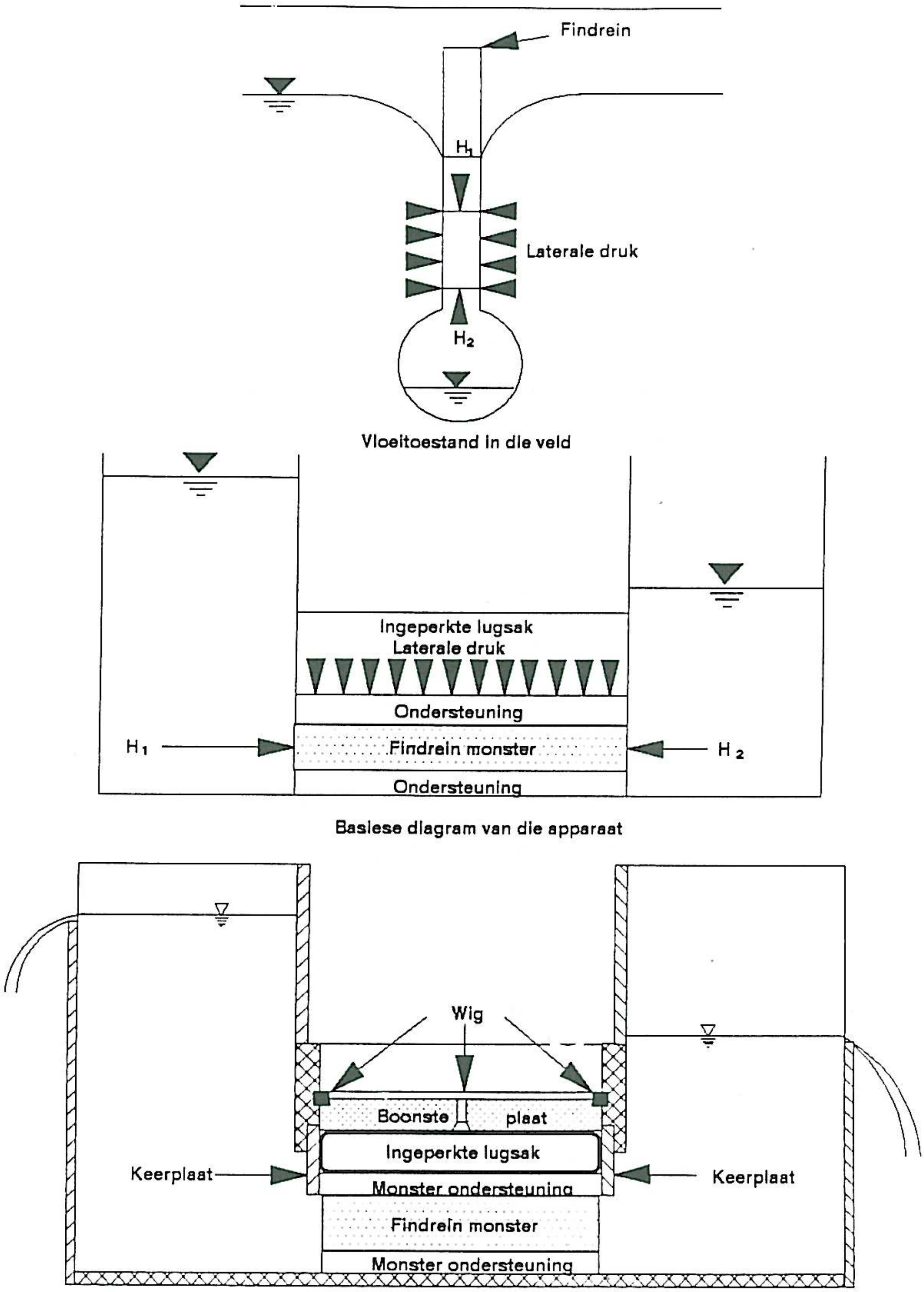
z= Diepte onder grondoppervlak (m)

τ= Eenheidsgewig van grond (kN/m^3)

Die eenheidsgewig van die grond is aanvaar as 25 kN/m^3 wat die maksimum waarde is wat verwag word, en die maksimum diepte is aanvaar as 5 m. Die horisontale spanning is dan 87,5 kPa indien aanvaar word dat dit 'n kleimateriaal is. Die apparaat is ontwikkel om veilig tot 100 kPa te toets wat ook die wettige perk is voordat die apparaat as 'n drukkamer getoets moet word.

Twee alternatiewe belastingskonfigurasies is ondersoek, naamlik 'n puntbelasting wat met gewigte op 'n plaat aangewend word en pneumatiese druk. Die massa benodig om 'n druk van 100 kPa te behaal is egter onprakties en daar is besluit op die lugdruk stelsel.

Las word oorgedra na die findreinmonster deur die twee ondersteuningskussings aan weerskante van die monster. Die ondersteuningskussings is rubber kussings of sandsakke wat uit 'n geotekstiel sak vervaardig is en met sand gevul is. Die doel van



Figuur 2: Snit deur findrein toetsapparaat.

die sagte ondersteuning is om die effek van matariaal wat deur laterale druk in die openinge van die findreinkern ingedruk word, te simuleer.

Indien die drukgradiënt as volg gedefiniëer word

$$I = \frac{\Delta h}{\Delta l}$$

waar

Δh = *Drukhoogteverandering* (m)

en

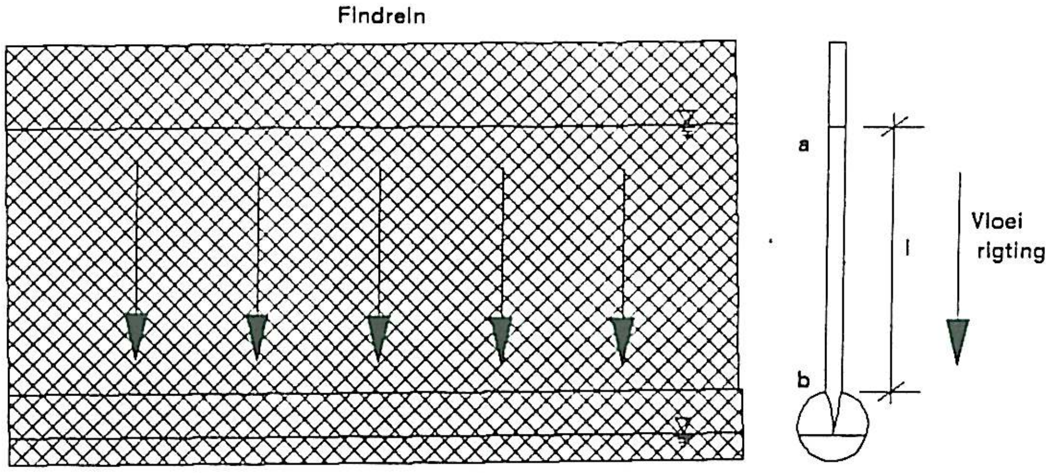
$\Delta\ l$ = *Die lengte waaroor die drukhoogte verander* (m)

kan verskillende waterdrukgradiënte verkry word deur die plate aan die stroomop en stroomaf kante van die apparaat, se hoogte te wissel. Die drukgradiënt sal dus die verskil in drukhoogte aan die voor en agterkant van die monster wees, gedeel deur die totale lengte van die monster waaroor die verlies plaasvind.

Indien 'n findrein egter onder werklike toestande bekou word, kan een van twee toestande geld. Die toestande word in Figuur 3 getoon.

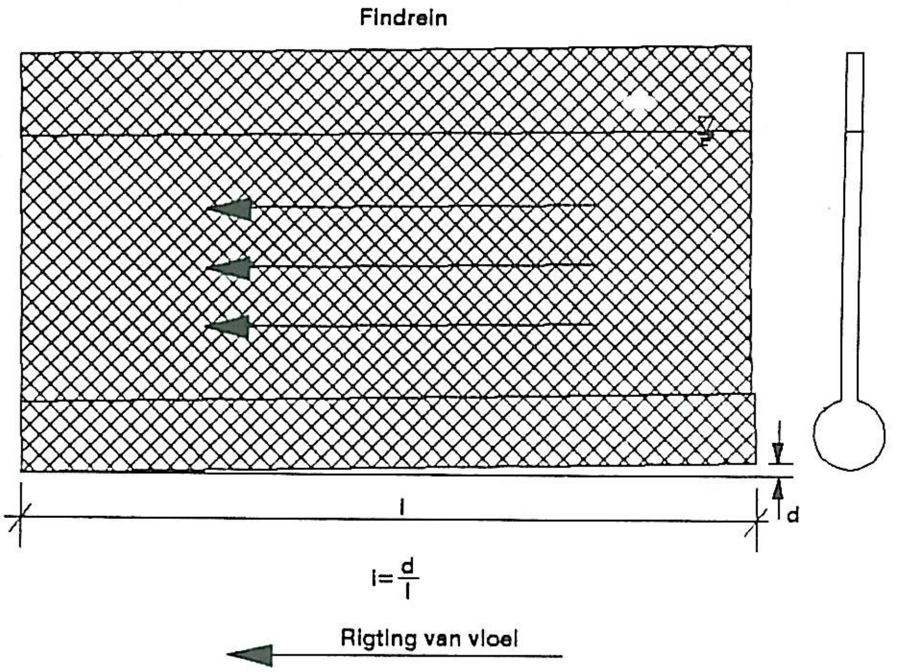
Die findrein kan vol vloei soos in Figuur 3(b) in welke geval kanaalvloei teorie sal geld, of net die regop dreineringsgedeeldte kan vol vloei en die versamelpyp halfvol soos in Figuur 3(a), in welke geval kanaalvloei in die versamelpyp geld. In die regop dreineringsgedeelte sal die totale energie verslies van punt a na punt b gelyk wees aan die verlies in potensiële energie (verskil in loodregte hoogte tussen punt a en b).

Die drukgradiënt sal dus die verlies in energie wees, gedeel deur die lengte van die findrein waaroor die verlies plaasvind. Hierdie twee waardes is egter ewe groot (die afstand tussen punte a en b) en gevolglik sal die drukgradiënt een wees. Murray,



$$i = \frac{\text{Verlies in energie}}{\text{Vloeilengte}} = \frac{l}{l} = 1$$

3(a) Versamel pyp vloei halfvol



$$i = \frac{d}{l}$$

3(b) Versamel pyp vloei vol

Figuur 3: Werklike vloeitoestande in findrein.

(1984) toon vloeikapasiteite vir 'n maksimum drukgradiënt van een, maar maak nie melding of dit die maksimum drukgradiënt is waarteen getoets is nie.

In die geval waar die hele drein vol vloei en waar kanaalvloei geld, word uniforme kanaalvloei teorie gebruik waar die drukgradiënt aanvaar word om gelyk te wees aan die bodemhelling van die kanaal (Ven te Chow, 1983). In hierdie geval sal die bodemhelling van die kanaal die helling wees waarteen die versamelpyp na die uitlaatpunt toe loop. Hierdie helling sal ongeveer 4% wees en Dempsey, (1988) het hierdie vorm van kanaalvloei vir findreins ondersoek.

Die apparaat moet dus oor drukgradiënte van 0,04 tot 1 die vloeikapasiteit van die findrein kan bepaal. Die vloeikapasiteit van 'n findrein kan met die huidige apparaat oor hierdie reeks van drukgradiënte bepaal word deur toetse teen 'n reeks van drukgradiënte groter as 0,3 te doen en dan met regressie modelle die kapasiteit teen drukgradiënte van kleiner as 0,3 te voorspel. Die vloeikapasiteit kan dus vir beide situasies soos in Figuur 3 getoon is, bepaal word.

Die apparaat is uit Perspex vervaardig en bestaan uit die hoofapparaat soos in Figuur 2 getoon, met 'n opvang en dreineringsbak. Die dreineringsbak is gekalibreer in eenhede van 5 l tot by 20 l om die volume vloei per tydseenheid te bepaal tydens elke toets.

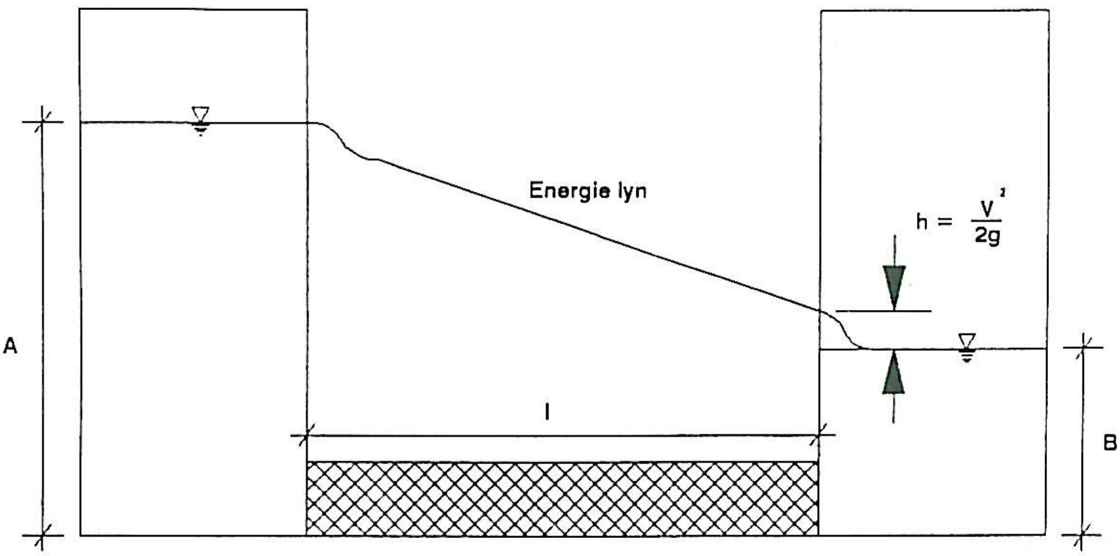
'n Statiese drukhoogte van 500 mm kan aan die stroomop kant gehandhaaf word indien die watertoevoer na die apparaat voldoende is. Die hoogte van die stroomop oorloop kan in inkremente van 100 mm afgestel word. Die hoogte van die oorloop aan die stroomaf kant kan vanaf 400 mm, in inkremente van 100 mm, afgestel word tot 0 mm in welke geval daar geen statiese druk aan die stroomaf kant is nie.

3. VERSKILLENDE BASIESE OPSTELLINGS.

Twee basiese opstellings kan onderskei word. In die eerste geval is daar 'n statiese druk aan die stroomaf kant van die apparaat soos in Figuur 4(a) getoon is. Tweedens kan die plaat aan die stroomaf kant verwyder word om sodoende geen statiese druk aan die stroomaf kant te hê nie. Die opstelling is in Figuur 4(b) getoon. Die twee opstellings moet egter verskillend evalueer word.

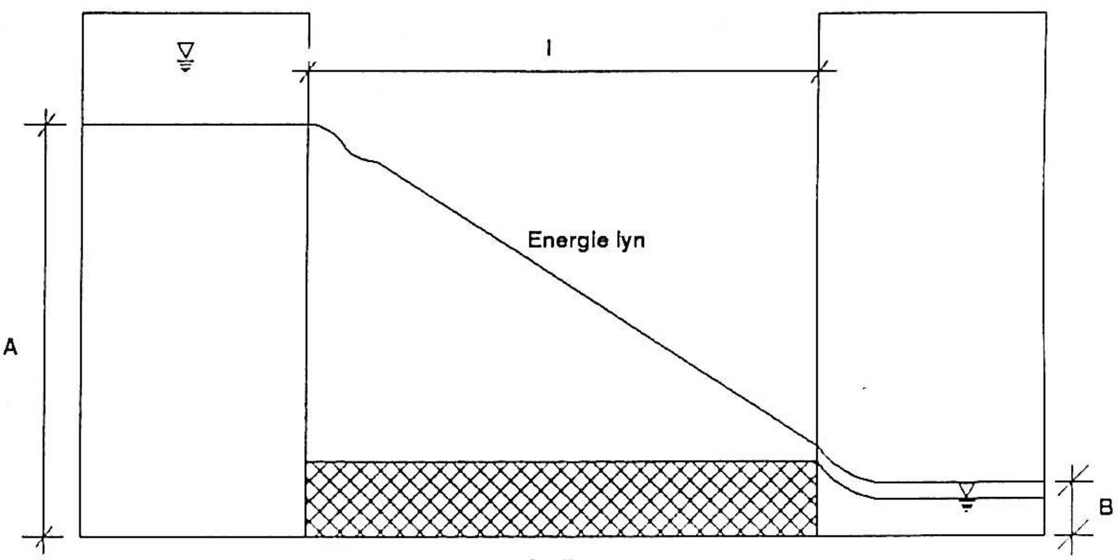
Die opstelling met statiese druk aan die stroomaf kant het 'n snelheidsdrukkomponent wat bydra tot die hoogte van die totale energie lyn net voor die findrein verlaat word. Indien aanvaar word dat die grootte van die stroomaf reservoir groot is in vergelyking met die findrein, sal die snelheidsdrukkomponent in die reservoir nul wees. Die totale verskil in energie wat die stelsel aandryf, is die verskil in statiese drukhoogte aan die stroomop- en stroomaf kant van die findrein monster.

In die geval van die opstelling sonder statiese druk aan die stroomaf kant gaan die snelheidsdrukkomponent egter nie verlore sodra die water die findrein monster verlaat nie. Die totale energie aan die stroomaf kant is dus effens hoër as die die datumlyn weens die snelheidsdrukkomponent. Hierdie snelheidskomponent moet in ag geneem word by die berekening van die drukgradiënt oor die findrein monster.



$$I = \frac{A - B}{l}$$

4(a) Statiese druk aan stroomaf kant



$$I = \frac{A - B}{l}$$

4(b) Geen statiese druk aan stroomaf kant nie.

Figuur 4: Verskillende basiese opstellings van die findrein toetsapparaat.

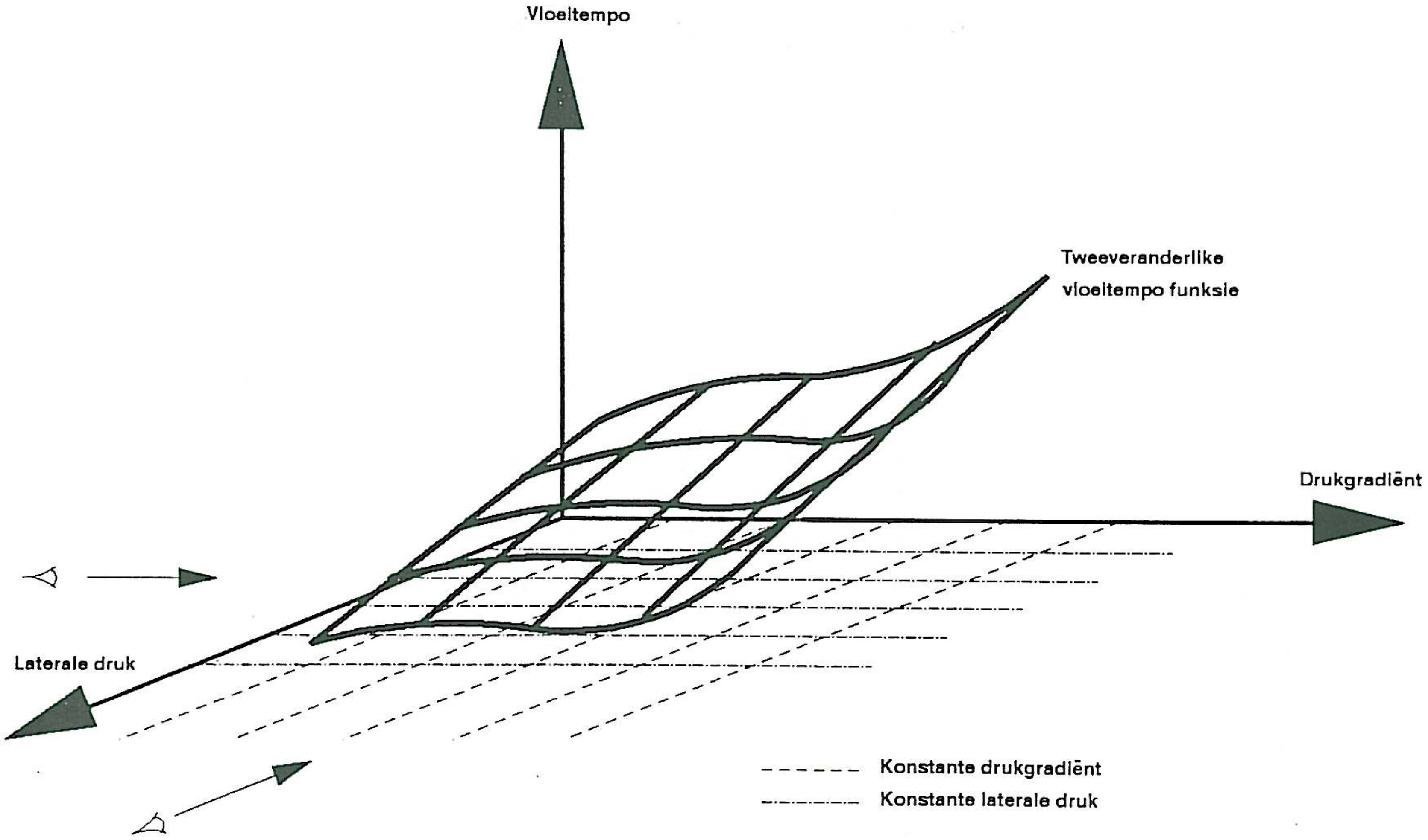
4. TWEEVERANDERLIKE FUNKSIES EN KONTOERKROMMES.

Die vloeitempo deur die findreinmonster word deur twee veranderlikes beïnvloed vir elke spesifieke opstelling. Die veranderlikes is die laterale druk en die water drukgradiënt oor die monster.

Die vloeitempo kan dus in 'n driedimensionele ruimte as 'n tweeveranderlike funksie voorgestel word. Die vloeitempo funksie sal 'n vlak in die driedimensionele ruimte vorm soos in Figuur 5 getoon.

Snitte wat in die driedimensionele ruimte gemaak word vir konstante laterale druk en drukgradiënt, sny die funksie langs lyne wat as kontoerkrommes bekend staan. Indien die driedimensionele ruimte loodreg op die drukgradiënt as beskou word, word die laterale druk kontoerkrommes as eenveranderlike funsies in 'n plat vlak gesien. Langs hierdie laterale druk kontoerkrommes word die vloeitempo slegs bepaal deur die drukgradiënt. Die laterale druk kontoerkrommes stel dus die verband tussen vloeitempo en drukgradiënt voor.

Indien die driedimensionele ruimte loodreg op die laterale druk as beskou word, word die drukgradiënt kontoerkrommes as eenveranderlike funksies in 'n plat vlak gesien. Die drukgradiënt kontoerkrommes stel die verband tussen vloeitempo en laterale druk voor, indien drukgradiënt konstant bly.



Figuur 5: Tweeveranderlike vloeitempo funksie en kontoerkrommes.

5. FINDREIN KAPASITEIT VIR ALGEMENE FINDREIN TIPES.

Drie findrein tipes was beskikbaar vir toetse. Die ondersteuningskussings is met sandsakke vervang vir die 5mm Geonet findrein en die 10 mm Eierkas findrein met die gevolg dat 'n totaal van vyf opstellings getoets is, naamlik:

i. 5 mm Geonet Findrein met rubber ondersteunings

ii. 10 mm Eierkas Findrein met rubber ondersteunings

iii. 37 mm Eierkas Findrein met rubber ondersteunings en

iv. 5 mm Geonet Findrein met sandsak ondersteuning.

v. 10 mm Eierkas findrein met sandsak ondersteuning.

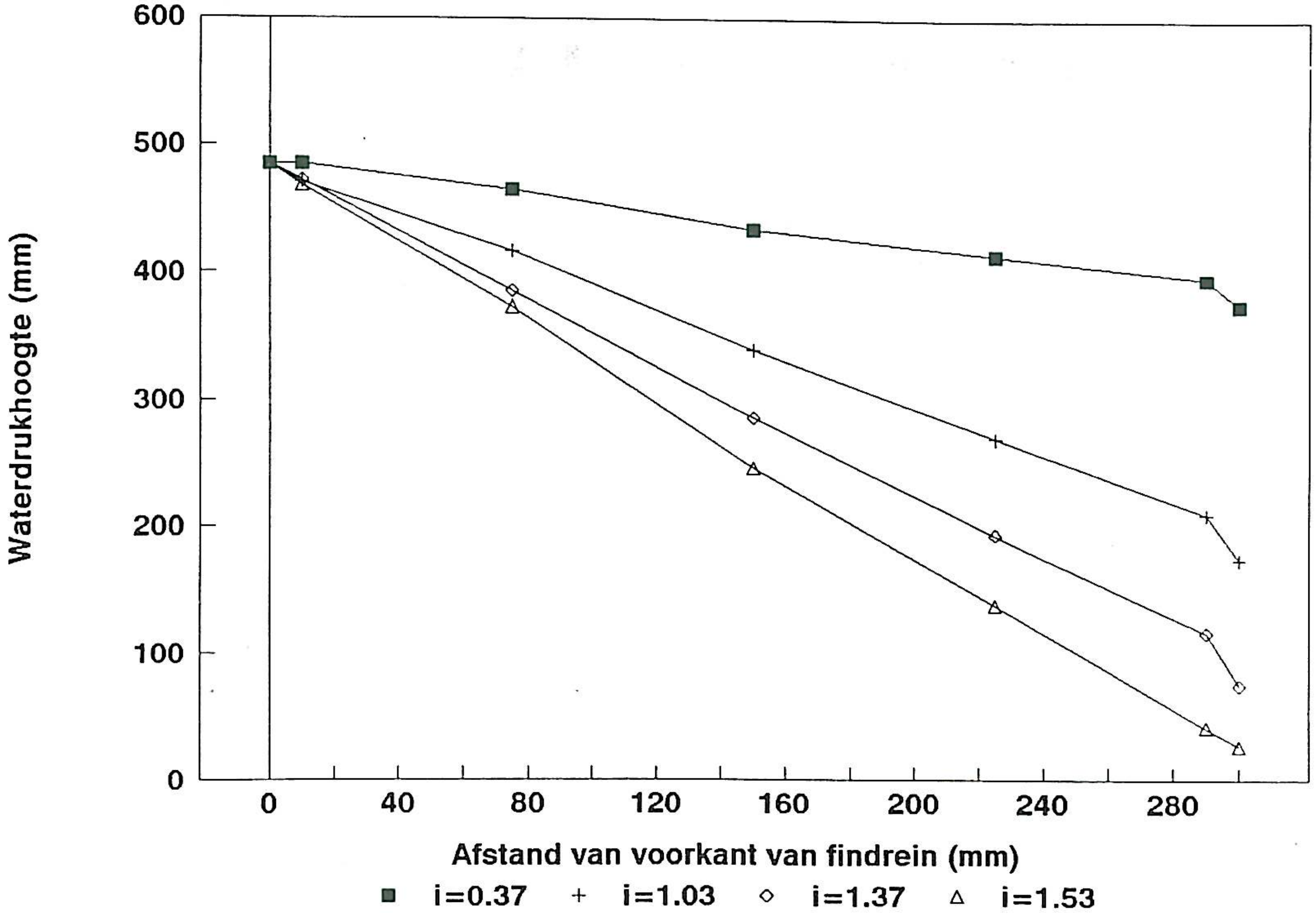
5.1 5 mm GEONET FINDREIN MET RUBBER ONDERSTEUNING.

Piezometerpypies is op gelyke afstande deur die findreinmonster geplaas vir hierdie opstelling. Die meetpunte is op 10, 75, 150, 225 en 290 mm vanaf die voorkant van die 300 mm monster geïnstaleer.

Die stroomop oorloop se hoogte is konstant gehou op 500 mm en die stroomaf oorloop is verlaag in inkremente van 100mm vanaf 400 mm tot 0 mm. Die Piezometervlakke wat in die findreinmonster gemeet is, is in Figuur 6 getoon.

