Envelope design for subsurface drains

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This book is published in the *ILRI Publication Series*. The main characteristic of this series is that the books are "practical" in the sense that they can be used in the day-to-day work of those professionals (mostly engineers) who are involved in irrigation and drainage.

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Forword

Since the beginning of the nineties ILRI has bilateral agreements with several research institutes in developing countries. The aims of these partnerships are to combine the common efforts in the better use of land and water resources in the following fields:

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Exchange of information
Development of joint projects.

This book is an example of the typical mission of ILRI; research of many years has been brought together in one publication for future use.
Work on drain envelopes at ILRI started as early as the mid seventies when staff of ILRI was involved at the East Khairpur Tile Drainage Project in Pakistan. Results of their findings were published in the proceedings of the First International Drainage Workshop held at ILRI in Wageningen, the Netherlands at the occasion of its 25 years of existence. In 1989 ILRI returned to Pakistan to assist the International Waterlogging and Salinity Research Institute, and one of the first requests was to investigate the troubles with the drain envelopes at the Fourth Drainage Project, Faisalabad, Pakistan. The authors of this book all three were involved with this activity. Their findings at the project and their experiences throughout their careers are brought together in this book.
The book is meant as a resource of work done to date, as well as, gives practical guidelines for successful implementation of drain envelopes.

Ir. A.W.H. van Weelderen
Director ILRI

Preface

This book has two fundamental objectives: (1) to serve as a practical guide for planners and designers; and (2) to serve as a resource book for envelope design and research. The second objective is the reason for the many cross-references in the book, which some reviewers found disruptive while appreciated by others. We decided to keep the cross-references. We ask those interested in design to kindly ignore the cross-references and footnotes in the design section of the book. However, for those planning a pilot area or laboratory research, or just wondering where a particular criterion came from, the information in parentheses in the grey shaded boxes, and in Chapters 5, 6 and beyond, is intended for you.

On behalf of my co-authors I would like to express our appreciation of the constructive remarks and contributions from the reviewers at McGill University, at USBR, at SCS and at ILRI. Although at times we might have been somewhat critical of some of their well-established procedures for drain envelope design, it was precisely this seeming lack of established criteria that sparked our search for the origins of the criteria and the limits that governed them. Meanwhile, new research and field experiences have redefined those limits and new criteria have been published by SCS, for instance.

Data collection for the book began in the early nineties and we updated each version when new information became available, modifying the criteria and incorporating our findings from Pakistan and elsewhere. All three of us were involved to different degrees in the search of the answers to gravel envelope failure in Pakistan. New synthetic envelope materials were already used extensively in Europe and North America and revision of gravel envelope design criteria did not seem necessary. Still, SCS refined theirs and we went one step further by applying the new, broad-based criteria of SCS to agricultural drains, enhanced by experience from the gravel envelope failure in Pakistan.

Extensive work for the appropriate design of synthetic envelope material continues. Yet, amongst the plethora of criteria for design of synthetic filters or geotextiles (presented in the resource section of the book) we found simple criteria developed by one of the authors of this book that satisfy agricultural conditions, which we can recommend. Although organic-based envelopes are losing ground to the synthetic materials, they are still applied. For the purpose of posterity we have included what we could find on criteria for their design. The book would not be complete without some costs however temporal they might be. Nor did we overlook the extreme importance of proper construction, which is essential for the successful functioning of drain envelope. Being researchers, all three of us could not fail to highlight laboratory and field-test-

ing procedures: the latter in the guidelines section, the former in the resource section of the book.
I would like to thank my wife and son for their understanding and patience when the going was tough and the progress slow. I feel it was worth the effort and I hope others will agree.
I consider the book to be a sound base for on-going work on design procedures for drain envelopes. New information is highly likely to surface via the Web. Your feedback and contributions, whether in the form of research results or based on case studies is most welcome. Please contact us via the ILRI website at ILRI@ILRI.NL

Willem F. Vlotman, Cairo, April 2000.

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Part 1 Guidelines

1. Introduction

1.1 Brief history of the drain envelope

Tile drains were first installed in the Netherlands in 1845 to remove excess water from agricultural soils. By 1850, 1140 ha had been successfully drained. The installation of subsurface tile drains in the USA also began round about the same time, where clogging was reported as early as 1859. Since then engineers have employed various means to protect drain openings from the entry of sediment, with varying degrees of success. Materials such as topsoil, sod, building paper, strips of tin, hay, straw, corncobs, cloth and burlap, leather, wood chips, sand, gravel, and other more modern materials have been used to prevent drains from clogging. While drainpipes can be safely installed in many soils without special protection, in some soils the pipes need to be fully protected by carefully designed drain envelopes.

Laid end to end with open joints, the earliest drainpipes were made of clay and concrete tiles. Water entered the drainpipes through the irregular gaps between the ends of the pipe sections. In the 1960s and 1970s perforated, smooth-walled plastic pipes were used as subsurface drainpipes. Corrugated plastic pipes made of polyvinyl chloride (PVC) and polyethylene (PE) were developed in the early 1960s and, since the 1980s, have become the most commonly used materials for drainage pipes. Corrugated plastic pipes with nominal inside diameters of up to 80 mm generally come in coils of 100 to 150 metres in length (in Europe, diameters of up to 200 mm can also be obtained). In the USA, 150 mm diameter pipes are provided in 30 m rolls. Drainpipes with larger diameters are produced in standard lengths of 6 or 9 metres.

Since the 1990s, gravel, pre-wrapped organic fibres (coconut, peat moss, straw), synthetic fibres and synthetic fabrics (geotextiles) have become the materials most commonly used to protect corrugated plastic drainpipes from sediment entry. Most of the porous synthetic materials used in drainage are specifically selected and designed for use as drain envelopes.

Criteria for the design of the gravel filters for use in hydraulic construction were first proposed by Terzaghi in the 1920s to protect a dam in Austria that had been built on a pervious foundation. The filter was designed to prevent soil movement as a result of piping from destroying the dam. Terzaghi's basic filter design guidelines have been used by many organisations such as the US Army Corps of Engineers, the US Bureau of Reclamation (USBR), the USDA Soil Conservation Service (SCS), and the Road Research Laboratory (RRL) in England. Terzaghi's original gravel filter criteria have been adapted for use in the design of envelopes for agricultural subsurface drains. Many of the scientists and engineers who initially established design guidelines for granular drain envelopes are no longer alive, and the research results and experience on which they based their recommendations are difficult to trace. As a result, drain envelope design guidelines are followed dogmatically, and questionable derivations of the original work appear. When serious problems occur with drain envelopes designed according to standard guidelines (Vlotman et al. 1990), the lack of information on basic principles is suddenly obvious, usually resulting in the initiation of new applied research on drain envelope materials.

1.2 This book

Overviews of current drain envelope design criteria are available (Willardson 1974, Dieleman and Trafford 1976, Fisher et al. 1990, Dierickx 1992b, 1993a, Stuyt and Willardson 1999, Giroud 1996), and three books on geotextiles and membranes in civil engineering have recently been published (Ingold 1994, Koerner 1994, Santvoort 1994). What was needed was a systematic overview of the work done, the experiences of the past with drain envelope design and construction, and a clear description of the reasons and backgrounds of current design criteria. This book consolidates past, present and newly developed drain envelope selection and design criteria. Information from the above-mentioned publications pertaining to the application of geotextiles in subsurface drainage has been incorporated into it. The book has been written for engineers, contractors, manufacturers, students, teachers, designers, researchers and other professionals who need to know about designing successful subsurface drainage systems. This includes preventing soil particle movement into drains as a result of the hydraulic conditions of the soil.

In Figure 1 the various steps in the selection and design process of drain envelopes is graphically shown, indicating the sections of the book where details can be found. A very brief description of the processes covered in this book - by way of questions and answers - is given in Box 1, the details of which can be found in the relevant chapters. The book is divided into two parts: Guidelines and Resources. The Guidelines are in Chapters 2, 3 and 4. The Resource part comprises Chapters 5, 6, the references, glossary and list of symbols.

Collecting appropriate soil information during the pre-drainage investigation is mandatory to determine drain envelope need. Designers and planners should be aware of the special needs for envelope design (Sections 2.4 through 2.6). The need for an envelope can be derived from soil field investigation in the laboratory, or can be based on special field trials (Figure 1, step 3).

Once the need for a protective envelope has been established, Section 3.1 will

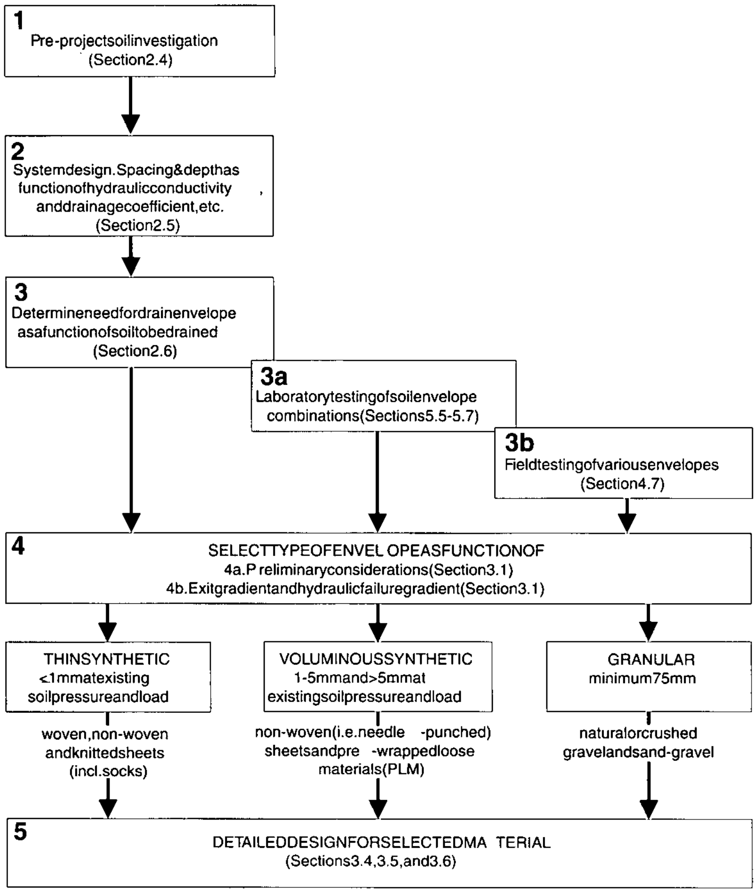


Figure 1 Steps in drain envelope d

guide the reader through the various options available among the range of suitable envelopes of different materials and functions.

Various sections in Chapters 3 and 4 provide practising engineers with useful procedures to specify the final requirements of the envelope and to make a cost evaluation of their chosen design. While one of the main objectives of this book is to give practical procedures for the determination of the need for a drain envelope and for the design of envelopes, it also documents the reasons for following certain procedures. For this reason copious use is made of forward references in Chapters 2 and 3 to sections where background and more detail can be found. For the same reason ample references to other publications are given. Details of laboratory investigations enhanced by field experiences are given in Chapters 5 and 6, together with suggestions for appropriate further research.

The numbers in front of many of the gradation labels in the figures refer to the appropriate serial number of the data set used to prepare the figures in the book. The data sets are available at ILRI.

The main purpose of subsurface drainage is for the removal of excess water in all types of situations, which are not all agricultural in nature, and include road drainage, urban drainage, temporary drainage of construction sites, drainage to protect hydraulic structures, and drainage to protect monuments. Various sources mentioned earlier cover these broader topics adequately. This book concentrates on the drain envelopes needed for agricultural drainage. Drain envelope design for agricultural purposes is different from many of the other filter applications because it usually has two objectives (amongst other functions), namely filtration and reduction of water entrance resistance, both of which need to be satisfied, but requiring solutions that achieve the opposite: a filter function and a permeability function.

The types of drains covered in this book are:

Agricultural subsurface drains:
- Laterals with free outflow conditions or submerged outlets;
- Laterals discharging into collectors and collectors in open drains; and
- Pumped subsurface drainage outlet systems;

Not covered in detail are:
- Road and rail drainage applications;
- Construction drainage;
- Pipeless drainage (French drains);
- (Vertical) strip drains; and
- Composite drains (a synthetic material covers a synthetic conveyance medium other than a plastic pipe).

Besides tables and figures this book includes boxes. The boxes contain material of interest to the reader but not needed for the general flow and understanding of the main text.

Box 1 Three questions, three answers.

Question:	*Do I need a drain envelope?*
Answer:	*A drain envelope will always improve the functioning of a drain unless the wrong envelope is used or if it is not installed properly. Hence, unless the percentage of clay in the soil is high and the soil stability around the drain is good, use an envelope.*
Question:	*How do I choose the right envelope?*
Answer:	*Calculate the exit gradient at the soil-drain envelope interface using the calculated drain spacing and the drainage coefficient to determine the inflow rate (Section 2.5) and with all water passing through only one-fourth of the surface area of the drain envelope-soil interface. Then determine the actual opening size through which water will flow into the drain in this one-quarter area (Section 5.1.4). Use the hydraulic conductivity of the soil to determine the gradient. Determine the Hydraulic Failure Gradient (Section 2.6) of the soil and compare the two gradients. If the exit gradient exceeds the Hydraulic Failure Gradient of the soil, increase the outside diameter of the drain envelope by using a gravel envelope with an appropriate grading curve (Section 3.3), or increase the drain diameter. If the Hydraulic Failure Gradient of the soil is still greater than the exit gradient of the water leaving the soil, choose a geotextile envelope material that meets the criteria based on the soil gradation curve and the proper characteristic opening size of the geotextile (Section 3.5). If gravel is also needed for the mechanical and/or bedding function, consider using a combination of granular and synthetic material.*
Question:	*How can I be certain the drain and the envelope are properly installed?*
Answer:	*The drain envelope should completely enclose the pipe without any holes or gaps. There should never be any free water or slurry in the trench in contact with the pipe and envelope material and the pipe should be completely covered with soil immediately. Filling of the trench can be delayed somewhat, but the pipe should be completely covered with soil before any water appears in the trench bottom. Consolidation of the backfilled trench material can be done later. If this is not possible consider trenchless drain construction (Section 4.3).*

2. The need for a drain envelope

2.1 Functions of a drain envelope

A drain envelope is porous material placed around a perforated pipe drain to perform one or more of the following functions:

Filter function: to provide mechanical support or restraint of the soil, at the drain interface with the soil, to prevent or limit the movement of soil particles into the drainpipe where they may settle and eventually clog the pipe. Initially, there might be some fine and colloidal material passing through the envelope into the drain. After construction when the soil-envelope combination has stabilised, it is expected and acceptable that a limited flow of clay and other suspended particles will remain in suspension in the drained water and leave the drain. The filter function may be temporary, i.e. long enough to allow the disturbed soil to stabilise (organic envelopes have been used successfully for this purpose in the Netherlands).

Hydraulic function: to provide a porous medium of relatively high permeability around the pipe to reduce entrance resistance at or near the drain openings.

Mechanical function[1]: to provide passive mechanical support to the pipe in order to prevent excess deflection and damage to the pipe due to soil load.

Bedding function[1]: to provide a stable base to support the pipe in order to prevent vertical displacement due to soil load during and after construction.

Over the years a number of terms have been used to describe drain envelopes. Dieleman and Trafford (1976) used the terms envelope, filter, and surround, to distinguish between types of envelopes based on their principal function. The term envelope has also been used as a generic name for any artificial material placed on or around a drain to improve its functioning without specifying the reason for its use. A filter is an envelope used specifically to keep fines from the soil from entering the drain. A surround is material specif-

[1] Can only be achieved with gravel and sand envelopes.

ically selected to provide a zone of high hydraulic conductivity around the drain, to minimise entrance losses and thus create an ideal hydraulic drain.

ILRI (Ritzema 1994) defined a drain filter as: "A layer or combination of layers of pervious materials, designed and installed to provide drainage, yet prevent the movement of soil particles in the flowing water." In the literature, drain envelopes are referred to under names and definitions that seem to be a mixture of descriptions of the materials used and the functions listed above. In this book, the different types of envelopes are classified according to the material from which they are made. An envelope appropriate for use under specified field conditions could perform one or more of the above functions. Granular envelope material, such as fine well-graded gravel or coarse sand, will perform all of the functions of an envelope. Synthetic envelopes may only perform the filter and hydraulic functions.

The main need for a drain envelope is to keep the pipe sediment free. This need only involves the <u>filter function</u> of the envelope. There are conditions where a drain will function more efficiently if an envelope is installed to improve the hydraulic function of the soil. A soil with a high clay content could be mechanically stable, but might have a low permeability. Since most of the potential energy moving water toward the drain is dissipated very close to the drain, the efficiency of drainage can be increased by placing some highly permeable envelope material around the drain to increase the effective diameter (hydraulic function). A larger effective drain diameter resulting from installation of a thick layer of highly permeable envelope material around the drain also decreases the exit gradient of water leaving the soil.

The <u>mechanical function</u> of a drain envelope is important when a flexible plastic pipe is used as a drain. From the mechanical function standpoint, the ideal drain envelope is a gravel envelope. The gravel fills the space between the pipe and the undisturbed soil of the sides of the trench with incompressible material that can enhance the full load-bearing strength of the pipe. It prevents flattening or deflection of the vertical pipe diameter that would reduce the hydraulic capacity of the pipe. A thick fibrous or geotextile drain envelope does not enhance the structural strength of a drainpipe.

When synthetic envelope material is used around a flexible plastic drainpipe (and also when no envelope is used on the same type of pipe) it is necessary to place the pipe in a groove, ploughed or cut into the undisturbed soil in the bottom of the trench, to provide structural support for the pipe. The groove can be a ninety-degree 'V' shape or it can be semicircular to conform to the outside diameter of the lower half of the pipe. Either bedding configuration results in a strong pipe support system, with a load-bearing capacity approximately equal to that of a pipe inside a gravel envelope.

The bedding function of a drain envelope is only accomplished by use of a gravel envelope. If the trench bottom is irregular, due to excavation by hand or by a backhoe type machine, gravel or sand is sometimes put into a trench and smoothed down by hand, so that the pipe will have a smooth uniform foundation (called bedding) that will not alter during backfilling. Installing special bedding material might also be necessary when unstable trench bottoms are encountered. Covering the base of the trench with gravel (envelope) material could give it sufficient weight to stabilise it. In some cases, a separate dewatering operation may be necessary to stabilise the trench bottom and sides so that the drainpipe and envelope can be properly installed and the trench backfilled without displacing or flattening the pipe.

2.2 Soils that require a drain envelope

When a new drainage project is being proposed, one of the first questions that arises is whether a drain envelope will be needed for any or part of the systems planned. Drain envelopes add an extra cost to a project, but if an envelope is necessary, drains installed without envelopes will fail.

Sedimentation and clogging of subsurface drains could result if:
1. the openings in the drain are too large and bridging of soil particles does not take place; or,
2. the soil itself is unstable under prevailing water flow gradients and the perforations/openings of the drain are not adequately protected by a drain envelope; and,
3. once the soil particles are in the drain, the grade of the drain or water velocity in the drain is not sufficient to flush the particles out of the drain.