Die verlies in drukhoogte, as gevolg van die verlies van die snelheidskomponent van die totale drukhoogte, is duidelik waarneembaar vir die opstellings met statiese druk aan die stroomaf kant.

Die drukhoogteverlies deur die monster is egter lineêr. Die verskil in drukhoogte tussen die voor en agterkant van die monster soos dit in Figuur 6



Figuur 6: Drukgradiënt oor 5 mm Geonet Findrein.

getoon is, is gebruik om die drukgradiënt oor die monster te bereken. Die drukgradiënte waarteen die toets gedoen is, is 0,37, 0,7, 1,03, 1,37 en 1,53.

Die laterale druk is gewissel van 20 kPa tot 100 kPa in inkremente van 20 kPa. Die drukgradiënt oor die monster het egter konstant gebly met veranderende laterale druk.

Figuur 7 toon die laterale druk kontoerkrommes vir vloeitempo oor 'n reeks van drukgradiënte vir die 5 mm Geonet findrein met rubber ondersteuning. Die regressie vergelyking vir die kontoerkrommes is 'n magsfunksie van die vorm

$$y = a^*x^b$$

Die vergelyking vir die druk kontoerkrommes van die vloeitempo is gevolglik

$$\text{Vloeitempo (l/s/m wydte)} = a^*(\text{Drukgradiënt})^b$$

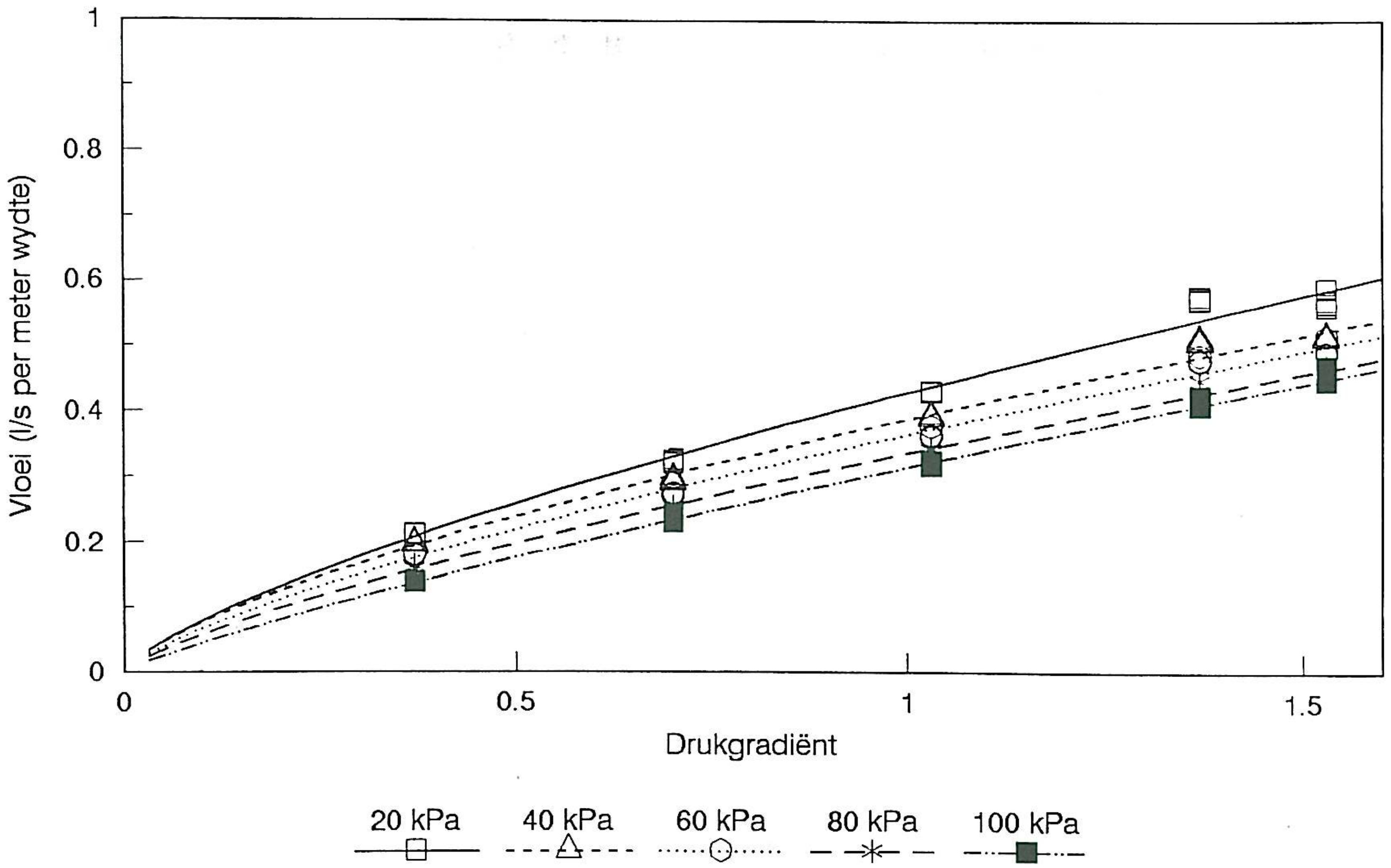
waar a en b afhanklik is van die waarue van die laterale druk vir die spesifieke kontoerkromme.

Die vloeitempos vir al die laterale druk kontoerkrommes neig na nul indien die drukgradiënt na nul neig. Die praktiese implikasie hiervan is dat daar geen vloei sal plaasvind indien daar nie 'n drukgradiënt is om die stelsel aan te dryf nie.

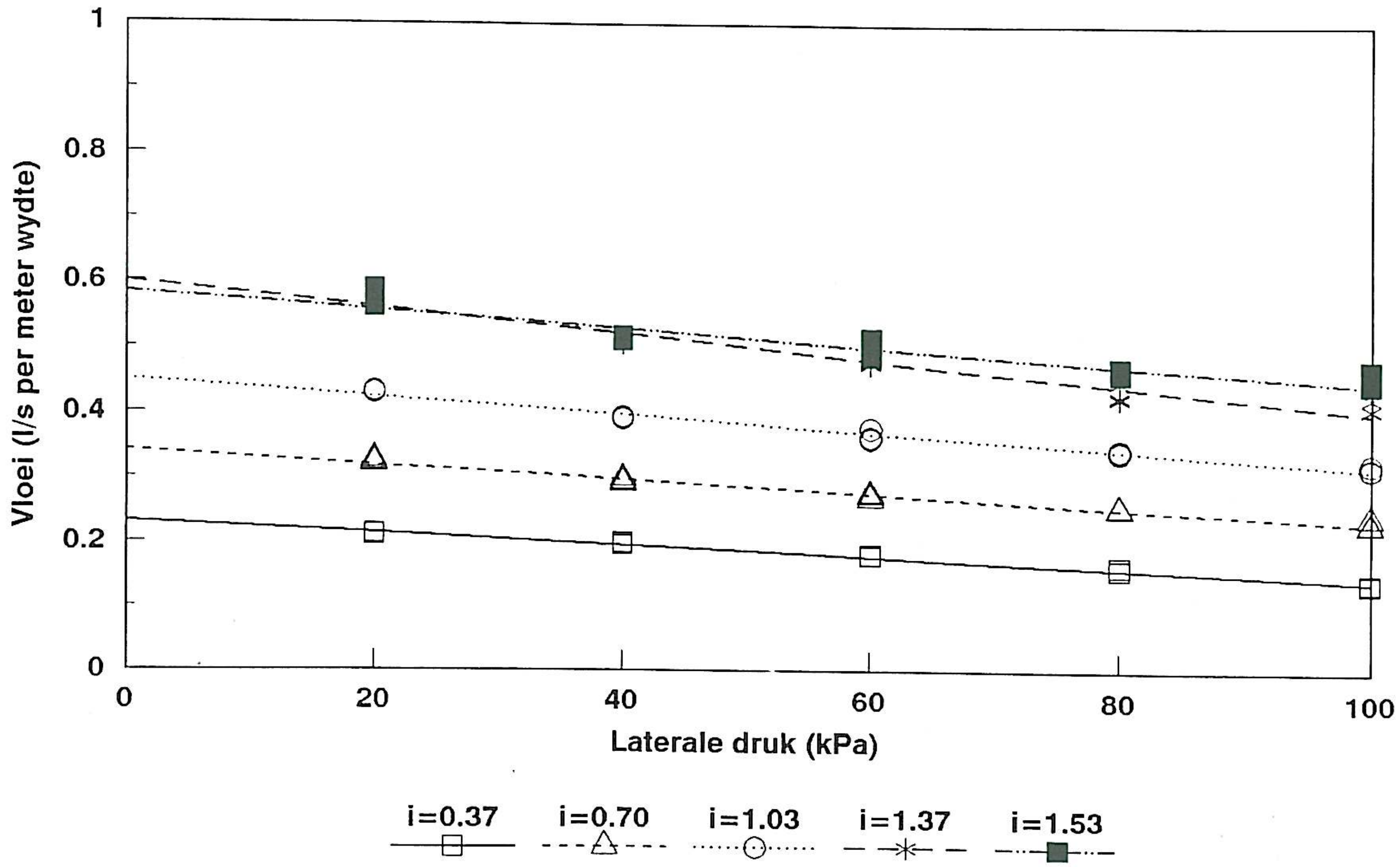
Drukgradiënt kontoerkrommes vir vloeitempo is in Figuur 8 getoon en die beste regressie model is 'n lineêre model. Die model is van die vorm

$$\text{Vloeitempo (l/s/m wydte)} = a + b^*(\text{laterale druk})$$

waar a en b afhanklik is van die waarde van die drukgradiënt vir die spesifieke kontoerkromme.



Figuur 7: Vloei deur 5 mm Geonet findrein met rubber ondersteuning



Figuur 8: Vloei deur 5 mm Geonet findrein met rubber ondersteuning.

Die vloeitempo neem af met toenemende druk maar teen 100 kPa is die findrein nog nie platgedruk nie, en vloei vind steeds plaas.

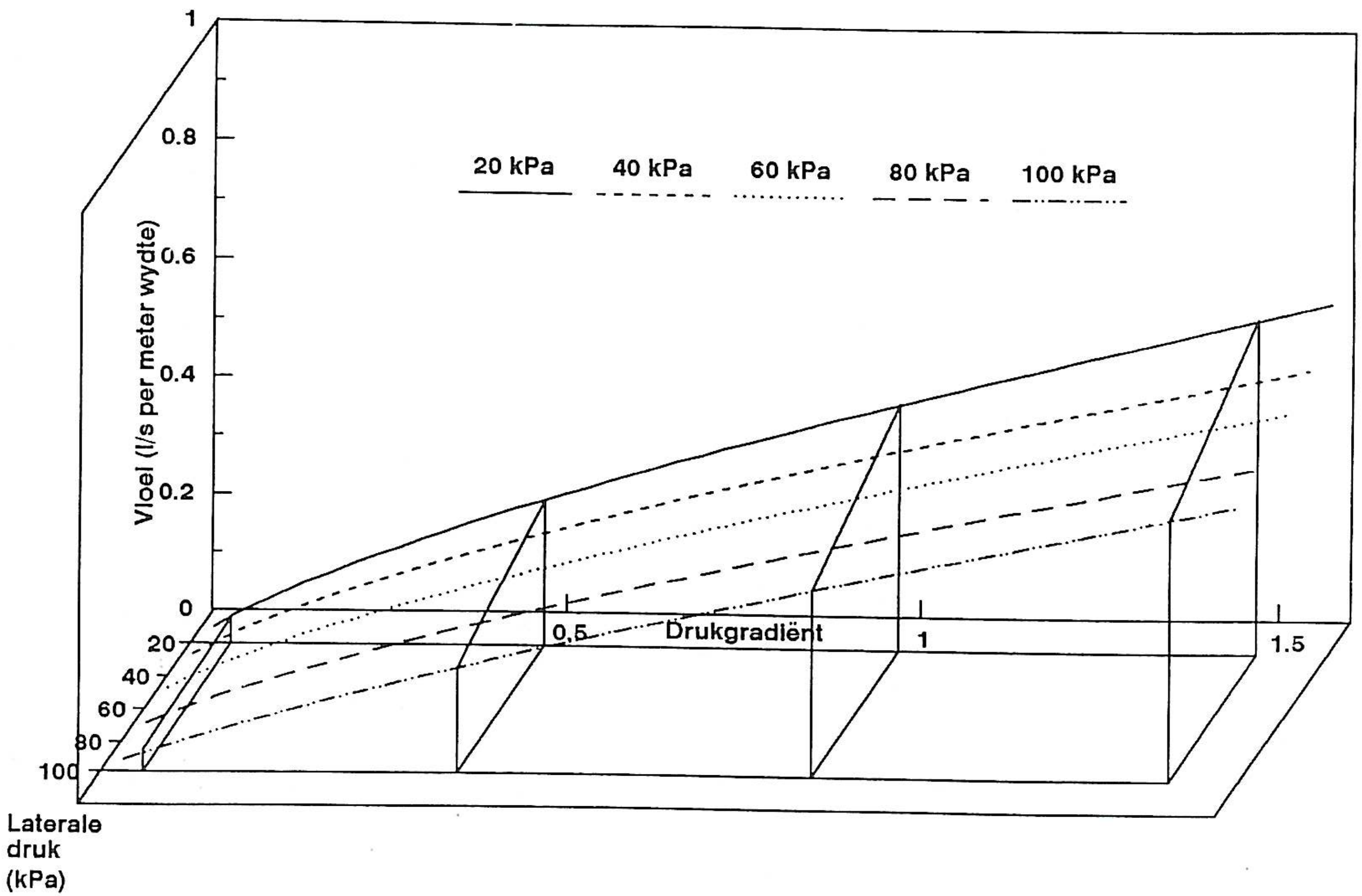
Tabel 1 toon die regressie modelle vir beide die laterale druk kontoerkrommes en die drukgradiënt kontoerkrommes. Die regressie modelle is baie goeie passings vir die gemete data.

Laterale druk kontoerkrommes				Drukgradiënt kontoerkrommes			
Vloeitempo = a*(Drukgradiënt)^b				Vloeitempo = a + b*(laterale druk)			
Laterale druk (kPa)	R^2	b	a	Druk gradiënt	R^2	b	a
20	0,992	0,7314	0,4302	0,37	0,99	-0,00092	0,2329
40	0,993	0,7018	0,3893	0,7	0,98	-0,00111	0,3415
60	0,994	0,7371	0,3660	1,03	0,98	-0,00136	0,4520
80	0,998	0,7635	0,3368	1,37	0,96	-0,00199	0,6013
100	0,998	0,8409	0,3155	1,53	0,89	-0,00140	0,5851

Tabel 1: *Regressie konstantes vir laterale druk kontoerkrommes en drukgradiënt kontoerkrommes vir die 5 mm Geonet findrein met rubber ondersteuning.*

Indien die resultate in Figure 7 en 8 gekombineer word, kan die tweeveranderlike vloeitempo funksie in die driedimensionele ruimte voorgestel word as 'n funksie van drukgradiënt en latarale druk. Die vloeitempo funksie vorm dan 'n vlak in die driedimensionele ruimte en die kontoerkrommes wat in Figure 7 en 8 getoon is, is lyne in die vlak waarlangs of die laterale druk of die drukgradiënt konstant bly.

Figuur 9 toon die vloeitempo as funksie van die laterale druk en drukgradiënt vir die 5 mm Geonet findrein met rubber ondersteuning.



Figuur 9: Vloeitempo as funksie van die drukgradiënt en laterale druk vir die 5 mm Geonet findrein met rubber ondersteuning.

5.2 10 mm EIERKAS FINDREIN MET RUBBER ONDERSTEUNING.

Die findrein is getoets met die stroomop oorloop op 500 mm en die stroomaf oorloop is laat sak vanaf 400 mm tot 100 mm in inkremente van 100 mm. Die waterdrukhoogtes aan die voor en agterkant van die 10 mm eierkas findrein is in Figuur 10 getoon. Die findrein is nie getoets sonder statiese waterdruk aan die stoomaf kant nie. Die drukgradiënte wat uit Figuur 10 bereken is, is 0,37, 0,7, 1,03 en 1,37.

Die laterale druk is gevariëer van 20 kPa na 80 kPa in intervalle van 20 kPa. Die findrein is platgedruk teen 100 kPa en nie teen die druk getoets nie.

Figuur 11 toon die laterale druk kontoerkrommes vir vloeitempo oor 'n reeks van drukgradiënte vir dir 10 mm eierkas findrein met rubber ondersteuning. Die regressie vergelyking vir die kontoerkrommes is weereens 'n magsfunksie van die vorm

$$y = a^{*}x^{b}$$

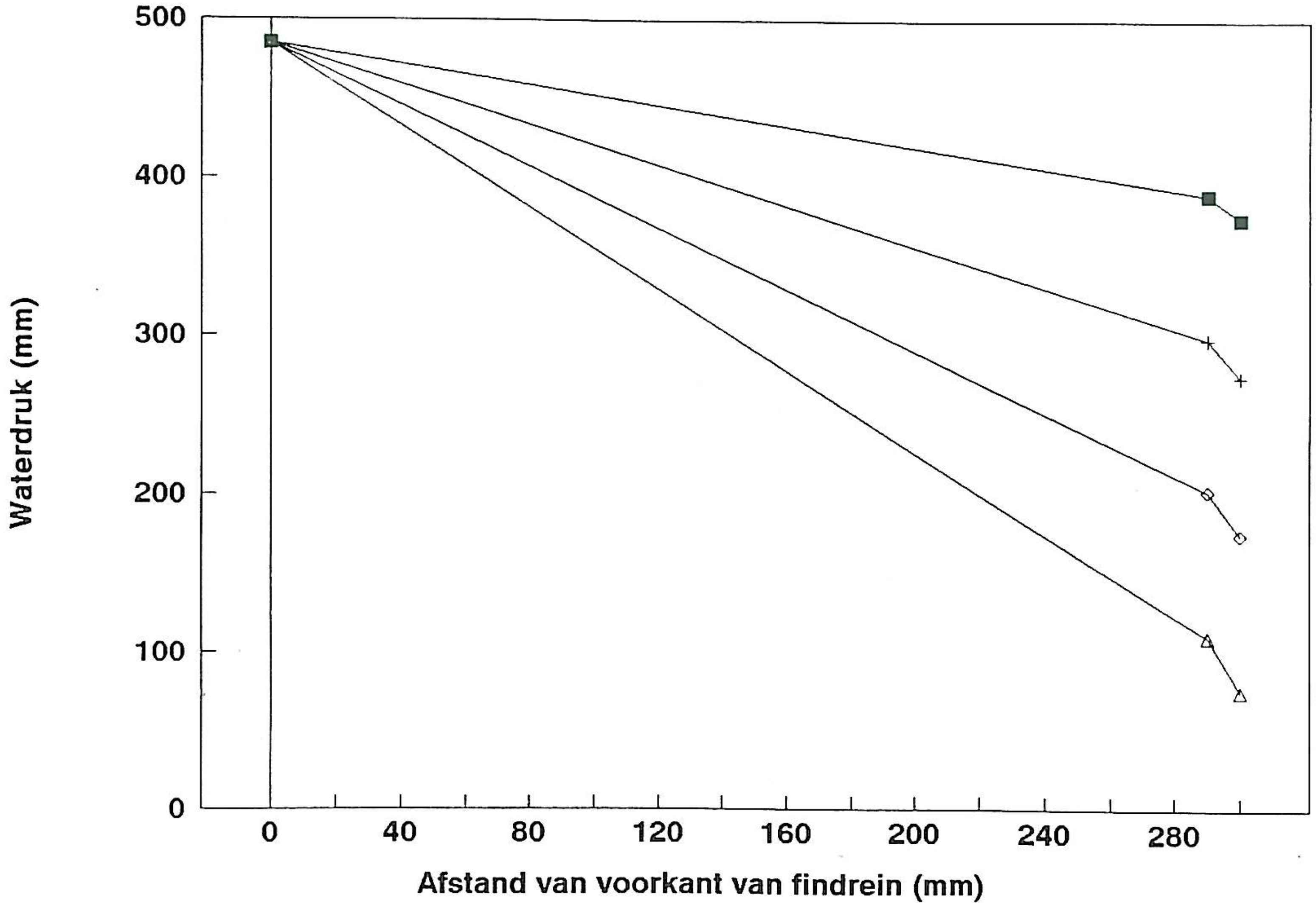
Die vergelyking vir die druk kontoerkrommes van die vloeitempo is gevolglik

$$\text{Vloeitempo (l/s/m wydte)} = a^{*}(\text{Drukgradiënt})^{b}$$

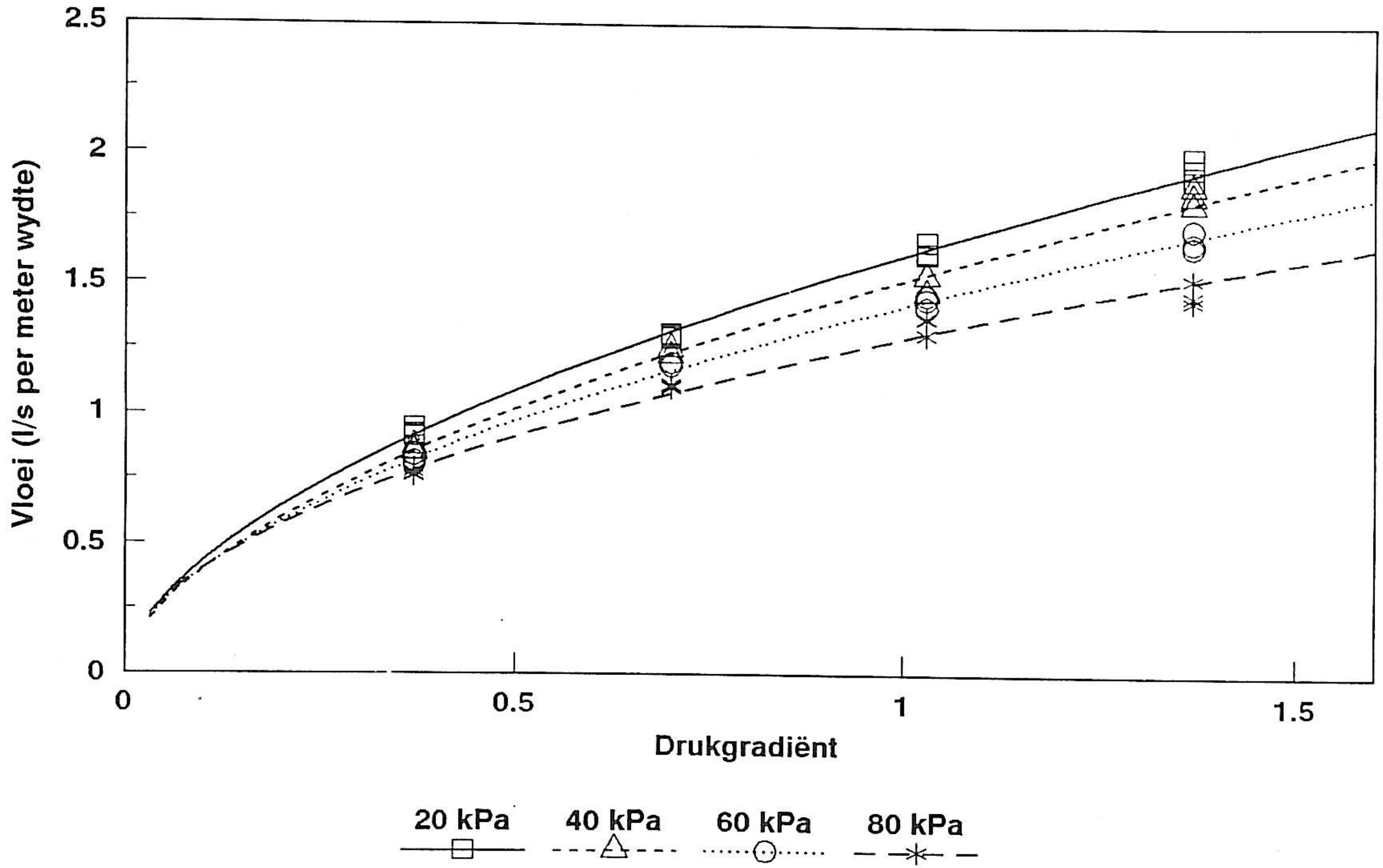
waar a en b afhanklik is van die waarde van die laterale druk waarvoor die druk kontoerkromme opgestel word.

Die vloeitempo vir al die laterale druk kontoerkrommes neem af met afnemende drukgradiënt en neig na nul indien die drukgradiënt na nul neig. Die praktiese implikasie is dat die vloei na nul neig indien daar geen drukgradiënt is om die stelsel aan te dryf nie.

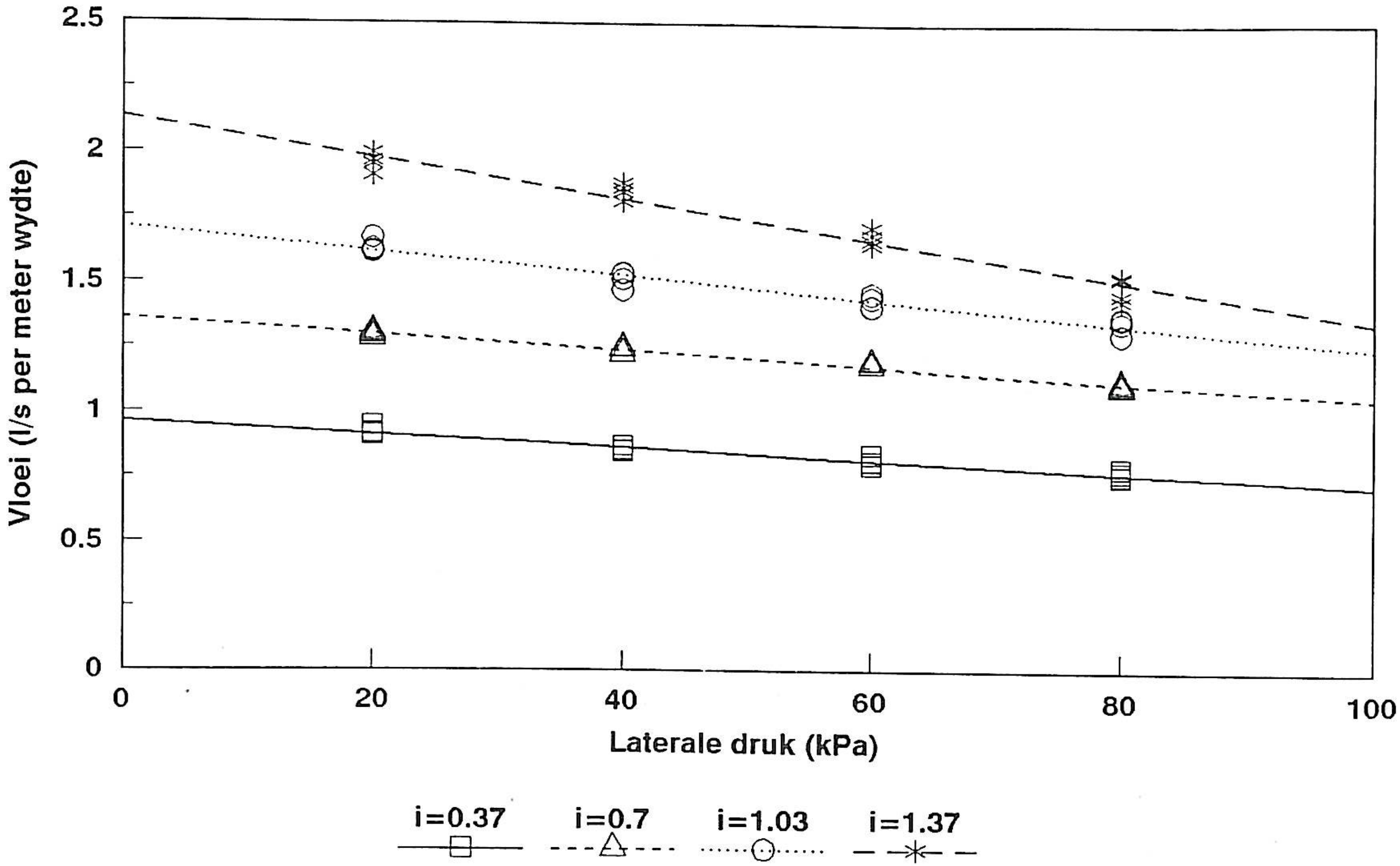
Drukgradiënt kontoerkrommes vir vloeitempo is in Figuur 12 getoon en die beste regressie model is 'n lineêre model van die vorm:



Figuur 10: Drukgradiënt oor 10mm Eierkas findrein met rubber ondersteuning.