The decision on the need for a drain envelope in a particular soil can be based on local experience or on empirical relations between measurable soil properties. Unless sandy, soils in humid areas, generally have a strong structural strength and drains can be installed in such soils without envelopes. Soils with a high clay and/or organic matter content also have higher structural strength. Simple correlation of soil structural strength with organic matter content or clay content have not been conclusive in determining whether drain envelopes will be needed for a particular soil, but this information, coupled with local experience can give dependable predictions.

Soils in humid areas do not share the same relationship between texture and stability as do soils in arid areas. For soil conditions in the humid areas found in the Netherlands, Van Zeijts (1992) developed relationships between clay and silt contents of soils and the need for a drain envelope, as well as the appropriateness of envelope types (organic, synthetic, thin or voluminous) for certain soil types (see Table 31). Properly designed gravel envelopes can be used for all soils. Geotextiles and fibrous envelopes have some limitations that are related to the mechanical and bedding functions. In the Netherlands drains can be installed without an envelope in soil with a clay content of more than 25%. In Egypt (DRI-staff, 1983 and Abdel-Dayem, 1985) it was found that clay and concrete drainpipes without envelopes remained sediment free in soils with a clay content of 30% or more, provided that construction of the loosely-connected concrete pipe segments had been satisfactorily carried out and no serious misalignment had occurred over the years. In clay soils with substantial amounts of the montmorillonite clay particles and consequently with potentially deep cracks, drains had sediment inside and it was postulated that sediment found inside the drains had been the result of surface erosion, and/or erosion of the cracks during irrigation. In India soils with clay < 30% and also soils with SAR > 8-13 and clay < 40% were found in need of drain envelopes (Rajad Project staff 1995). More information on the possible effect of SAR is given in Box 2. The philosophy of the US Bureau of Reclamation (USBR) is that in settings where normally drains are to be constructed there will always be soils that require an envelope if not for filtering, than for hydraulic capability or for bedding of the plastic pipe. Even extensive investigations might not reveal the exact location of where problem soils could be encountered along the drain line. Therefore, the USBR automatically uses a granular envelope appropriate to the soil conditions on virtually all pipe drains.

Arid region soils are generally less stable than humid area soils, so clay content alone is not a good indicator of soil strength and stability. A parameter called Hydraulic Failure Gradient (HFG) has been developed (see Section 5.3) to determine the resistance of soils to flowing water. The expected inflow to the drains and the area of openings in the drains can be used to calculate the exit gradient (related to the velocity of water in the soil, Section 5.1) of water entering the drain openings. If the exit gradient exceeds the HFG of the soil, an envelope is needed. The Plasticity Index and the Saturated Hydraulic Conductivity of the soil are used in an empirical equation to determine the HFG of a particular soil for comparison with the computed exit gradient. Placing a properly designed envelope around the drain to protect the drain openings will also reduce exit gradients providing additional protection. Increasing the drain diameter, increasing the area of perforations, or decreasing the drain spacing are other alternatives for decreasing exit gradients.

Drains installed in soils that are mechanically unstable when they are wet will need the additional protection of a drain envelope. Drainage contractors refer to these soils as problem soils. Unstable soils are non-cohesive or weakly cohesive soils such as fine sands and silts. Sodium-affected soils can also be unstable (Box 2). Coarse-textured soils that have a uniform fine particle size are especially troublesome. Soils containing a high clay percentage tend to be more stable and many have sufficient inherent cohesion that they do not need envelopes. If excavated auger holes or trenches collapse shortly after they are opened below a water table, drains installed in that soil are most likely to require a drain envelope.

Various other physical factors related to installation conditions affect the decision on the need for drain envelopes in a particular soil. If the soil is dry or moist (not saturated) during installation an envelope might not be needed, whereas a soil that is wet (fully saturated) at the time of installation (and the installation depth is below the water table) would. Yet, dry soil clods can loose their structural strength upon wetting due to splitting caused by the escape of enclosed air. Dry soil can therefore also pose sedimentation problems when the escaping air causes dispersion.

Furthermore, the speed of the trencher has an effect on the type of drain envelope selected in situations where speed and immediate backfilling are essential to prevent uplift of the pipe laid. Installing a fabric envelope in a trench containing a slurry could result in instantaneous failure of the envelope if the fabric used is too fine (blocking of the envelope). For questionable soils and installation conditions, it might be necessary to make an initial decision based on the best information available, followed by a pilot installation to check procedures and performance of the drain and envelope and the installation procedure. Stuyt (1992) concluded that the best method to determine whether an envelope is needed is to install test drains and monitor sedimentation over time.

Huinink (1992) and Van Zeijts (1992) in the Netherlands, and Samani and Willardson (1981) and the Soil Conservation Service (SCS 1991) in the USA have developed procedures for assessing the need for a drain envelope. However, all of the prediction methods are presented with limiting qualifiers such as the subjective statement of ripened or unripened soil (Table 31, see Glossary for definition), clay and silt combinations and the term instability, or the all-encompassing statement of 'unstable soil'. A basic understanding of the flow of water towards the drain and the soil mechanics in action at the soil-envelope and envelope-drain interface is therefore essential for making proper judgements on the need for a drain envelope and the design requirements thereof. Whether or not a drain envelope is needed for a specific situation depends on both the hydraulic conditions and the stability of the soil near the drains. Early indicators for the need of a drain envelope are collapse of the auger hole during the pre-drainage soil investigation, and anticipated construction of the drains below the water table.
Some indirect methods of assessing the need for a drain envelope in a particular soil are presented in Section 2.6 (direct methods would be field-testing of drains). Specific soil tests and calculation of expected flow gradients are needed for further evaluation.

Box 2 Use of SAR, EC and ESP to assess likelihood of dispersion.

The Sodium Adsorption Ratio (SAR) of the soil water is a measure to judge whether soil is in danger of becoming alkaline when it comes into contact with certain water qualities. When the SAR > 13 and the Electrical Conductivity of irrigation water (EC_w) < 2 dS/m the soil may disperse (deflocculate), which will cause considerable reduction in permeability due to the movement of very fine particles into small pores. The adjusted SAR (SAR_{adj}) is used to assess the permeability hazard based on irrigation water quality (Hoffman et al. 1980). If the SAR of the soil water at drain depth is high the clay will be dispersed only once all the salt has been leached and the EC_w is low.

Usually intrinsic soil permeability is defined in terms of saturated hydraulic conductivity of the soil along with physical properties of the flowing solution, such as its viscosity and density. These fail to describe the dependence of permeability on effective soil porosity (function of clay swelling properties) and the concentration and composition of the soil solution. Swelling of soil clay particles (Smectite/Montmorillonite) in a confined system decreases the size of large pores, while dispersion and movement of clay platelets further blocks the soil. Many researchers predicted changes in permeability as a function of salt concentration and ionic composition changes in swelling soils (Bresler et al. 1982).

A typical example of sodium-induced deflocculation given by McNeal and Coleman (1966) showed that saturated hydraulic conductivity decreased as the salt concentration decreases or as the Exchangeable Sodium Percentage (ESP) of the soil increases. The effect is normally greatest in soils with high clay contents and/or high contents of swelling clay minerals, although exceptions in Hawaiian soils have been documented. McNeal and Coleman presented a figure which showed the saturated hydraulic conductivity (K_s) related to salt concentration and SAR (the latter as curve parameters) for Pachappa sandy loam and Waukena clay loam. The K_s of the Pachappa sandy loam with a SAR of 40 did not begin to decrease until EC_w (Electrical Conductivity of the [irrigation] water) dropped below 6 dS/m, while for the Waukena Clay Loam K_s started dropping when the EC_w dropped below 17 dS/m. Since the finer soil would be the least stable, relating the EC_w and SAR of the Waukena Clay Loam would present a conservative approach for assessing the potential of reduction in K_s because of dispersion:

$$EC_b = -2.3 + 0.5 * SAR \qquad \text{Eq. 1}$$

where,

EC_b *is the dispersion or deflocculation boundary of the electrical conductivity of soil water in dS/m*

At the boundary value or at lower EC_w values soil particles become detached and the soil tends towards dispersion for the given SAR value. If the EC_w of the soil water is high enough there is no danger of deflocculation which would clog the envelope or drain. To assess the danger, evaluation of the SAR and ground water quality at drain depth would seem appropriate.

The SAR and ESP relationship can be obtained by (Jurinak and Suarez 1990):

$$SAR = ESP \ / \ (k'_g * (100 - ESP)) \qquad \text{Eq. 2}$$

where,

k'_g *is the modified Gapon selectivity coefficient in $(mmol/l)^{-1/2}$, typical values 0.016 - 0.008; a value of 0.015 is widely used for Illite clays.*

Ayers and Westcot (1976, 1985) and Hoffman et al. 1980 give additional guidelines to judge potential dispersion/permeability problems based on EC_w and the SAR_{adj}.

2.3 Soils that do not require a filtering drain envelope

Where an envelope is not needed to satisfy the filtering function to keep fine sediments out of the pipe, there might still be certain hydraulic or installation conditions that necessitate the use of envelopes. Here, specific criteria pertaining to the hydraulic, mechanical, or the bedding function apply. The soils that do not require a filtering envelope are:

- heavy clay soils (heavy clay soils can be defined as having a clay percentage > 60% and hydraulic conductivity < 0.1 m/d);
- clay soils with the percentage clay exceeding 25 - 30% in humid climates;
- soils with a Plasticity Index (PI) greater than 15;
- soils with a Coefficient of Uniformity (C_u) > 12; and
- coarse soils with 90% of the particle sizes larger than the maximum drainpipe perforation width.[2]

2.4 Soil characteristics to determine envelope need, and for design and functionality

To determine the need for a drain envelope, to design it properly, and to anticipate any problems after its installation a number of soil properties need to be known. These will be described briefly. A section follows this on how to deal with the potentially large amount of soil data available, making it manageable for design purposes.

The standard reference in the text to particle size of base or soil material and granular envelope material respectively will be d_{xx} and D_{xx}. The lower case d refers to the base or soil material at drain depth, whether used with granular, organic or synthetic material, while the capital D denotes the size of the granular envelope material. The number following each letter (xx) is the percentage of the sample, by dry weight (cumulative percentage passing) that is finer than the d or D in mm as determined by a sieving test (Section 5.5.1).

There are two traditional methods to display the soil data: the soil texture triangle, which serves to give a unified name to a particular soil; and the semilogarithmic plot, or particle size distribution curve (PSD curve). Examples of these graphs can be found throughout the book.

2 This criterion is rather conservative. More precise and more flexible bridging criteria are described in Section 3.3.4 & 5.2.

2.4.1 Required soil investigation

The following data need to be collected:

1. Soil samples from drain depth in sufficient number and with sufficient material to perform the following tests:
   a) sieve analysis (Section 5.5.1). The analysis is needed to assess the particle diameter at certain cumulative percentages of material passing for use in the design formulas (Section 3);
   b) Plasticity Index (PI) determination (Section 5.5.2), as an indicator of soil stability and to determine the Hydraulic Failure Gradient (HFG) without the need for direct laboratory determination;
   c) the soil texture analysis using the hydrometer method (sand, silt and clay percentages, Section 5.5.5). The clay percentage provides the first indication for the need for an envelope, and the finer particle ranges are important in granular envelope design;
   d) chemical analysis to determine the susceptibility of soils to disperse (deflocculate): EC_e, EC_w, SAR, etc.;
   e) iron ochre, calcium carbonate, sulphur and manganese content in the soil (Section 5.5.5).
2. Saturated hydraulic conductivity (K_s) at drain depth. If the hole from which the soil samples were taken for the sieve analysis etc. remains open until the next day, the same hole can be used for the auger hole method for the determination of the Saturated Hydraulic Conductivity. Instability of the auger hole is a first indication of potential construction problems and the need for an envelope. The auger hole method is one of the standard pre-drainage investigation tests. Not only is it used for the drain envelope design but also for drain spacing determination. In unstable soil, a screen will need to be used in order to perform the auger hole method of hydraulic conductivity determination (Oosterbaan and Nijland 1994);
3. Depth of the impermeable layer. This information is needed to determine the gradients to be expected at the drain and to compare these with the HFG, and is part of the standard pre-drainage investigation. The accuracy of the depth of the impermeable layer is less critical than the determination of the hydraulic conductivity (Box 5).

The amount of soil collected using an auger 8 - 10 cm in diameter over a 30 cm length will usually be sufficient to perform all the necessary mechanical and chemical analyses. The density of the measurements depends on the particular needs, but for soil investigation purposes a density of one measurement per 10 - 25 ha (grids of 300 x 300 m or 500 x 500 m) is commonly used. Gallichand et al. (1992) found that for preliminary surveys a grid of 900 x 900 m would provide adequate information on hydraulic conductivity, while optimum results were obtained with grids that had distances between 400 - 600 m.

Certain assessments (e.g. the critical gradient) might require the void ratio and the specific gravity of the soil, in which case undisturbed samples need to be taken. The void ratio (porosity) is not used in standard drain envelope design, except to compare the Hydraulic Failure Gradient, but is more commonly used to determine water-holding capacity. Data from pre-project investigations or indicator values from laboratory tests (Sections 5.5 and 5.6) with typical soils can be used.

2.4.2 Base soil bandwidths

Several sources (see Figure 2 and 3) have displayed bandwidths of problematic soils (or soils in which drains are known to need envelopes). Some of these bandwidths seem to define a narrow range of soils that always need envelopes. However, close perusal shows this to be far from true and that the narrowness was more a function of how the graphs were presented rather than a true well-defined band. There is, therefore, a need for a methodology to define the best representative bandwidth for a soil. Classical design procedures give ranges of soils covering more than is necessary for agricultural conditions. Some examples from the literature follow in the next paragraphs.

SCS (1994) presented four classifications of soils (Table 1) each with a slightly different granular filter design. SCS filter design includes soils with clay percentages of over 25 – 30 %, which do not need a drain envelope for agricultural applications. Detailed design of synthetic envelopes with Egyptian Delta soils showed a range of soils that is too large is not practical (Vlotman and Omara, 1996), and it was found that several classes are needed to limit the required number of standard synthetic envelopes (Table 2). Regardless of the standardisation effort, it is desirable to design geotextile envelopes for the coarser soils individually. In the Netherlands, the choice of envelope material classes (prewrapped loose material PLM) went from three to two, with one being rather dominant in use (75% of the cases based on typical soils).

Soils in the Netherlands that were in need of drain envelopes were represented in a rather narrow band (Figure 2C), while in Canada and Germany the bandwidth of problematic soils was rather wide (Figure 3C). The US never produced a bandwidth recommendation, but guidelines by the USBR (1993, Table 34, p. 270), the SCS (1994), and the chart using the Plasticity Index (PI) of the Unified Soil Classification system (Figure 74, p. 250) clearly divide the full range of soils into certain classifications, each of which requiring a slightly different design (for general purpose filters).

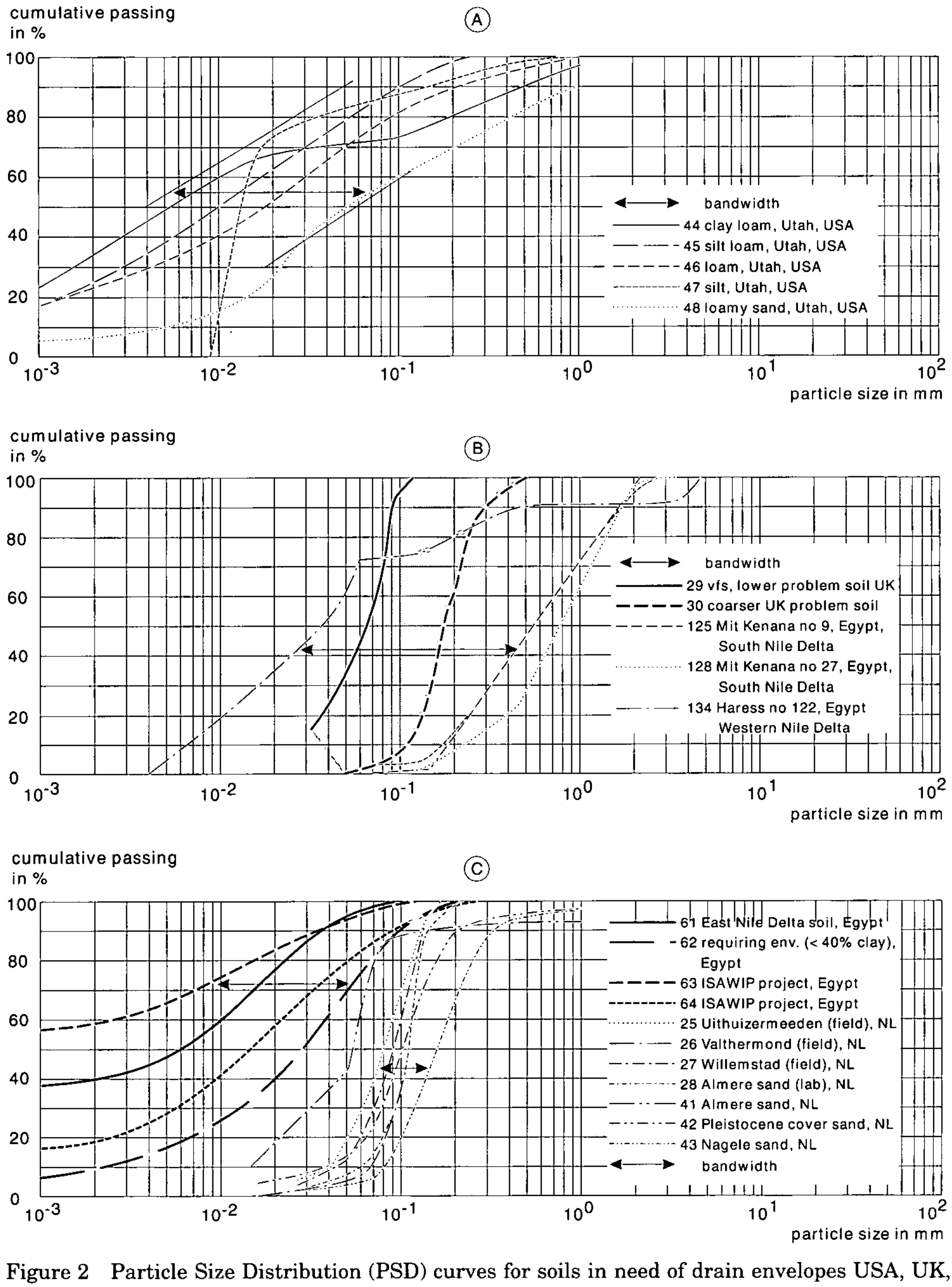


Figure 2 Particle Size Distribution (PSD) curves for soils in need of drain envelopes USA, UK, Egypt, the Netherlands (NL).

A USA soils, Cache Valley, Utah, USA (Willardson and Ahmed 1988).

B UK (Davies *et al.* 1978), Egypt (Abdel Hadi 1996).

C Egypt (Metzger *et al.* 1992), the Netherlands (NL): Stuyt (1992) and Scholten 1988.