Figuur 11: Vloei deur 10mm Eierkas findrein met rubber ondersteuning.



Figuur 12: Vloei deur 10mm Eierkas findrein met rubber ondersteuning.

Vloeitempo (l/s/m wydte) = a + b*(laterale druk)

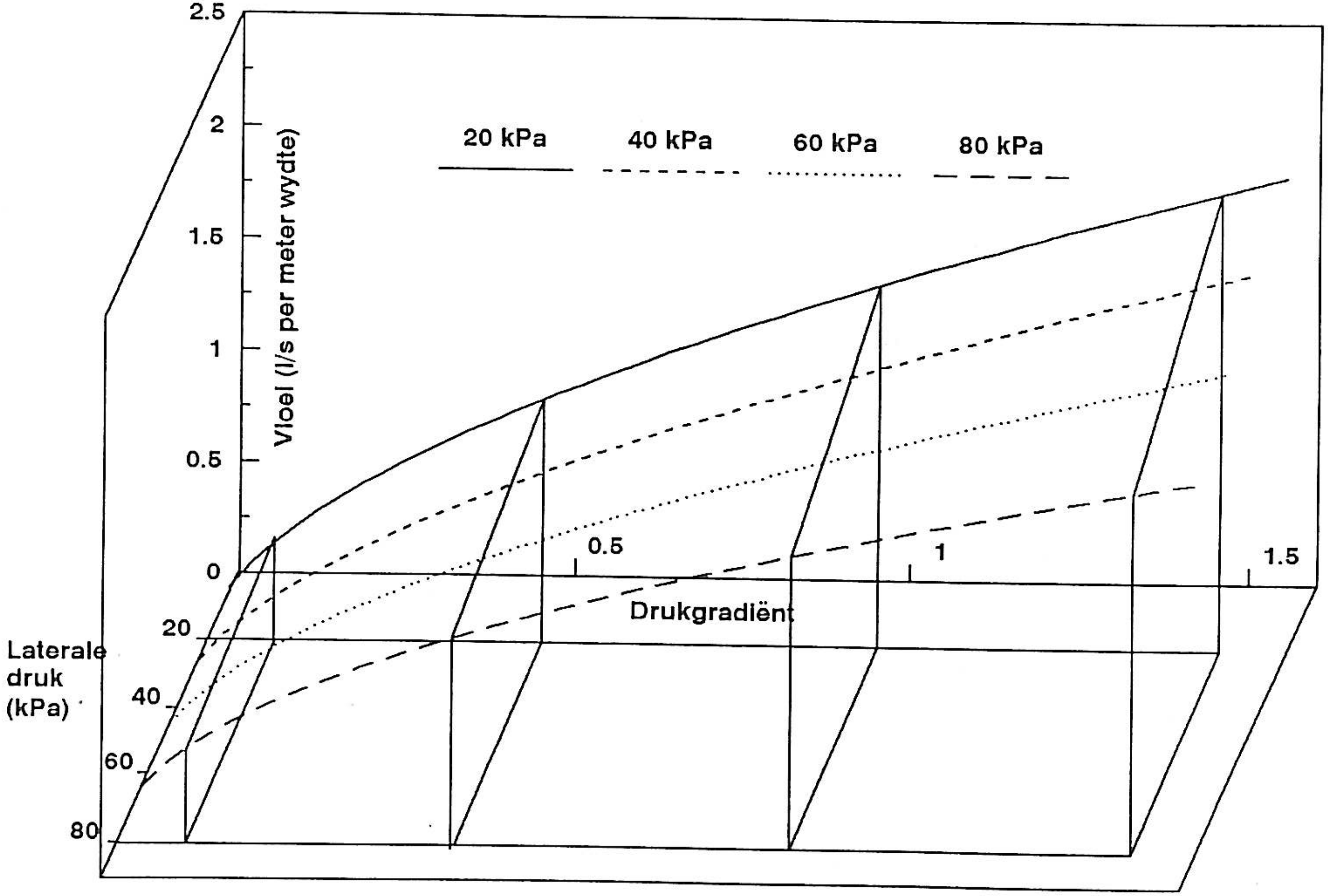
waar a en b afhanklik is van die die waarde van die drukgradiënt waarvoor die drukgradiënt kontoerkromme opgestel word. Die vloeitempo neem af met toenemende druk.

Tabel 2 toon die regressie modelle vir beide die laterale druk kontoerkrommes en die drukgradiënt kontoerkrommes. Die regressie modelle is baie goeie passings vir die gemete data.

Laterale druk kontoerkrommes				Drukgradiënt kontoerkrommes			
Vloeitempo = a*(Drukgradiënt)^b				Vloeitempo = a + b*(laterale druk)			
Laterale druk (kPa)	R^2	b	a	Druk gradiënt	R^2	b	a
20	0,997	0,5709	1,6148	0,37	0,934	-0,00241	0,9625
40	0,996	0,5771	1,5174	0,7	0,967	-0,00299	1,3596
60	0,996	0,5487	1,4202	1,03	0,940	-0,00460	1,7110
80	0,987	0,5062	1,2956	1,37	0,955	-0,00787	2,1364

Tabel 2: *Regressie konstantes vir laterale druk kontoerkrommes en drukgradiënt kontoerkrommes vir die 10 mm Eierkas findrein met rubber ondersteuning.*

Indien die resultate in Figure 11 en 12 gekombineer word, kan die tweeveranderlike vloeitempo funksie in die driedimensionele ruimte voorgestel word as 'n funksie van drukgradiënt en laterale druk. Figuur 13 toon die vloeitempo as funksie van drukgradiënt en laterale druk vir die 10 mm Eierkas findrein.



Figuur 13: Vloeitempo as funksie van drukgradiënt en laterale druk vir die 10 mm Eierkas findrein met rubber ondersteuning.

5.3 37 mm EIERKAS FINDREIN MET RUBBER ONDERSTEUNING.

Die resultate vir die 37 mm Eierkas findrein is in Tabel 3 getoon. Die vloeikapasiteit van die findrein is so hoog dat die toevoer na die die apparaat slegs in staat was om lae drukgradiënte, in die omgewing van 0,23 tot 0,32 te onderhou.

Laterale druk (kPa)	20	20	40	40
Dukgradiënt	0,24	0,23	0,29	0,32
Vloeitempo (l/s per m wydte)	8,8496	9,0909	8,1301	9,4697
	8,7719	8,5106	7,7519	8,9686
	8,6059	8,6009	8,0128	8,8080
	8,6281	8,7198	7,9808	8,8535

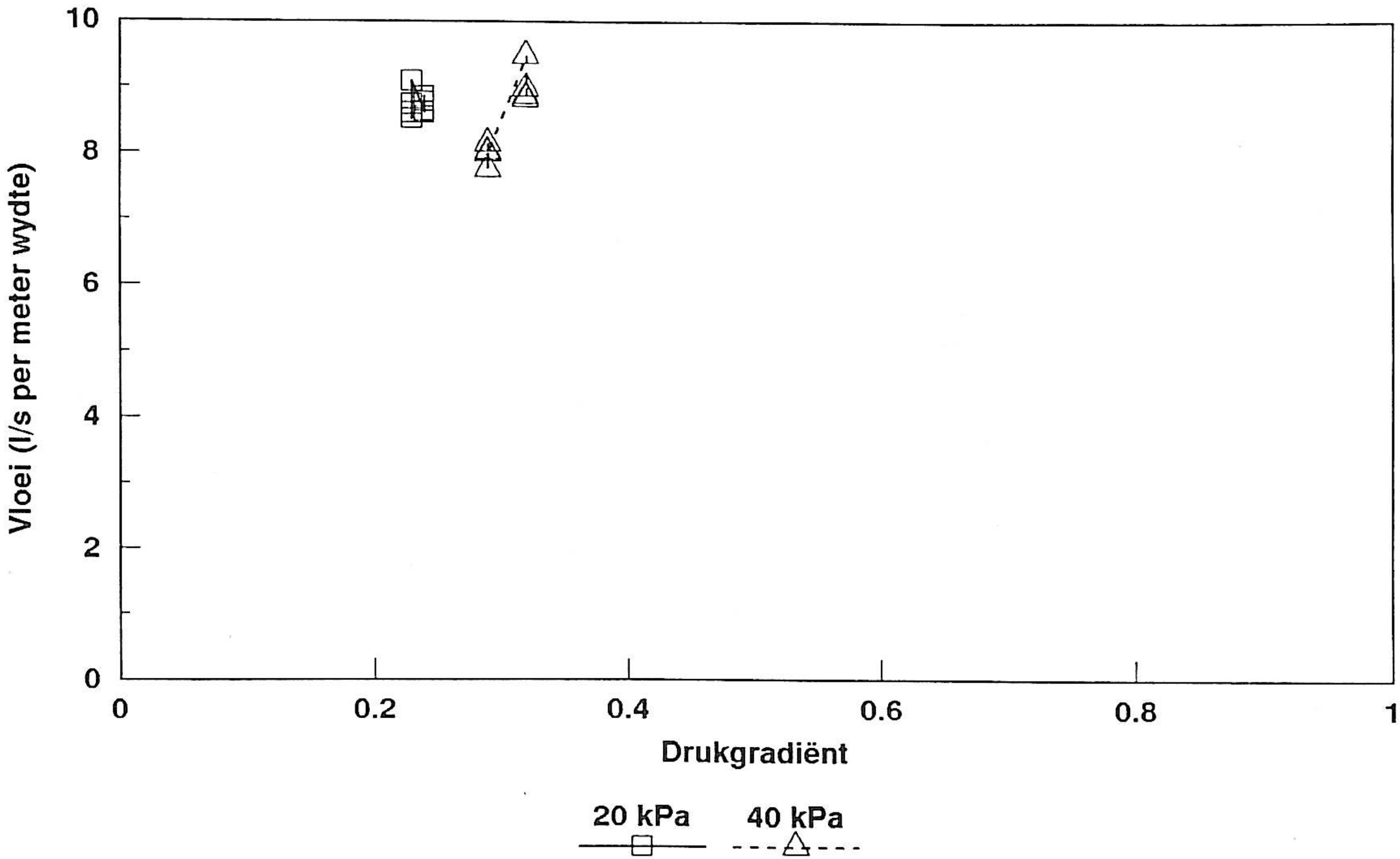
Tabel 3: *Vloeitempo vir die 37 mm Eierkas findrein met rubber ondersteuning.*

Die resultate van Tabel 3 is in Figuur 14 getoon maar geen regressie is gedoen nie omdat daar nie 'n voldoende reeks van drukgradiënte is nie. Die kapasiteit van die findrein is egter baie hoog.

Die maksimum laterale druk waarteen getoets is, is 40 kPa en teen hoër drukke het die findrein platgedruk.

5.4 5 mm GEONET FINDREIN MET SANDSAK ONDERSTEUNING.

Piezometerpypies is weereens op gelyke afstande deur die findreinmonster geplaas. Die stroomop drukhoogte is konstant gehou op 500 mm en die stroomaf drukhoogte is gewissel van 400 mm na 100 mm. Die drukgradiënt



Figuur 14: Vloei deur 37 mm Eierkas findrein met sandsak ondersteuning.

wat oor die monster gemeet is, is 0,37, 0,72, 1,05 en 1,39 en is in Figuur 15 getoon.

Die laterale druk is gewissel van 20 kPa tot 80 kPa in inkremente van van 20 kPa.

Figuur 16 toon die laterale druk kontoerkrommes vir die 5 mm Geonet findrein met sandsak ondersteuning oor die reeks van drukgradiënte waarteen die toets gedoen is.

Die regressie vergelyking vir die kontoerkrommes is 'n magsfunksie van die vorm:

$$\text{Vloeitempo (l/s/m wydte)} = a^{*}(\text{drukgradiënt})^{b}$$

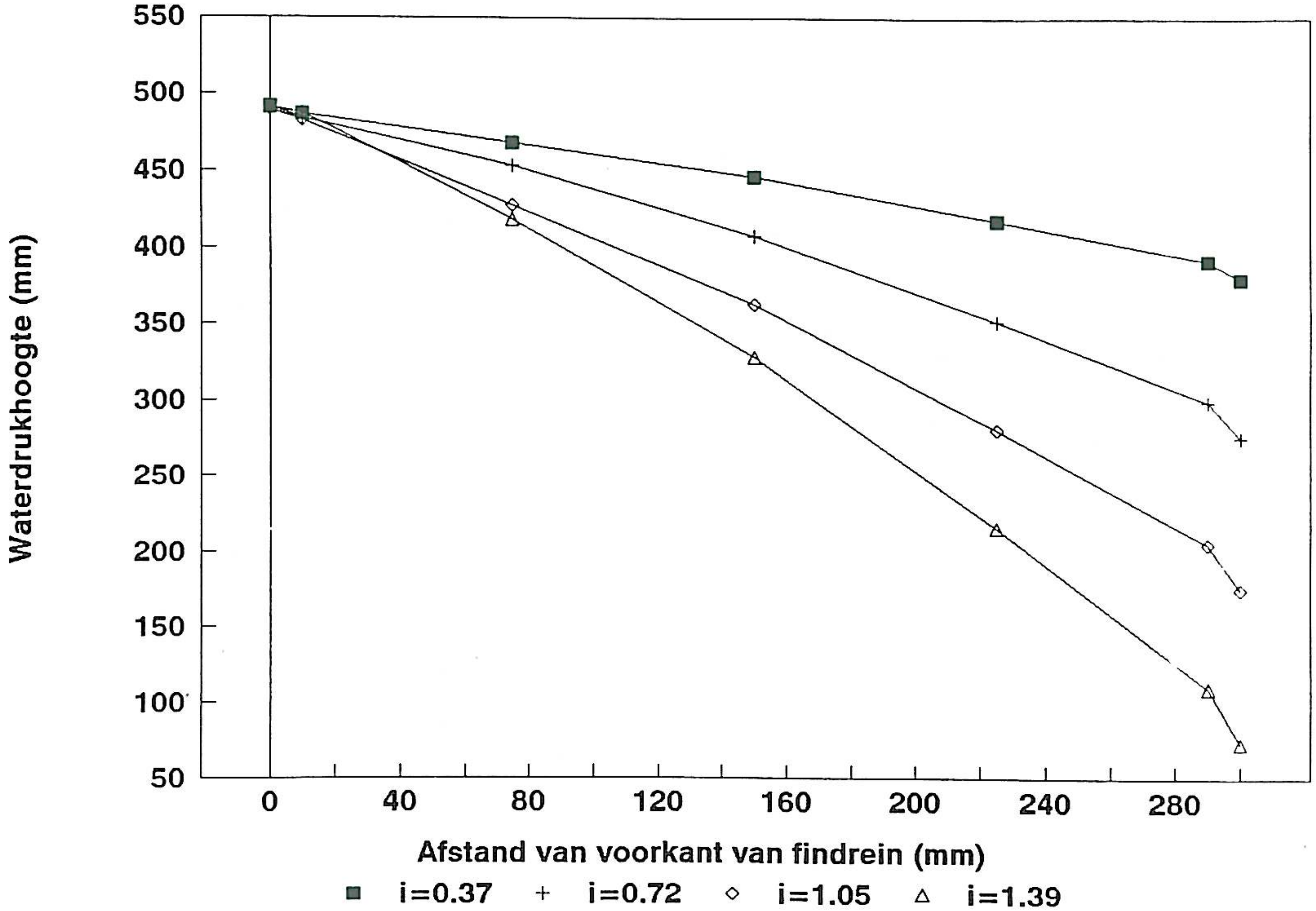
waar a en b regressie konstantes is waarvan die grootte afhanklik is van die die waarde van die laterale druk waarvoor die spesifieke kontoerkromme geld.

Drukgradiënt kontoerkrommes vir vloeitempo is in Figuur 17 getoon en die beste regressie model is 'n lineêre model. Die model is as volg:

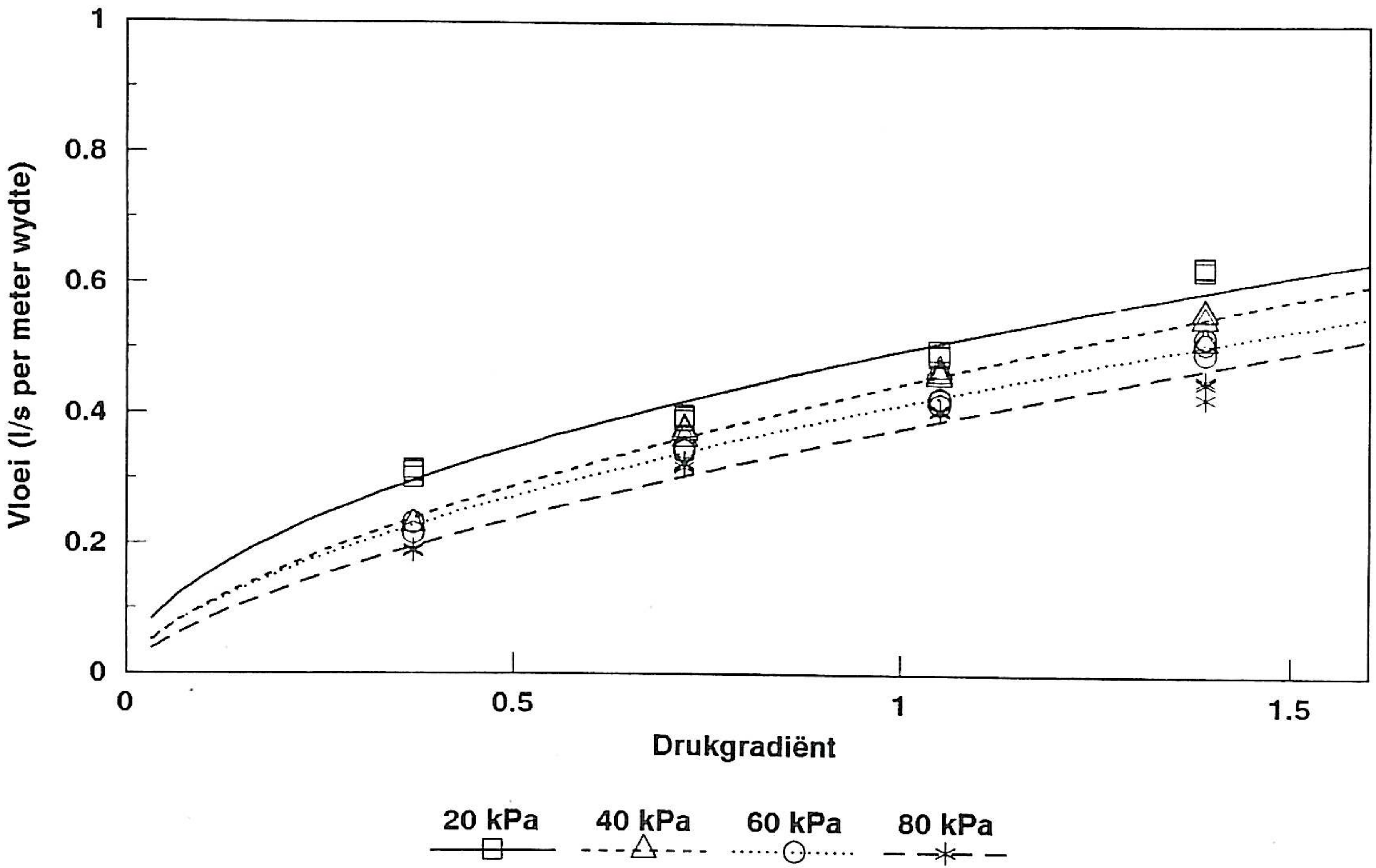
$$\text{Vloeitempo (l/s/m wydte)} = a+b^{*}(\text{laterale druk})$$

waar a en b afhanklik is van die waarde van die drukgradiënt waarvoor die kontoerkromme geld.

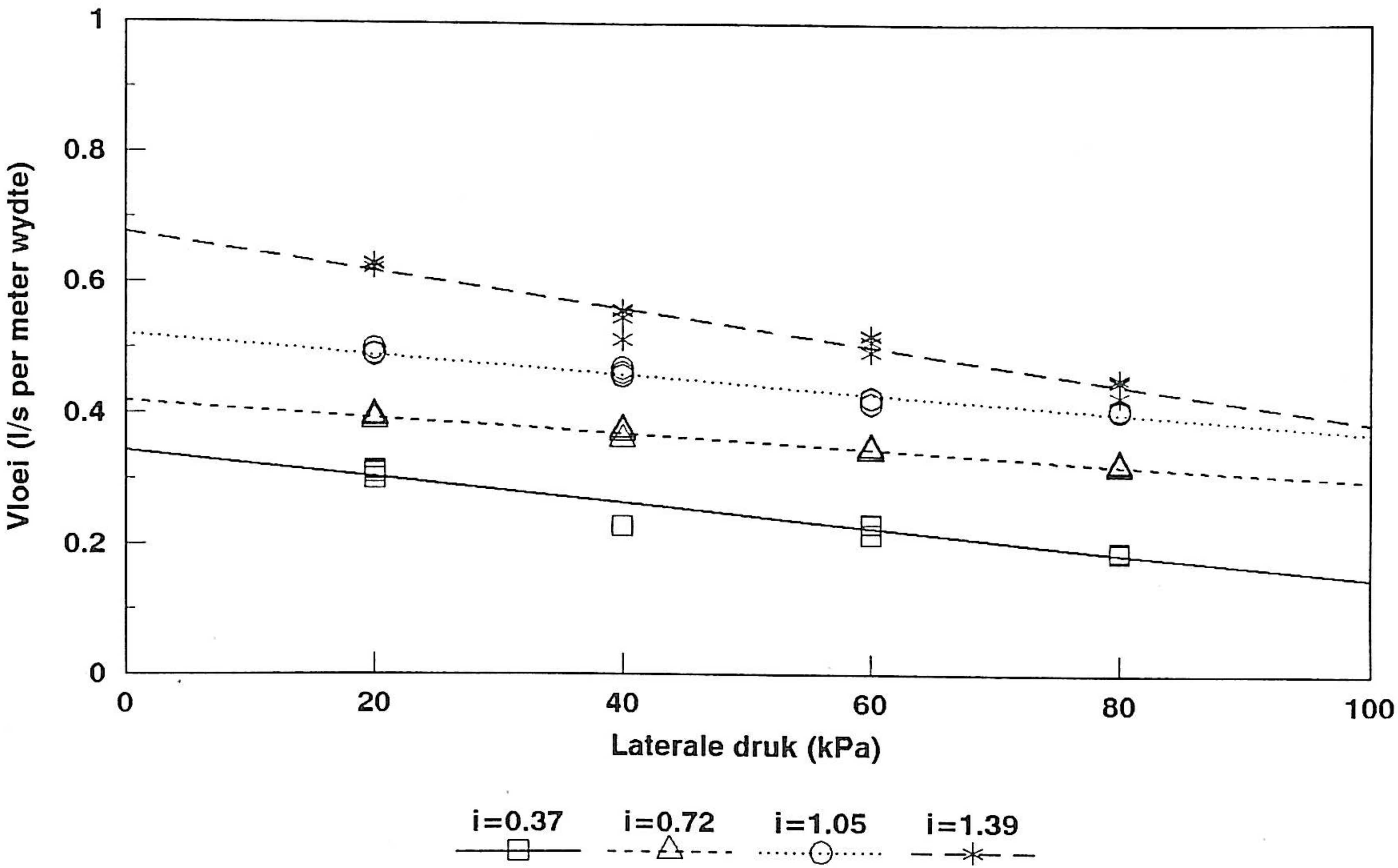
Tabel 4 toon die regressie modelle vir beide tipes kontoerkrommes vir die 5 mm Geonet findrein met sandsak ondersteuning.



Figuur 15: Drukgradiënt oor 5 mm Geonet findrein met sandsak ondersteuning.



Figuur 16: Vloei deur 5 mm Geonet findrein met sandsak ondersteuning.



Figuur 17: Vloei deur 5 mm Geonet findrein met sandsak ondersteuning.

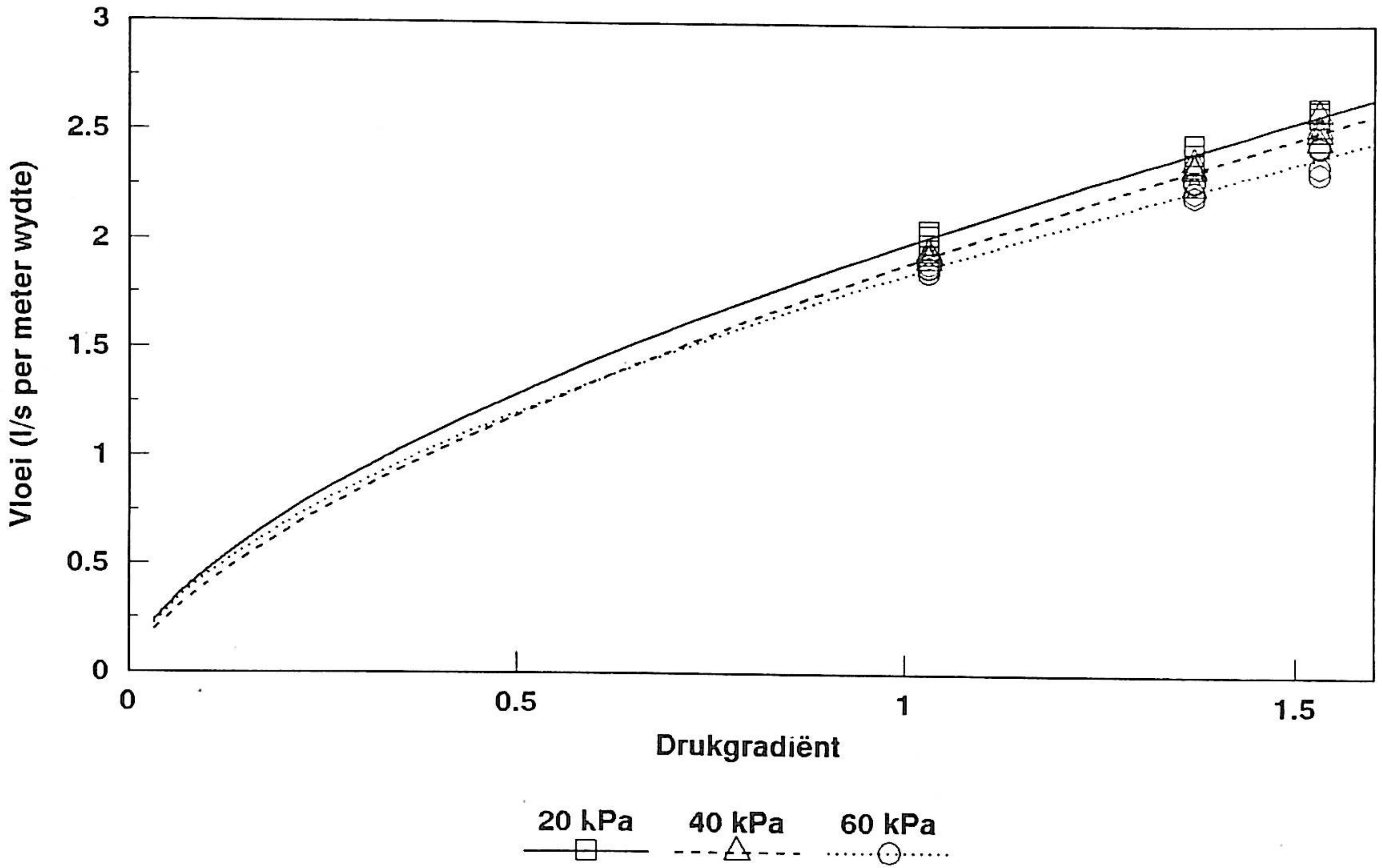
Laterale druk kontoerkrommes				Drukgradiënt kontoerkrommes			
Vloeitempo = a*(Drukgradiënt)^b				Vloeitempo = a + b*(laterale druk)			
Laterale druk (kPa)	R^2	b	a	Druk gradiënt	R^2	b	a
20	0,961	0,520	0,498	0,37	0,943	-0,00195	0,3422
40	0,986	0,631	0,447	0,72	0,979	-0,00121	0,4184
60	0,994	0,607	0,416	1,05	0,955	-0,00150	0,5205
80	0,978	0,668	0,379	1,39	0,943	-0,00289	0,6765

Tabel 4: *Regressie konstantes vir laterale druk kontoerkrommes en drukgradiënt kontoerkrommes vir die 5 mm Geonet findrein met sandsak ondersteuning.*

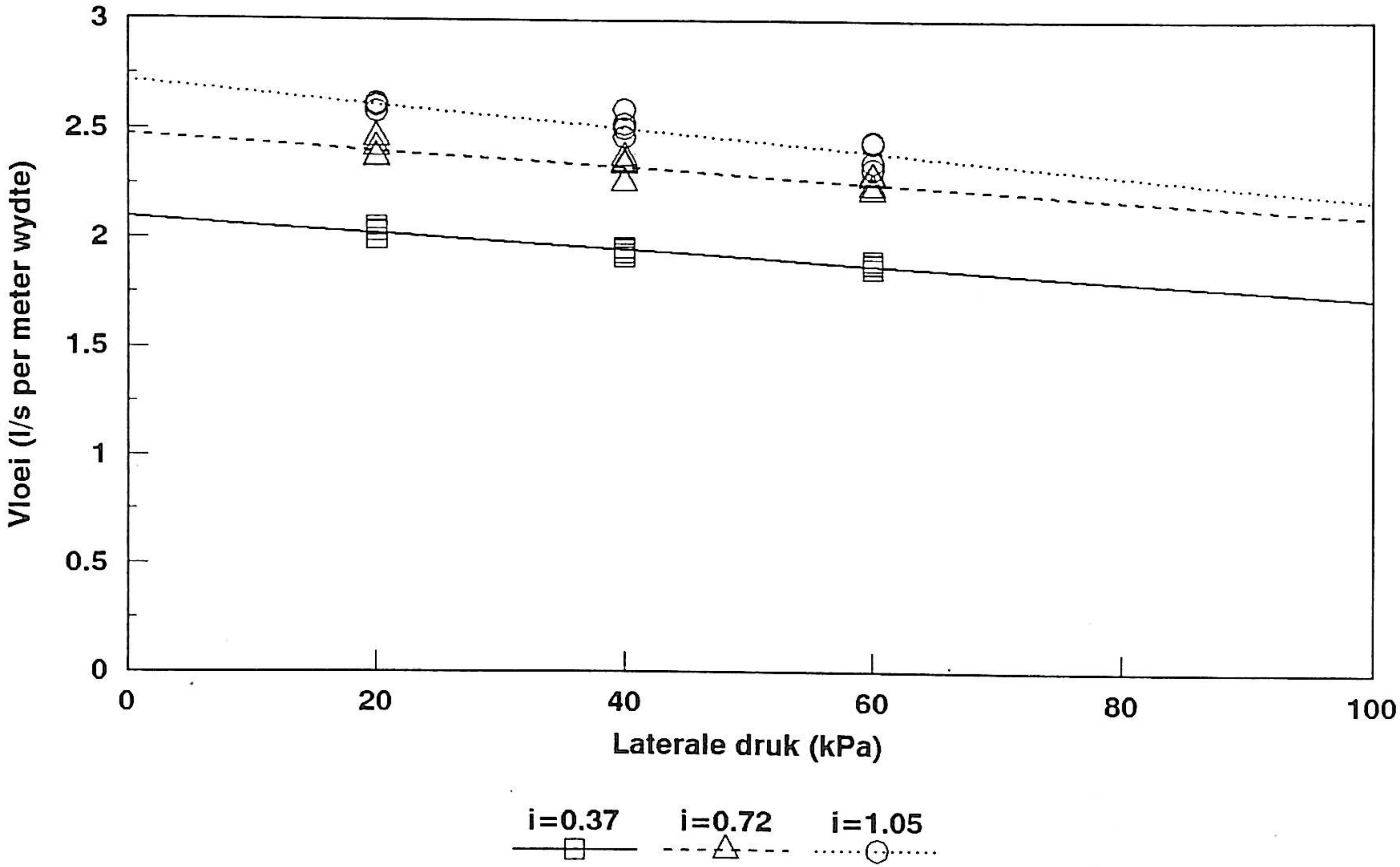
5.5 10 mm EIERKAS FINDREIN MET SANDSAK ONDERSTEUNING.

Figuur 18 toon die laterale druk kontoerkrommes as 'n funksie van drukgradiënt vir die 10 mm Eierkas findrein met sandsak ondersteuning. Die regressie vergelyking is weereens 'n magsfunksie.

Figuur 19 toon die drukgradiënt kontoerkrommes vir die vloeitempo funksie. Die regressie mode is weereens lineêr. Tabel 5 toon die regressie modelle vir beide die laterale druk en drukgradiënt kontoerkrommes.



Figuur 18: Vloei deur 10 mm Eierkas findrein met sandsak ondersteuning.



Figuur 19: Vloei deur 10 mm Eierkas findrein met sandsak ondersteuning.

Laterale druk kontoerkrommes				Drukgradiënt kontoerkrommes			
Vloeitempo = a*(Drukgradiënt)^b				Vloeitempo = a + b*(laterale druk)			
Laterale druk (kPa)	R^2	b	a	Druk gradiënt	R^2	b	a
20	0,985	0,621	1,987	0,37	0,907	-0,00372	2,096
40	0,979	0,663	1,896	0,72	0,726	-0,00371	2,475
60	0,974	0,611	1,848	1,05	0,779	-0,00545	2,723

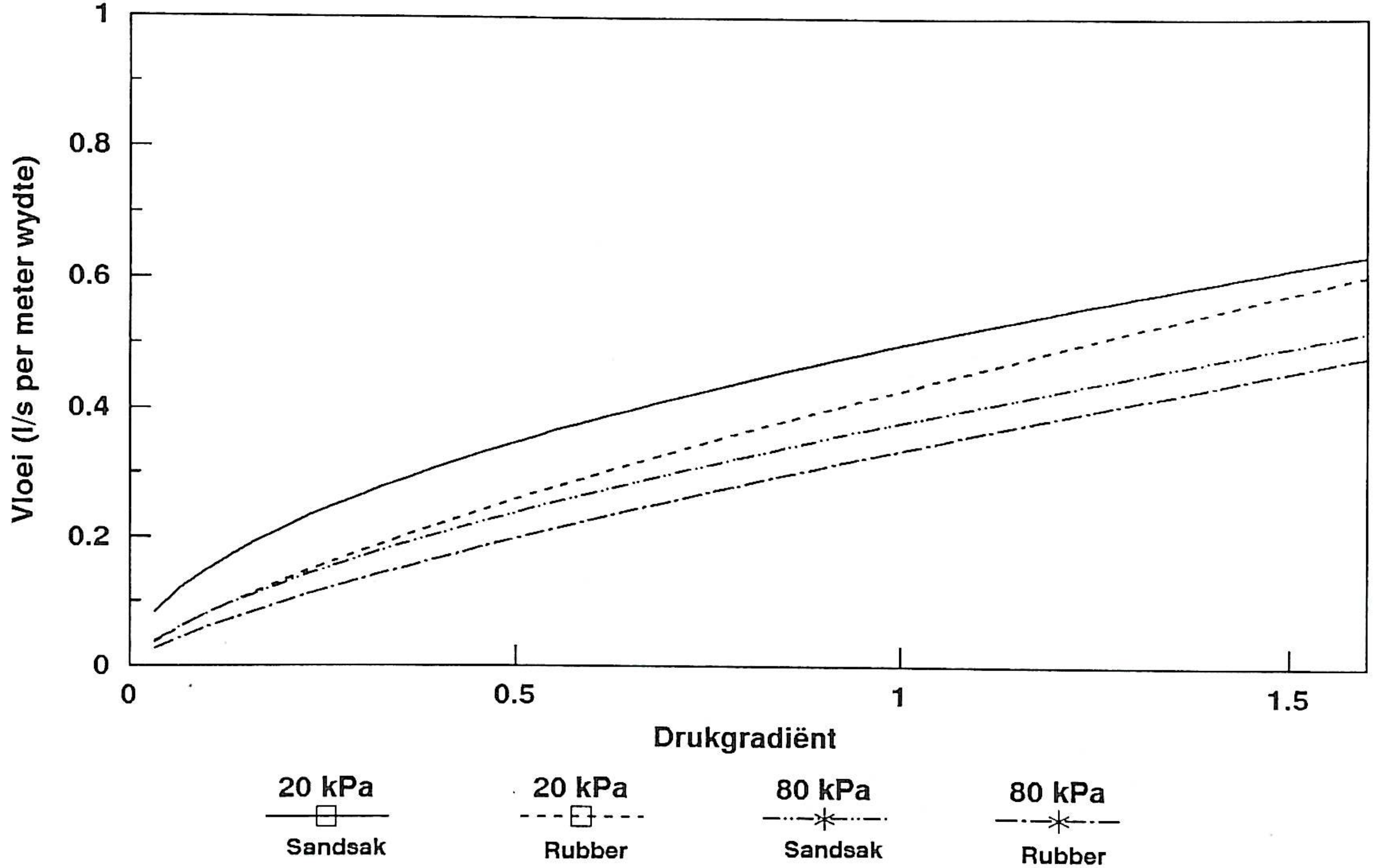
Tabel 5: *Regressie konstantes vir laterale druk kontoerkrommes en drukgradiënt kontoerkrommes vir die 10 mm Eierkas findrein met sandsak ondersteuning.*

6. VERGELYKING TUSSEN SANDSAK EN RUBBER ONDERSTEUNING.

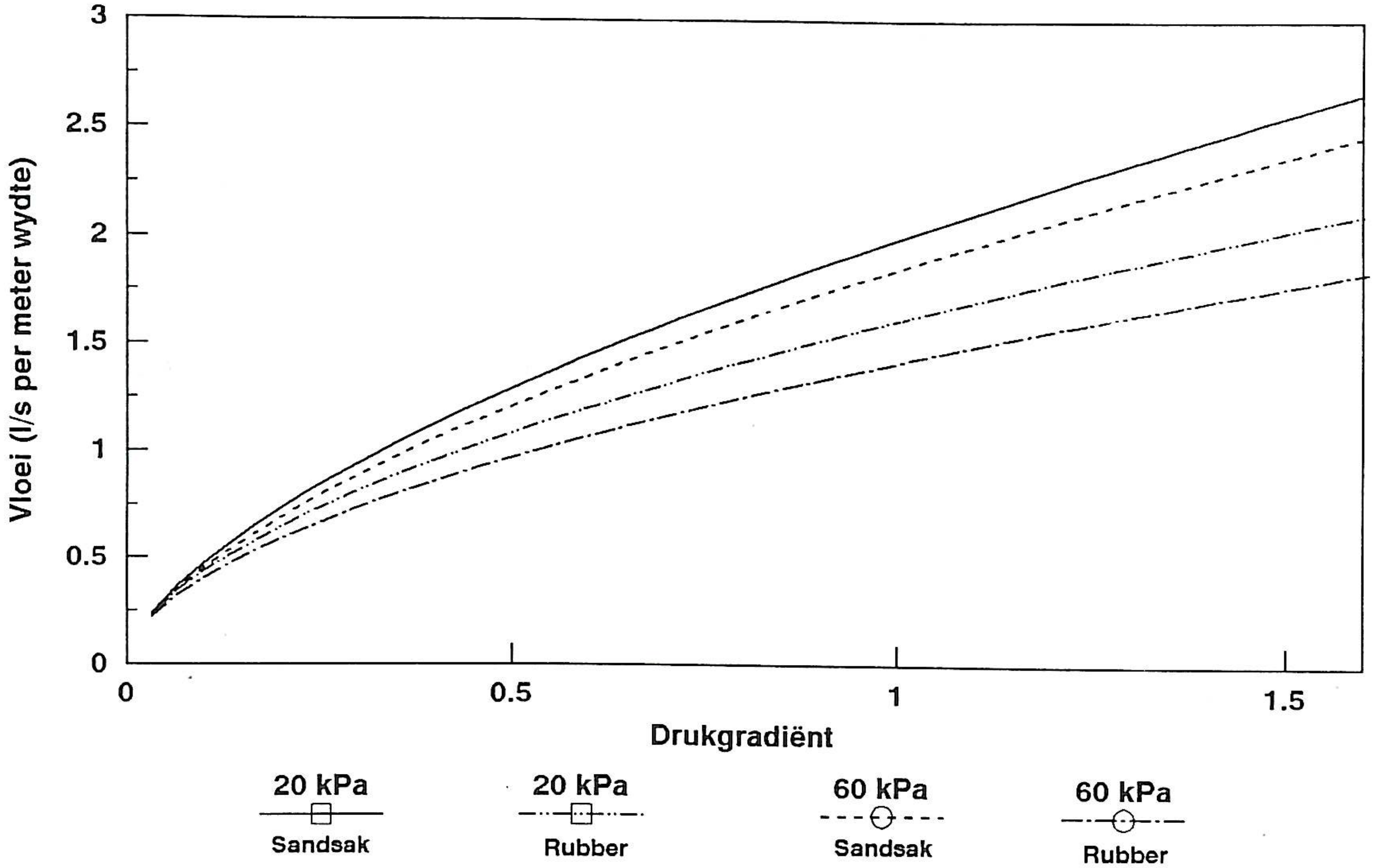
Twee findrein tipes is getoets met beide rubber en sandsak ondersteuning, naamlik die 5 mm Geonet findrein en die 10 mm Eierkas findrein. Die vergelykende resultate vir die twee opstellings is in Figure 20 en 21 getoon.

In beide gevalle is die findrein kapasiteit hoër vir die sandsak opstelling. Hierdie waarneming is moontlik deur twee redes veroorsaak. Eerstens is dit baie moeilik om die opstelling met die sandsak ondersteuning goed te seël en 'n klein persentasie vloei kan aan die kante van die opstelling deurvloei wat aanleiding sal gee tot hoër findrein kapasiteit. Tweedens is dit makliker om die sagter rubber ondersteuning met laterale druk in die openinge in die findrein kern in te druk wat 'n kleiner vloei-area en laer kapasiteit tot gevolg het.

Die onderskatting van findrein kapasiteit met die rubber ondersteuning opstelling is egter konserwatief.



Figuur 20: Vergelyking tussen vloei deur 5 mm Geonet findrein met rubber en sandsak ondersteuning.



Figuur 21: Vergelyking tussen vloei deur 10 mm Eierkas findrein met sandsak en rubber ondersteuning.

7. VERGELYKING VAN FINDREIN KAPASITEIT MET VERWAGDE VLOEITEMPOS.

Van der Merwe, (1988) het 'n volskaalse dreineringstelsel met 'n geotekstiel filtermedium gemonitor. Hierdie is slegs een voorbeeld van die moontlike praktiese vloeitempo wat uit 'n dreineringstelsel verwag kan word en is slegs ter illustrasie hier gebruik.

Die maksimum effektiewe deurlaatbaarheid was $10{,}6 \cdot 10^{-6}$ m/s per vierkante meter van die area wat onder die watertafel gelê het. Die filter is gebou tot op 'n diepte van 2 m met die gevolg dat die maksimum hoeveelheid water wat in die regop dreineringsgedeelte van 'n findrein sou afloop, 0,0212 l/s per m wydte sal wees. Hierdie waarde is baie laer as die kapasiteit teen 'n drukgradiënt van een vir al die findrein tipes wat ondersoek is.

8. GEVOLGTREKKINGS EN AANBEVELINGS.

i. Die maksimum laterale druk waarteen die onderskeie findreinmateriaal effektief kan opereer is as volg:

a. 100 kPa vir die 5 mm Geonet findrein.

b. 80 kPa vir die 10 mm Eierkas findrein.

c. 40 kPa vir die 37 mm Eierkas findrein.

Indien die grondparameters soos in afdeling 2 gebruik word, is die maksimum dieptes waarop die onderskeie findreinmateriale gebruik kan word:

d. 5 m vir die 5 mm Geonet findrein.

e. 4.57 m vir die 10 mm Eierkas findrein.

f. 2.29 m vir die 37 mm Eierkas findrein.

Hierdie waardes dien slegs ter illustrasie. Die korrekte benadering sal wees om die laterale druk koëffisiënt K_0, en die eenheidsgewig van die grond vir elke spesifieke ontwerp te bepaal en dan die veilige diepte te bereken vanaf die toelaatbare laterale druk waardes. Verder moet onthou word dat die terugvulling en kompaktering van materiaal gedurende die konstruksie van die dreineringstelsel hoër laterale spannings tot gevolg sal hê as die statiese laterale spanning.

ii. Vergelykende vloeitempo's tussen die verskillende tipes findreine by dieselfde toestande is as volg (Laterale druk = 20 kPa, Drukgradiënt = 0,3, rubber ondersteuning):

5 mm Geonet findrein: 0,2 l/s per m wydte

10 mm Eierkas findrein: 0,8 l/s per m wydte

37 mm Eierkas findrein: 9 l/s per m wydte

iii. Die vloeitempo blyk 'n magsfunksie van drukgradiënt te wees teen konstante laterale druk met die regressie konstantes afhanlik van die waarde van die laterale druk.

iv. Die vloeitempo is 'n lineêre funksie van laterale druk teen konstante drukgradiënt met die regressie konstantes afhanklik van die die waarde van die drukgradiënt.

v. Die kapasiteit van die findrein materiaal is hoog genoeg om nie die kapasiteit van die dreineringstelsel te beïnvloed nie. 'n Algemene analitiese metode moet egter ontwikkel word om vloeistelsels te analiseer. Weens die aard van die basiese vergelyking, 'n tweede orde differensiaal vergelyking ten opsigte van ruimtelike en tyd koördinate wat vloei in 'n poreuse medium beheer, behoort 'n numeriese metode soos eindige element metodes die aangewese oplossing te bied. Eindige element metodes kan ook ongestadigde (randvoorwaardes verander met tyd), onversadigde vogbeweging in 'n poreuse medium hanteer.

Met behulp van hierdie model sal voorspel kan word hoeveel vog uit die versadigde en onversadigde sone in 'n filtermedium sal inbeweeg. Op grond hiervan sal die nodige kapasiteit van die filtermedium dan bepaal kan word.

vi. Weens praktiese redes word aanbeveel dat die rubber ondersteunings in die vervolg gebruik word om die findrein kapasititeit te bepaal. Die rubber ondersteuning seël makliker in die apparaat en die findrein kapasiteit word konserwatief bereken met die rubber ondersteuning.

9. VERWYSINGS.

Freeme C R (1983). Evaluation of Pavement Behaviour for Major Rehabilitation of Roads. Tegniese verslag Rp/19/83, NIVPN, WNNR, Suid-Afrika, September 1983.

Van der Merwe C J (1988). Some Aspects of Road Subsurface Drainage in South Africa. Ph. D. Verhandeling, Departement van Siviele Ingenieurswese, Universiteit van Pretoria, 1988.

Murray H R (1984). Composite Geotextiles for Soil and Structure Drainage. "International Man-made Fibres Congress", Dornbirn, Oostenryk, September 1984.

Ven te Chow (1983). Open Channel Hydraulics. McGraw-Hill Inc. Japan, 1983.

Dempsey B J (1988). Core Flow Capacity Requirements of Geocomposite Fin Drain Materials Utilized in Pavement Subdrainage. "67th Annual Transportation Research Board Meeting", Washington D.C, Maart 1988.

APPENDIX F

DURABILITY OF GEOTEXTILES - A LITERATURE SURVEY

INTERIM REPORT IR 88/29/2

INTERIM REPORT **IR 88/29/2**

SOUTH AFRICAN ROADS BOARD
RESEARCH AND DEVELOPMENT ADVISORY COMMITTEE

Durability of geotextiles - a literature survey

RESEARCH DONE FOR
AND ON BEHALF OF THE
SOUTH AFRICAN ROADS BOARD BY
DIVISION OF ROADS AND TRANSPORT TECHNOLOGY
CSIR
P O BOX 395
PRETORIA
March 1991

TITEL/TITLE DURABILITY OF GEOTEXTILES - A LITERATURE SURVEY			
VERSLAG NR: **REPORT NO:** IR 88/29/2	**ISBN:**	**DATUM:** **DATE:** March 1991	**VERSLAGSTATUS:** **REPORT STATUS:** Interim Report
NOAK NR/RDAC NO: IR 88/29/2			
GEDOEN DEUR: **CARRIED OUT BY:** Division of Roads and Transport Technology, CSIR P O Box 395 Pretoria 0001		**OPDRAGGEWER:** **COMMISSIONED BY:** Chief Director : National Roads P O Box 415 PRETORIA 0001	
OUTEUR(S): **AUTHOR(S):** C J VAN DER MERWE		**NAVRAE:** **ENQUIRIES:** Navplan Private Bag X5 ALKANTRANT 0005	
SINOPSIS: Die verslag bespreek die bevindings van 'n literatuurstudie oor die duursaamheid van geotekstiele. Duursaamheid word gedefinieer en die mate waarin dit geotekstiele beïnvloed in hul onderskeie toepassings word bespreek. 'n Oorsig word gegee oor die eienskappe van polimere, geotekstiele en geosintetiese materiale aangesien dit 'n direkte invloed het op duursaamheid. Duursaamheid van geotekstiele word bespreek onder grond, sonlig, temperatuur en die bepaling van duursaamheid.		**SYNOPSIS:** The report describes the results of a literature survey on the durability of geotextiles. Durability is defined and its applicability to the various functions of geotextiles described. An overview is given of the properties of polymers, geosynthetics and geotextiles, as these directly affect durability. Durability is described under the headings of soil, sunlight, temperature and evaluation of durability.	
TREFWOORDE: Geotekstiele, ondergrondsedreinering, duursaamheid. **KEYWORDS:** Geotextiles, geosynthetics, subsurface drainage, durability.			
KOPIEREG **COPYRIGHT**			**VERSLAGKOSTE** **REPORT COST**

LIST OF CONTENTS

1. INTRODUCTION

Geotextiles and other related geosynthetic products are used fairly widely in road subsurface drainage applications in South Africa. Since they were introduced in the 1970s, their usage has increased steadily and, broadly speaking, so have the users' confidence in the performance of these materials. There is, however, still a degree of uncertainty about the durability of geotextiles over the long design lives that they are required to perform. This stems partly from the fact that these products have only been in use for about 20 years and very little information has been gathered and documented on their long-term field performance.

This input is intended to summarise, from the available literature, various aspects that relate to the durability of geotextiles and to indicate how these affect the South African road building industry. Some recommendations are also given on the evaluation of geotextile durability in South Africa.

2. GENERAL

2.1 THE NEED FOR DURABILITY

It is interesting that one of the main reasons for the development of the geotextile industry was in fact the discovery of polymers from which fibres were manufactured, of which the durability was far superior to that of natural fibres (Giroud, 1986). The synthetic fibres could withstand rot and chemical attack orders of magnitude better than their natural counterparts. Despite this, the durability of geotextiles still needs to be carefully evaluated for the following reasons (Leflaive, 1988):

- The duration of civil engineering works is much longer than those usually required for synthetic textiles;
- working conditions are particularly severe, especially in reinforced systems where stress is permanent;
- the soil environment, while being favourable under some aspects (protection from light, temperature stability) is different from one site to another, of complex nature and not always well-known.

The required life of a geotextile varies considerably, depending on the application and function thereof. A short life of two years could be expected in temporary works and, on the other extreme, in excess of 100 years in bridge abutments, retaining walls, etc. (Murray, 1988). Road applications, including subsurface drainage, fall somewhere in between, say 20 to 30 years.

Materials that are unaffected by their environment over periods of longer than 100 years are very scarce indeed. All the widely-used construction materials (steel, concrete, timber) can be susceptible to degradation over such long periods (John, 1987). The same can be said for road-building materials, but it is fair to say that geotextiles will generally be required to last for normal road design lives, i.e. 20 to 30 years.

2.2 DEFINITION

The general understanding of geotextile durability is "the ability of the geotextile to maintain its integrity, and a high degree of initial mechanical performance, over a long period of time when subjected to its operational environment" (Ingold, 1988).

A wider definition is (Gamski, 1988): "the term durability expresses the preservation in time of the characteristics or performance of a material, which are checked at the time of delivery or before placing and service".

These definitions refer to the geotextile alone, but it is important to realise that a successful design is dependent on a durable system, i.e. soil/geotextile system, not only a durable geotextile (Ingold, 1988).

Terminology referring to the life of a geotextile also needs to be defined: (Rigo, Ingold, 1988).

- "Service life" is the time during which the geotextile performs as it was designed to.

- "Design life" is the life of the larger system, i.e. the filter, the dam, the soil structure, the road, etc.

It should be the aim, therefore, that the service life of the geotextile be equal to or greater than the design life of the system.

The durability of a textile is determined by changes that occur due to one or more of the following (Gamski, 1988):

- spontaneously in the composition or in the internal structure of the material

- due to the action of placing the geotextile, e.g. mechanical loads, or

- due to the actions during service, e.g. loads, chemicals, temperature, etc.

These stages are interrelated and the changes due to each action add up. For example, a spontaneous reduction in strength due to sunlight exposure could result in rupture during placing, which in turn could adversely affect the geotextile's performance, e.g. its filtration characteristics.

The changes that occur in the geotextile over a period of time can be grouped under the following headings (Leflaive, 1988):

- Mechanical, which refers mainly to creep and behaviour under repeated loading.

- Hydraulic refers to the risk of clogging and the resultant change in permeability.

- Physical-chemical changes which refer to the aging of materials under the influences of chemicals, biological attack, sunlight, etc.

These mechanisms of change do not occur in isolation and interactions are numerous, e.g. stress may modify aging, aging may have an effect on creep, etc.

The hydraulic durability of geotextiles as determined by the soil/geotextile interaction and compatibility has been investigated fairly extensively and reported on in previous stages of this research project (Van der Merwe, 1988, 1989(a)). The main findings were that, in general, the long-term performance of geotextiles as filters was similar to that of crushed stone filters in that the permeability reduction was determined by a filter zone in the soil rather than in the filter itself. However, a few cases of geotextile clogging in the laboratory were observed and compatibility tests have been recommended to select geotextiles for filters (Van der Merwe, 1989(b)).

2.3 FUNCTIONS AND APPLICATIONS

Geotextiles perform different functions in a variety of civil engineering and other applications. The durability requirements differ for the various functions. The functions performed by geotextiles are (Murray et al., 1982; Giroud, 1986):

Filtration: The geotextile acts as a filter by allowing water to pass through and retaining the soil particles. This function is of particular importance in drainage applications.

Fluid transmission (drainage): The geotextile acts as a water carrier by allowing flow in the plane. This function is important in certain drainage applications, e.g. where the geotextile is wrapped around a pipe without a natural (granular) or synthetic (geonet) water-carrier.