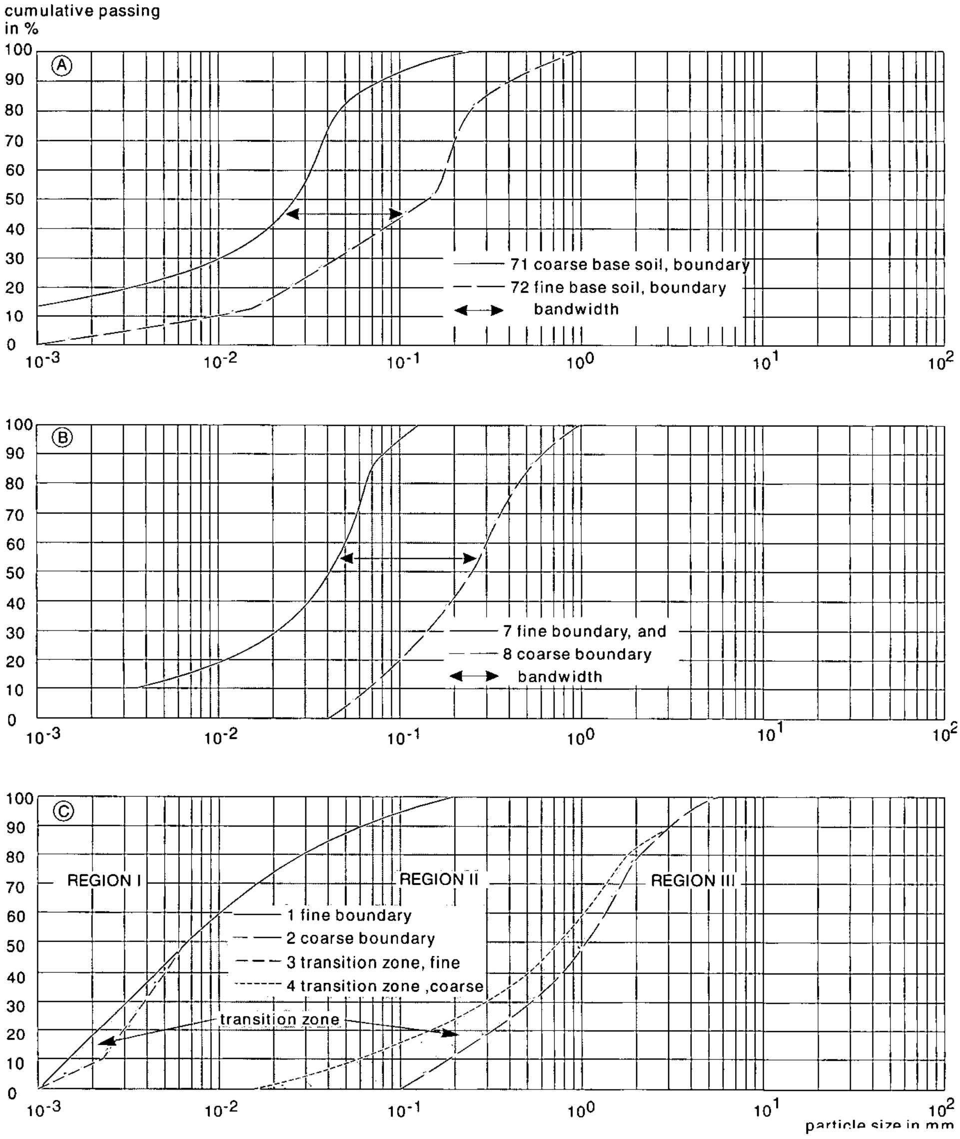


Figure 3 Soils in need of drain envelopes Pakistan, Canada, Germany.

A Pakistan soils requiring envelope.

B Canadian problems soil range (Irwin and Hore 1979, Cavelaars et al. 1994).

C German problem soil range (FGSV 535, 1994).

Table 1 Soil classification for filter design by the Soil Conservation Service (SCS 1994).

% finer than 0.074 mm after regrading*, where applicable	Base soil descriptions
> 85	Fine silt and clay
40 – 85	Sand, silt, clay, and silty and clayey sand
15 – 39	Silty and clayey sand and gravel
< 15	Sand and gravel

* Regrading: using d < 4.76 mm sieve results only.

Table 2 Ranges of selected d90 values for use with Egyptian soils. (after Vlotman and Omara 1996).

d_{90} in μm	Base soil description
< 60	fine soils > 30% clay: no need for envelope
60 – 200	Soil range 1; standard synthetic envelope no.1 with O_{90} = 200 – 600 μm
200 – 500	Soil range 2; standard synthetic envelope no.2 with O_{90} = 500 - 1250 μm
500 – 3000	soil range 3: broad range individual design necessary
> 3000	gravel soils; no envelope needed

The smallest sieve size most commonly used (standard sieve no. 200, 0.074 mm) is often used as the classification criterion (SCS 1994, Wilson-Fahmy *et al.* 1996).
Wilson Fahmy et al. (1996) use three basic soil types to judge geotextile filter performance and filter criteria (Table 3).

Table 3 Example of soil classification used to group assessment of geotextiles. (Wilson-Fahmy et al. 1996).

% finer than 0.074 mm	Base soil description
≥ 50	fine grained
13 – 49	mixed soils
≤ 12	Granular

USBR (1993) employs a distinctly different method that is similar to traditional divisions into silt, and sand ranges; the range within which the d_{60} falls is used for the classification: 0.02 – 0.05, 0.05 – 0.1, 0.1 – 0.25, and 0.25 – 1.0 mm.

From the foregoing it is clear that there is neither a unified approach to classify soils for drain envelope design, nor one for filter envelope design. Based on sieving techniques, some criteria stipulate the range of particle sizes that fall within or outside certain percentages of the soil, while others stipulate actual particle sizes regardless of the percentage of the soil it represents.

To make a first assessment of envelope need it is important to know the percentage of clay particles (particle sizes smaller than 0.002 mm). In addition, as clearly shown in Section 3.3, it is necessary to determine the finer boundary of the base soil bandwidth for filtration purposes and the coarser boundary for use with the permeability/hydraulic criteria (granular envelopes only). The boundaries at the percentages most commonly used to describe certain Particle Size Distribution (PSD curve) characteristics must be determined on some rational basis. In other words the finest curve found is not necessarily the one to be used for the fine boundary. A frequency analysis of the occurrence of the d_5, d_{10}, d_{15} , d_{60}, d_{85} and d_{90} sizes is recommended for the coarse boundary. Then, depending on the shape of the frequency distribution one can use quartile percentages, standard deviation sizes, and the like (Section 5.5.1) to determine the actual boundary for use in the design.

Based on pre-drainage investigation results, it is expected that the data for one or more representative base soil bandwidths are available for the area where subsurface drains are to be constructed. Broad bandwidths (with coarse/fine boundary ratios > 10, such as in Figure 2A and C, and Figure 3C) will not result in satisfactory drain envelopes that will meet the opposing requirements of filtering and high permeability (See also Box 3). Hence, one should check whether bandwidths representative for sub-areas from topographical point of view or sub-areas with similar soil types can be produced. Bandwidths with coarse/fine boundary ratios of 3 - 6 (Figure 2B and Figure 3A and B) will result in appropriate granular envelope bandwidths. Bandwidth could be based on the 25% and 75% quartiles of the full set of base soil samples or other reasoning.

Figure 4 gives the final result of an example of bandwidth determination using the 25 and 75 % quartile methodology. Soil samples were collected from different depths, based on a grid of 150 x 150 m, in anticipation of construc-

Box 3 Soil boundaries and re-grading.

Base soils at drain depth should be represented by a bandwidth on the Particle Size Distribution (PSD) curve. The finer boundary of the band represents the PSD curve that needs to be retained for filtering purposes, providing that the percentage clay is less than 25 - 30%. The coarser boundary represents the soil PSD line that should be used in assessing the desirable hydraulic properties of the envelope.

Regrading of the soil sample, which is removing large particle sizes to better reflect the true hydraulic properties of the soil under investigation, as suggested by SCS 1994 and Sherard et al. 1984a, is generally recommended when a substantial amount of the sample has particles larger than 4.75 mm. Most examples of agricultural soils used in this book have d_{100} < 4.75 mm and hence regrading is generally not needed for agricultural soils. Regrading is adjusting the amounts retained on each sieve by excluding the combined weight of the particle sizes greater than 4.75 mm, which means the weight retained of particles on sieve no.4 of the Standard US Sieve set (Table 15, p 164) and higher.

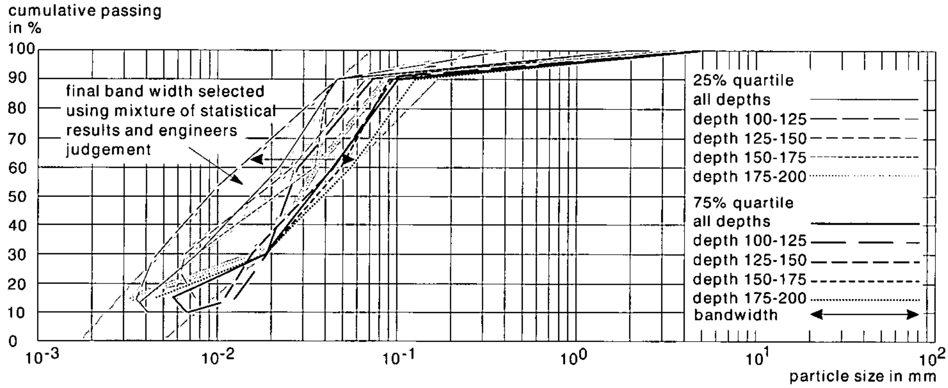


Figure 4 Results of quartile analyses of particle size distributions at various drain depth and recommended final bandwidth.

tion drain depths ranging between 1.2 and 1.9 m. Four different depth ranges were statistically analysed. Soils at depths of 100 – 125 cm were heavy textured with a bandwidth ratio (25%/75%) ranging between 1.7 and 3.4.m. Soils deeper down were lighter textured with bandwidth ratios of around 1.5. Figure 4 shows little difference between the bandwidths at the various depths. It is the reason for the decision to recommend that the final bandwidth for designing the drain envelopes should encompass all possible soils based on the quartile analyses. The fine boundary of the soils at a depth of 100 - 125 cm at d_{90}, d_{85}, ...d_{15} (the 25 % quartile values) and the coarse boundary of the soils of the 175 - 200 range (the 75% quartile values) were used to set a smooth, fine, and coarse boundary of the final bandwidth. This final bandwidth is the one used for the design of the drain envelope. The bandwidth ratios of the final boundaries in Figure 4 range between 2.3 at d_{100} and 4.3 at d_{60}.

2.5 Determination of drainage discharge, drain depth and drain spacing.

The drainage coefficient, drain depth and drain spacing must be determined prior to using certain drain envelope need determination methodologies. In theory, once these have been determined and the system designed, one can estimate the maximum discharge that would cause the highest gradients at the soil-envelope interface. While details of these design items go way beyond the scope of this book, some general and helpful descriptions can be found in Figure 5 and Box 4.

The drainage coefficient is the result of a comprehensive water balance determination. It can be defined as the amount of water that will recharge to the

Box 4 Components of a water balance.

Net Recharge *is the amount of water that causes the water table to rise or drop. It is the result of all factors affecting the water table behaviour, including the effect of existing drainage systems and subsurface in- and outflow of the area of interest.*
Drainable Surplus *is the amount of water that must be removed from an area within a certain period to avoid an unacceptable rise in the levels of groundwater or surface water. Vertical or horizontal subsurface drainage systems or surface drainage systems might remove this surplus.*
Drainage Coefficient *is the discharge of a particular (subsurface) drainage system, expressed as a depth of water that must be removed within a certain time. It is usually the* ***Design Discharge.*** *Its magnitude will depend on the* ***Dewatering Criterion,*** *which is the period in which the drainable surplus needs to be removed (typically 7 days in humid climates, and 3 - 5 days in semi-arid regions).*
Drainage Discharge *is the actual volume of water removed by a drainage system expressed as a rate. This may be higher or lower than the Drainage Coefficient. Drainage discharge can be used to determine design drainage coefficients and drainable surplus.*

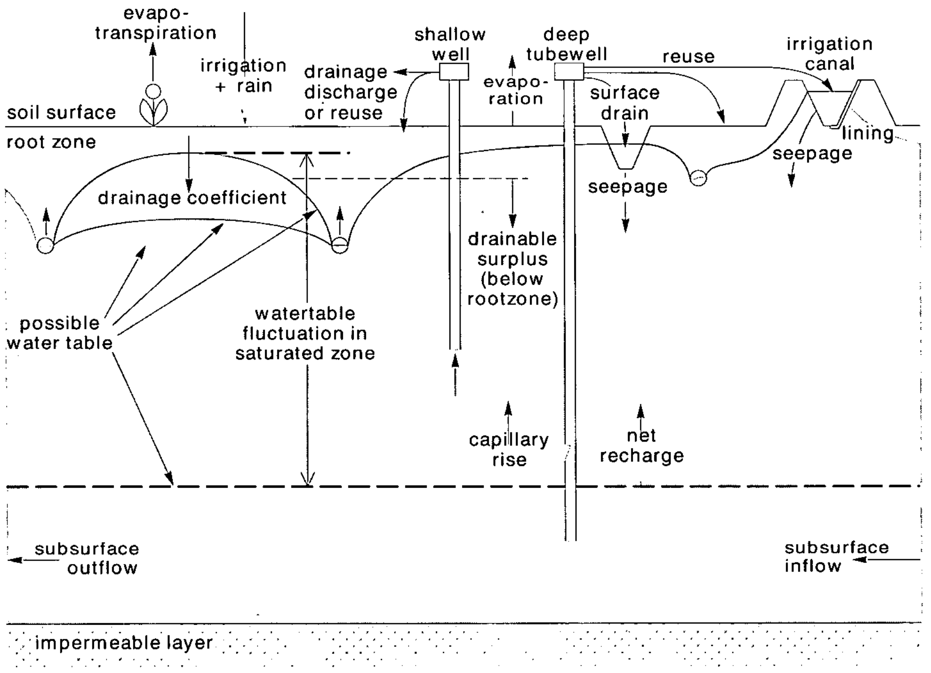


Figure 5 Schema of water balance components in unconfined aquifer.

ground water in a certain period, or the amount of water that will drain from the rootzone over that period. All factors such as evaporation directly from the soil, evapotranspiration from the vegetation, rainfall, irrigation, subsurface inflow into the area of interest, and pumping from existing systems should be taken into consideration. Some typical values of design drainage coefficients (q_d) are:

Humid climates	7 - 14 mm/d
Moderate climates	4 - 7 mm/d
Irrigated areas with some rainfall:	2 - 4 mm/d
Arid irrigated areas	1 - 2 mm/d

The second parameter that requires judgement before drain spacing can be determined is the water table depth between two drains. Some typical values to consider are:

Shallow rooting crops, humid climate	0.5 m with drains at 0.7 - 1 m depth
Grain crops, humid climate	0.7 m with drains at approx. 1 m depth
Food crops irrigated areas, no salinity danger	0.9 m with drains at 1.2 - 1.5 m depth
Arid irrigated areas, with salinity danger	0.9 - 1.5 m with drains at 1.2 - 3 m depth

As may be clear from the above listing, these guidelines are very, very, broad, and one should really check local experience before deciding on a combination of midway water table depth and drain depth. Main considerations to decide on depth are:

- typical crop root depth;
- number of days in which a certain water table depths is to be reached (dewatering criterion) after a recharge event (rain or irrigation);
- potential of secondary salinisation (salinisation due to evaporation or root extraction of water from the soil) which is primarily a function of irrigation water quality, soil type, water table depth, evapotranspiration and land use; and
- availability of construction equipment.

The depth of the impermeable layer is another parameter to be considered, or rather one that needs to be known, before applying drain spacing equations. Although no approximate value can be given, in most drain spacing formulas, if beyond a certain depth the actual depth is larger, it will have a negligible effect on the spacing calculation. The depth of the permeable layer is used to calculate the equivalent depth for use in the Hooghoudt equation. The equivalent depth takes into account the radial flow resistance near drains in deep homogeneous soils, and becomes approximately constant when the depth to the impermeable layer is approximately one quarter of the drain spacing ($D = \pm \frac{1}{4} S$) or greater.

To decide on the appropriate hydraulic conductivity to be used in the drain spacing equation, a map with contour lines of the hydraulic conductivity (K_s) should be prepared and areas with distinctly different values should be marked appropriately. If the area to be drained is fairly homogeneous the maps are not needed. Hydraulic conductivity should be divided into classes and for each of these classes the corresponding Plasticity Indices (PI), as well as the percentage clay are to be grouped and the average, standard deviation as well as the maximum and minimum values determined. The geometric mean of the measured K-values gives the best representation of the hydraulic conductivity for further drainage design (Van der Sluys and Dierickx 1986, Oosterbaan and Nijland 1994). The PI and % clay values are needed later when the various envelope need criteria are applied (Section 2.6).

If hydraulic conductivity values are not known they may be estimated from particle sizes (see Section 5.4). In that case it is necessary to have a map which indicates the areas represented by a certain particle size (the size of the particle for which 10 or 15 percent of the sample soil particles are smaller, d_{10} or d_{15}).

The relative importance of the above-mentioned parameters in drainage design is given in Box 5. When the spacing and depth have been selected, the subsurface drain discharge is determined from the design drainage coefficient (q_d), the drain length and the drain spacing, resulting in the design discharge (Q_d, Figure 6) for determination of pipe size. This discharge is used for determination of drain diameter after one or more safety factors have been applied (Cavelaars *et al.* 1994). The design discharge, however, is not the discharge we are interested in for the purpose of envelope need determination.

Field observations of drain discharges (either collector drains or lateral drains) will show large variations above and below the design discharge (q_d or Q_d). Lower discharges of the collector drain, for instance, can occur if not all of the area is draining into the collector, simultaneously. The same will happen on a smaller scale with laterals in irrigated areas, if not all of the lateral catchment is irrigated at the same time. Higher discharges can temporarily occur if the water table rises above the design water table depth, or if the drain spacing deviates (smaller) from the design. These higher discharges (Figure 6) are of interest to drain envelope design: they will expose the envelope to higher than normal design gradients.

The simplest way to determine the maximum expected discharge is to use the same drain spacing equation as was used for the design, but this time solve the equation for the discharge using the design drain depth and spacing, and assume that the midway water table depth is at the surface! This will be referred to as the possible maximum discharge (Q_{dmax}). Another way of deter-

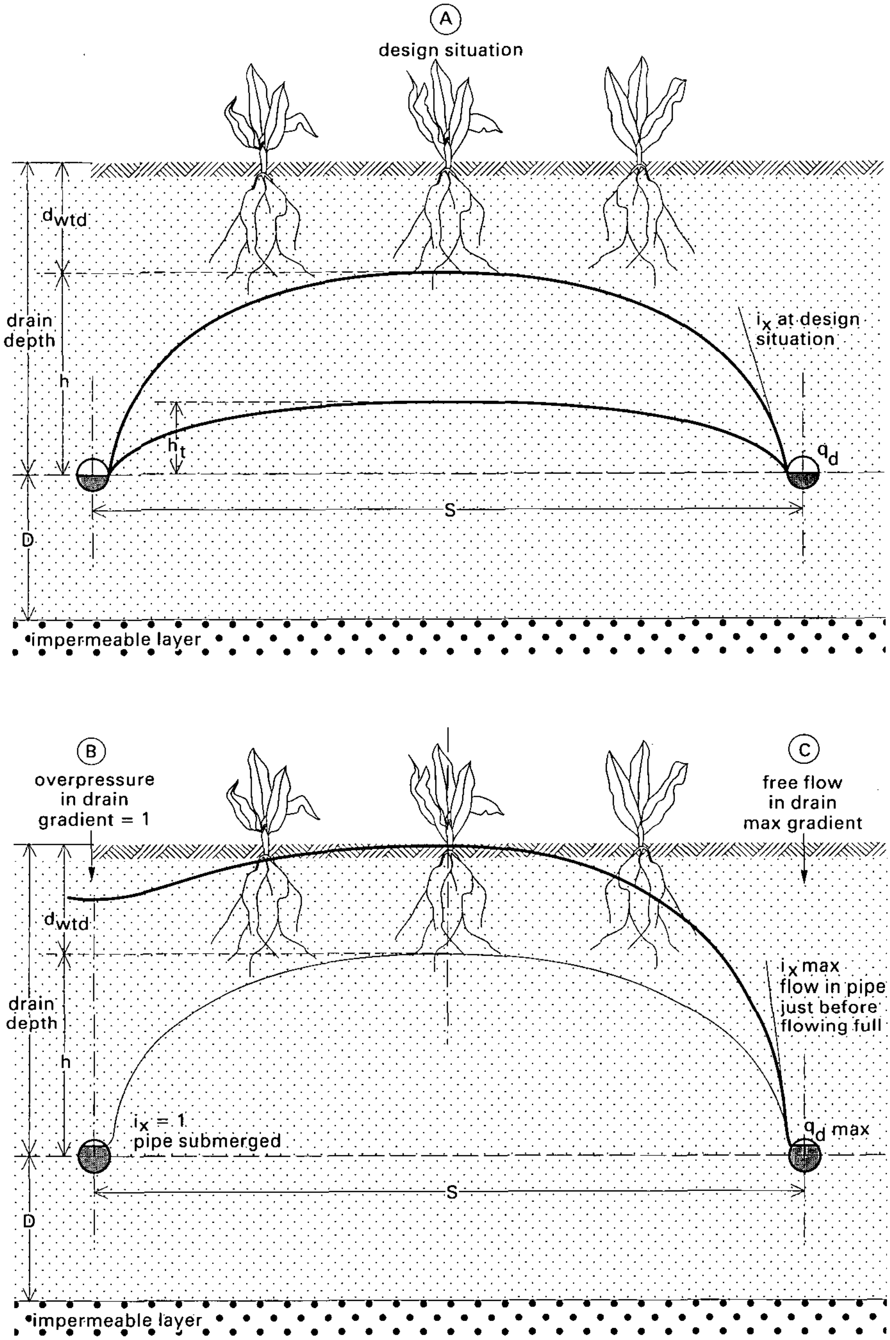


Figure 6 Possible discharges from subsurface drains.
A Typical design situation.
B Over-pressure in the drainpipe, maximum gradient at the pipe is one.
C Maximum gradient under free flow conditions in the pipe.

Box 5 Sensitivity of drainage design to certain parameters.

The spatial and temporal variability of soil properties and climatic conditions introduce a considerable amount of uncertainty in subsurface drainage design. Much work is necessary to incorporate the uncertainty of design parameters into the stochastic and deterministic analysis of drainage systems.
Wu and Chieng (1990) carried out a detailed sensitivity analysis of homogeneous and two-layered soils: sensitivity being the rate of change of the selected parameter with respect to the change in drain spacing. They concluded that in both steady and transient drainage design, the drain spacing is very sensitive to: 1. the midpoint water table height; 2. hydraulic conductivity; and, 3. the drainage coefficient. Drain spacing is not highly sensitive to the depth of the impermeable layer below drains and the drain radius.
In transient state design, the initial water table height and the water table drawdown over a given period are the most sensitive parameters in the drain spacing determination. The sensitivity analysis was executed using the following range of parameters: $q = 0.001 - 0.02$ m/d, $K = 0.1 - 6.0$ m/d, D or d_e (depth to permeable layer) = 0 - 10.0 m and h (the midway water table height) = 0.1 - 1.2 m.

Box 6 Standard drainage design books.

Skaggs, R.W. and van Schilfgaarde, J. (Eds.). 1999. Agricultural Drainage. No. 38 Agronomy Series. American Society of Agronomy (ASA), Crop Science Society of America (CSSA), Soil Science Society of America (SSSA), Madison, Wisconsin, USA. 1328 pp.

ASAE Standards 1997. Standards (S), Engineering Practices (EP), Data. See: S526.1, Mar 95, EP479 Dec 95, EP 407.1 Dec 95, EP 369.1, Dec 94, EP302.4, Aug 93, EP463.1 Nov 95, EP260.4 Dec 96.

FAO. Irrigation and Drainage Papers numbers:
9, 1972. Drainage Materials
15, 1973. Drainage Machinery
28, 1976. Drainage Testing
38, 1979. Drainage Design Factors
51, 1995. Prospects for the Drainage of Clay Soils
55, 1996. Control of Water Pollution from Agriculture

Luthin, J.N. (Ed.). 1957. Drainage of Agricultural Lands. American Society of Agronomy (ASA), Agronomy Vol. No. 7. Madison, Wisconsin, USA, 620 pp.

Ochs, W.J. and Bishay, B.G. 1992. Drainage Guidelines. World Bank Technical Paper No. 195, World Bank, Washington D.C., USA, 186 pp.

Ritzema, H.P. (Ed.). 1994. Drainage Principles and Applications. ILRI Publication 16, 2nd Ed. Wageningen, the Netherlands. 1125 pp.

SCS 1971. National Engineering Handbook, Section 16: Drainage of Agricultural Land, USDA-SCS, Washington DC, USA.

Smedema, L.K. and D.W. Rycroft 1983. Land Drainage: Planning and Design of Agricultural Drainage Systems. Batsford, London, 376 pp.

USBR. 1993. Drainage Manual. (Revised Reprint). United States Department of the Interior, Bureau of Reclamation. Denver Colorado, USA, 321 pp.

van Schilfgaarde J. (Ed.). 1974. Drainage for Agriculture. Agronomy Vol. No. 17, American Society of Agronomy. Madison, Wisconsin, USA, 700 pp.

mining a possible maximum discharge is by calculating the full flow capacity of the drain using the Manning equation[3]. The disadvantage with this method is that it does not include the potential restricting effect of flow towards the drain in soils with low hydraulic conductivity.

The design of the drainage system, including the layout, hydraulic capacities and the drain envelope, is an iterative procedure. For instance, to determine the need for the drain envelope decisions on spacing, depth and equivalent depth need to be made (for details see references in Box 6). During the drain spacing determination a pipe diameter is assumed, but the actual one may not be known until the pipe diameter has been determined based on design discharge. Drain discharge depends on spacing and the drainage coefficient. Therefore, educated guesses are made during the first calculations, but these need to be checked when finalising of the overall design.

2.6 Determining the need for an envelope

With the assumption that the various soil tests and other parameters such as tentative drain spacing, drain diameters, and depth to barrier are available, the process of determining the need for a drain envelope can begin as described in this section (steps 1, 2 and 3 in Figure 1). Some general considerations of the various components of this process are given, followed by more detailed explanations (Figure 7).

The first indicator of the need for a drain envelope is the percentage of clay in the soil. From experience in the Netherlands, Egypt and India, a clay percentage higher than 40% would most likely indicate that there is no need for a filtering drain envelope (Figure 7). The safe clay content was actually found to be closer to 17.5% in the Netherlands (Section 6.1.3) but safety factors resulted in a recommendation of clay > 25% at drain depth[4].

A recent report (Rajad Project staff 1995) indicated that unfavourable combinations of the SAR in the soil with certain electrical conductivity values of irrigation water could lead to dispersion of clay particles at drain depth. This was found in soils with clay percentages of up to 40% and hence this factor was built into the flowchart for drain envelope need (Figure 7).

3 USBR has measured flows up to 1.2 times the maximum, computed by the Manning equation, when there was a head of water above the pipe.

4 Table 31 (p 256) mentions that an envelope is recommended if soil layers above drain depth have less than 25% clay.

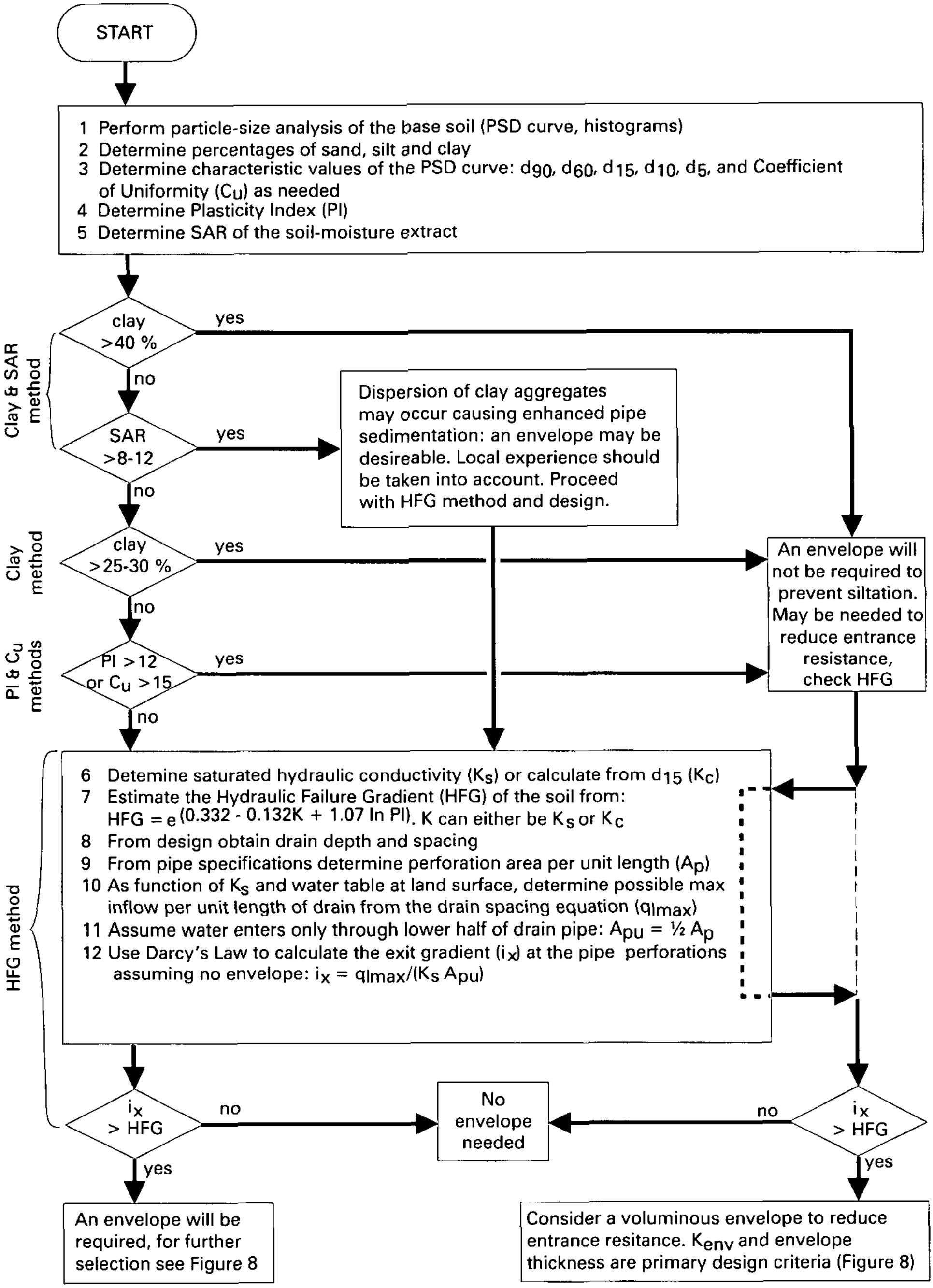


Figure 7 Flow chart to determine the need for an envelope.

Closely related to the clay percentage indicator is the use of the Plasticity Index (PI) method (Dieleman and Trafford, 1976), this in itself is an indicator of soil stability. It was found that when the PI > 12 a filter for the purpose of retaining fine particles was not necessary. The same source reports the use of the Coefficient of Uniformity (C_u) also as a possible indicator; when $C_u > 15$ there is no need for a filtering envelope. This last indicator cannot be used in many cases, as most agricultural soils do not have such high C_u values. Nevertheless, it is a parameter that is readily available when particle size distribution analysis has been performed, and hence it can be considered in the overall assessment.

When the representative saturated hydraulic conductivity of the soils is between 0.02 m/d and 4.5 m/d, the Hydraulic Failure Gradient (HFG) method is useful for determining the envelope need as well as the selection of the type of drain envelope. At present the HFG method has really only been proven for Utah and Michigan soils (Figure 43), although research in Egypt is underway to test the applicability to Nile Delta soils. The uniformity coefficients of Utah soils are generally higher than most other soils that have been tested for drain envelopes or in which envelopes have been applied (Figure 43, p 148, Figure 79 and 80, p 259, 262). Since the HFG method may be the only one that is globally applicable it is recommended for use as one of the guidelines to determine the need for envelopes. HFG is compared with the expected exit gradient ix at the pipe-soil and the envelope-soil interfaces: a smaller HFG of the soil indicates that an envelope is needed.

The HFG method is applied as follows:

1. Determine the possible maximum discharge (Q_{dmax}) as function of the hydraulic conductivity of the soil (K_s), the midpoint water table height above drain level (H_{max}) taken at the land surface (so not the design water table depth but rather the maximum head above the drain, which in many cases means equal to drain depth, that will cause the maximum gradient, i_{max} shown in Figure 6), and the drain spacing (S) from the Hooghoudt equation (with equivalent depth) or other drain spacing equation, e.g.:

$$Q_{dmax} = q_{dmax} * S * L = \frac{8K_1 d_e(h + d_{wtd}) + 4K_2(h + d_{wtd})^2}{S^2} S * L \qquad \text{Eq. 3}$$

where,

Q_{dmax}	is the maximum possible discharge under free flow conditions in the drainpipe (m^3/d);
q_{dmax}	the maximum possible discharge per unit area (m/d);
S	drain spacing (m);
L	drain length (m);

d_e	equivalent depth as function of depth to impermeable layer (D), S and the outside pipe radius (m), Eq. 6 see Box 7;
h	design head (m) see Figure 6;
d_{wtd}	design water table depth below surface (m);
H_{max}	$= h + d_{wtd}$ drain depth (m);
K_1	hydraulic conductivity of soil layer below the drain (m/d), usually K_s; and
K_2	hydraulic conductivity of soil layer above the drain (m/d), usually K_s.

2. Calculate the exit gradient either at the soil-pipe interface (no envelope) or the soil-envelope interface, using appropriate opening size and permeability coefficients, assuming the validity of the Darcy equation for the situation (Sections 5.1.3 and 5.1.4). For example, the discharge per unit length is:

$$q_{lmax} = q_{dmax} * S = \frac{Q_{dmax}}{L} \qquad \text{Eq. 4}$$

where,

q_{lmax} in $m^3d^{-1}m^{-1} = m^2/d$, and

$$i_x = \frac{q_{lmax}}{K_2A_{pu}} \qquad \text{Eq. 5}$$

where,

K_2	may be taken as K_s at drain depth; and
A_{pu}	actual area of inflow into the drainpipe (m^2/m, section 5.1.4).

3. Calculate the HFG using Eq. 34 (Section 5.3). The HFG formula should only be used if $0.02 < K_s < 4.5$ m/d. If K_s is outside this range, use the other criteria (such as percent clay, PI, C_u and local field experience to determine the envelope need.

4. Compare the HFG and i_x. If $i_x >$ HFG then an envelope will be needed.

Note, even if a filtering envelope is not needed (Figure 7), it is still advisable to calculate the exit gradient and compare it with the HFG. If the exit gradients are high, this indicates high entrance resistance, which is not desirable (see Section 5.1). Here, HFG is not used as a measure of stability (actually K_s is probably smaller than the 0.02 m/d) but merely as a boundary to limit exit gradient and the dependent entrance head loss. Close perusal of the calculation of the exit gradient will show that the exit gradient is independent of K_s in the methodology outlined above. This is because K_s is used to determine the maximum drain discharge, and subsequently the same value is used again in the Darcy equation; eliminating the K_s in the final determination of the exit gradient. For reasons of clarity the K_s has been left in the methodology, since it is needed to determine HFG. Rather than using the HFG as the limit for acceptable gradients, the ratio of entrance head loss to the midway head loss has been suggested as a possible indicator. However, as this indicator is so dependent on site (spacing, drain depth) and location (country preferences), we suggest not using it.

Box 7 Method to determine equivalent depth.

The equivalent depth can be calculated from the following formulas (Ritzema 1994):

$$d_e = \frac{\dfrac{\pi S}{8}}{\dfrac{S}{\pi r_o} \ln \qquad + F(x)} \qquad \textit{Eq. 6}$$

where,

$$x = \frac{2\pi D}{S} \qquad \textit{Eq. 7}$$

and

$$F(x) = \sum_{n=1}^{\infty} \ln \coth(nx) \qquad \textit{Eq. 8}$$

where,

- *ro* — *the equivalent radius of the drain trench in m,* $= \dfrac{u}{\pi}$
- *u* — *wetted perimeter in m, for pipes in trenches: b + 2 r*
- *b* — *width of the trench in m.*
- *x* — *Eq 7*

The above solutions are based on a series solution of the Hooghoudt equation. Traditionally, tables were used before calculators included the coth(x) function. The method assumes drains running half-full and no entrance resistance and water entrance area equal to the wetted perimeter.

From the above it would appear that there are five indicator methods for assessing the need for a drain envelope: the clay and SAR method; the clay percentage method; the PI method; the C_u method; and the HFG method. In Figure 7 these methods have been aligned sequentially, so that once a method has been used it should not be necessary to backtrack. This, however, needs verification as the methods have never before been presented together (Table 4). In particular, the clay and SAR method and the position of the PI and C_u methods at the end of the line might raise questions.

Table 4 Limitations of methods to assess envelope need.

Clay and SAR method:	Reported since 1995 (Rajad Project staff 1995) but not much field data available in the literature. SAR value > 8- 12 may occur with clay % > 40% or clay % < 25%. Nothing is reported on these ranges. Also electrical conductivity of the soil played a role, but is not further specified.
Clay method:	Few hard data available and results reported from the Netherlands and Egypt are partially based on personal (albeit long-term) experience of authors referenced in articles. No data sets since early to mid-1980s presented in the literature that show unambiguous boundary values. In arid climates soils are less stable than in humid climates and clay content alone may not be enough as indicator of soil stability → HFG method and inclusion of SAR in the judgements.
PI method:	No limitation reported, little actual field data available in the literature. Since 1957 (Sherard 1957) and 1976 (Dieleman and Trafford, FAO 28).
C_u method:	Few agricultural soils seem to have C_u values greater than 15. C_u values are generally not reported in the literature in this context; little data presented since 1976 (FAO 28).
HFG method:	Only tested in Utah and Michigan (USA) with $0.02 < K_s < 4.5$ m/d.