Separation: The geotextile separates two different materials and ensures that one material does not contaminate the other. If a granular subbase is placed on a clayey subgrade, a geotextile can be used to ensure there is no movement of fine particles into the subbase material.

Protection: The geotextile protects a material from damage, e.g. where it is used for erosion control.

Reinforcement: The geotextile provides tensile strength to soil or asphalt, e.g. in embankments or in road structures. Giroud (1986) distinguishes between a tensioned membrane and a tensile member, indicating the two reinforcement functions. In an embankment, the geotextile is a tensile member, and in reducing large deflections in an unpaved road it is a tensioned membrane.

The applicability and relative importance of the various functions in some of the most important applications are shown in Table 1 (Murray et al., 1982) and Table 2 (Giroud, 1986).

The durability of geotextiles affects their properties in the long-term, which in turn affect their ability to perform the various functions. Table 3 (Murray et al., 1982) gives a summary of the importance of the various properties needed to perform the various functions. Durability as a property is shown as the last item on Table 3, but it should be remembered that durability also affects the other properties listed in the table.

The most important aspects of durability that affect the various functions are described below for each function (Ingold, 1988):

Filtration: The geotextile must maintain its physical integrity throughout its service life, but the greatest demand is during installation when the geotextile must resist the forces applied by handling, placing and filling. The polymers must be resistant to attack by any chemicals that are present in the liquids that pass through the filter. This is important in waste containment applications.

One of the biggest demands on durability occurs prior to installation when the geotextile is exposed to sunlight and therefore ultraviolet (UV) attack. The time of exposure should be kept to a minimum. This applies to all the different functions and applications.

If the geotextile is used under dynamic loading conditions such as in a pavement structure or under wave action, it must be able to resist abrasion and fatigue. In some long term applications a continuous normal stress may occur, e.g. behind a retaining wall. In such cases the geotextile must resist compressive creep.

The durability of a geotextile as a filter is very dependent on its ability to maintain the required level of permeability, which in turn is dependant on soil/geotextile compatibility and clogging, as described above in Section 2.2. The possibility of chemical clogging, e.g. by iron oxides, should

Table 1: Various applications and functions of geotextiles (Murray et al., 1982)

Application	Function			
	Separation	Filtration	Drainage in the plane	Reinforcement
Roads, railways and area sub-grade stabilisation	D	M	N.I.	M
Drainage	M	D	N.I.	N.I.
Wet-fill embankments	M	N.I.	D	M
Coastal and river protection	D	N.I.	N.I.	M
Land reclamation	M	N.I.	N.I.	M
Asphalt reinforcement	N.I.	N.I.	N.I.	D
Earth reinforcement	N.I.	N.I.	N.I.	D

D = Dominant function
M = Minor function
N.I. = Not important

Table 2. Relationships between applications and functions of geotextiles (Giroud, 1986)

Application category	Application area	Application type	Function: Fluid transmission	Function: Protection	Function: Separation	Function: Tensioned Membrane	Function: Tensile Member
Hydraulic applications	DRAINAGE	Geosynthetic drains without filter	X				
		Geosynthetic drains with filter (geocomposities)	X	X			
		Gravel drains, pipes		X			
	EROSION CONTROL	Bank revetment		X			
		Erosion mat			X		
		Silt fence, silt curtain		X			
Geosynthetic construction	CONTAINERS	Concrete forming, sandbags (hydraulic fill)		X			
		Gabions, sandbags					
	GEOMEMBRANE SUPPORT	Bridging					
		Cushion			X		
Geotechnical structures	ROADWAYS**	Asphalt overlay			X		
		Unpaved road (large deflection)				X	
		Base course (small deflection), ballast				X	X
	SOIL REINFORCEMENT	Reinforced walls, slopes and embankments					X

* For each application type, only the most important functions are indicated (e.g. the table does not show that the fluid transmission function may be involved when a geotextile is placed under flat concrete slabs in a bank revetment, or when a geotextile is used in a railroad track on a saturated subgrade, etc.)

** Here roadways are considered to include all types of traffic supporting structures such as roads, parking lots, staging areas, railroad tracks, etc.

Table 3. Properties important in various functions of geotextiles (Murray et al., 1982)

GROUP	PROPERTIES	FUNCTIONS			REMARKS
		SEPARATION	FILTRATION	REINFORCEMENT	
1. Constructional and basic physical properties	Material constituents	Not important	Not important	Minor - dominant	Constituent materials can be the dominant factor controlling stess-strain behaviour and surface friction
	Method of manufacture	Minor - dominant	Dominant	Dominant	This determines the structure which controls many properties, hence it must be carefully controlled
	Mass per unit area	Dominant	Dominant	Dominant	A good measure of the consistency of the product quality
	Pore sizes	Dominant	Dominant	Not important	Of particular importance for separation and filtration, and determines the capability of geotextiles to retain soil internally or at the soil-geotextile interface
	Open area	Dominant	Dominant	Not important	
	Thickness	Dominant	Dominant	Not important	Compression in soil and reorientation of fibres during straining changes thickness and structure, which modifies many of the geotextiles' properties
	Rigidity and structure	Dominant	Dominant	Dominant	

Table 3. Properties important in various functions of geotextiles (Murray et al., 1982), continued

GROUP	PROPERTIES	FUNCTIONS			REMARKS
		SEPARATION	FILTRATION	REINFORCEMENT	
2. Mechanical and hydraulic properties	Overall stress-strain behaviour	Minor	Minor	Dominant	Reinforcement function is dependent on all these properties. Also the entire stress-strain-time relationship and not just peak load at a standard strain rate is significant
	Overall extension to rupture	Minor	Minor	Dominant	
	Creep and stress relaxation	Not important	Not important	Dominant	
	Local burst strength	Dominant	Dominant	Minor - dominant	The ability to resist and redistribute local loads are two of the principal benefits of geotextiles and must be associated with burst and tear resistance
	Tear propagation	Dominant	Dominant	Dominant	
	Surface friction/adhesion	Not important	Not important	Dominant	There must be sufficient surface friction/adhesion to mobilise tensile strength
	Fluid capacity per unit area or unit thickness	Not important	Dominant	Not important	The capability of geotextiles to allow water to flow through them is fundamental to all filtration and drainage functions
3. Environmental properties	Durability Temperature stability Chemical stability UV light stability	Dominant	Dominant	Dominant	Geotextiles must be capable of maintaining their functional performances with time,and therefore must accommodate the environment conditions in which they are required to perform.

also be considered in some areas. Severe clogging can also be caused at the construction stage by clay or cement slurry, which could result in the total clogging of the geotextile.

Fluid transmission (drainage in the plane): Only very thick felts or mats are used for this function and the applications in South Africa are very limited. However, meshes, mats and formed sheets (e.g. "egg boxes") are used widely for this purpose in fin drains and are typically wrapped in geotextiles. In these applications the largest physical stresses can again be expected to occur during installation. For the long term effectiveness, the core material must be able to transmit the fluid in adequate volumes under the prevailing pressures, i.e. the transmissivity must be maintained. Furthermore, the polymers must resist any attack from chemicals that may be in the fluid.

Separation: Once again the biggest demands are usually before and after installation (UV, handling, placing, filling). In marine applications, the geotextiles may be subjected to salt water attack which can be devastating. Where the separation function is performed under dynamic loading conditions, e.g. traffic or wave action, resistance to abrasion and fatigue is required. It should be remembered that the separation function rarely applies alone and usually one or more of the other functions apply simultaneously.

Reinforcement: Reinforcement applications can vary from relatively short-term, low-risk applications, such as unpaved roads to long-term high risk applications, such as vertical walls and bridge abutments. On some applications, the geotextile is only needed to give stability over the short to medium term, e.g. in the case of embankments on soft ground that are designed to be stable on their own once the foundation soil has consolidated. The durability requirements of such geotextiles will be considerably different to those applied in a vertical reinforced soil wall with a design life of up to 120 years in some countries.

The basic criteria for a geotextile to be successful in a reinforcement function are the same, however, whether it be short or long term. These are:

- the reinforcement must not rupture, and
- the reinforcement must prevent excessive movement.

In road applications, the geotextile must also resist the effects of cyclic loading and, in asphalt reinforcement, high laying temperatures.

Geotextiles used for reinforcement are also subjected to the pre-installation and installation exposures and forces.

3. PROPERTIES OF POLYMERS, GEOSYNTHETICS AND GEOTEXTILES

The long-term behaviour of geotextiles is determined by the following categories of factors:

- Polymer properties, i.e. chemical composition
- Geotextile characteristics, i.e. method of manufacture
- External factors, i.e. stresses, soil, chemicals, sunlight, etc.

To understand the effect of the external factors, it is necessary to know the fundamental properties of geotextiles, i.e. their chemical and manufactured characteristics.

3.1 POLYMERS

The four polymer families most widely used throughout the world as raw materials for geotextiles are (John 1987):

- Polyester
- Polyamide
- Polypropylene
- Polyethylene

In South Africa polyester and polypropylene are used in most geotextiles.

The oldest of the polymers is polyethylene, which was discovered in 1931 by ICI and commercially produced in 1939. The polymer was first called polythene and is still known under that name by the general public. Polyethylene is more correct, however, and indicates that the members of this polymer family are all polymers of ethylene. Polyethylene has a very simple molecular structure, very good chemical resistance, excellent electrical insulation properties and low cost. However, compared to some of the more modern polymers, polyethylenes have relatively low strength and high creep characteristics. High density polyethylenes are a refinement of the original products and were developed in the 1950s.

The first polyamide was discovered in 1935. The best known of this family of polymers is nylon, of which there are several variations. The two main groups of polyamides are the aliphatic and the aromatic polyamides. The aromatic polyamides or aramids are more sophisticated and considerably more expensive than the aliphatics. The aliphatics, of which nylon is one, are generally the only polyamides used in geotextiles. These polymers are more expensive than polyethylene but have better strength and creep characteristics. Their resistance to UV light is low, however, and they are known to be susceptible to high temperature and moisture (CPA, 1990).

The next family of polymers to be developed was polyester in 1941. Well-known trade names such as Dacron and Terylene are all polyesters. Polyesters are widely used in South African geotextiles. They are the most expensive of the polymers that are in general use, but also have the highest strengths and low creep characteristics.

The youngest of the relevant polymer families is polypropylene, discovered in 1954, and marketed in 1957. Polypropylene and polyethylene both fall under the larger grouping of polyolefins, and there are some similarities between polypropylene and high density polyethylene. Polypropylene is very inert against chemicals and its low cost makes it one of the most widely-used polymers in the geotextile industry.

The main properties of the four groups of polymers are shown in Figure 1 (John, 1987). It should be noted that a large variety of polymers exist within each group or family, and those shown on Figure 1 merely represent those that are the most widely-used in geotextiles. Also, the properties are often affected by the environment and by the type and manufacture of geotextile, which may be of overriding importance relative to that of the polymers.

3.2 GEOSYNTHETICS

This report focuses mainly on geotextiles as such, but the principles of durability apply equally to geosynthetics in general and a brief description of the various products used in this field is useful.

Table 4 (Giroud, 1986) shows a proposed classification of all products used in civil engineering applications, not only synthetic but also conventional materials like steel and glass. Giroud proposed the term "geoproducts" for this purpose.

COMPARATIVE PROPERTIES \ POLYMER GROUP		polyester	polyamide	polypropylene	polyethylene
strength		⬤	●	•	•
elastic modulus		⬤	●	•	•
strain at failure		●	●	⬤	⬤
creep		•	●	⬤	⬤
unit wieght		⬤	●	•	•
cost		⬤	●	•	•
RESISTANCE TO :					
U.V. light	stabilised	⬤	●	•	⬤
	unstabilised	⬤	●	●	•
alkalis		•	⬤	⬤	⬤
fungus, vermin, insects		●	●	●	⬤
fuel		●	●	•	•
detergents		⬤	⬤	⬤	⬤

⬤ high

● medium

• low

FIGURE 1 COMPARATIVE PROPERTIES OF GENERAL POLYMER FAMILIES (John, 1987)

POLYMER.DRW/1991/01

TABLE 4

PROPOSED CLASSIFICATION OF GEOPRODUCTS (Giroud, 1986)

<table>
<tr><th>SCALE</th><th>DIMENSIONS</th><th colspan="2">TYPES</th><th>MATERIALS</th></tr>
<tr><td rowspan="2">Micro-geoproducts</td><td>Uni-dimensional</td><td colspan="2">Fibres, filaments, yarns</td><td rowspan="3">Synthetics
Steel
Glass</td></tr>
<tr><td>Bi-dimensional</td><td colspan="2">Microgrids</td></tr>
<tr><td rowspan="5">Macro-geoproducts</td><td rowspan="2">Uni-dimensional</td><td colspan="2">Cables
Strips</td></tr>
<tr><td colspan="2">Rods ("Nails")</td><td>Steel</td></tr>
<tr><td rowspan="3">Bi-dimensional</td><td rowspan="3">Geofabrics*</td><td>Grids (Geogrids)</td><td rowspan="2">Usually synthetics
("Geosynthetics")</td></tr>
<tr><td>Geotextiles
Webbings
Mats (Geomats)
Nets, waffles
Formed sheets</td></tr>
<tr><td>Geomembranes</td><td>Asphalt (usually with synthetic fabric)</td></tr>
</table>

* Here the term "geofabrics" is not limited to fabrics but is applicable to all planar structures made of polymers.

The following is a discussion of the various products:

Micro-geoproducts: The uni-dimensional products are usually the fibres or filaments from which geotextiles are made. These can be used for reinforcement by randomly placing them in the soil mass (e.g. De Garidel et al., 1986; Gray et al., 1986). The bi-dimensional products are small (± 10 x 10 mm) grids also used for reinforcement by random distribution in the soil mass (e.g. De Garidel et al., 1986).

Grids, geogrids, webbings: These are relatively stiff nets with symmetrically shaped openings used for reinforcement.

Geotextiles: Woven, nonwoven and knitted geotextiles are planar, fabric-like products used for reinforcement, separation, filtration and fluid transmission. The term "geofabrics" is often used synonymously with geotextiles, sometimes referring to nonwovens where "geotextiles" refers to wovens. The term "geofabrics" referring to all planar structures in Table 4 can therefore be misleading.

Mats, geomats, formed sheets: These are usually special application products used for surface protection, e.g. erosion control. They sometimes incorporate special features such as cells for establishing plant growth.

Nets, waffles: Nets and waffles usually have a definite third dimension and are typically used for fluid transmission, e.g. in drains as a substitute for granular filters and/or pipes. The term "geospacer" (Hoekstra et al., 1986) has been used for these products. Nets are made of randomly distributed fibres and differ from nonwoven geotextiles in that the openings between fibres are much larger, resulting in a very high permeability. The fibres are also thicker and stronger, resulting in a more rigid structure. Waffles usually have a larger third dimension than nets, resulting in a larger fluid capacity. They are typically made from planes or sheets of polymers, rather than fibres, which are cast into various forms. One example is the so-called "egg box" waffle.

Geomembranes: These are impermeable sheets used for linings and as separators. In an earlier classification (Rankilor, 1981) the term "membrane" was used to describe geotextiles, as well as impermeable sheets.

Geosynthetics: This term usually refers to all synthetic materials used in civil engineering applications.

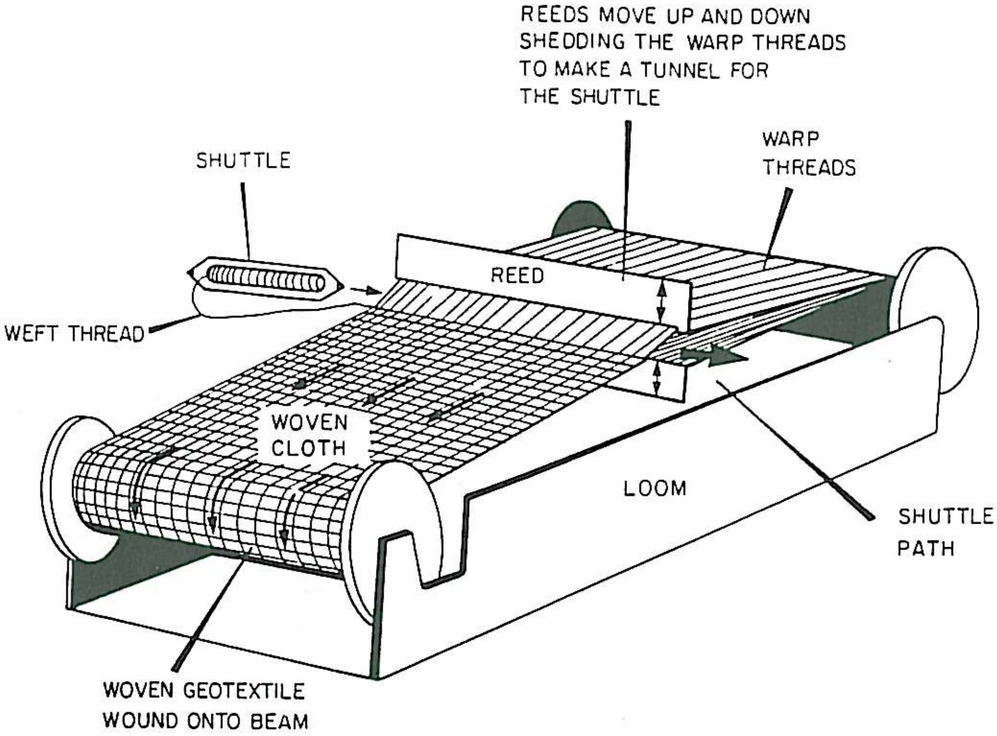
3.3 GEOTEXTILES

Geotextiles are the most widely-used geosynthetics. The following is a more detailed description and classification of these products. The four basic types of geotextile are:

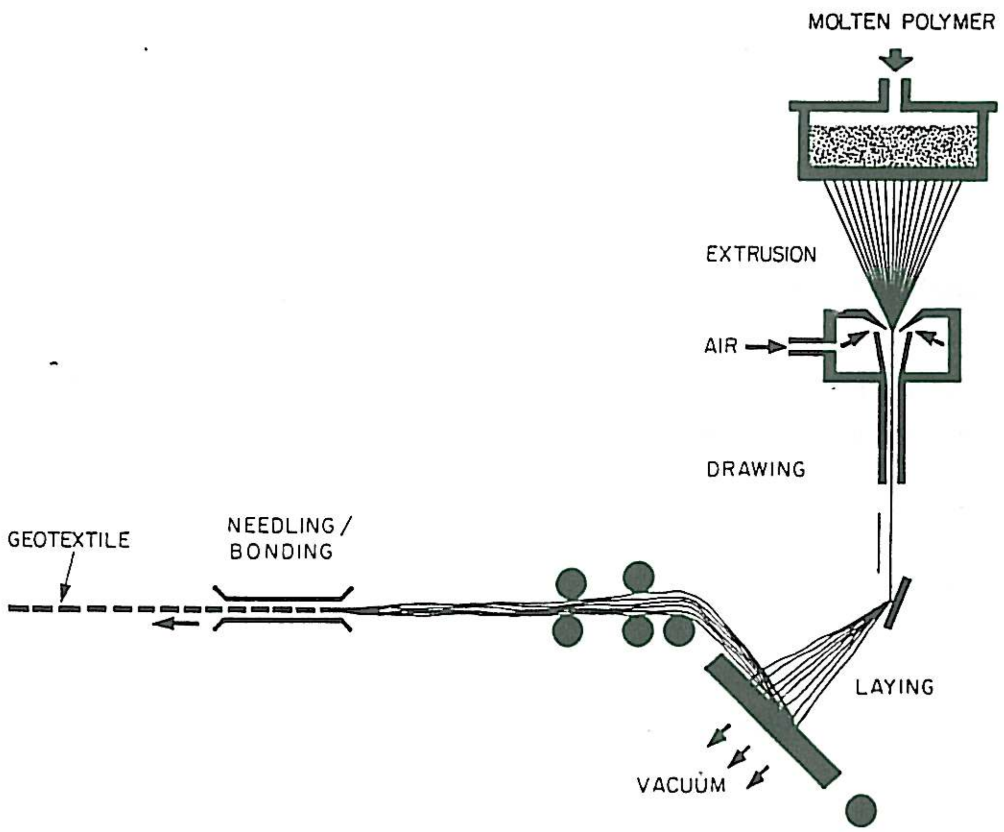
Wovens
Nonwovens and
Composites

3.3.1 Woven Geotextiles

The manufacture of woven geotextiles is based on traditional textile weaving methods. The geotextile is manufactured on a loom, as shown in Figure 2. Warp threads run along the length of the loom and are lifted and lowered by reeds. The weft threads are usually stronger and stiffer than the weft which have to be flexible enough to allow the weaving process.



a) Woven geotextiles



b) Continuous filament nonwoven geotextiles

FIGURE 2

SCHEMATIC ILLUSTRATION OF GEOTEXTILE MANUFACTURE

The end product obtained by weaving depends on the type of thread used, the method of weaving and after-treatment (stabilising) of the woven product. A classification of woven geotextiles is given in Figure 3(a), and discussed below.

(a) Thread type

Slit film (ribbon filaments, thin strips, tapes): Slit film geotextiles are woven from thin strips, usually less than 5 mm wide, which are split from an extended sheet of synthetic material. These geotextiles are the most common wovens and are usually less permeable than those woven with monofilaments.

Monofilament: Monofilament threads are single fibres of constant diameter. Geotextiles that are woven with these threads can have larger openings, resulting in higher permeability.

Multifilament: In multifilament woven geotextiles, the warp and/or weft threads consist of a number of single strands twisted together. These products are the most expensive and have the highest strength. Permeability is usually higher than that of slit film wovens.

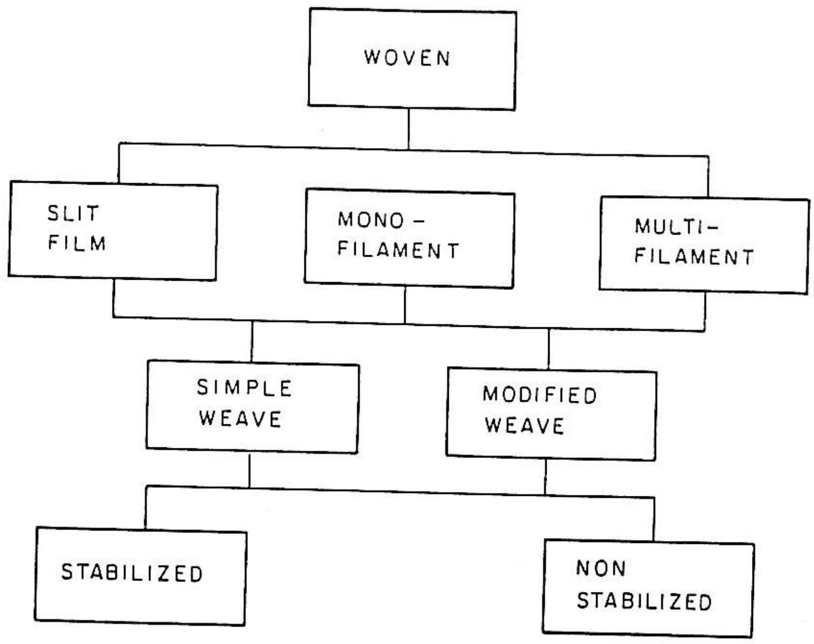
(b) Weave pattern

Simple weave is the basic weaving pattern where every warp thread intersects every weft thread, and both threads are similar.

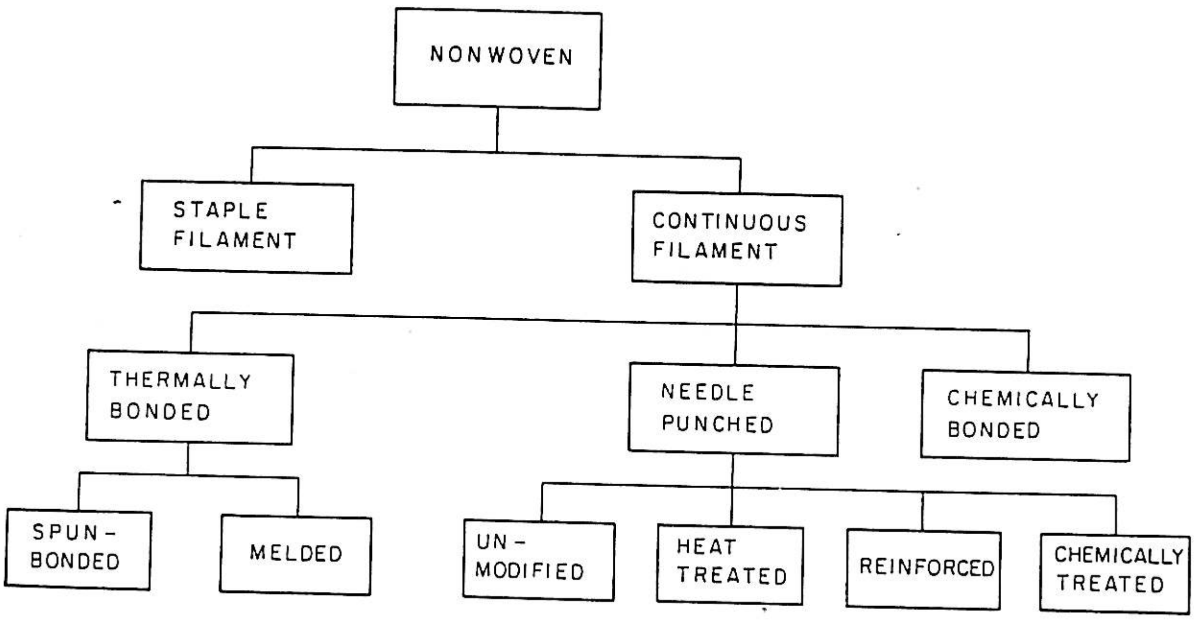
Modified weave is applied to alter the opening sizes, permeability and strength of the geotextiles. This can be done by varying the insertion patterns (e.g. a warp thread stepping over two weft threads at a time), using different types of thread for the warp and weft directions, or by simply reducing or increasing the number of warp or weft threads.

(c) Stabilizing

If a geotextile has an open weave with large openings, the threads can move relative to each other, thus distorting the openings. This is overcome by stabilising the geotextile either by heat or by chemical coating.



a) Woven geotextiles



b) Nonwoven geotextiles

FIGURE 3

CLASSIFICATION OF GEOTEXTILES

3.3.2 Nonwoven geotextiles

A classification of nonwoven geotextiles is shown on Figure 3(b). The first subdivision is staple filament and continuous filament geotextiles.

(a) Staple filament (Drylaid)

The term staple filament refers to relatively short (50 to 100 mm) polymer filaments or fibres which are bound together by needling, and secured by chemical bonding.

(b) Continuous filament (molten process)

Figure 2(b) shows the manufacturing process of continuous filament nonwovens. Continuous filaments are extruded from the molten polymer. The filaments or fibres are then laid in a random manner on a moving belt prior to a bonding process. Three different processes of bonding are used, namely, thermal bonding, mechanical bonding (needle punching) and chemical bonding.

Thermally bonded (heat bonded)

The laid fibres pass through a heating chamber and are heated to a point where the polymer is in a semi-liquid state. The fibres then bond to each other at the points of contact. When pressure is combined with heat, the term "calendering" is used. Depending on the types of fibre used, the geotextile is termed "spun bonded" or "melded".

Spun bonded: The process described above is applied to "monofilament" fibres, or fibres made of a single polymer.

Melded: The fibres are "heterofilaments", i.e. consists of two different polymers, one forming a core and the other an outer sheath. In the heat bonding process only the outer sheath reaches a semi-liquid state allowing the fibres to bond. The inner core remains in a solid state and its molecular structure is not altered.

Mechanically bonded (Needle punched)

The "web" of filaments is passed through a needling chamber. Needles with spikes perforate the web, thus bonding the fibres by interweaving (interlocking) them. The "web" then becomes a sheet, which is the needle-punched geotextile (unmodified). Further treatment is sometimes applied, e.g. reinforcement by inserting continuous threads in the geotextile, a certain amount of heat treatment, or chemical treatment.