3. Material selection and design of envelope

Steps 1 - 3 of Figure 1 have now been completed and it has been determined that an envelope is needed. Before proceeding to the actual design of the envelope as far as its required functions are concerned (filtering, hydraulic, mechanical, bedding), step 4 should be performed comprising a number of considerations:

- the selection of the material as function of availability (cost being a main factor);
- whether or not the envelope should act as filter or any of the other functions or a combination thereof;
- the conditions that the envelope material will be exposed to during transport and placement;
- the environment in which the envelope will function; and
- whether a thick (voluminous) or thin envelope should be used.

In the following sections of Chapter 3 a description is given of the steps that will lead to a successful selection of an appropriate envelope material and detailed design once the envelope material is selected. The key elements of the complete design process are given in:

1. Figure 1 Steps in drain envelope design.
2. Figure 7 Flow chart to determine the need for an envelope.
3. Figure 8 Flow chart for selection of voluminous or thin envelopes.
4. Section 3.3 Granular Envelopes.
5. Section 3.4 Organic Envelopes (Table 5 and 6).
6. Section 3.5 Synthetic Envelopes.

Additional information may be found at the following locations elsewhere in the book:

7. Box 10 Summary of desirable properties of synthetic materials for drain envelopes.
8. Figure 9 Typical open area of water entry per unit length of pipe for different envelope materials.
9. Figure 58 Compressibility of geotextiles as function of load.

One may also wish to consult the following:

10. Table 34 Overview of existing design criteria for the use of sand and gravel around drainpipes.
11. Table 35 Existing filter criteria for geotextiles.
12. Table 27 Relationships between different pore sizes for non-woven geotextiles.
13. Figure 74 Unified Soil Classification Chart.

3.1 Selection of type of material for the envelope

Based on the material used the following types of envelopes can be distinguished:

- Granular envelopes. Gravel or sand/gravel combinations.
- Organic envelopes. A wide range of organic materials have been used in the past, such as peat, top soil, sod, building paper, hay, straw, corn cobs, cloth, burlap, leather, wood chips, etc. More recently, wire coir or coconut fibre, have primarily been used. Coconut fibre combined with synthetic fibres are used too. In Europe, Pre-wrapped Loose Material (PLM) are commonly used.
- Fabric envelopes. In recent years, wide ranges of (synthetic) fabric material have been used as drain envelopes (besides limited use of natural fabrics like jute and cotton). These are the above-mentioned PLM envelopes with synthetic fibres only, thin knitted materials (also known as socks, or available as sheet material), and a variety of non-woven materials, mostly thin to thick needle punched.

3.1.1 Environmental conditions

To select the type of envelope material a set of general conditions should be checked first:

1. the availability of materials, and hence the likely cost;
2. the expected function: hydraulic, filter, mechanical, bedding;
3. loading on the pipe and envelope;
4. handling characteristics during transport and transportation;
5. danger of biochemical fouling (iron ochre);
6. ripening process of the soil;
7. organic matter and pH of the soil;
8. calcium carbonate content (of soil and granular envelope) and pH of the water; and
9. the climatic conditions.

ad.1. First of all a list of the available materials must be obtained, including the source, and the distance from the source to the construction site. Most regions in which drainage projects are executed will have ready access to some form of granular material, which may or may not be suitable for a drain envelope. Even where granular material is readily available, transportation and handling costs can easily result in higher total envelope costs than when imported synthetic materials are used (see Chapter 4). Recent experience has shown that in most cases synthetic materials are cheaper than gravel and sand.

ad.2. To effectively improve the hydraulic function, a voluminous envelope is generally necessary. The Hydraulic Failure Gradient (HFG) determination and estimation of the gradient at the pipe-envelope and the envelope-soil interface (Figure 8) can assist with the determination of the required thickness. Several example calculations are given in Section 5.3.2, while the steps are shown as a flow chart in Figure 8. Based on the perforations and their arrangement along the pipe circumference, it may be necessary to create flow along the pipe (in case of perforations alternately in parallel corrugations, Figure 15). Longitudinal flow is readily achieved in granular filters, but with synthetic envelope materials due attention should be paid to the flow characteristics in the plane of the material (Sections 5.1 and 5.6.8). The relative openness of a particular envelope material is shown in Figure 9. To accommodate the filter function, thin synthetic filter fabrics might be considered in addition to the traditionally used gravel filters. To avoid possible clogging and loss of permeability of these thin materials special criteria is given in Section 3.5. For the mechanical function only granular material will suffice. For bedding any gravel material will generally be acceptable.

ad.3. Loading conditions are to be considered for several reasons: (1) gravel might be selected to hold the pipe down immediately following construction when uplifting water forces are expected; (2) gravel could be selected to enhance the strength of the pipe-envelope combination when excessive loads are expected (for instance in unstable soils where soil wedges of collapsing trench walls may damage the pipe); and (3) where loading plays a role with the compressibility of the potential synthetic/organic envelope material (Section 5.6.6). The required thickness as determined with the HFG method should be achieved under compressed conditions. Actual loads in the field (Box 8) may range from 15 kPa to 40 kPa when a drainpipe is at a depth of between 0.75 m and 2.0 m (Figure 58, p 197). Standard testing of voluminous materials is done at 2 kPa (equivalent to the pressure of a wet, saturated, soil at an approximate depth of 0.1 m).

ad.4. Proper handling of envelopes during transport and placement is critical for both granular and non-granular fabric envelopes. For granular envelopes, guidelines to prevent segregation are necessary, while for synthetic envelopes strength criteria are important (puncture strength, tensile strength, etc.). Gaps in overlaps or seams can be prevented when stitching is done properly and the strength of the seam is adequate (see Sections 4.2 - 4.4 and 5.6.4 - 5.6.7).

ad.5. Thin synthetic envelopes should not be used in areas where problems with ochre can be expected. In the Netherlands, pipes with perforation widths ranging from 1.4 to 2 mm are recommended in such situations[4].

4

Box 8 Determining load on drainpipe and envelope.

The load on the drain and hence the envelope may be calculated from (ASAE 1997, Luthin et al. 1968, Manson 1957[5]):

$$W_c = C_d \rho_s g B_c B_d \qquad \textit{Eq. 9}$$

where,

W_c the pipe load in N/m

C_d the load coefficient, dimensionless. Typical values as a function of the ratio H/B_d are:
$C_d = A \ln (H/B_d) + B$
where,
H is the head above the top of the pipe, and for A and B the following values may be used

damp top soil and dry wet sand:	$A = 1.3$	$B = 0.94$
saturated top soil:	$A = 1.08$	$B = 0.98$
damp yellow clay:	$A = 0.95$	$B = 0.99$
saturated yellow clay:	$A = 0.83$	$B = 0.99$

ρ_s the bulk density of saturated soil in kg/m^3. Typically $\pm$ 2000 kg/m^3

g gravity acceleration in m/s^2 (9.81 m/s^2)

B_c the pipe width in m

B_d the trench width in m

Box 9 gives additional information for dealing with situations where iron ochre is a problem;

ad.6. Soils that are have recently been reclaimed and have not yet ripened (oxidised) have a low hydraulic conductivity ($K_s \leq 0.1$ m/d). Therefore, voluminous envelopes (thickness > 2 mm without load) are recommended. Since this might be a temporary situation until the soil has ripened as a result of oxidation and biological activity, and since hydraulic conductivity could increase 100-fold in 1 to 1.5 years, voluminous organic materials are suitable, provided that soils do not become unstable after ripening;

ad.7. Deterioration of organic envelope material is enhanced when there is much organic matter (humus) in the soil, which will stimulate biological activity and affect the lifetime of an organic envelope. This could be particularly true in areas with peat soils. Oxidised remains readily clog thin envelopes - as was experienced in the Netherlands - so larger pipe perforations are recommended[6]. Organic matter in clay soils with high pH will deteriorate faster and therefore organic envelopes will not last long.

ad.8. Soils rich in calcium cause more rapid deterioration of organic matter, hence organic envelopes are less suitable. A quick test can be performed by applying a few drops of a 10% HCl solution; if no visible or audible

[5] Manson used B_d^2 in Eq. 3 rather than $B_c B_d$.

[6] Class B of NEN 7036, which prescribes perforation size between 1.4 - 2 mm width: Dutch standard on corrugated PVC drainpipes.

reaction takes place the soil is low in calcium. The calcium of the soil in combination with a pH < 5.8 may cause limestone deposits to affect the functioning of fine filters. However, solubility of gypsum (a different form of calcium in the soil) is not a function of pH, and will not cause additional movement of calcium in the soil (Section 5.5.5);

ad.9. Organic envelopes last longer in temperate than in tropical climates. Synthetic envelope material (PVC more than PE) exposed for extended periods of time to sunlight, deteriorates. However, hot weather and an abundance of sunlight should not preclude the use of certain synthetics; protective measures during storage are necessary.

Box 9 Dealing with iron ochre during design, operation and maintenance of pipe drains.

When the possibility of iron ochre problems[7] has been identified the following options to prevent and/or reduce the impact are available to the designer:

1. *Include easy pipeline flushing features in the design.*
2. *Do not use thin woven synthetics, or fibre glass or similar materials with small openings.*
3. *Use drainpipes with maximum allowable perforation size (i.e. 1.4 - 2 mm in the Netherlands, Type B pipe).*
4. *Apply materials containing copper in or around the drain as copper is bactericidal, but it is not an environmentally sound solution and therefore not recommended.*
5. *Apply tannic acid to the envelope material (see Section 4.5). This is however a temporary solution with also possible negative environmental side effects, and is not recommended.*
6. *Construct the drains deep so they are submerged most of the time. Although in theory this should prevent oxidation, in practice it was found to work only partially in the Netherlands (Scholten 1989), the USA and Canada (McKyes et al. 1992). See Section 5.5.5 for details.*

Finally, with drains in situ, aeration of the soil by deep ploughing or by constructing mole drains has been tried but met with limited success, so considered only a temporary solution. The idea was to stimulate oxidation and deposit iron in the soil before it reaches the drain.

3.1.2 Required envelope thickness

Once the nine factors above have been considered, an idea might have been formed about which type of envelope is desirable. Now the material thickness should be considered. The required thickness of the envelope could play a role in the selection of the type of material (synthetic thin or voluminous, or granular natural material).

To create the most favourable hydraulic condition at the soil-envelope interface, namely, the lowest possible gradient that is lower than the HFG, further consideration of the exit gradient at the soil-envelope interface is helpful (Figure 8, Steps 13 – 17):

[7] Apart from iron ochre problems, bio-chemical problems can also relate to sulphur and maganese reducing bacteria and their corresponding deposits (see Section 4.5).

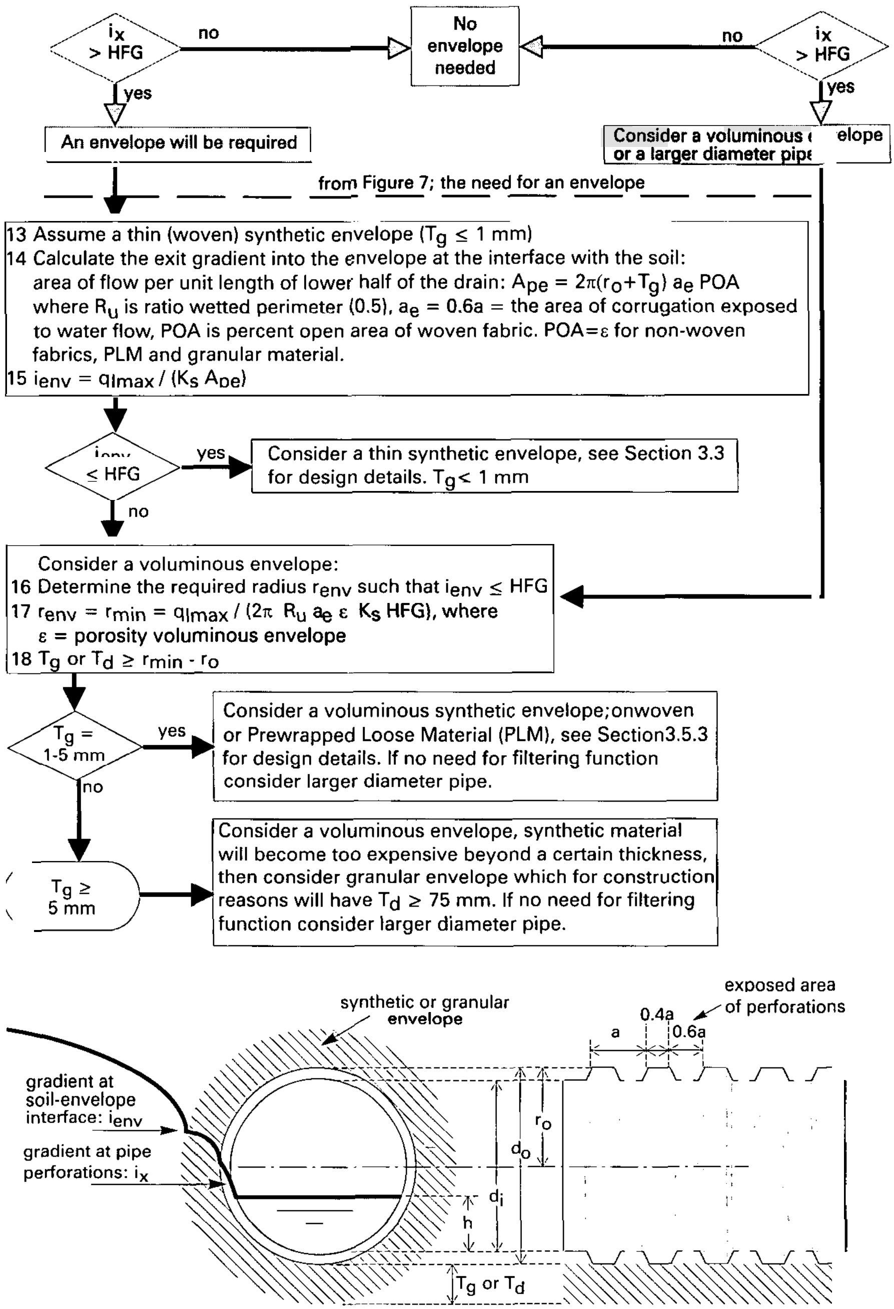
1. Assume that it was already determined that i_x > HFG (steps 1-12, Figure 7, implying an envelope is needed) and that selecting the next larger pipe size is not economical (should be checked again after the following calculations have been made). Even when a thin envelope is applied the ix will already be reduced simply because of the larger open area (Figure 9). Calculate the new exit gradient at the soil-envelope interface (i_{env}), using (Figure 8, steps 13 – 15):
   - the appropriate envelope porosity or percent open area (POA) obtained from published data or test results (Sections 5.1.4, 5.6.8 and 5.6.9);
   - the possible maximum discharge (as determined in Section 2.6); and
   - the drain spacing and the soil permeability coefficient (K_s or K_c when calculated from particle size distribution).

   Assume the validity of the Darcy equation for the situation at the soil-envelope interface (Sections 5.1.3 and 5.1.4). Darcy's equation is valid for laminar flow conditions. When flow is turbulent or in transition between laminar and turbulent, head losses will be higher and likewise the exit gradient for the same discharge.
   At this stage whether or not a thick envelope will be necessary will not be known, so assume a thin woven synthetic envelope, which is less favourable than some of the other materials of 1 mm thickness as shown in Figure 9. Calculate A_{pe} (step 14, Figure 8).

2. If i_{env} < HFG then a thin envelope may be considered.

3. If i_{env} > HFG then a voluminous envelope needs to be considered, and the required thickness can be determined (steps 15-16, Figure 8). The HFG is the maximum allowable exit gradient and the minimum radius required can be calculated to achieve this (Figure 8, item 17). Rather than using the POA for this calculation, the porosity (ε) of either the non-woven synthetic or the granular envelope needs to be considered (Sections 5.1.4, 5.6.8). Determine the minimum envelope thickness from the difference between the minimum radius and the outside radius of the drainpipe.

4. These are the options available:
   If the minimum thickness required is between 1 and 5 mm a voluminous synthetic envelope is the likely choice. This is the actual envelope thickness at drain depth and at the appropriate soil loading pressures. Depending on the synthetic material selected, use one of the compression ratios in Figure 58 (p 197) to determine the required thickness at 2 kPa. If no filtering function is required selecting a larger diameter pipe would reduce the exit gradient.
   If the minimum thickness is larger than 5 mm, then the costs of the synthetic material could be very high and the point at which a granular envelope becomes cheaper (see Chapter 4) can be determined. A granular enve-



A_p = area of perforations per unit length
A_{pu} = ½ A_p,or actual area as function of wetted perimeter (u)
A_{pe} = open area of envelope material
T_d = envelope thickness granular material
T_g = thickness of synthetic material at ground pressure at drain depth

Figure 8 Flow chart for selection of voluminous or thin envelopes.

lope will always have a minimum thickness of 75 mm for construction reasons, regardless of what the calculation shows. For a thickness between 5 and 75 mm a larger pipe diameter might be considered as an economical alternative if sedimentation in the drainpipe is not a problem.

We now know which envelope is preferred and whether or not a filter function is one of the requirements. The next sections will give details of finalising the design, such that the retention, hydraulic and miscellaneous criteria are met.

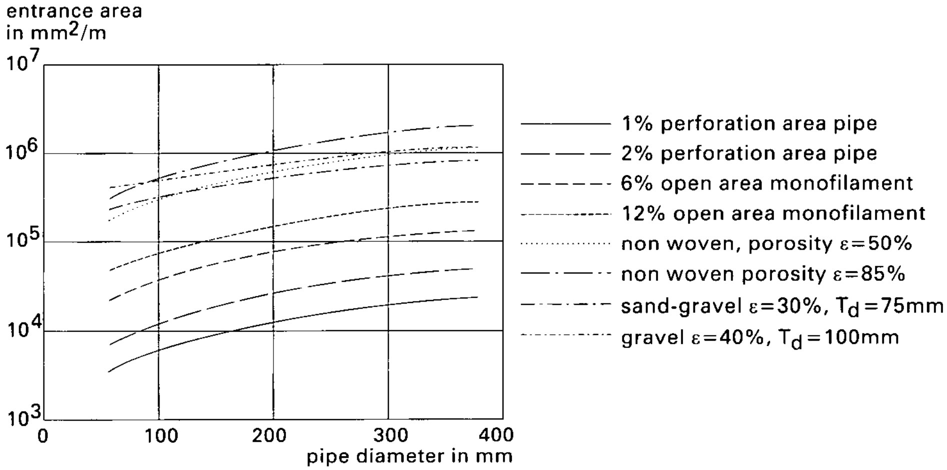


Figure 9 Typical open area of water entry per unit length of pipe for different envelope materials.

3.2 Method of a drain envelope design

There are two approaches for designing drain envelopes. One can either use laboratory indicator tests or use generalised criteria derived from laboratory and field observations. The difference is that laboratory indicator tests are performed for each material and situation considered, while generalised criteria give guidance without further laboratory tests. Both are briefly described in the next two sections.