Chemically bonded (resin dipped)

In this method of bonding, epoxy resins or similar chemicals bond the fibres together. The geotextile is normally dipped in the chemical and fans or vacuums are sometimes used to remove the excess bonding agent.

3.3.3 **Knitted geotextiles**

Knitted geotextiles consist of a single strand systematically intertwined with itself, and is manufactured with a knitting machine, instead of a weaving loom.

3.3.4 **Composite Geotextiles (multi-layered geotextiles)**

These are manufactured by combining layers of different material. In most cases, a nonwoven (usually staple filament) is needle punched onto a woven geotextile.

4. SOIL-INDUCED DEGRADATION

The changes in the characteristics of a geotextile in the soil may be either in its hydraulic or its mechanical properties. In most drainage applications, small changes in the in situ strength will not be of great concern, but changes in the hydraulic properties can have a significant effect on the performance of the system. In reinforcement applications, however, the mechanical properties must be maintained at the levels for which they were designed.

The degree of degradation that can be expected to occur will largely depend on whether the geotextile is used in commonly-encountered ordinary soil or in one of the less common aggressive soil types. Chemical pollution can also result in fairly aggressive soil-associated degradation of any building material, including geotextiles.

All indications from the literature are that the loss of strength of geotextiles in commonly encountered soils is not significant when these geotextiles are manufactured from the most widely-used polymers (John, 1987). In a study in France (Sotton et al., 1982), geotextile samples were recovered after they had been in use for more than 12 years. The loss in strength was typically about 5%. In a few cases, strength losses of up to 30% were recorded, but these were attributed to puncture damage during installation, rather than degradation.

In a study where geotextiles were recovered after 10 years of service in coastal revetments in Florida (Christopher, 1983), the strength loss was generally found to be between 4% and 11%. At two sites where abrasion of the geotextile had occurred and where the geotextile was partially exposed to sunlight, strength losses of up to 30% and 50%, respectively, were recorded.

In a study on geotextiles recovered from road drainage systems in Pennsylvania (Hoffman and Turgeon, 1983), the loss of tensile strength was generally less than 25% after six years. In some cases, strength losses of up to 45% were recorded, but these were also attributed to other factors, e.g. UV degradation.

Geotextile samples recovered from a soil reinforcement applications (Brady, 1986) showed no strength loss at all after seven years.

In a laboratory study in Romania (Ionescu, 1982), geotextiles were submerged in a variety of potentially hazardous media for 5-17 months in typical solutions that can be expected in soils. The media included iron bacteria, desulfovibrious, levansynthesizing bacteria, liquid mineral medium, sea water, compost and garden soil. No change in permeability or strength was obtained with any of the samples.

A very comprehensive study is currently being undertaken in the form of an international weathering programme (Rankilor, 1988, 1990). Various geotextiles and geogrids have been set up in four substantially different environments, namely, a desert site in New Mexico, a temperate site in the UK, a permafrost site in Sweden and a tropical site in Indonesia. Variables include type of geotextile, type of polymer, temperature, degree of sunlight exposure, as well as soil type for those samples buried beneath the surface. To date the study has resulted in some guidelines on sunlight exposure (see Section 5 below), but no conclusive results have been obtained on soil-induced degradation.

A few less common soils form particularly aggressive environments for geotextiles (Rankilor, 1981; John, 1987). Particular care should be taken when aggressive environments are expected and special tests should be carried out prior to design. This applies not only to geotextiles but to all construction materials. Problem soils include:

(i) Salt-affected soils
(ii) Highly organic soils
(iii) Sulphate soils
(iv) Ferruginous soils

Salt saturation of the soil can create a potential danger for some polymers but, in general, even in soils that are known to have high salinity, the salt concentration is usually low enough not to cause a threat. If any doubt exists, the soil salinity should be tested.

Highly organic soils have microbes that can grow on plastic materials and also organic acids and solvents. The degree to which these will be a threat will vary widely from one place to another, and as yet no incidents have been recorded of serious geotextile degradation in organic soils.

Acid sulphate soils occur naturally in flat swampy or marshy areas filled with peat and also in industrial areas from soil pollution. High levels of sulphate can be detrimental to synthetic materials and special tests should be carried out if problems are expected.

Ferruginous soils are probably the biggest threat and lead to clogging of geotextiles and pipes. Iron, either in the form of iron oxide or some other ferruginous compound, can accumulate on the geotextile, especially when affected by the presence of iron bacteria. Both hydraulic and mechanical properties of geotextiles are affected by this process.

Any significant pollution of the soil with chemicals that are not normally part of the soil environment can potentially affect geotextiles. The presence of high or low pH conditions or chemical solvents will affect most polymers but to different degrees (John, 1987). In the case of polyester, for example, hydrolysis occurs and is aggravated by high pH, high temperature and high humidity (Ingold et al., 1986). Prolonged exposure to uncured concrete can result in a significant strength loss in polyesters (Wewerka, 1982), but in the normal situation on a construction site, the concrete solidifies to a less aggressive form before damage can be caused.

5. SUNLIGHT DEGRADATION

Sunlight degradation is probably the biggest threat to geotextiles and exposure to sunlight should be kept to a minimum. Most polymers will degrade when exposed to sunlight owing to the presence of ultraviolet (UV) rays in sunlight. In general, the rate of degradation is dependent on the material thickness, and geotextiles which are made of thin fibres degrade fairly rapidly. Most geotextiles are, however, made with polymers, the UV resistance of which is increased considerably by the addition of stabilisers which either absorb the UV energy or prevent it from penetrating far into the polymer fibres (John, 1987; van Wijk et al., 1986).

The rate of degradation is determined by the intensity of UV radiation which is measured in kilo Langleys (kLy). One kLy is 1 kcal/cm^2 irradiated energy. UV radiation varies considerably across the world. In Northern and Central Europe, the radiation varies between 60 and 80 kLy per year, and in Southern African between 140 kLy on the east coast to over 180 kLy in the arid western parts of the subcontinent. In addition to the degree of radiation, the degradation of geotextiles is also affected by factors of temperature, humidity, rain, wind, dust and other atmospheric components, e.g. ozone, nitrous oxides, hydrocarbons, etc. (Raumann, 1982). The type of polymer, method of manufacture and stabilisers used also has a major effect on the rate of degradation. All these factors are interrelated, thus making the prediction of rate of degradation very difficult.

Sunlight exposure can be simulated in the laboratory by exposing geotextiles to light from a xenon tube. Various test procedures exist world-wide, e.g. Din 53387 (German) and ASTM G26-77 (American)(Van Wijk et al., 1986). Each procedure uses specific times and intensities of exposure, as well as wet and dry cycles. It is difficult, however, to relate degradation values obtained in the laboratory to those expected in the field, where large variations occur from one place to another and from one year to another.

In a series of outdoor exposure tests in the USA (Raumann, 1982), geotextile samples were exposed to normal sunlight in Florida, Arizona and North Carolina. In this study, polypropylene geotextiles performed worst, with the majority having a 50% strength loss in about 4 to 24 weeks, whereas polyester geotextiles showed the same loss after approximately one year. The tests showed, however, that the method of manufacture and stabilisers used had a considerable influence on the polypropylene geotextiles, and performance similar to that of polyester was in fact obtained on some polypropylene samples.

The study by Rankilor (1988, 1990), described in Section 4 above, has produced some useful results relating to UV exposure. In the study, degradation is not only defined as the decrease in maximum tensile strength as is usually the case, but is based on the change in the shapes of the stress/strain curves in the first 5-10% of strain. According to Rankilor, these change significantly with time, even if the maximum shear strength remains unchanged. Based on the results obtained to date, the recommended time limits for exposure of geotextiles to ultraviolet are as follows:

Climate	Temperate	Arctic	Desert	Tropical
Summer	8 weeks	4 weeks	2 weeks	1 week
Winter	12 weeks	6 weeks	2 weeks	1 week

These values are generalised for all geotextiles and polymers and are probably a conservative guideline.

6. TEMPERATURE RESISTANCE

In general, higher temperatures result in lower strength, creep and durability properties of polymers. It is unlikely, however, that geotextiles will be subjected to soil temperatures that significantly exceed those at which laboratory tests are normally performed (John, 1987).

High temperatures do occur, however, in paving applications where the application temperatures of premix laying and binder spraying are considerably higher than normal testing temperatures (20-25°C). For premix reinforcement applications, polypropylene grids have been found to be more temperature resistant than polyethylene grids (Porraz et al., 1986).

On the other end of the scale, geotextiles may be subjected to freezing and thawing in the cold regions of the world. A study by Allan et al. (1982) failed to detect any adverse effects on creep or tensile strength of both woven and nonwoven geotextiles under these conditions.

7. EVAULATING GEOTEXTILE DURABILITY

The properties that my be affected by degradation include mechanical properties, e.g. tensile strength, puncture resistance etc. , as well as hydraulic properties, mainly permeability. Deterioration of one of the properties will probably be an indication of deterioration of other properties, but not necessarily so and also not always to the same degree. The parameter most widely used for evaluating durability is the percentage reduction in maximum tensile strength. The study by Rankilor (1990) has shown that the maximum tensile strength may remain unchanged, whilst the shape of the stress/strain curve may change considerably. This may be important in some applications and less important in others. In drainage applications, the change in mechanical properties will in general be less important than the change in hydraulic properties. It is evident therefore that a meaningful evaluation of durability can only be made by considering those properties that directly affect the performance of the geotextile in the particular application.

In broad terms, durability can be evaluated in the laboratory or in the field or with a combination of both. Unfortunately, both laboratory and field methods have severe limitations.

Field evaluation is time-consuming since there is no acceleration involved. The performance of a particular geotextile in a certain application and in a specific environment after, say, 20 years can only be determined after 20 years, at which time the polymers and method of manufacture of geotextiles may have changed considerably. To monitor field performance over a short period and then extrapolate can also be misleading.

The factors that cause degradation can be duplicated and accelerated in the laboratory. However, it is virtually impossible to duplicate field conditions in the laboratory, especially since many factors usually affect the durability simultaneously, and not one or two in isolation as is usually the case in a laboratory. No meaningful assessment of the geotextile behaviour can be made without taking full account of its application and environment (Ingold, 1988) and laboratory methods can therefore at best be used for comparison of geotextiles and not for field simulation (Rigo, 1988).

To determine the expected long-term performance of a geotextile is therefore no easy task. Ingold (1988) suggests that the degree of testing should be determined by the consequences of failure. If the consequences are expected to be serious, it should be attempted to accurately predict the long-term performance by a variety of laboratory tests. Where the consequences of failure are slight, no laboratory testing should be done, and the designer should rely on past experience of other users of the same product.

8. CONCLUSIONS AND RECOMMENDATIONS

In general, the polymers that are used in geotextiles, particularly those widely used in southern Africa, have proved to be durable in commonly encountered environments. The consequences of failure in most road applications can probably be considered as relatively slight. These factors would suggest that actual laboratory durability testing is not really warranted, provided that geotextiles of well-known and proven polymers are used. Furthermore, if any aggressive environment is expected to occur, some laboratory testing may be considered.

Ultraviolet degradation is still a threat to polymers and geotextiles and sunlight exposure should be kept to a minimum.

Confidence in the durability of geotextiles can only be obtained and improved by monitoring of actual long-term performance in the field. This should be pursued by the road industry in general and specifically by road authorities.

9. REFERENCES

1. ALLEN, T, VINSOM, T S and BELL, J R. 1982. Tensile strength and creep behaviour of geotextiles in cold regions applications. Proc. 2nd Int. Conf. on Geotextiles, Vol III, Las Vegas.

2. BRADY, K. 1986. Reinforced and anchored earth. Ground Engineering, 19,7. (As referred to by John, 1987)

3. CAPE PROVINCIAL ADMINISTRATION. 1990. Standard project specifications. Roads and Traffic Branch, Cape Town.

4. CHRISTOPHER, B R. 1983. Evaluation of two geotextile installations in excess of a decade old. Transportation Research Record 916, TRB, Washington, DC.

5. DE GARIDEL, R and MOREL, G. 1986. New soil strengthening techniques by textile elements for low-volume roads. Proceedings of the Third International Conference on Geotextiles, Vol. I, Vienna, 153-158.

6. GAMSKI, K. 1988. Introduction. Rilem Seminar on durability of geotextiles. Chapman and Hall, 222p.

7. GIROUD, J P. 1986. From geotextiles to geosynthetics: a revolution in geotechnical engineering. Proceedings of the Third International Conference on Geotextiles, Vol I, Vienna, 1-18.

8. GRAY, D H and Al-REFEAI, T. 1986. Behaviour of fabric-versus fiber-reinforced sand. Journal of Geotechnical Engineering, Vol 112, No 8, 804-820.

9. HOEKSTRA, S E and BERKHOUT, H C. 1986. Geotextile/"geospacer" composites for environmental projects. Proceedings of the Third International Conference on Geotextiles, Vol II, Vienna, 343-347.RANKILOR, P R. 1981. Membranes in ground engineering. Chichester: Wiley, 377p.

10. HOFFMAN, G L and TURGEON, R. 1983. Long-term in situ properties of geotextiles. Transportation Research Record 916, TRB, Washington, DC.

11. IMGOLD, T S and MILLER, C S. 1986. Short, intermediate and long term stability of geotextile reinforced embankments over soft clays. Proceedings of the 3rd International Conference on Geotextiles. Vol II, Vienna, Austria.

12. IMGOLD, T S. 1988. Civil engineering requirements for long-term behaviour of geotextiles. Rilem Seminar on durability of geotextiles. Chapman and Hall, 222p.

13. IONESCU, A et al. 1982. Methods used for testing bio-colmatation and -degradation of geotextiles manufactured in Romania. Proceedings of the Second International Conference on Geotextiles, Vol II, Las Vegas.

14. JOHN, N W M. 1987. Geotextiles. Blackie: UK, Chapman and Hall: USA, 347 p.
VAN DER MERWE, C J. 1988. Some aspects of road subsurface drainage. PhD thesis, University of Pretoria, Pretoria.

15. LEFLAIVE, E. 1988. The different aspects of long term behaviour of geotextiles. Rilem Seminar on durability of geotextiles. Chapman and Hall, 222p.

16. MURRAY, R T. 1988. The users' point of view. Rilem Seminar on durability of geotextiles. Chapman and Hall, 222p.

17. MURRAY, R T and MCGOWN, A. 1982. The selection and testing procedures for the specification of geotextiles. Proceedings of the Second International Conference on Geotextiles, Vol II, Las Vegas, 291-2965.

18. PORRAZ, J L M and SOLAR, S A. 1986. The use of geogrid (polyethylene mesh) in the reinforcement of asphalt mix at the international airport of Mexico. Proc. of the 3rd Int. Conf. on Geotextiles, Vol I, Vienna, Austria.

19. RANKILOR, P R. 1988. Problems relating to light degradation and site testing of geotextiles - interim results of an international weathering programme. Rilem Seminar on durability of geotextiles. Chapman and Hall, 222p.

20. RANKILOR, P R. 1990. The weathering of fourteen different geotextiles in temperate, tropical, desert and permafrost conditions. Proceedings of the 4th International Conference on Geotextiles, Geomembranes and related Products. Vol 2, The Hague, Netherlands.

21. RAUMANN, G. 1982. Outdoor exposure tests of geotextiles. Second Int. Conf. on Geotextiles. Vol II, Las Vegas.

22. RIGO, J M. 1988. The researchers' point of view. Rilem Seminar on durability of geotextiles. Chapman and Hall, 222p.

23. SOTTON, M, LECLERCQ, B, PAUTE, J and FAYOUX, D. 1982. Some answers on the durability problem of geotextiles. Proceedings of the Second International Conference on Geotextiles, Vol II, Las Vegas. (In French)

24. VAN DER MERWE, C J. 1989(a). A laboratory evaluation of soil/geotextile compatibility for road subsurface drainage systems. DRTT Research Report DPVT93, CSIR, Pretoria.

25. VAN DER MERWE, C J. 1989(b). Recommended tests and tentative criteria for geotextiles in road subsurface drainage applications. DRTT Research Report DPVT 92, CSIR, Pretoria.

26. VAN WIJK, W and STOERZER, M. 1986. UV Stability of polypropylene. Proc. 3rd Int. Conf. on Geotextiles. Vol III, Vienna, Austria.

27. WEWERKA, M. 1982. Practical experience in the use of geotextiles. Proc. Symp. Recent Developments in Ground Improvement Techniques. Bangkok, 1982. (As referred to by John, 1987)

APPENDIX G

FIELD STUDY OF SUBSURFACE DRAINAGE SYSTEMS: SITE SELECTION AND OBJECTIVES

INTERIM REPORT IR 88/29/3

INTERIM REPORT **IR 88/29/3**

SOUTH AFRICAN ROADS BOARD
RESEARCH AND DEVELOPMENT ADVISORY COMMITTEE

Field study of subsurface drainage systems: site selection and objectives

RESEARCH DONE FOR
AND ON BEHALF OF THE
SOUTH AFRICAN ROADS BOARD BY
DIVISION OF ROADS AND TRANSPORT TECHNOLOGY
CSIR
P O BOX 395
PRETORIA
March 1991

TITEL/TITLE FIELD STUDY OF SUBSURFACE DRAINAGE SYSTEMS: SITE SELECTION AND OBJECTIVES

VERSLAG NR: **REPORT NO:** IR 88/29/3	**ISBN:**	**DATUM:** **DATE:** Maart 1991	**VERSLAGSTATUS:** **REPORT STATUS:** Interim Report

NOAK NR/RDAC NO: IR 88/29/1

GEDOEN DEUR: **CARRIED OUT BY:** Division of Roads and Transport Technology, CSIR P O Box 395 Pretoria 0001	**OPDRAGGEWER:** **COMMISSIONED BY:** Chief Director : National Roads P O Box 415 PRETORIA 0001
OUTEUR(S): **AUTHOR(S):** C J VAN DER MERWE	**NAVRAE:** **ENQUIRIES:** Navplan Private Bag X5 ALKANTRANT 0005

SINOPSIS:

Hierdie verslag beskryf die eerste fase van 'n veldstudie om die effektiwiteit van ondergrondse dreineringstelsels te moniteer.

Die doelstellings van die studie word uiteengesit en die verskeie aspekte rakande ontwerp en materiale wat ondersoek sal word, word aangetoon. Die metodes van ondersoek wat beoog word, word ook beskryf.

Die terreine wat geïdentifiseer is word gelys en detail omtrent die dreingeringstelsels en toestand van die plaveisels word beskryf.

SYNOPSIS:

This report describes the first stage of a field study to monitor the efficacy of installed subsurface drainage systems.

The objectives are outlined and the various aspects of design and materials that are to be monitored are given. This includes the drains as well as the pavement behaviour. The method of monitoring that is envisaged is also described.

The sites that have been selected for monitoring are listed and details concerning the drainage system and pavement condition are given.

TREFWOORDE: **Ondergrondse dreinering, geotekstiele, findrein, plaveiselgedrag.**
KEYWORDS: Subsurface drainage, geotextiles, fin drains, pavement performance.

KOPIEREG **COPYRIGHT**	**VERSLAGKOSTE** **REPORT COST**

LIST OF CONTENTS

1. INTRODUCTION

The aim of the subsurface drainage project has been to provide standard tests, design criteria and specifications for geotextiles, drainage systems and related products and also to give general guidelines for subsurface drainage practice. The emphasis to date has mostly been on laboratory work. There is a great need, however, to verify laboratory research findings with actual field results and thereby calibrate the laboratory work and set realistic criteria. The laboratory work has also been limited to certain aspects of subsurface drainage, whereas field work can be used to evaluate a variety of situations.

As part of the research activities of the current year, a field study has been launched. This report describes the objectives of the field study, sites that have been selected for monitoring and, in broad terms, the method of investigating that is envisaged.

2. OBJECTIVES

The primary objective of the field investigation is to evaluate the efficacy of installed subsurface drainage systems in terms of the following:

- drain geometry, i.e. width, length, depth and placing/spacing of the side drains and herringbone drains
- geotextile durability
- soil/geotextile compatibility
- calibration of laboratory work
- applicability of generic geotextile types, e.g. woven versus nonwoven
- granular filters versus geotextiles
- fin drains versus conventional
- flow net and egg-box fin drain cores: capacity and efficacy
- pipes: pitch fibre versus synthetic, capacities
- drain outlet construction
- effects of blocking of system
- maintenance of subsurface drainage systems
- effects of installing side drains in all cuttings or only where problems are encountered.

In addition to the above, the investigation also provides an opportunity to monitor the deterioration of pavements that are subjected to excess moisture in those cases where subsurface drainage has not been installed or is ineffective and also where drainage is installed after the road condition has deteriorated considerably. Opportunities to monitor pavement behaviour under these conditions have been identified (see Section 3 below) and it is intended to include these in the study. Pavement performance under wet conditions has been studied in Heavy Vehicle Simulator (HVS) tests, but field verification by means of long term pavement performance monitoring has not been done. The evaluation of pavement performance where subsurface drainage has been installed also forms an integral part of the evaluation of the drainage system.

3. SITES SELECTED

The main objective for the past twelve months was to identify and select potential subsurface drainage sites that may be monitored over a number of years. It was attempted to select sites covering a wide range of systems and materials and also over a large geographical area.

Sites that have provisionally been selected are listed and described in Appendix A. A total of 26 sites in three provinces are included at this stage that cover a reasonably wide range of types of geotextiles, pipes, core material, as well as drain types, e.g. fin drains, conventional drains, side drains and herringbone drains. Included are also examples of successful drains and drains that have failed, as well as sections of road that apparently do not have subsurface drainage systems installed but that show signs of severe water-related failures. It was attempted throughout to select sites where there is definite evidence of excessive ground water, since these represent the most critical cases.

The various sites are described in Appendix A under the headings of site (i.e. designation), location, date installed, reason for installation, description of drainage system, road condition, distinct features of the particular system, dates visited with observations made and names/telephone numbers of contact persons. Figures are also given of site layouts in cases where they are deemed necessary. The dimensions of the drains, as well as the dates installed, are in many cases only approximate estimations and further refinements may be made in future.

4. MONITORING PROCEDURE

The monitoring of a subsurface drainage system can be done in varying degrees of detail. At one end of the spectrum one may choose to select a single drain and to monitor it intensively by instrumenting it with flow meters, psychrometers, etc. and monitoring every aspect of the drain in detail. The other extreme would be to select a wide variety of sites and to simply note any visual changes that may take place. While neither approach is wrong, there are advantages and disadvantages in both cases. A detailed assessment of one site would be expensive and only a few parameters could be evaluated, but the accuracy and degree of certainty would be favourable. An approach of less detail and more sites would result in a higher degree of uncertainty but more variables can be evaluated.

The current situation with subsurface drainage research is that fairly detailed laboratory research has been done that may be regarded as controlled but covering relatively few variables. A field investigation was also conducted at one site (Van der Merwe, 1988) that can also be described as detailed. The need at this stage is therefore seen to be more for an approach of less detail and more variables. The resulting uncertainties will be offset to some degree as a result of experience gained thus far. Furthermore, monitoring should not be restricted to visual observations only, but should be supplemented with some measurements, especially pavement condition surveys. It is envisaged that some of the sections that have been identified will be monitored with a fair amount of detail, whilst others will merely be observed visually.

The following is a list of measurements/observations that can be made to evaluate installed subsurface drainage systems in the field:

- visual determination of the existence of flow
- measurement of outflow
- visual assessment of the condition of the drainage system, e.g. condition of outlet, blockages, surface water, vegetation, etc.
- measurement of the ground water table at various positions
- visual assessment of the pavement condition
- instrument-based evaluation of pavement condition, including the following:
    - riding quality (e.g. LDI)
    - deflection and deflection bowl (e.g. IDM)
    - in depth deflections (MDD)
    - irregularities (also riding quality), using rod and level survey techniques
    - rut depth and profilometer measurements
    - DCP measurements
- discussions with persons involved, e.g. Regional Engineers and Road Superintendents.

5. CONCLUSIONS AND RECOMMENDATIONS

Sites have been identified and selected that cover a wide range of variables. These sites should all be monitored to some extent, even if only observed visually from time to time. The process of identifying and physically finding an installed system is often time-consuming and costly, and it would therefore be worthwhile to continue observing all those that have been "found". It is recommended that some sites be monitored with a reasonable amount of detail, though, as described in Section 4 above.

It is also recommended that further sites be included in the study, should appropriate ones be found. It would be extremely valuable if all new and recently installed sites are documented, marked and monitored, if only visually, on a network basis.

6. REFERENCES

VAN DER MERWE, C J, 1988. Some aspects of road subsurface drainage in South Africa. PhD thesis, University of Pretoria.

Appendix A

Site:
P5-3. Km 47,5.

Location:
TPA Road P5-3, km 47,5. Road between Ermelo and Bethal in South Eastern Transvaal, Route R29. First cutting west of Kaffirrivierspruit, approximately 10 km outside Ermelo.

Date installed:
May 1990.

Reason for installation
Local pavement failure was evident that was believed to be due to excess (subsurface) water. Regular maintenance was applied but deterioration continued. Installation of drain halted deterioration.

Drainage system:
Fin drain utilising egg-box type spacer and non-woven geotextile. See Figure A1.

Road condition:
Local failure evident on first visit. On second visit after 8 months, distress still evident but not worse. Distress in form of rutting, undulations, shear failures, poor riding quality.

Distinct features:
Egg-box type fin drain. Installed to halt pavement deterioration, evidently successful. Continuous flow of water from drain evident.

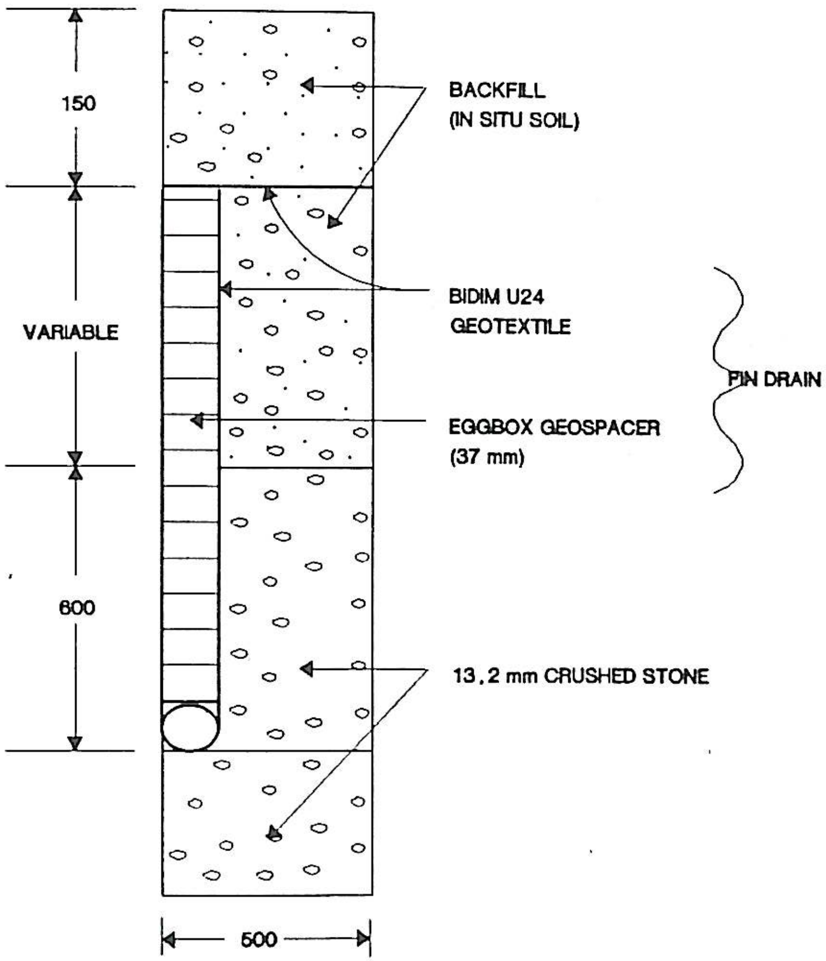
Date(s) visited and comments:

30 July, 1990:	With Ermelo Regional Engineer, Mr G Erlank. Localised pavement distress evident but has ceased to deteriorate, according to Mr Erlank. Water flowing from drain outlet.
8 March, 1991:	Pavement condition similar to that on previous visit. Flow from outlet also similar.