3.2.1 Laboratory indicator tests

Indicator tests are performed in the laboratory and aim at simulating field conditions. Laboratory simulations and results are not often directly transferable to field conditions (Section 5.7). There are four primary tests that can be done:

1. Permittivity tests. These tests determine the hydraulic properties of the envelope material to be used. For granular material, the constant head and the falling head methods (Section 5.5.4) are the most common. No distinction is made between vertical and horizontal hydraulic conductivity as this is not relevant to disturbed soil, sand, and gravel analysis in the laboratory. For geotextile permittivity (hydraulic conductivity including material thickness) distinction is made in cross-plane permeability, with and without load, and in-plane permeability (also called transmissivity). The measurements determine water conveyance capabilities. When performed under standard conditions results can be compared with set values (see Sections 5.6.10 - 12);
2. The Gradient Ratio test (GR, Section 5.7.2). This test aims at measuring the clogging potential of the proposed material proposed and there are several standards available. The duration of the test is approximately 500 hrs. Standard guidelines for testing do not recommend indicator values. The GR test compares hydraulic gradient in the soil with a geotextile to that of the soil alone. An indicator often quoted is that GR_{25} should be < 3 for a geotextile to be acceptable, provided testing conditions are exactly as specified. Serious doubts have been raised about the applicability, reproducibility, and best testing procedure (Shi *et al.* 1994, Li *et al.* 1994 and Section 5.7.2);
3. The Long-Term Flow test (LTF) also investigates clogging potential, but specifically the gradual clogging over time, which can be a problem with some non-woven synthetics. The test could last up to 2000 hrs. No indicator values or specific indicator parameters have been reported and judgement is based on interpretation of graphs that show permeameter results over time. Tests of a shorter duration are under development (Section 5.7.1);
4. The Hydraulic Conductivity Ratio test (HCR) uses permeability ratios to assess the clogging potential (Section 5.7.3). The test has not been used for agricultural drainage conditions, hence the reported boundary value of HCR = 0.2 should be applied with caution. High values of HCR suggest soil loss through the fabric, low values indicate clogging of the envelope material, and intermediate values suggest soil-to-geotextile equilibrium.

From the brief descriptions above, and from the details given in Section 5.7, it might be apparent that laboratory indicator tests are far from straightforward solutions to questions about desirable soil-envelope combinations. For a number of years various researchers have been using permeameters for testing soil-drain envelope combinations (Dierickx and Yüncüoglu 1982, Stuyt 1992, Lennoz-Gratin 1987, 1992, Vlotman *et al.* 1990, DRI staff 1992, Koerner 1994, Li *et al.* 1994). These tests often combine the various aspects tested in the four indicator tests described above. Based on permeameter testing for research purposes an upward-flow permeameter is recommended as an indicator test for drain envelopes (Sections 5.7.4 and 5). However, as indicator tests are time-consuming they are only recommended when criteria - described in the following sections - do not result in satisfactory drain envelopes, or when anticipated field conditions are distinctly different from those on which the criteria were based.

3.2.2 Generalised design criteria

The second approach to designing drain envelopes is the use of generalised design criteria based on previous experience and as reported in the literature. Proposed criteria for design of filters for a variety of hydraulic structures, including in some cases agricultural drains are given in Chapter 6. All of the criteria rely heavily on the assessment of the base soil PSD curve.

Few of the generalised criteria were specifically derived for selection of agricultural drain envelopes. In particular, the retention and/or filter criteria are primarily based on those derived for other civil engineering applications of filters, such as bed and side slope protection, dams and hydraulic structures, silt fences, road drainage and vertical drainage. Nevertheless, the experience gained from those applications is readily adaptable for use with agricultural drains (Box 10 and 11). The main differences between filter or retention criteria between agricultural drain envelopes and other civil engineering applications are:

1. Hydraulic and retention criteria both need to be satisfied at the same time, while with other civil engineering works, one or the other may be dominant.
2. In civil engineering applications multiple step filters are common, but are not practical for agricultural drains.
3. For civil engineering applications the criteria need to cover the full range of possible soils, from heavy clays to coarse gravel. For agricultural applications heavy clays and coarse material generally do not need filters, hence criteria that cover these soil ranges are not needed;
4. Civil engineering applications of filters are normally constructed in building pits that are kept temporarily well drained, which allows for optimal construction conditions. This is usually not the case in agricultural drain construction.
5. For civil engineering applications, the soils that need to be protected can often be represented by a single PSD curve, or by a very narrow bandwidth. For agricultural conditions this bandwidth is usually much broader.

Of the recent publications containing design criteria for sand and gravel filters such as SCS 1994, ASCE 1994, and USBR 1993, and for geotextile filters such as in Koerner 1994 and Santvoort 1994, only ASCE 1994 and Santvoort

1994 present criteria specifically intended for horizontal subsurface drains. Perusal of the criteria, however, clearly shows reliance on earlier criteria presented for other civil engineering applications, with minor adjustments for use under agricultural conditions (Box 10). USBR 1993 recognises that most drains are constructed in the wet but recommends construction of the drains in the dry, if possible, whenever accurate placement of the drains can be determined before high water table problems develop. The SCS 1994 publication shows an example that requires a multiple step filtering envelope.

Box 10 Comparing and contrasting.

When studying the literature on drain envelope design eventually one will be struck by the observation that in granular drain envelope design particle sizes of the base soil are related to particle sizes of the envelope, while in fabric design the particle size of the base soil is compared with the opening size of the fabric.
The majority of fine particles need to be retained in place by the drain-envelope combination. The opening sizes in the envelope therefore need to be smaller than the smallest particle not likely to remain suspended in flowing water. If the opening size is larger than the soil particle, then the soil particles need to bridge across openings. It was found that in many cases the particles would bridge. Hence bridging plays an important role in envelope hydraulic resistance and stability.
As there is no direct means of measuring the pore size in granular filters a relationship has been determined between representative particle sizes of the granular envelope material and the expected characteristic pore size (See Sections 5.2 and 5.4 for more details). The basic principle is that the smaller particles of the granular filter are an indication of the characteristic diameter of the pores: they may be considered as being equivalent to an O_{85} - O_{95} *size of the pores. As with the soils and the granular envelopes the figures 85 or 95 refer to the percentage of pore sizes that have smaller openings than the* O_{85} *or the* O_{95}.
Therefore, when Terzaghi presented his original filter criterion in which he refers to the D_{15} *of the filter, he actually was referring to a characteristic pore size of the filter in the range* O_{85} - O_{95}. *Because bridging by smaller particles over the opening is likely, the actual pore size (opening) is allowed to be larger by ratios that vary from 4 - 7. For example: if* D_{15} *represents the* O_{85} *of the granular filter then Terzaghi's original criterion would read* $O_{85} < 4d_{85}$ *instead of* $D_{15} < 4d_{85}$.
The reason why D_{15} *and* d_{85} *are used is not given. It might imply that if soil particle sizes were normally distributed, sizes that are one standard deviation from the mean both smaller and larger are the* d_{16} *and* d_{84} *sizes respectively (see Figure 53, p98). The bridging ratio is determined based on theory and practice (Sections 5.2 and 6.2.1).*

Drainage of agricultural lands is proposed because water tables are high for prolonged times. Construction pumping to install drains with envelopes under dry conditions is likely to be costly and time-consuming. The arrival of trenching and trenchless techniques, where excavation, pipe laying and envelope placement are done in one procedure, allows successful construction of drains below the water table. This puts extra demands on the type of envelope used, and creates conditions that are hard to simulate in the laboratory. The generalised indicator criteria presented in the sections below are based on those presented in the literature (Box 14), and on recent specific experi-

Box 11 Which comes first, the envelope particle size or the base soil particle size?

The particle size of the granular envelope (or the opening size in case of organic and synthetic materials) is the dependent variable in relationships with base soils. Hence they should either be on the left side of the equation (equal sign or greater than sign), or in the enumerator of a ratio. For instance:

$$D_{15} < 4\ d_{85} \quad \text{or} \quad \frac{D_{15}}{d_{85}} < 4$$

The above formulation is recommended. Occasionally it was observed (in the literature) that inconsistent use of this notation led to specifying that the soil had to be smaller or greater than the envelope opening size. So, although $d_{85} > 0.25D_{15}$ is mathematically equivalent to the expression given above, d_{85} is generally not a function of the envelope material.

ences primarily in Pakistan, where adverse conditions in unstable soils put high demands on the retention properties of granular envelopes.

3.3 Granular envelopes

Design of a granular (sand-gravel) filter for use as a drain envelope is different from design of granular filters for hydraulic structures in that a drain envelope needs to satisfy both the demand for the filtering function and the demand for high permeability, simultaneously. This is not always easy and often designers will give preference to one or the other. For instance, one of the criteria will be relaxed to accommodate gradations as close as possible to those found in nature. This is quite acceptable if one carefully considers the consequences of such action. Crushing and mixing (or blending of materials from different sources) is acceptable provided some additional criteria are considered. Also, with crushed material, the perfect blend might be out of reach due to economic constraints.

Another consideration in proposing the best set of criteria for agricultural drain envelopes is the need for a one-step granular filter which can be placed with trenchers at the same time as the pipe: here, two- or three-step filters are not practical (Section 6.2.1). Trenchless techniques are generally not suitable for granular envelope construction. Two- and three-step filters are common with highway drainage, construction drainage and with bed and slope protection of hydraulic structures (dams, etc.); when a trencher is not used.

The most challenging part of a granular drain envelope design is meeting the conflicting criteria at the D_{15} particle size range. The prescriptions of D_{15} are primarily based on the $d_{85f(ine)}$ and $d_{15c(oarse)}$ sizes of the base soil bandwidth. Depending on the base soil bandwidth, one might find that hydraulic criteria based on the coarse boundary prescribes a size very close to the maximum size resulting from the filter criteria. The result is a very narrow impractical bandwidth at the 15% passing level. At the same time one may also find that bridging criteria (relating perforation width or diameter to a characteristic particle size) result in D_{15} sizes that are close to or larger than the maximum allowable size following from the filter criterion. The only way to resolve this is by using common sense, and by carefully weighing up the consequences of relaxing one or several of the criteria.

During the initial project stages, when base soil is analysed and materials for use as drain envelope are selected, it is important that all sieves indicated in a particular set are used for envelope selection rather than just 7 or 9 sieves deemed necessary to produce a semi-logarithmic gradation curve (21-sieve analysis in case of the US Standard Sieve Set, Table 15). The 21-sieve analysis will identify missing particle sizes, or ranges of particle sizes. These missing particles could be the result of sedimentation conditions in the past or the type of crushing machinery used. In Pakistan, a common indicator test, such as a 7-sieve analysis, was proved not sufficient to assess the cause of failure of gravel envelopes that occurred. Problems were overcome by: (1) assessing the potential envelope material with the full set of sieves (21 sieve analysis); (2) reducing the largest particle size allowed; and (3) relaxing the criterion for the amount of fines allowed in the envelope material. All these measures served the purpose of reducing the excessively high hydraulic conductivities of the original (failing) envelope material.

Once a suitable envelope has been designed using all 21 sieves, then during construction, for quality control, one can resort to using seven to nine sieves. It is assumed that at that moment a working granular envelope has been selected, and sieve analysis is then purely to check for segregation that may or may not have occurred during transport.

Segregation during transport and storage on-site should be prevented. To prevent segregation it is more important that certain ranges of particle sizes be present in the proposed material than individual particle sizes. Disallowing large particles (> 19 - 38 mm) will help in preventing segregation[8]. Various researchers showed that the particle size gradation curves of the envelope material do not need to be parallel to the base material particle size gradation curve, as long as the individual Particle Size Distribution curve stays within the selected bandwidth of the envelope material. Boundaries of the bandwidth are controlled by prescription of the Coefficient of Uniformity (C_u) for the boundary curves.

[8] Note that SCS (1994) prescribes a maximum D_{90} as function of D_{10} which ranges from 20 - 60 mm (Table 34).

Agricultural drainage conditions can be rather severe for short periods as is shown in Section 5.1. In particular, when the subsurface drainage system is a pumped system, high gradients can exist for short periods. Cyclic and reverse flow conditions will be more common in the future when there is greater demand for more flexible operation and management of subsurface drainage systems to save water and to lessen the impact of (poor) drainage water quality on the downstream environment.

When the drainage system is used for sub-irrigation, the envelope will be exposed to reverse flow conditions[9] which may destroy the arching (bridging) that might have occurred at the soil envelope interface. The natural filter that might have built up will be destroyed and fine particles are likely to collect at the envelope-base material interface. When flow into the drain starts again these particles will move back and with a proper filter envelope new arches and new natural filter will build up again. If a filter fails it will do so almost immediately after construction. Clogging over time is a far more gradual process that may take many years and is not common with granular envelopes.

Based on the foregoing and the conclusions presented in Sections 5.7.5 and 6.2 a mixture of guidelines and criteria are thought best for the design of gravel envelopes. This resulted in establishing control points on the Particle Size Distribution (PSD) curve through which the coarse and fine boundaries of a granular envelope bandwidth can be drawn. Gradation curve guides and bandwidth guides determine the recommended shape of the gradation band. The guides are intended to create a realistic gradation bandwidth, which can be implemented in practice. The guides are not criteria.

3.3.1 Control points for the coarse boundary of the envelope material bandwidth

The subscripts c and f denote coarse and fine boundary, respectively. Examples of the application of the control points are shown in Figure 11 and Figure 12.

1. $D_{15c} < 7^*d_{85f}$ Control point 1: the filter criterion d_{85} is that of the fine boundary of the base soil. Filters with a ratio greater than 9 always failed according to Sherard et al. (1984b). Retention criteria that specify that D_{15c} should be greater than or equal to (not less than) 0.6, 0.3 or 0.2 mm were mostly based on the fact that samples tested

[9] also a form of cyclic flow, a term which is more commonly used to describe condotions under wave actions.

did not have any smaller material, yet they worked well as a filter. Hence, there is no need to have smaller particles, but they are allowed provided the hydraulic criterion allows it (control point 4).

2. $D_{60c} = 5*D_{15c}$ — Control point 2, the gradation curve guide. Based on SCS (1994) guideline that $D_{10} = D_{15}/1.2$ and that $C_u = 6$.

3. $D_{100c} < 9.5$ mm — Control point 3, the segregation criterion. Particles larger than 9.5 mm (Sieve no. 3 of the Standard US sieve set) seemed to cause segregation and handling problems (flowability of granular material in trencher boxes, see Section 4.3). Boundaries as suggested by SCS 1994 are not deemed applicable for the typical agricultural soils. The 9.5 mm accommodates the boundary for crushed material suggested by Rehman (1995).

3.3.2 Control points for the fine boundary of the envelope material bandwidth

4a. $D_{15f} > 4*d_{15c}$ — Control point 4a, hydraulic criterion[10], d_{15c} is that of the coarse boundary of the base soil. Assuming that D_{15} and d_{15} are the primary characteristic particle sizes that control the hydraulic conductivity (pore space, see Section 5.4), this criteria assures that the D_{15} is larger than the d_{15} of the coarse boundary of the base soil bandwidth, such that the hydraulic conductivity of the envelope may be expected to be one magnitude bigger than that of the coarsest base soil. This criterion, however does not work well for soils with particles $d_{15c} >$ 0.09 mm (such as the Dutch, Canadian and UK soils in Figure 2 and 3). Prescribing a D_{15f} based on a practical bandwidth ratio (such as step 5 below) will provide a more realistic value, but the result will be a lower hydraulic conductivity of the envelope.

4b. $D_{15f} = D_{15c}/5$ — Control point 4b, bandwidth guide. Control point 1 (filter criterion) and control point 4a (hydraulic criterion) could possibly have conflicting results. Hence, control point 4b gives a control point based on the filter criterion. This control point is similar to control point 2 combining the control of the bandwidth and $C_u \leq 6$. If control point 4b > 4a use 4b. If 4b < 4a the decision on what

[10] when $D_{15} <$ factor $* d_{15}$ are mentioned it is a filter criterion else it is a hydraulic criterion!

to do will depend on the degree by which 4b is smaller than 4a. Use 4a if the bandwidth ratio remains within practical limits (i.e. check readily available material to see if they match the narrow bandwidth) and as long as $C_u > 2$. If $C_u < 2$ the material will be too uniformly graded preventing natural filter build-up under the adverse construction conditions often encountered at drain depth. Here, use 4b or something acceptable between 4b and 4a. This will result in a lower than desirable hydraulic conductivity, but may not be a problem if the perforation area of pipes selected is the maximum that will give the lowest possible entrance resistance. In the end it will be the engineer who decides on the final shape of the boundary of the gravel bandwidth. Examples of both situations are given later in the book.

5. $D_{5f} > 0.074$ mm — Control point 5, hydraulic criterion. This was originally intended to prevent a hydraulic conductivity that would be too low, but will also control the amount of fine sediment passing into the pipe of the envelope immediately after construction. The D_{5f} is not likely to bridge the perforation of the pipe. It is likely that this criterion conflicts if control points 4a and 4b are smaller than control point 5. This criterion can then be relaxed somewhat for very fine base soils when the final minimum D_{15f} is less than 0.074 mm (resulting from control point 4a, b or c) and when removing fines of the envelope material is too expensive. Control point 4c stems from the first bridging criterion mentioned in Section 3.3.5.

6. $D_{60f} = D_{60c}/5$ — Control point 6, bandwidth guide. The bandwidth of the specified envelope gradation should not exceed the ratio of 5 below the 60% passing level.

7. $D_{85} > D_{opening}$ — Control point 7, retention (bridging) criterion. This control point generally means $D_{85} > 2$ mm, and there is no difficulty satisfying this[11]. Only for very fine soils (soils in Egypt, Figure 2) when $d_{85} < 0.074$ mm will this criterion result in too much restriction above the 60% passing level (for further considerations see below).

[11] Most US manufacturers provide pipes up to 250 mm in diameter with 5 mm round holes and pipes > 250 mm with 10 mm round holes. The larger hole size is to meet the 1 sq. inch per foot open area criteria (see Box 15). It does compound envelope design (generally only for non-agricultural drains when these larger drain diameters are more common).

The foregoing steps and control points should result in an envelope bandwidth with a high likelihood of success, provided that the material has been checked for missing particle sizes. The material of the envelope bandwidth will have a higher hydraulic conductivity (K_{env}) than that of the base soil, but by how much is not quantified. It is more important to select the highest possible unit perforation area of the pipe, or the unit open area of the envelope to reduce entrance resistance (Figure 9). As far as the pipe is concerned, the prime concern when recommending a higher unit perforation area will be to maintain the pipe strength. We are unable to confirm that the commonly quoted 1- 2% open area of the pipe is based on hydraulic or pipe strength considerations. Perhaps it is a mixture of both.

3.3.3 Miscellaneous criteria

1. All openings should be covered by at least 75 mm (3") of granular filter material (construction criterion).
2. The envelope material should not contain deleterious materials, such as plant materials or soil.

3.3.4 Additional requirements for the use of crushed material

The use of crushed rock for granular envelope material has been acceptable in most cases, except when it failed to function as a filter with subsurface drains in Pakistan. Therefore, crushed materials are acceptable provided the following provisions are adhered to:

1. There should be no particles that are (disproportional) larger in one direction by a factor 2 of the shortest dimension. This is a long time requirement in specifications originating from USBR and the Corp of Engineers criteria. Its merit lies in that it prevents segregation and large pore spaces. Missing particle sizes in the crushed rock envelope material was one of the factors that caused envelope failure in Pakistan).
2. A statistically satisfying number of samples should be analysed from the crushing plant with the full set of US standard sieves (21-sieve analysis) to see whether any particle ranges might be missing. The missing particle ranges are not apparent as gap-graded material in standard semi-log PSD curves; histograms representing the results of sieving of the individual sieves should be produced (Figure 50).
3. The hydraulic conductivity of the crushed rock should be assessed in the laboratory using permeameters, and should remain below 300 m/d to be acceptable.