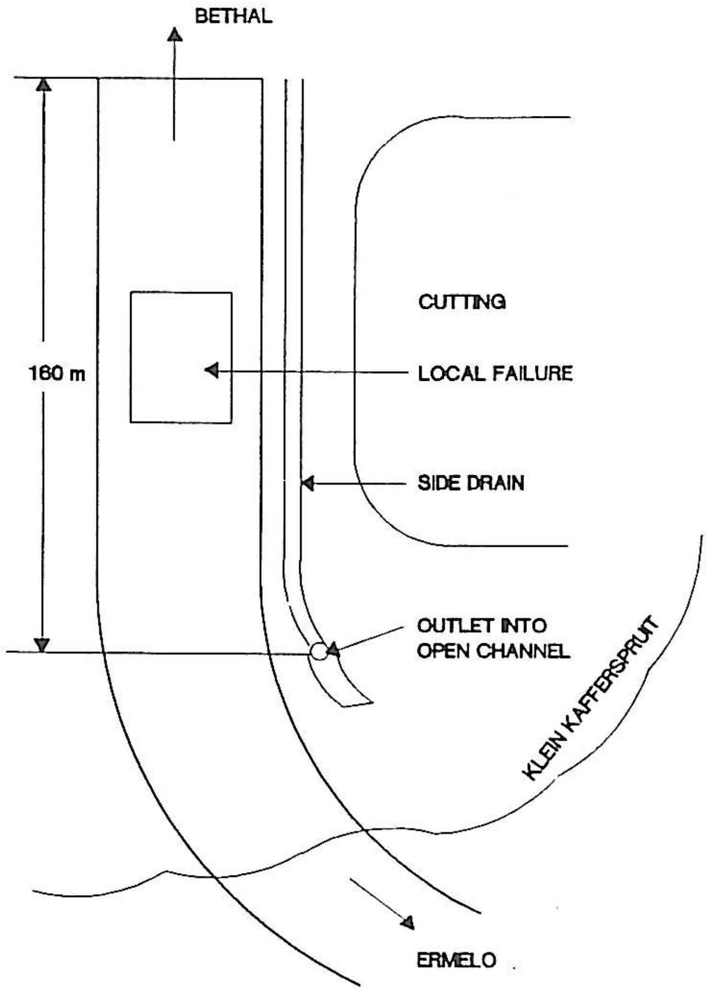
Contact person(s):
Mr Gerhard Erlank*, TPA Regional Engineer, Ermelo.
Mr Johan van der Merwe, TPA Road Superintendent, Ermelo.

Tel: TPA Ermelo: (01341) 2011.

ROADP5-3 DRW. 1991



(a) Typical section through side drain



(b) Site Layout

FIGURE A1 : P5–3, km 47,5

Site:
N4-7. Km 7,5.

Location:
National Road N4-7, km 7,5. TPA Road P154-7. Just outside Nelspruit on Komatipoort Road in Eastern Transvaal. First uphill after Rietspruit.

Date installed:
1988.

Reason for installation:
As part of major rehabilitation. Road was in very bad condition due to subsurface water resulting from localised springs and possibly seepage from higher ground.

Drainage system:
Side drain on southern side of road and herringbone drainage. Elements used in drains unknown, probably a variety. Some form of fin drain (Figure A2) possibly also used. Outflow of all drains are from one pipe which flows into the Rietspruit River. There is a very strong flow of water, apparently throughout the year.

Road condition:
Very good. The road was in a poor condition before installation of the drainage system. Drainage system has operated successfully for 3 years.

Distinct features:
Very high, continuous flow of water. Successful drainage system. Details of system uncertain but can probably be obtained.

Date(s) visited and comments:

23 August 1990: With engineer from Lydenburg region (Mr Johan de Villiers) and Mr Paul Sanders, DRTT. Road in good condition and outflow very strong.

Contact person(s):

Mr Johan de Villiers*, TPA : Lydenburg (now with Peri-Urban).
Mr Tommy du Plessis,

Telephone: TPA Lydenburg (01323) 2211
TPA Nelspruit (01311) 23211.

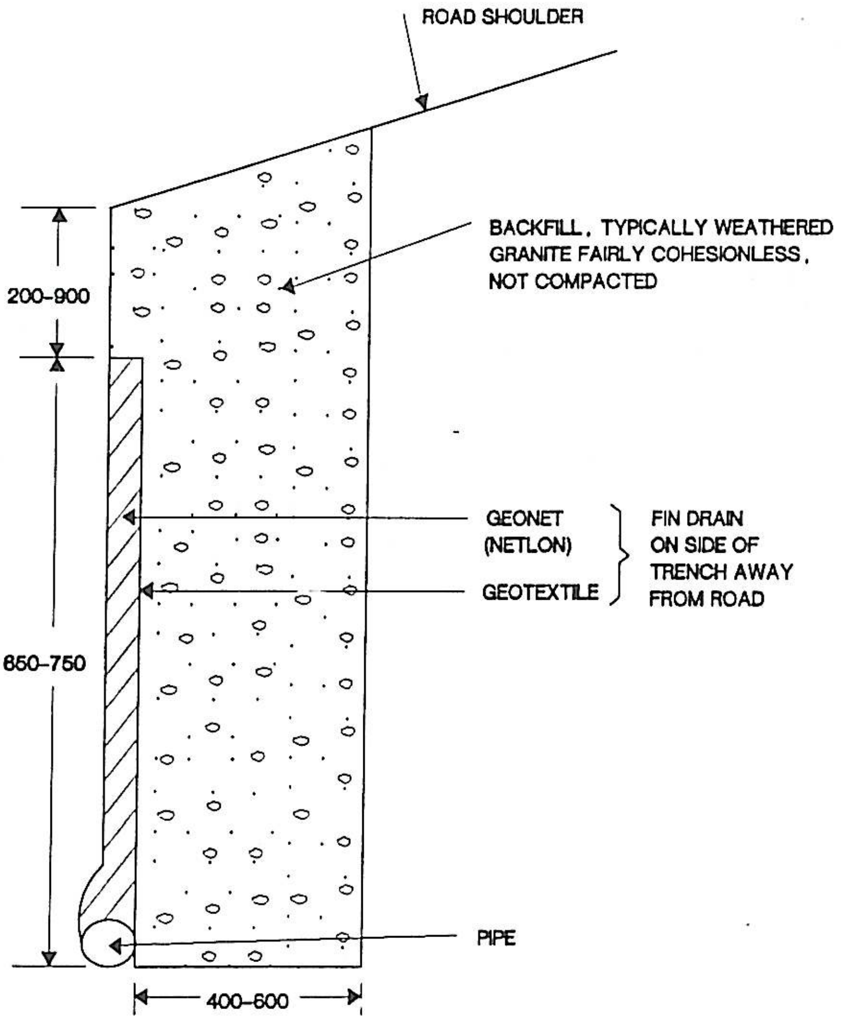


FIGURE A2 : TYPICAL SIDE DRAIN : N4-7, ROAD 1657, ROAD 725

ROAD725 DRW. 1991

Site:
N4-7. Km 13,5.

Location:
National Road N4-7, km 13,5 TPA Road P154-7. Outside Nelspruit on Komatipoort Road in Eastern Transvaal. Outflow at culvert S1678. Just before turn-off to Hotel Mabalel.

Date installed:
1988.

Reason for installation:
As part of major rehabilitation. In most cuttings in this region (TPA Lydenburg) subsurface drainage is installed because this is an excess moisture region.

Drainage system:
Fin drain as shown in Figure A2 and comprising Bidim U24 geotextile, Netlon geonet and Netlon geopipe.

Road condition:
Good.

Distinct features:
Fin drain. Netlon geopipe can be compared to pitch fibre pipes (km 13,840 and km 17, 140). Evaluation of pavement performance can be done against control section (km 15,140 - 5,880), where there is no drainage (none needed).

Date(s) visited and comments:

23 August 1990:	With Mr Johan de Villiers (Engineer, TPA Lydenburg) and Mr Paul Sanders (DRTT). Road in good condition. Flow from system combines with flow from surface drain, probably from a local spring, fountain or irrigation. Flows into culvert.

Contact person(s):
Mr Johan de Villiers*, TPA Lydenburg.

Telephone: TPA Lydenburg (01323) 2211.

Site:
N4-7. Km 13,840.

Location:
National Road N4-7, km 13,840 TPA Road P154-7. Outside Nelspruit on Komatipoort Road in Eastern Transvaal. Outflow at culvert S1679.

Date installed:
1988.

Reason for installation:
As part of major rehabilitation. In most cuttings in this region (TPA Lydenburg) subsurface drainage is installed because this is an excess moisture region.

Drainage system:
Fin drain as shown in Figure A2 and comprising Bidim U24 geotextile, Netlon geonet and Santar pitch fibre pipe.

Road condition:
Good.

Distinct features:
Fin drain. Pitch fibre pipe can be compared to Netlon geopipe (km 13,5). Evaluation of pavement performance can be done against control section (km 15,140 - 5,880), where there is no drainage (non needed).

Date(s) visited and comments:

23 August 1990:	With Mr Johan de Villiers (Engineer, TPA Lydenburg) and Mr Paul Sanders (DRTT). Road in good condition. Flow into culvert evident.

Contact person(s):
Mr Johan de Villiers*, TPA Lydenburg.

Telephone: TPA Lydenburg (01323) 2211.

Site:
N4-7. Km 17,140.

Location:
National Road N4-7, km 17,140 TPA Road P154-7. Outside Nelspruit on Komatipoort Road in Eastern Transvaal.

Date installed:
1988.

Reason for installation:
As part of major rehabilitation. In most cuttings in this region (TPA Lydenburg) subsurface drainage is installed because this is an excess moisture region.

Drainage system:
Fin drain as shown in Figure A2 and comprising Bidim U24 geotextile, Netlon geonet and Santar pipe.

Road condition:
Good.

Distinct features:
Fin drain. Pitch fibre pipe can be compared to Netlon geopipe (km 13,5). Evaluation of pavement performance can be done against control section (km 15,140 - 5,880), where there is no drainage (none needed).

Date(s) visited and comments:
23 August 1990: With Mr Johan de Villiers (Engineer, TPA Lydenburg) and Mr Paul Sanders (DRTT). Road in good condition. Strong flow evident.

Contact person(s):
Mr Johan de Villiers*, TPA Lydenburg.

Telephone: TPA Lydenburg (01323) 2211.

Site:
Road 1657.

Location:
TPA Road 1657, approximately 5 km from Road P80-1 (Route R570) at Letuba River. Nearest town is Hectorspruit in the Eastern Transvaal.

Date installed:
June 1990.

Reason for installation:
New construction, ground water evident.

Drainage system:
Fin drain, as shown on Figure A2, and comprising Bidim U24 geotextile, Netlon geospacer and Santar pitch fibre pipe.

Road condition:
New, good.

Distinct features:
Fin drain. New road and new drainage system. Performance of both can be monitored from the start of their respective lives.

Date(s) visited and comments:

23 August, 1990:	With Mr Johan de Villiers, TPA Lydenburg and Mr Paul Sanders, DRTT. No flow evident but there was flow during and after construction, according to Mr de Villiers. No pipe or outflow structure visible, only crushed stone. Wing walls still to be built.

Contact person(s):
Mr Johan de Villiers*, TPA Lydenburg.

Telephone: TPA Lydenburg (01323) 2211.

Site:
Road 725.

Location:
TPA Road 725, 160 m from road P17-7 (Route R40 between Nelspruit and Witrivier). Located just outside Nelspruit.

Date installed:
June 1990.

Reason for installation:
New construction, ground water evident.

Drainage system:
Fin drain: Industex S110 geotextile, Netlon flow net and Santar pitch fibre pipe (see Figure A2).

Road condition:
Under construction during first visit.

Distinct features:
Fin drain, woven geotextile, new construction which can be monitored from beginning.

Date(s) visited and comments:
23 August, 1990: With Mr Johan de Villiers, TPA Lydenburg, and Mr Paul Sanders, DRTT. No flow evident but there was flow soon after construction, according to Mr de Villiers.

Contact person(s):
Mr Johan de Villiers, TPA Lydenburg.

Telephone: TPA Lydenburg (01323) 2211.

Site:
Road 2629.

Location:
TPA Road 2629 - new road between Pietersburg and Seshego in Northern Transvaal. Dual road next to dam immediately after last street crossing in Pietersburg.

Date installed:
1989.

Reason for installation:
During construction, subsurface flow evident, possibly seepage from dam.

Drainage system:
Fin drain, Bidim. Layout as shown on Figure A3.

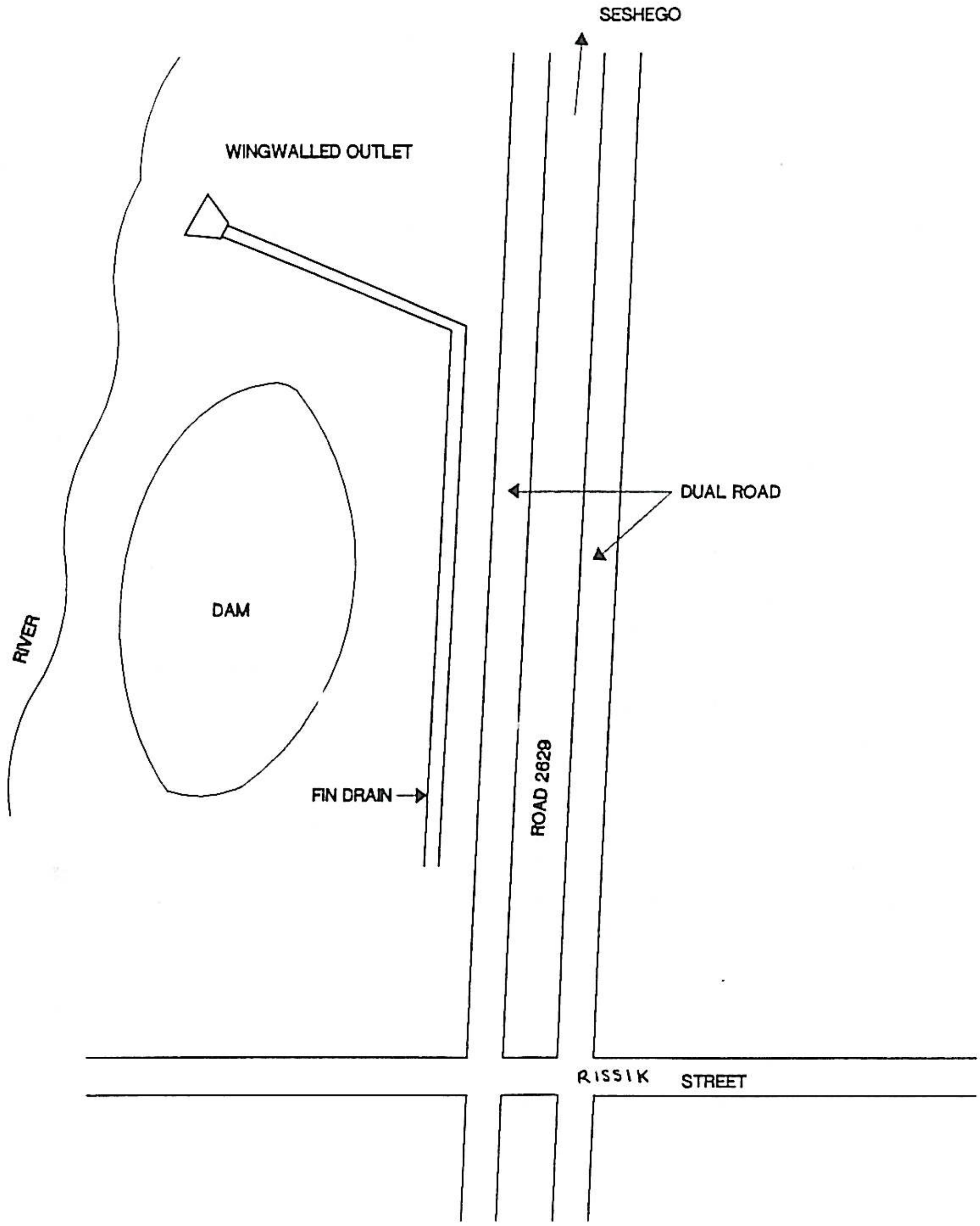
Road condition:
New, completed but not opened to traffic.

Distinct features:
Dual road, single drain can give indication of effectiveness of draw down of water table. Fin drain. New construction.

Date(s) visited and comments:
3 September, 1990: With Mr Theuns Kruger of TPA, Pietersburg. Flow evident, has been continuous, according to Mr Kruger.

Contact person(s):
Mr Theuns Kruger*, Chief Materials Technician, TPA, Pietersburg.

Telephone: TPA Pietersburg: (01521) 2061.



ROAD2629 DRW. 1991

FIGURE A3 : SITE LAYOUT ON ROAD 2629

Site:

Road 447.

Location:

Intersection of Road 447 and entrance road to Ga-Kgapane Hospital in Modjadji township, north-east of Tzaneen in Northern Transvaal. Road 447 branches off Road 43-2 (Route R36) between Duiwelskloof and Mooketsi.

Date installed:

Unknown, possibly during construction of road.

Reason for installation:

Excess ground water evident during construction.

Drainage system:

Fin drain with Bidim and Netlon flow net. Crushed stone (19 mm) was placed at the bottom of the trench. Layout is as shown on Figure A4. It is not known where the outlet of the system is situated. It is highly likely that the outlet is blocked - ponding is evident on the shoulder and even on the road surface. Fairly heavy growth of vegetation also indicates excess water.

Road condition:

Local failures, patching evident in outer wheelpath of right lane, i.e. wheelpath closest to water of the lane that carries heavy traffic. The lane adjacent to water problem does not carry heavy traffic and does not show any signs of distress (yet).

Distinct features:

Probably a blocked side drain which can be monitored for deterioration of pavement due to excess moisture and/or the halting of distress if the blocked drain is opened. Alternatively, an example of an ineffective drainage system.

Date(s) visited and comments:

3 September, 1990: With Messrs Pretorius and Bothma of TPA, Tzaneen. Evidence of blocked drain and pavement distress.

Contact person(s):

Mr Frikkie Richter*, Chief Roads Superintendent. Mr Hendrik Pretorius. Mr Louis Bothma.
All from TPA, Tzaneen. Telephone: (01521) 22033/4.

Mr Theuns Kruger, Chief Materials Technician, TPA, Pietersburg.

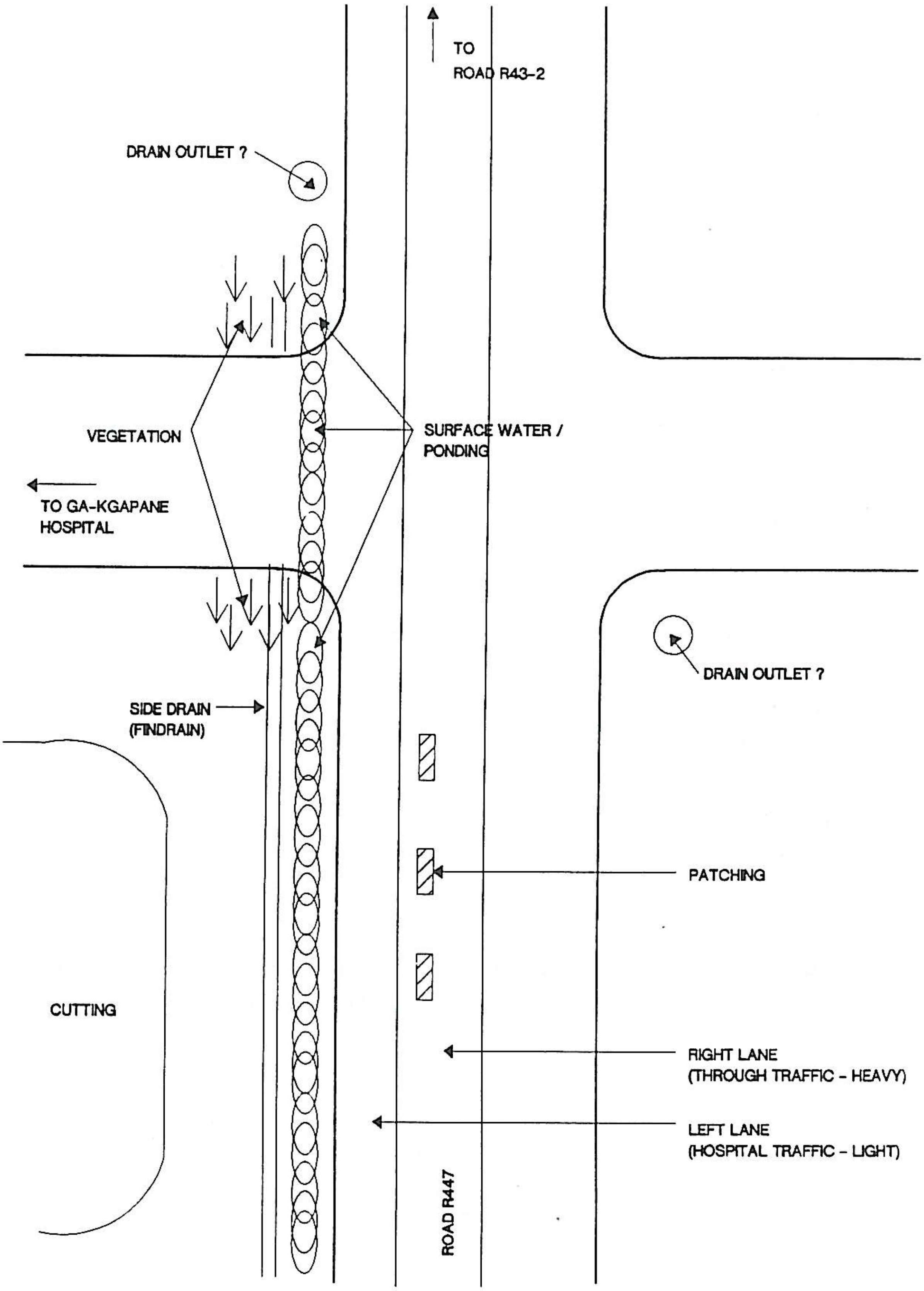


FIGURE A4 : SITE LAYOUT ON ROAD 447

ROAD 447 DRW. 1991

Site:

Road 1292.

Location:

TPA Road 1292 which branches off Road P43-3 (R71 between Tzaneen and Phalaborwa) towards Giyani in Northern Transvaal.

Date installed:

September/October, 1990.

Reason for installation:

New construction, ground water evident.

Drainage system:

Fin drain, Bidim, Netlon geonet, Santar pitch fibre pipe.

Road condition:

Under construction.

Distinct features:

Fin drain, high water level, observed during construction.

Date(s) visited and comments:

3 September, 1990: With Messrs Pretorius and Bothma of TPA Tzaneen. Road and drainage system under construction. Evidence of considerable amount of ground water.

Contact person(s):

Mr Frikkie Richter*, Chief Roads Superintendent. Mr Hendrik Pretorius. Mr Louis Bothma.

All from TPA, Tzaneen. Telephone: TPA Tzaneen (01521) 22033/4.

Mr Theuns Kruger, Chief Materials Technician, TPA, Pietersburg.

Site:

P52-3.

Location:

On TPA Road P52-3 at intersection (T-junction, southernmost) with Road 1651. Road P52-3 (Route 545) runs between Ogies and Bethal in South-Eastern Transvaal. Side drain is on the eastern side of the road, i.e. opposite the intersection.

Date installed:

1988/1989?

Reason for installation:

During rehabilitation - evidence of ground water.

Drainage system:

Conventional type side drain with Bidim U24 wrapped around 19 mm stone and Santar pitch fibre pipe. Outflow in culvert.

Road condition:

Good.

Distinct features:

Conventional drain. Uncertainty exists, however, about the extent of the flow.

Date(s) visited and comments:

24 September, 1990: With Mr van Vreden of TPA, Ermelo. No flow from system. Road in good condition.

Contact person(s):

Mr Gerhard Erlank, TPA Regional Engineer, Ermelo.

Mr Hennie van Vredan, TPA, Ermelo.

Mr Terrence Milne, Van Wyk & Louw, Consulting Engineers, Pretoria.

Telephone: TPA Ermelo: (01341) 2011.

Van Wyk & Louw: (012) 3253400.

Site:

P90-1.

Location:

On TPA Road P90-1 (Route R547 between Kinross and Kriel) in South-Eastern Transvaal. Located at first cutting north-east of intersection with TPA Road 637, approximately 1 km after intersection.

Date installed:

August, 1989.

Reason for installation:

During rehabilitation - major road distress and evidence of seepage from adjacent cutting, as well as springs in the road. Pavement materials were saturated, in the words of the RE: "A wet mess".

Drainage system:

Layout as shown on Figure A5(a). Conventional side drains with Bidim U24, 19 mm crushed stone, and pitch fibre pipe. Fin drain beneath road surface (not herringbone) consisting of Bidim U24 and Netlon geonet. Neat, effective wing walls at outlets.

Road condition:

Good. Rehabilitation consisted of removal of 950 mm of soaked material and reconstruction as shown on Figure A5(b).

Distinct features:

Drainage system is effectively controlling what was a very serious problem. Wing walls very neat and effective.

Date(s) visited and comments:

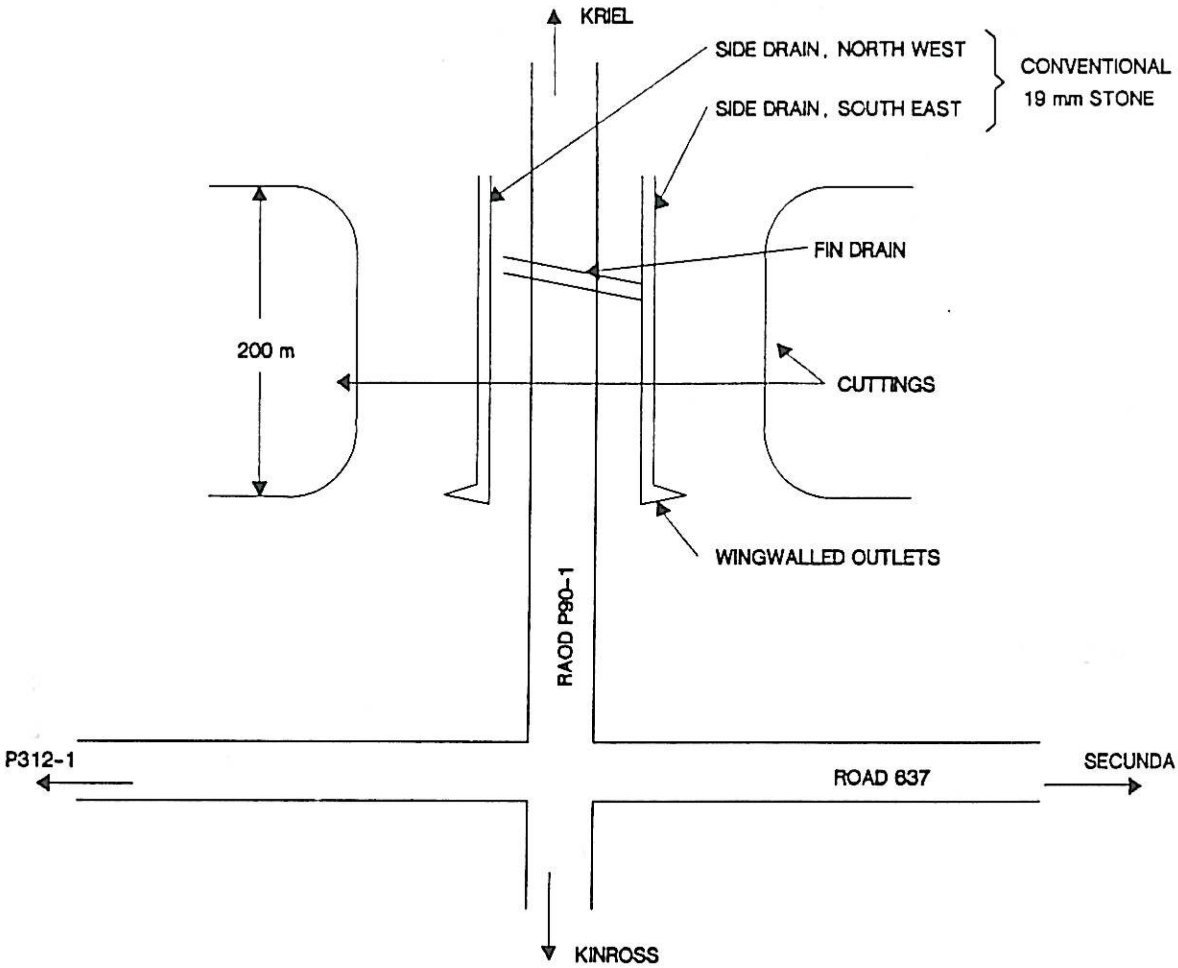
24 September, 1990: With Mr van Vreden of TPA, Ermelo. Outflow at south-eastern outlet but none at north-western outlet. Road in good condition.