3.3.5 Optional bridging criteria

Lacking among the present criteria are direct bridging criteria. The main problem with bridging criteria is the decision about which representative particle size of the soil (d_{xx}) or envelope (D_{xx}) to use: d_{10}, d_{15}, d_{50}, or d_{85}. SCS (1994) uses D_{85} for non-critical situations (where surging or gradient reversal is not anticipated) and D_{15} for critical situations. For agricultural conditions, where reversal of flow can occur, it is advisable to follow critical guidelines. The diameter of the circular perforations or the maximum width of slotted perforations is given by $D_{opening}$. Bridging criteria should only be considered for the pipe-envelope or pipe-soil interface, or else use the retention criteria. Based on review of various bridging criteria (Section 5.2) the following control points for the fine boundary of the envelope bandwidth have been selected by us:

1. $D_{15} > 0.25*D_{opening}$ Control point 4c, retention (bridging) criterion. The ratio will vary (see two items below and Table 11, p 145) depending on the selected representative particle size of the envelope material (D_{15}, D_{50} or D_{90}). Based on perusal of Section 5.2 a factor of 0.25 is recommended when using D_{15}. However, although this presents a seemingly generous allowance, it will probably be found to be in conflict with control point 4a or 4b, because generally it will mean $D_{15} > 0.5$ mm (in case max. opening width is 2 mm). When unstable soils are encountered and filtering is critical using a granular envelope, select pipes with maximum width of the opening to be less than 2 mm (see Box 15, p 141).
2. $D_{50} > 0.5*D_{opening}$ Additional check/control point, retention (bridging) criterion, based on personal judgement on considering the description in Section 5.2. Except for very fine soils ($d_{85c} < 0.074$ mm), this criterion will result in a control point right in the middle of the envelope bandwidth resulting from the first six control points.
3. $D_{90} > 1.5*D_{opening}$ Additional check/control point, retention (bridging) criterion, based on the material presented in Section 5.2. It might be noted that the greater the characteristic particle size selected, the less likely the bridging will be a function of that particular particle size. Rather, most of the particle sizes will be smaller; hopefully they bridge. The only reason to include this questionable checkpoint is for comparison with the design criteria of synthetics, described in Section 3.5.3. Note however that control point 7 above is similar but allows finer envelope material to be used.

3.3.6 Final remarks on granular envelope design and examples

Gap-graded soils are not common in agricultural soils. Gap-grading is mainly a phenomenon in poorly graded gravel envelopes and only found in examples of US guidelines or as constructed laboratory soils (Figure 52, p 168, Figure 80, p 262). It never refers to the curves as a PSD curve based on an actual soil/gravel sample.

There are a number of acceptable standard aggregate gradations used for concrete and bituminous mixtures that can serve just as well for agricultural drainage envelopes. They are the fine aggregate gradations prescribed by the America Society for Testing of Materials in ASTM C33-93 and ASTM D-1073 (Figure 10). They may be readily available in countries where these standards are used. Care must be taken to assure there are no particle size gaps.

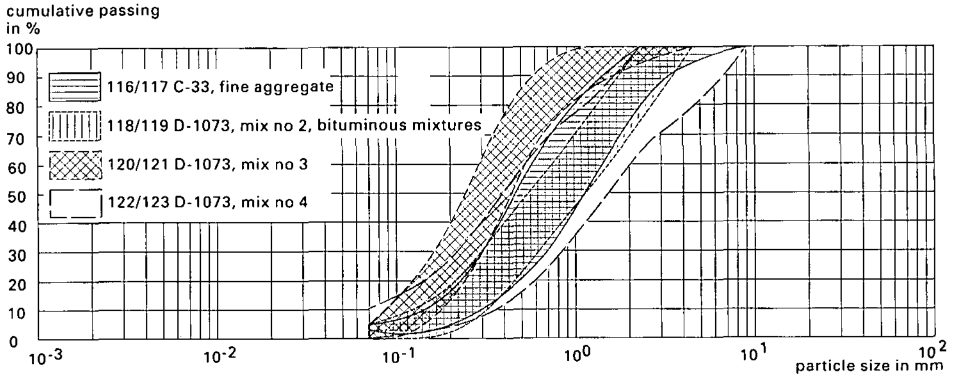


Figure 10 Standard aggregate gradations of ASTM appropriate for use as drain envelopes.

Example of granular envelope design with typical problem soils in Pakistan.

The soil bandwidth displayed in Figure 11 is synthesised for the various bandwidths shown in Figure 86 (p 299) and is therefore somewhat wider than would normally be encountered. It is perhaps wider than desirable, but as shown the control points result in an envelope range without too much conflict between the criteria. For the final curve control point 7 is somewhat relaxed and smaller particles are allowed. There is no major conflict with control points 4a and 4b.

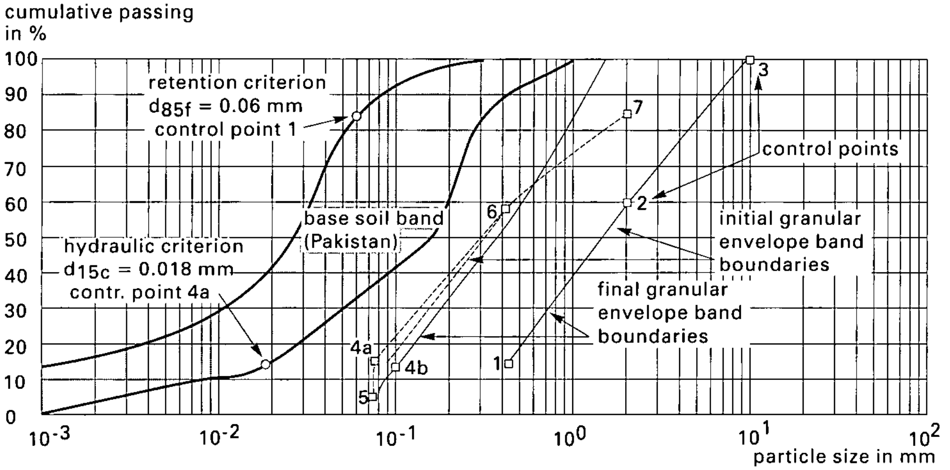


Figure 11 Base soil and envelope design in Pakistan.

Example of granular envelope design with UK problem soil.

The UK base soil band (Figure 12) gives rise to the typical conflict between control points 4a and 4b. The hydraulic criterion (control point 4a) results in a point very close to that of control point 1, the retention criteria. As it is more important to retain the fines, the hydraulic criterion is relaxed, i.e., smaller particles are allowed. Control point 4is the final selection of 4a, b or c. Control point 4, which coincides with the d_{15c} serves as a guide, together with control point 5. A curve is drawn that seems natural in shape, resulting in a final control point number 4 for the fine boundary somewhere in between 4a and 4b.

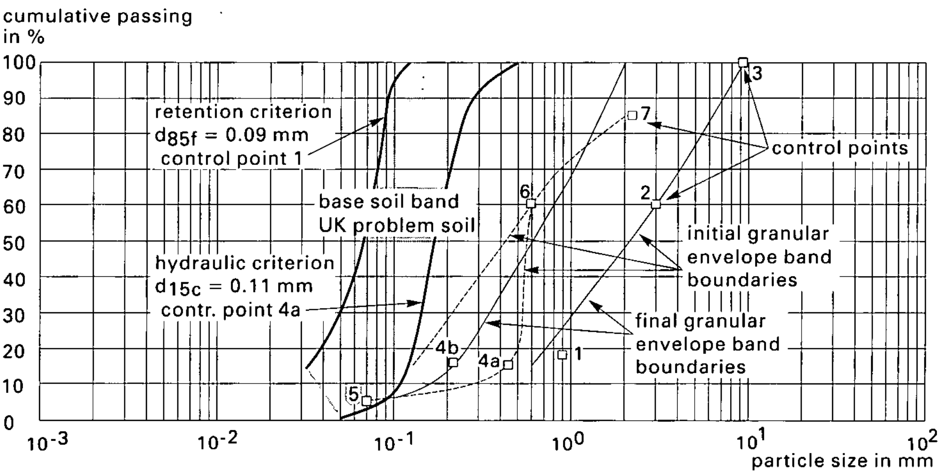


Figure 12 UK problem soil band and granular envelope design.

Verification is necessary that materials for the drainage system under consideration are indeed available without too much blending or special screening.

3.4 Organic envelopes

Organic envelopes are mainly used in Northern and Western Europe to prevent soil invasion into drain tubes but they also contribute to improving the performance of the drainage system. Of all the organic materials in use, the pre-wrapped loose coconut fibres and mixtures of coconut and synthetic fibres are the most common today (saw dust and wood chips may still be in use in the Scandinavian countries). Recent price increases (1994 - 1995) in the imported coconut fibres (from Sri Lanka) has led to a major shift towards loose, pre-wrapped synthetic materials in the Netherlands.

The best known organic materials in Western Europe were fibrous peat, flax straw and coconut fibres, while wood chips and sawdust were used in Northern Europe (Jonsson 1986). In addition, a whole range of other organic materials such as heather bushes, hay, cereal straw, rice straw, reeds, grass sods and corn cobs have also been in use in the temperate climate zones of Europe and the Americas. Jute has found limited application in Asia. However, most of these materials never gained the popularity of the pre-wrapped loose coconut fibres. Most of these materials were applied as loose cover and have gradually been replaced by materials that can be formed into strips. Just covering the top of drainpipes, as practised in the past, does not prevent sedimentation. Effective protection can only be assured when the envelope surrounds the drain tube. To achieve this, some of the organic materials were pre-wrapped as a strip or as loose fibres (Figure 13).

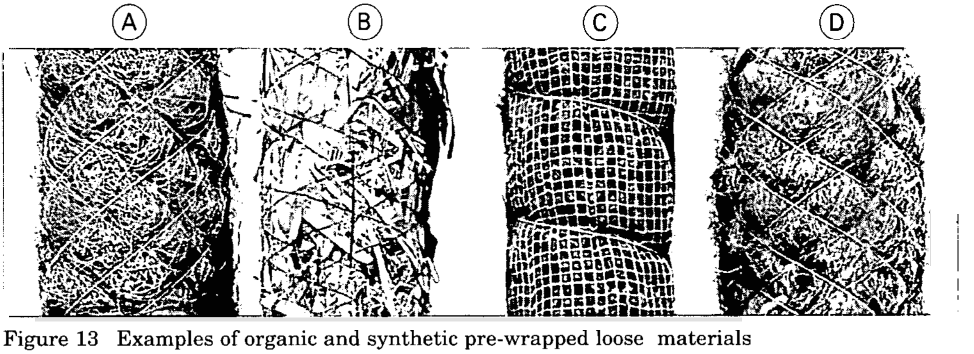


Figure 13 Examples of organic and synthetic pre-wrapped loose materials
A Coconut fibre.
B Flax or rye straw fibre.
C Peat fibre.
D PLM synthetic fibre.

Design criteria for organic envelopes are limited; determining permeability and characteristic opening sizes is not convenient. It is assumed that the considerable thickness will compensate for the rather coarse structure of most organic materials and thus provide filtration. As the permeability is generally much higher than that of the surrounding soil, it is not considered as a design parameter. Besides the visual judgement of regularity of the material, weight and thickness are the only two design parameters for pre-wrapped organic envelopes.

Organic materials are usually voluminous with a minimum thickness of 4 mm at a load of 2 kPa or thickness determined according to Annex B of CEN/TC 155 N 1261 (1994) which describes a method using a tape under load to measure the circumference at a minimum of five locations. The mean average thickness depends on the material, but it must not deviate by more than 25% of the thickness specified on the manufacturer's label. Table 5 shows the thickness as required by some of the European and American Standards. Table 6 shows the mass required for flax, straw and coconut fibres for materials delivered as strip or loose pre-wrapped material in Belgium and the Netherlands (BRBV 1978 and NEN7047 1981). Both weight and thickness guarantee a proper functioning of the organic envelope. Organic materials are vulnerable to deterioration because of microbiological activity, which is more pronounced in alkaline soils and soil high in organic matter. High temperatures and oxygen facilitate microbiological decomposition. Certain organic materials under certain conditions have been observed to decay within one year. Organic materials are not recommended for use in arid and semi-arid regions, unless the application is of a temporary nature only, or the envelope is expected to be needed only during the construction (i.e. the soil is expected to stabilise over time).

Table 5 Prescribed minimum thickness for synthetic and granular envelopes.

Description	Minimum thickness in mm	Remarks
Vegetative material	150	ASAE EP260.4 (ASAE 1984) recommends use only for rigid pipes and not plastic which depends on lateral support provided by granular envelope
Pre-wrapped Loose Material (PLM):		
synthetic fibrous	3	CEN/TC 155 N 1261 (1994)
synthetic granular	8	thickness determined according to ISO 9863
organic fibrous	4	or as in Annex B of this standard. Deviation
organic granular	8	of no more than 25% from declared thickness by manufacturer. PLM classes (F: fine, S: standard): PLM-F 300 µm $\leq O_{90} \leq$ 600 µm; PLM-S 600 µm $\leq O_{90} \leq$ 1100 µm
Coconut fibre:		NEN 7047, 1981 (Netherlands standard)
Type 750 g (mass > 750 g/m^2)	6	but not greater than 10*actual mass/750
Type 1000 g (mass > 1000 g/m^2)	8.5	but not greater than 13*actual mass/1000. Maximum thickness to prevent too loose a structure and danger of clogging, minimum is to prevent too high density and high entrance resistance.
Polypropylene PLM:		NEN-7090, 1989 (Netherlands standard)
PP450 (O_{90} = 450 µm)	3	
PP 700 (O_{90} =700 µm)	4	
PP 700 heavy duty (O_{90} = 700 µm)	6	
Granular material	75	USBR 1978. Minimum thickness may be 75-150 mm depending on type of equipment used for installation. Dierickx (1980) found that beyond 5 mm, additional thickness did not reduce entrance resistance. Sherard *et al.* (1984b) reported that filtering action took place in first 2 mm of the fine filter.

Table 6 Guidelines for required mass of organic material around drainpipes.

	Flax straw		Coconut fibre	
	strip	pre-wrapped	Strip	pre-wrapped
Nominal mass	2000 g/m^2	1500 g/m^2	1000 g/m^2	750 g/m^2
Minimal mass	1800 g/m^2	1350 g/m^2	900 g/m^2	675 g/m^2

3.5 Synthetic envelopes

The design of synthetic drain envelopes is different from the design of granular envelopes because in most cases the envelope material can easily be selected from manufacturer's lists and there is no need to build an envelope from the beginning. This is an advantage and a disadvantage. Filters cannot be easily adjusted to local conditions. Also, primarily one size criterion is used, namely the O_{90}/d_{90} ratio, which is a retention criterion only. The commonly used hydraulic criteria will be given, although generally not needed since the permeability of most fabrics are considerable higher than of the surrounding soil.

As opposed to granular envelope design, the mechanical strength of the geotextile fabric needs to be considered. For details on mechanical and hydraulic properties of commercially available geotextiles[12] it is advisable to get a copy of the latest annual issue of the Specifier's Guide of the journal: 'Geotechnical Fabrics Report, Engineer's guide to Geosynthetics'. The only time that you are likely to design your own synthetic envelope material is when pre-wrapped loose materials (PLM) are used and production is taken on by a public drainage authority rather than a private company. PLM envelopes are made from a mixture of fibres of different thickness and manufacturers have established their own formulae to calculate the characteristic opening size (O_{90}) from the mixture of fibres (Vlotman and Omara 1996). This process, however, is very much manufacture-related (Figure 14) and many different formulae exist, which are often the production secrets of the manufacturers. Attempts to calculate O_{90} from mass and thickness have also met with little success because of the many different production paths that can have been followed to come to the final product.

Synthetic envelopes include the loose synthetic fibres wrapped around a drainpipe (pre-wrapped loose material - PLM) as well as woven, non-woven and knitted fabrics. These can all be grouped under the generic name of **geotextiles**.

Approximately 11 different synthetic materials have been used worldwide especially for drain envelopes. Each of these materials can be made from one of nine polymers (Figure 14). The manufacturing process consists of four distinct steps: 1. fibre preparation; 2. web formation; 3. web bonding; and 4. post treatment. The result is a vast amount of different quality envelope materials as shown in Figure 14 for non-woven needle punched envelope material. Figure 14 also shows that recommending a particular mixture of fibres for

[12] Note that the GFR Specifier's Guide uses the AOS = O_{90} for the Characteristic Opening Size of the synthetic fabric rather then the O_{90} recommended herein. The Guide is US based and the USA prefers to use AOS.

PLM envelopes in order to achieve a certain O_{90} value is subject to many variables. Moreover, recent developments in related drainage applications given in Figure 87 (p 306) and the application of vertical drainage screens called strip drains used for sport field drainage - which could probably be used in agricultural applications - increases the options of usage of geotextiles with drainage. A brief overview of materials available is presented in the next section.

3.5.1 Geotextiles, geonets, geopipes, geocomposites

Koerner (1994) describes four major applications of drainage and synthetic fabrics: geotextiles, geonets, geopipes and geocomposites (see also Box 12).

A geotextile can be any permeable textile used with foundation, soil, rock, earth, or any other geo-technical engineering related material as an integral part of a man-made project structure or system. Drainage water movement takes place within the material (planar flow, transmissivity, see Section 5.6.12). All geotextiles are capable of draining water in their in-plane direction, but their effectiveness varies widely as a function of their manufacturing style (Figure 14) and thickness. To convey water to the drainpipe perforations their in-plane water carrying capacity is of importance. Apart form pore sizes, the compressibility of the material plays a key role in conveying the water.

A geonet serves a similar purpose and is made of a net-like polymeric material formed from intersecting ribs integrally joined at junctions. A geonet is essentially a flat, water conveyance medium, which needs to be protected by a geotextile (to prevent soil invasion) on one side and a geomembrane on the other to guide the water. It is commonly used for slope protection of (hydraulic) structures, and cannot be used with drainpipes.

A geopipe is another expression of plastic drainage pipe. The fact that it is made of materials similar to that of the geotextiles is why it is grouped under the name geopipe. Drainage is only one of its many potential uses.

Geocomposites are manufactured drainage media using geotextiles, geonets and geomembranes in laminated or composite form. For drainage purposes they are used as wick drains or strip drains[13] (instead of vertical sand drains), as sheet drains (behind retaining walls) and with highway edge drainage

[13] In Europe these materials are referred to as prefabricated vertical drains (PVDs). In the US the name wick drain has caught on. However, although they may bear some resemblance to candle wicks, they do not wick (Koerner 1994, p. 735). Wick is not officially a verb, but implies the capillary action to draw a liquid to where it is used. Wick drains do not rely on capillary action to drain.

(Figure 87, p 306). To date they have not been used with agricultural drainage, but experience with these drains from a soil retention point of view is useful.