Contact person(s):

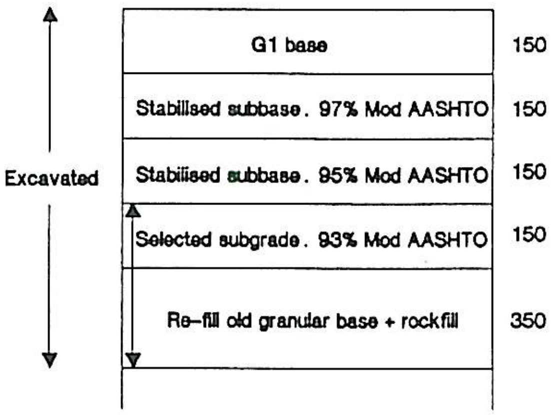
Mr Gerhard Erlank, TPA Regional Engineer, Ermelo. Tel: (01341) 2011

Mr Hennie van Vredan, TPA, Ermelo.

Mr Terrence Milne, Van Wyk & Louw, Consulting Engineers, Pretoria. Tel: (012) 325 3400.



(a) SITE LAYOUT



(b) PAVEMENT STRUCTURE AFTER REHABLITATION

FIGURE A5 : SITE ON ROAD P90-1

ROAD P90.DRW.1991

Site:

Road 637. Km 10,0.

Location:

TPA Road 637 at km 10,0, approximately 2 km south-east of intersection with Road P90-1 (Route R547 between Kinross and Kriel). Road 637 connects Road P132-1 with Secunda in the South-Eastern Transvaal.

Date installed:

1985.

Reason for installation:

Serious water-associated failures, uncertain whether part of major rehabilitation.

Drainage system:

Side drain and herringbone system as shown on Figure A6. Separate outlets for each, not wing walled but open and protected, approximately 500 m from road in river. All drains conventional with Bidim U24, 13,2 mm stone and Santar 150 mm pipe.

Road condition:

Reasonably good. There is no evidence of local failure in the vicinity of the subsurface drainage systems, but the whole road is showing signs of distress.

Distinct features:

Conventional drain. An example of both side drain and herringbone drain that carry continuous large volumes of water and have been successful for more than 5 years.

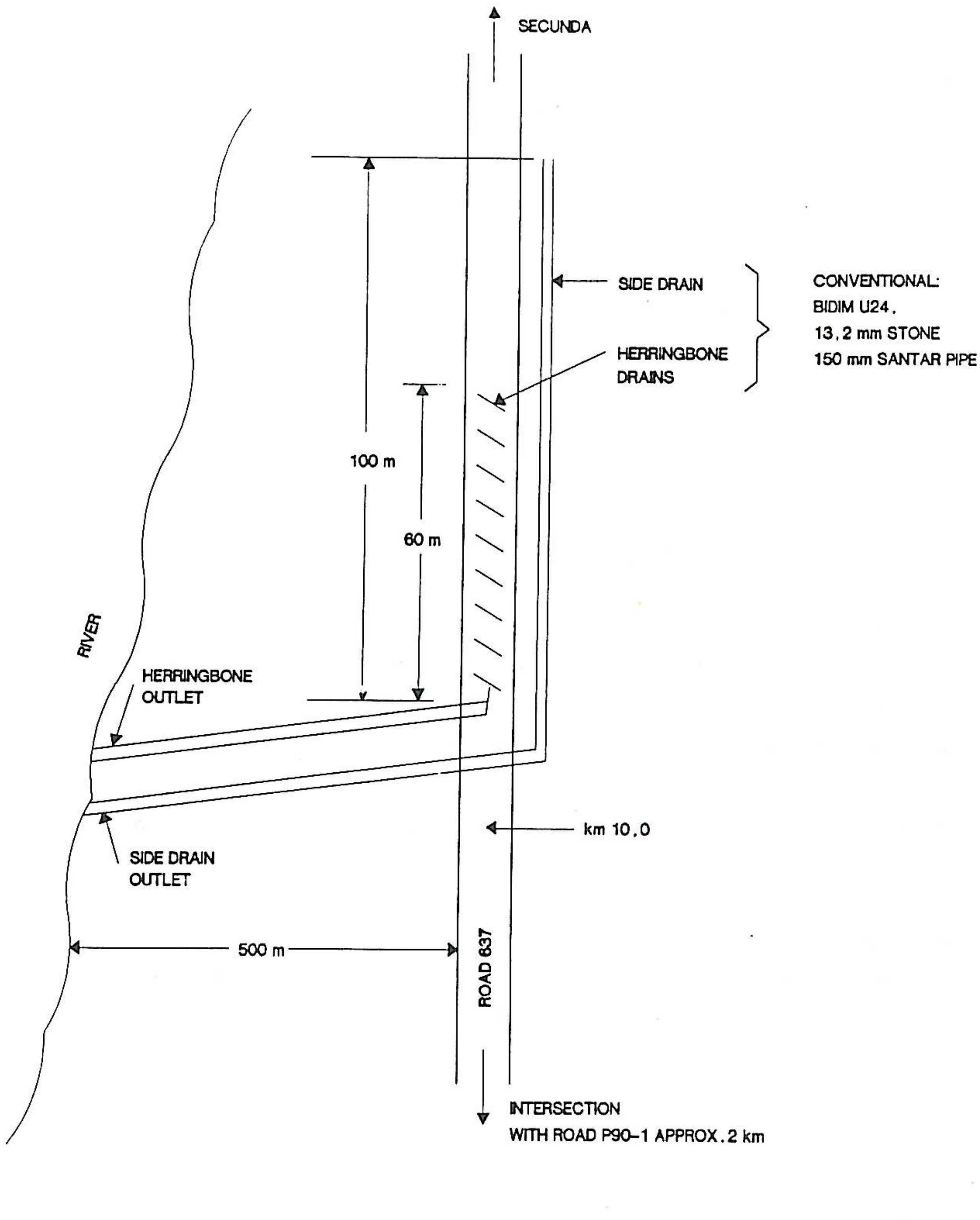
Date(s) visited and comments:

24 September, 1990: With Mr van Vreden of Ermelo TPA. Flow from both outlets, road condition good.

Contact person(s):

Mr Hennie van Vreden, TPA Ermelo.

Telephone: TPA Ermelo: (01341) 2011.



ROAD637 DRW. 1991

FIGURE A6 : SITE LAYOUT ON ROAD 637

Site:

P109-1. Km 19,8.

Location:

TPA Road P109-1. Freeway that runs parallel to P5-1, P5-2, P5-3 (Route R29) from Springs to Ermelo in the South Eastern Transvaal. Site is located at km 19,8, approximately 1 km west of intersection with Road 77, about halfway between Leandra and Springs.

Date installed:

1987.

Reason for installation:

Ground water encountered during construction, springs in the road.

Drainage system:

Conventional side drain with Bidim U24, 13,2 mm stone and Santar 150 mm pipe. Drain is on northern side of the road in first cutting west of large earth dam. The outlet is at the dam. No wing wall has been provided. The outlet has been blocked with soil but water is still seeping through and flowing towards the dam.

Road condition:

Good.

Distinct features:

An example of a poor outflow system that has become blocked and is a potential threat to the system and the road.

Date(s) visited and comments:

24 September, 1990: With Mr Van Vreden of TPA Ermelo. Outlet blocked but water still seeping out. Road in good condition.

Contact person(s):

Mr Hennie van Vreden, TPA Ermelo.

Site:

P81-1. Km ?

Location:

TPA Road P81-1, Route R540 between Lydenburg and Dullstroom, approximately 20 km from Lydenburg.

Date installed:

Unknown.

Reason for installation:

Unknown, possibly ground water encountered during construction.

Drainage system:

Unknown, probably side drain and herringbone drains.

Road condition:

Local failure. On first visit (May 1990) the road was in a bad condition with shear failures, potholes, patching, undulations, rutting and poor riding quality. According to Mr de Villiers of TPA Lydenburg, this was due to the outlet of the drainage system being blocked. The outlet was opened and cleared in August 1990. On the next visit there was still evidence of patching and poor riding quality, but no new failures. The road was subsequently repaired over a large area and on the last visit it was in a good condition, with only a different coloured surfacing marking the area of previous failure.

Distinct features:

An example of a drainage system that failed due to blocking of the outlet, which led to a serious pavement failure. Indications are that the problem has been solved by clearing the blocked outlet.

Date(s) visited and comments:

29 May, 1990:	Road in very bad condition (local failure). No drain outlet observed, apparently blocked.
1 November, 1990:	Flow from opened drain outlet. Road still in poor condition but no new failures.
10 January, 1991:	Road totally repaired, different coloured seal. Flow from outlet similar to November 1990, despite heavy rains. Outlet still cleared and effective.

Contact person(s):

Mr Johan de Villiers, TPA Lydenburg. Telephone: (01323) 2211.

Site:

P7-2. Km 77.

Location:

TPA Road P7-2 (Route R543) between Piet Retief and Wakkerstroom in the South-Eastern Transvaal. The site is located at km 77, approximately 8 km from Piet Retief, with the outlet at the intersection with TPA Road 2550 (turnoff to Lüneburg).

Date installed:

1986.

Reason for installation:

Water-associated failure in cutting.

Drainage system:

Conventional side drain with Bidim and stone. Uncertain whether Bidim is only at the top, as at km 65,6 (see description of P7-2 km 65,5). Side drains on both sides of the road in cutting, length 60 m.

Road condition:

Local failures still visible under new seal, but deterioration process evidently halted.

Distinct features:

Example of a failure halted with the installation of side drains.

Date(s) visited and comments:

8 March, 1991: With Mr Snyman of TPA, Piet Retief. Flow from side drain on southern side of the road. Flow mixes with surface water, outlet not very clear. Evidence of failure still visible.

Contact person(s):

Mr Gerhard Erlack, TPA Regional Engineer, Ermelo.

Mr Sakkie Fourie*, Chief Materials Technician, Ermelo.

Mr Leon du Plooy, Road Superintendent, TPA Piet Retief.

Mr Boet Snyman*, Foreman, TPA Piet Retief.

Telephone: TPA Ermelo: (01341) 2011.
TPA Piet Retief (01343) 4357.

Site:

P7-2. Km 65,5.

Location:

TPA Road P7-2 (Route R543) between Piet Retief and Wakkerstroom in the South-Eastern Transvaal. The site is located at km 65,5, approximately 20 km from Piet Retief.

Date installed:

1988.

Reason for installation:

Water-associated failure in cutting.

Drainage system:

Side drain consisting of large stone (> 75 mm) with Bidim only over the top of the trench, not on the sides. This is apparently done due to earlier reported problems with clogging of geotextiles. Uncertain whether pipes have been installed in the system. According to Mr Snyman of TPA, the installation was done in a hurry. Drain 200 m long in cutting.

Road condition:

Good.

Distinct features:

Problem has been solved, road is now in a good condition after serious failures. Side drain is unconventional with no geotextiles on the sides. If clogging of geotextiles in the area can be confirmed, laboratory clogging tests will be valuable to calibrate earlier laboratory research.

Date(s) visited and comments:

8 March 1991: With Mr Snyman of TPA Piet Retief. Strong outflow evident, road condition good.

Contact person(s):

Mr Gerhard Erlack, TPA Regional Engineer, Ermelo.
Mr Sakkie Faurie*, Chief Materials Technician, Ermelo.
Mr Leon du Plooy, Road Superintendent, TPA Piet Retief.
Mr Boet Snyman*, Foreman, TPA Piet Retief.
Telephone: TPA Ermelo: (01341) 2011.
TPA Piet Retief (01343) 4357.

Site:

P49-1.

Location:

On TPA Road P49-1 (Route R65) between Hendrina and Middelburg in the South Eastern Transvaal. Site is located just outside Hendrina in the first cutting.

Date installed:

N.A.

Reason for installation:

N.A.

Drainage system:

None.

Road condition:

Very poor. Local failure in cutting due to water ingress. Potholes, shear failures, patches and poor riding quality evident.

Distinct features:

An example of a water-associated failure without drainage that can be monitored for deterioration.

Date(s) visited and comments:

8 March, 1991: Road condition very poor.

Contact person(s):

Unknown.

Site:

P81-5.

Location:

On TPA Road P81-5 (Route R29) between Ermelo and Piet Retief in South-Eastern Transvaal. The site is located approximately 40 km from Ermelo in a deep cutting, going down the escarpment.

Date installed:

N.A.

Reason for installation:

N.A.

Drainage system:

Surface water side drains only.

Road condition:

Very poor. Serious water-associated failures over 500 m.

Distinct features:

Example of pavement failure in undrained cutting in the presence of excess ground water. Deterioration can be monitored, as well as improvement should drainage system be installed.

Date(s) visited and comments:

30 July, 1990:	Road in poor condition.
8 March, 1991:	Road in similar condition - cleaning of surface side drains in progress.

Contact person(s):

Mr Gerhard Erlank, TPA Regional Engineer, Ermelo.

Telephone: (01341) 2011.

Site:

MR 27.

Location:

CPA Road Main Road 27 between Somerset West and Stellenbosch in the Western Cape. Site is at km 1,7 at Somerset West.

Date installed:

Not yet.

Reason for installation:

This portion of road in a cutting showed signs of water-associated failures after major rehabilitation during a rainy season. A subsurface drainage system was designed and geotextiles tested for compatibility by ourselves. The drainage system will probably be installed when funds become available.

Drainage system:

Side drains on both sides, as well as in the median of the dual carriageway, either conventional or fin drain. Three geotextiles were tested for compatibility (woven and non-woven) and were all found to be satisfactory. It is uncertain which will be used.

Road condition:

Some failures, but not very serious.

Distinct features:

System can be monitored from the beginning. Laboratory work has been done by ourselves on soil/geotextile compatibility for this site.

Date(s) visited and comments:

N.A.

Contact person(s):

Mr Mike White, Uhlmann, Witthaus and Prins, Cape Town.

Telephone: (021) 212507.

Site:

Kyalitsha.

Location:

At Kyalitsha in the Cape Flats. Exact location unknown.

Date installed:

Probably August 1991.

Reason for installation:

New construction - high water table.

Drainage system:

Not finalised.

Road condition:

New.

Distinct features:

Cape Flat sands are free draining and are not expected to give any problems. High permeability materials can give an indication of high flows that need to be designed for.

Date(s) visited and comments:

N.A.

Contact person(s):

Mr Mike White, Uhlmann, Witthaus and Prins, Cape Town.

Telephone: (021) 212507.

Site:

TR41.

Location:

CPA Trunk Road 41 between Cookhouse and Somerset East in the Eastern Cape.

Date installed:

1990/1991.

Reason for installation:

As part of major rehabilitation of TR41/2 & 3 between Cookhouse and Bruintjeshoogte. Approximately 20 km between Cookhouse and Somerset East is in areas of agricultural irrigation with high water tables, and water-related distress evident in many cuttings.

Drainage systems:

Various. Three types of geotextiles used, namely, Bidim, Typar and Industex. These geotextiles were tested by ourselves for soil/geotextile compatibility.

Road condition:

Good.

Distinct features:

Soil/geotextile combinations tested by ourselves in the laboratory. Various geotextiles and drainage systems.

Date(s) visited and comments:

N.A.

Contact person(s):

Mr Darryl During, Bruinette, Kruger and Stoffberg, Cape Town.

Telephone (021) 221070.

Site:

TR 33/2.

Location:

CPA Trunk Road 33/2 between Hartenbos and Eight Bells in the southern Cape.

Date installed:

Not yet, construction to start soon.

Reason for installation:

As part of major rehabilitation (reconstruction) of approximately 14 km of road. Indications of ground water, drainage to be installed in cuttings.

Drainage system:

Various. Soil/geotextile compatibility test done by ourselves on three different geotextiles.

Road condition:

Reconstruction.

Distinct features:

Soil/geotextile combinations tested in the laboratory (27 combinations).

Date(s) visited and comments:

N.A.

Contact person(s):

Mr Mervin Henderson, CPA Roads Department, Cape Town.

Telephone: (021) 258 2020/258 2019.

Site:

Kestell-Glen Lennie.

Location:

On the Kestell-Harrismith PAO road in the eastern OFS. Various sites on 28 km between Kestell and Glen Lennie.

Date installed:

1987.

Reason for installation:

New construction in cuttings where signs of ground water were evident.

Drainage system:

Fin drains.

Road condition:

In some areas good, but some cuttings show signs of premature failure. It is not certain whether these cuttings were not provided with drainage systems or whether drains have failed.

Distinct features:

Examples of drains that have failed and/or comparisons between drained and undrained cuttings.

Date(s) visited and comments:

N.A.

Contact person(s):

Mr Gerhard Verwey, Vorster, van der Westhuizen and Partners, Bloemfontein.

Telephone: (051) 477036.

Site:

Fouriesburg-Caledonspoort.

Location:

Fouriesburg-Caledonspoort PAO road in the eastern OFS. Various sites in cuttings.

Date installed:

1988-1991.

Reason for installation:

New construction - installed in various applications, typically cuttings where evidence existed of ground water.

Drainage system:

All conventional with Industex S110T woven geotextile, 19 mm stone and 150 mm Santar pipes. A total of 13,5 km of side drains installed in sands, dispersive soils and shales.

Road condition:

New.

Distinct features:

Variety of systems and soils: sands, dispersive soils and shales. Woven geotextiles. Soil/geotextile compatibility tests done in our laboratory.

Date(s) visited and comments:

N.A.

Contact person(s):

Mr Gerhard Verwey, Vorster, van der Westhuizen and Partners, Bloemfontein. Telephone (051) 477036.

Mr Anton Kloppers, Vorster, van der Westhuizen and Partners, Harrismith. Telephone (01436) 21577.

Site:

N1/24. Km 21,8.

Location:

On National Road N1, Section 24 at km 21,8 on the Kranskop toll road between Nylstroom and Middelfontein in the Northern Transvaal.

Date installed:

1985/86.

Reason for installation:

Substantial ground water encountered during construction.

Drainage system:

Side drain, 840 m long. Layout shown on Figure A7 and cross-section on Figure A8.

Road condition:

Good.

Distinct features:

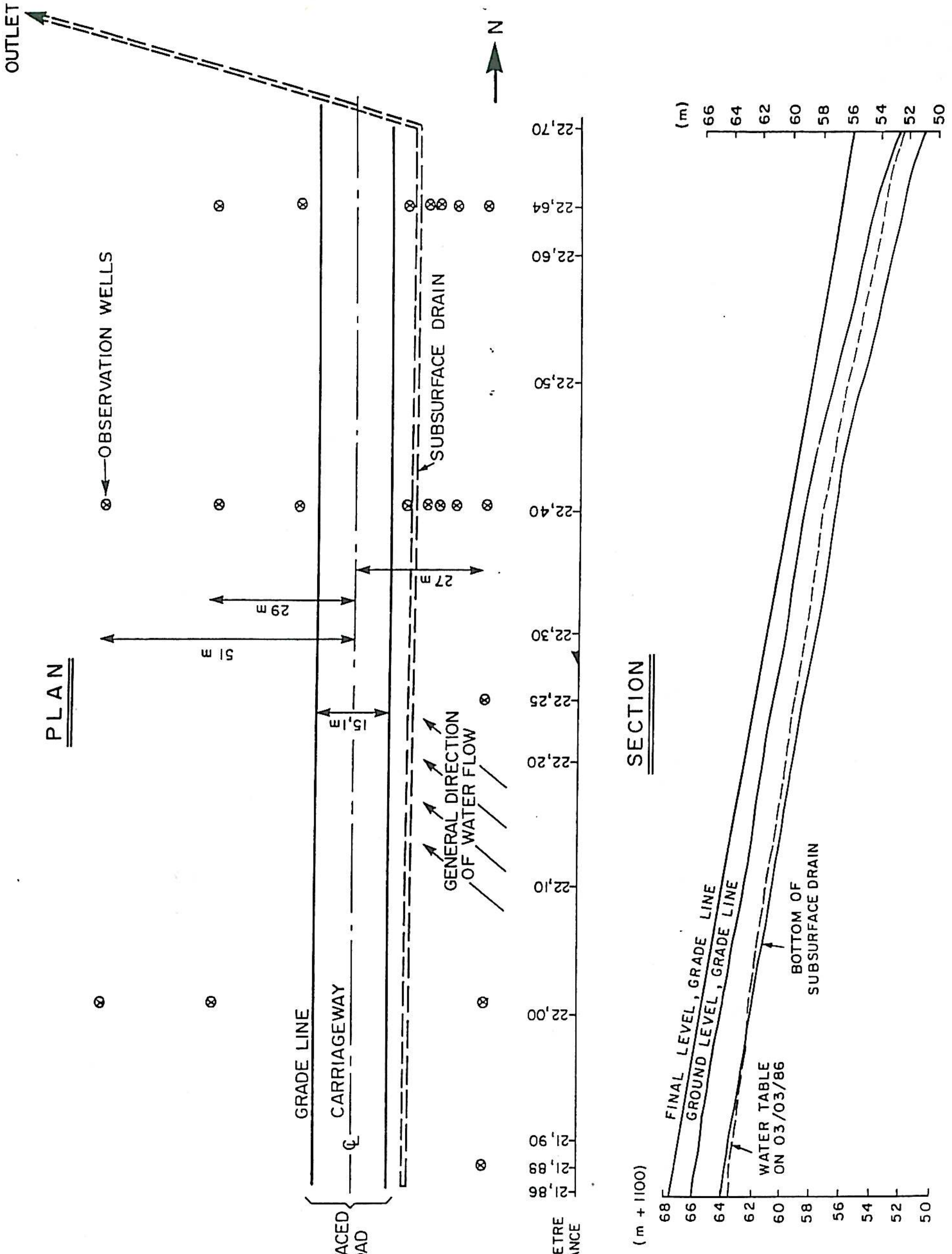
This installation was monitored in detail between 1985 and 1988 (Van der Merwe, 1988). To date, the section has performed well and is ideally suited for further monitoring.

Date(s) visited and comments:

Regularly since 1985.	Road still in good condition and water has flowed from system continuously.

Contact person(s):

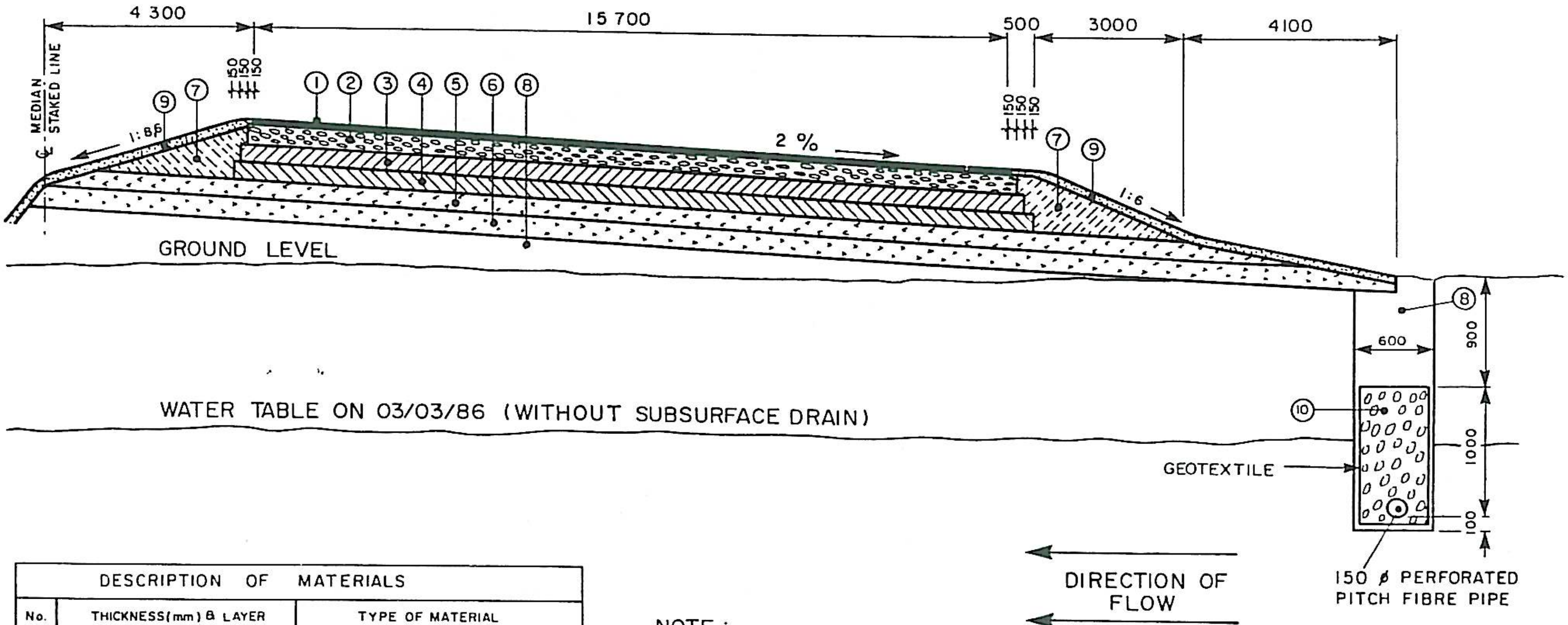
Department of Transport.



940-4-526/1 BS

FIGURE A7

LAYOUT OF SITE N1/24 km 21,8



DESCRIPTION OF MATERIALS		
No.	THICKNESS (mm) & LAYER	TYPE OF MATERIAL
1	40 SURFACING (AS)	SEMI-GAP GRADED ASPHALT - 19mm NOM SIZE ROLLED IN PRECOATED CHIPS
2	150 BASECOURSE (G1)	CRUSHED STONE
3	150 UPPER SUBBASE (C3)	NATURAL GRAVEL
4	150 LOWER SUBBASE (C3)	NATURAL GRAVEL
5	150 UPPER SEL SUBGRADE (G7)	SAND OR NATURAL GRAVEL
6	150 LOWER SEL SUBGRADE (G9)	SAND OR NATURAL GRAVEL
7	SHOULDER	NATURAL GRAVEL
8	FILL	SAND OR NATURAL GRAVEL
9	TOP SOIL	TOP SOIL
10	FILTER MATERIAL	19 mm AGGREGATE

NOTE :

VERTICAL SCALE = 2,67 HORIZONTAL SCALE

FIGURE A8

CROSS-SECTION N1/24 km 22,4