Box 12 Definitions of geotextiles.

What are Geosynthetics?
(Adapted from the Geosynthetic Research Institute (GRI) Web site, posted 19/1/96).

Geosynthetics are materials manufactured from various types of polymers used to enhance, augment and make possible cost effective environmental, transportation and geotechnical engineering construction projects. They are used to provide one or more of the following functions: separation, reinforcement, filtration, drainage, and liquid barrier.

The most common types of geosynthetics are:

geotextiles – flexible, textile-like fabrics of controlled permeability used to provide all of the above functions, except liquid barrier, in soil, rock and waste materials. Note, natural fibre geotextiles (e.g. using jute) are manufactured in some parts of the world and these products are also considered to fall within the geotextile classification.

geomembranes – essentially impermeable polymeric sheets used as barriers for liquid or solid waste containment.

geogrids – stiff or flexible polymer grid-like sheets with large apertures, used primarily to reinforce unstable soil and waste masses.

geonets – stiff polymer net-like sheets with in-plane openings used primarily as a drainage material within landfills or in soil and rock masses.

geosynthetic clay liners – prefabricated bentonite clay layers incorporated between geotextiles and/or geomembranes and used as a barrier for liquid or solid waste containment.

geopipes – perforated or solid wall polymeric pipes used for the drainage of various liquids.

geocomposites – hybrid systems of any or all of the above geosynthetic types which can function as specifically designed for use in soil, rock, waste and liquid related problems.

other – the geosynthetics industry has exhibited much innovation that has led to other speciality products. These include threaded soil masses, polymeric anchors, and encapsulated soil cells. As with geocomposites their primary function is product-dependent and can be any of the five major functions of geosynthetics.

The generic name to describe all the (drainage) materials in the preceding paragraph is **geosynthetics**.

Geotextiles do not exhibit the same tendency to decay as organic envelopes and once they are buried (out of direct sunlight), they can be considered as viable substitutes for the traditional sand-gravel envelope materials. Geotextiles similar to the ones recommended for drainage applications have been successfully used in dam construction for over 15 years.

Small diameter pipes (up to 80 mm nominal diameter) can easily be prewrapped with geotextile materials. Larger diameter pipes, supplied in six-metre-long sections, need to be provided on-site with the geotextile. The material is wrapped around the pipe then sewn or thermally bonded on-site, or, if delivered as a sock, can be pulled over the joined pipe section prior to instal-

MANUFACTURING PROCESS OF GEOTEXTILES (most common options to produce needle punched nonwoven: **bold face**)

1 FIBRE PREPARATION		2 WEB FORMATION	3 WEB BONDING	4 POST TREATMENT *	USER REQUIREMENTS
TYPE OF POLYMERS (% indicates usage)	POLYMERS MADE INTO FIBRES OR YARN	WOVEN/NONWOVEN	BONDING		USER SPECIFICATIONS

(specs. in Denier , tex or dtex)

WOVEN
1 textiles
2 industrial applications
2.1 plain weave
2.2 basket weave
2.3 twill weave
2.4 satin weave

NONWOVEN

Type of polymers

1 **polypropylene (PP) 83 %**
2 **polyester (PET) 14 %**
3 high-density polyethylene (HDPE) } 2%
4 low-density polyethylene (LDPE) } 2%
5 polyamide, NYLON (PA) 1%
6 linear low-density polyethylene (LLDPE)
7 polystyrene (PS)
8 polyvinyl chloride (PVC)
9 acrylonitrile butadiene styrene (ABS)

2 out of 9 options

Polymers made into fibres or yarn

1 monofilament
2 **multifilament**
3 staple fibres
4 **stapel yarn**
5 slit film monofilament
6 slit film multifilament

2 out of 6

Nonwoven web formation

1 **rotation of spineret**
2 electrical charges
3 controled airstreams
4 **conveyor belt speed**
5 **mixing of ratios fibres**

3 out of 5
excl. mixing ratios

Bonding

1 thermal (heat bonding or setting)
2 chemical (resin spraying or impregnation)
3 **mechanical (punching)**
3.1 needle thickness, length
3.2 number and type of barbs
3.3 needle density per area
3.4 entanglement (during up- or downstroke, or both)
4 **thickness of final product**

5 out of 7
excl. thickness options

User specifications

1 retention criterion
O_{90}/d_{90}
2 hydraulic criterion
$K_f > K_s$
3 mechanic specs. (strength, mass, thickness, chemical resistance,etc.)

at least 60 options
excl. post treatment

* EXAMPLES OF POST TREATMENT

1 UV stabilisation additives
2 impregnation of webs to achieve water resistance, chemical resistance
3 production of:
- geo nets
- geo grids
- mats
- composites

TYPES OF MATERIALS FOR DRAIN ENVELOPES

1 **prewrapped loose material (PP)**

2 **knitted sock (PA)**
3 **polystyrene granules in netting (PS)**

4 nonwoven continuous filament heat bounded
5 **nonwoven continuous filament needle punched (PP)**
6 **nonwoven staple needle punched (PP)**
7 nonwoven resin-bonded

8 woven monofilament
9 woven multifilament
10 woven slit-film monofilament
11 woven slit-film monofilament

Figure 14 Manufacturing processes of non-woven geotextiles and an indication of the number of possible end products.

lation. Alternatively, the pipe sections can be pre-wrapped in the factory, except for the joints. The disadvantage here is that a small section of the pipe is lost to water entry, and there is an increased risk of damage to the envelope during transportation and handling. The advantage is that quality control of sewing, in case of strip material, is done in the factory where control is more convenient to execute and hence potentially better.

For small diameter pipes, a cord of porous fibre wound around the valley of the spiral-corrugated pipe has been used also to cover the perforations. However, as the type of material used (multi-filament cord) was prone to clogging this method did not gain popularity. Nevertheless, this idea could be come in useful when using large diameter pipes (250 mm and up), which have large corrugations (wave length of 80 mm or more, Figure 15) into which materials suitable to the expected soil conditions could be fixed under factory conditions. Substantial material savings could be achieved and it might reduce or do away with the need for criteria for tensile strength and puncture resistance.

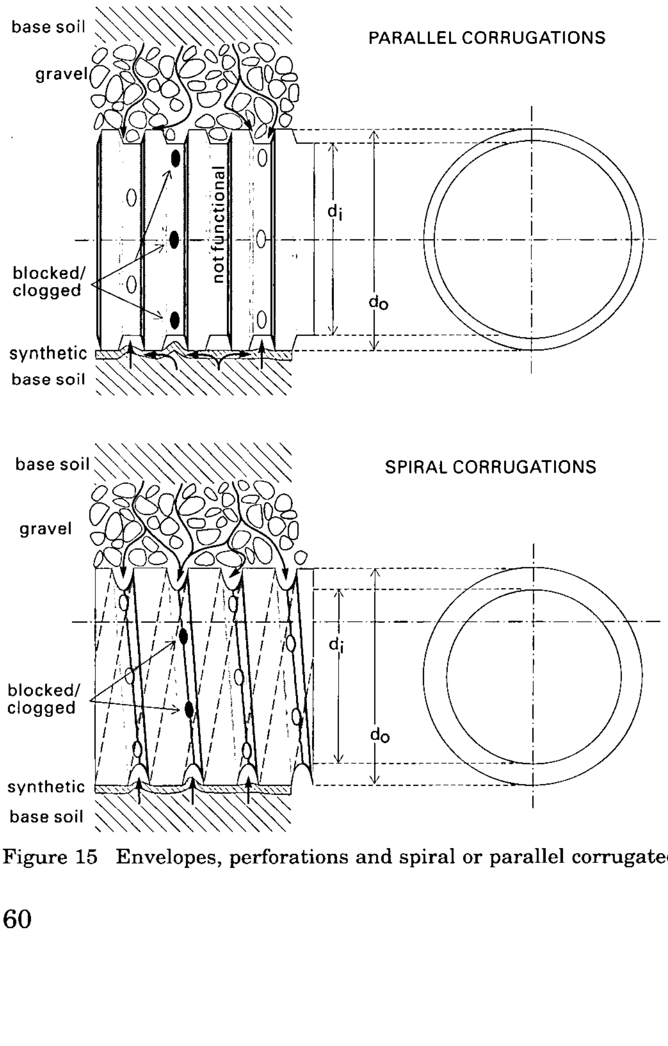


Figure 15 Envelopes, perforations and spiral or parallel corrugated plastic pipes.

There are four classes of criteria for the design of synthetic (fabric) envelopes: 1. retention; 2. hydraulic; 3. the requirement to avoid long-term clogging; and 4. mechanical and strength. The thickness of the material needed as shown in Section 3.1.2 was determined as a function of one of the hydraulic requirements: to create an exit gradient from the soil which is less than the HFG. It is here that the soil hydraulic conductivity plays an important role. The hydraulic requirements that follow are related to the envelope material only.

3.5.2 Retention criteria

The COS of a geotextile is used to assess its capacity to retain soil particles. The COS, however is subject to many definitions (AOS, EOS, FOS, O_{90}, O_{95}, O_{98}, etc. see Section 5.6.9). Many retention criteria have been drafted (Section 6.2.4) and many more are likely to follow. They are the result of different situations and test conditions. In civil engineering applications the stability of structures generally requires far greater attention to safety criteria. The application of filters in agricultural engineering requires less demanding criteria and simple rules are preferred.

Based on the review in Chapter 6.2.4 and the conclusions of the review (Section 6.2.5) we recommend the following set of design criteria for use at this point in time:

1. $O_{90}/d_{90} \leq 2.5$ for envelopes thickness ≤ 1 mm.
2. $O_{90}/d_{90} \leq 5$ for envelopes thickness ≥ 5 mm.

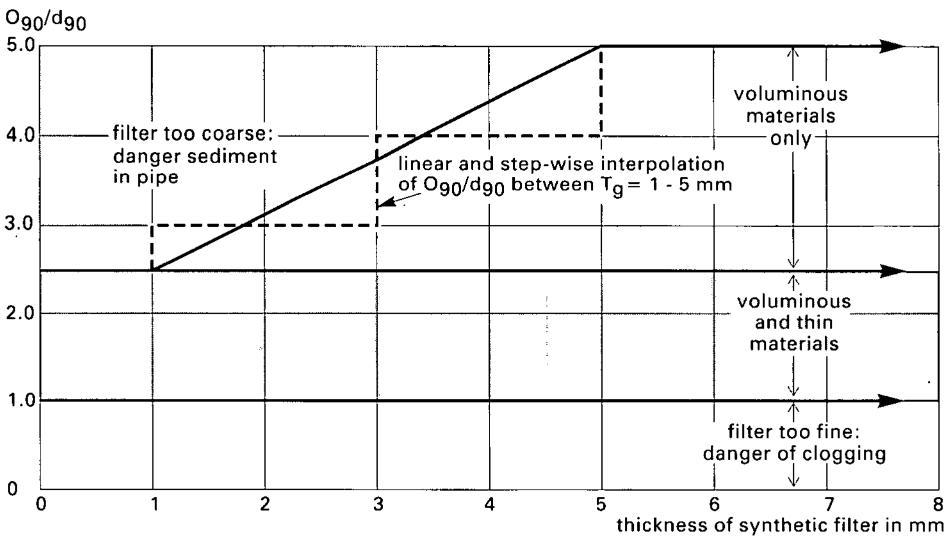


Figure 16 Retention criteria for synthetic material with thickness between 1 and 5 mm.

3. For envelopes with a thickness between 1 and 5 mm, it is advisable to linearly interpolate the ratio, or use step-wise interpolation as shown in Figure 16, which also shows the effect of both procedures on the materials. The thicker the synthetic material, the coarser the filter allowed, based on the assumption that the likelihood of bridging of soil particles is greater in a thicker material than in a thinner envelope and larger pore sizes may be allowed. The linear approach described above allows generally slightly coarser filters than with the step-wise approach of Dierickx (1992a).

Note: unless otherwise stated, the values of O_{90} in the above criteria and throughout this book are assumed to have been determined using the wet sieving method (Section 5.6.9)! If another method of COS determination has been used, or other characteristic opening sizes, or characterising particle dimensions at certain percentage passing are given in the literature, these will have to be converted using the approximate factors given in Table 26 and 27 and for soil in Table 18, before the O_{90}/d_{90} criteria or any relationship with O_{90} can be used!

3.5.3 Hydraulic criteria

The hydraulic characteristics of a particular synthetic envelope material depends on the following characteristics:

- characteristic opening size (Section 5.6.9)
- percent open area, or porosity (Section 5.6.8)
- compressibility (Section 5.6.6)
- water permeability normal to the plane with and without load (permittivity, Sections 5.6.10 and 11)
- water flow characteristics in the plane with and without load (transmissivity, Section 5.6.12)
- water penetration resistance (Section 5.6.13)

The characteristic opening size (COS) is generally not used in hydraulic criteria, except that sometimes a minimum O_{90} value is prescribed. The percent open area (POA) and/or the porosity of the envelope material are very important, and were used when the minimum envelope material thickness was determined in Section 3.1.

When hydraulic conductivity (permittivity normal to the geotextile plane or transmissivity in the geotextile plane) is considered it is critical to evaluate which part of the fabric can really contribute to the flow (Figure 8 and 15). Compressibility primarily affects the porosity and/or COS, which in turn affects the hydraulic properties of the material. The thickness of certain types of envelope materials as a function of drain depth using a specific mass of a saturated soil of 2000 kg/m^3 is shown in Figure 58. No data are reported in the literature on the effect of compressibility on the COS. The effect of compression of the fabric on water permeability normal to the geotextile plane is minimal (Section 5.6.11), and typical values given for permittivity without load may be used for conditions under load as well (Box 13). Permeability (or hydraulic conductivity) in-plane (transmissivity) is affected by load as shown in Figure 64 (Section 5.6.12). Koerner (1994) concluded that for most geotextiles a constant value for transmissivity was reached for loads > 24 kPa which is roughly equivalent to drain depths > 1.2 m. Generally, government-regulating agencies do not require permittivity and transmissivity results in their specification, instead, they specify hydraulic conductivity ratios. It should be noted that these hydraulic conductivity ratios imply (or should) expected soil pressure and the thickness of the envelope material (see also Section 6.2.4). Some typical values of transmissivity and permittivity are given in Box 13.

Water penetration resistance problems were reported from France (Lennoz-Gratin 1987) and investigated (Section 5.6.13) in the laboratory. Standard tests exist. In the literature the phenomena is also referred to as wettability of the geotextile. Laboratory tests on a range of geotextiles demonstrated that to overcome the penetration resistance typical initial heads varied between 5 - 30 mm, which is not a major problem with agricultural drains (the reported problems in France were with sport fields).

Although it is clear that the factors described above have an effect on the hydraulic functioning of the geotextiles, they do not reflect directly on the most common criteria used to prescribe the hydraulic characteristics of materials considered for agricultural drains. Rather, they rely on relative comparison of the clear water permeability of soil and envelope (tests described in Section 5.6) and minimum opening size of the characteristic opening. The criteria are applicable to all soils types encountered in agricultural drainage:

1. $O_{90} \geq 200$ µm (= 0.2 mm) to prevent the risk of a reduced permeability as a result of clogging/blocking of the envelope. Geotextiles with O_{90} between 100 and 200 µm may be considered for use, but under certain soil conditions clogging/blocking may be more likely (e.g. gap-graded soils, unstable structured soils). To be safe the 200 µm boundary is recommended.
2. $K_e > 10\ K_s.i_s$ (Giroud 1996) where is is the hydraulic gradient in the soil near the envelope. This formula could apply for all three K_e/K_s ratios (incl. the two following) if the severity of the actual situation at the soil-envelope interface is described by ranges of the (expected) hydraulic gradient is (Section 5.1.4 and Box 25, p 289).
3. $K_e > K_s$. This criterion is generally recommended for use when conditions are neither critical (economically and life threatening) nor severe (hydraulically; i.e. high gradients). It is prescribed in most states in the US (Koerner 1994).

4. $K_e > 10\ K_s$. This criterion is recommended when cyclic hydraulic loading can take place or when reversal of flow direction may occur. Some states in the US prescribe this (Koerner 1994).

From the above it may be clear that the discussion concerning hydraulic conductivity ratios is far from over. Criteria 2 and 3 originate from the field of civil engineering. We recommend criteria 1 and 4 for agricultural conditions. More data are needed on actual field experiences than were available at the time of writing this book and perhaps further laboratory testing can clarify the issue in the future.

3.5.4 Anti-clogging criteria

1. $O_{90}/d_{90} \geq 1$. Dierickx (1993a) proposed this criterion. For coarser soils ($d_{90} \geq 200$ µm) the O_{90}/d_{90} ratio may be less than 1 provided that the O_{90} of the geotextile is at least 200 µm and that it does not hamper the water flow.
2. $O_{90} \geq 100 - 200$ µm. Dierickx (1993a) recommends the 200 µm, but in the USA and elsewhere, lower values have been allowed (Koerner, 1994).

3.5.5 Mechanical, strength, and miscellaneous criteria

Geotextiles used as an envelope for drainpipes in subsurface land drainage must satisfy certain requirements related to their hydraulic properties. Other properties are useful for identification purposes and supply information on the regularity of the product. Mechanical properties are important for handling and installation. Drainpipes wrapped with geotextiles are sometimes exposed to natural weathering and chemical deterioration, which may affect their functioning. Therefore the following properties of geotextiles, used as envelopes for drain tubes, are to be considered (in parentheses are the sections where details on testing and/or determination of the property are described):
- thickness at 2 kPa (Section 5.6.2);
- denier or dtex[14] of the base material (Section 5.6.2);
- mass per unit area (Section 5.6.3);
- strength of the material (tensile, grab, tear, Section 5.6.4);
- strength of the joints (seam strength, Section 5.6.4);
- static puncture resistance (Section 5.6.5);
- abrasion damage (Section 5.6.7); and
- resistance to material deterioration (section 5.6.14).

[14] Denier is the weight in grams of 9000 m of a single fibre, while dtex is the same but for 10,000 m, Koerner 1994 defines tex as weight per 1000 m! 1 tex = 10 dtex.

The thickness is generally intended for identification purposes, for verification of the regularity of the product and for evaluation of the variability in the other required properties of geotextiles. As was shown earlier, it is also important to know its properties at certain soil (and loading) pressures after installation. Thickness is determined at a standard pressure of 2 kPa.

The dtex or Denier is a measure of the weight of a single fibre of certain length, and plays a key role with PLM materials: manufacturers use the fibre thickness in certain formulae to calculate the COS as a function of certain mixtures of fibres with different dtex.

Also the mass per unit area is used for identification purposes and for verification of the regularity of the product. It is a critical factor in determining the price of the geotextile.

The tensile strength of the material is characterised by force elongation characteristics, and grab strength, which is the tensile strength when only part of the width of the material is subjected to tension, are key indicators to judge the strength of the material. Typical values are given in Box 13. The tear strength is a measure to determine the resistance of the material against tearing during its installation (Box 13). The strength of the joints (seam strength) is generally given as percentage of the tensile strength. The static puncture resistance measures the resistance of a geotextile against punctures by rocks, gravel and other sharp objects. Values typically prescribed for road and construction drainage applications are given in Box 13.

Abrasion damage may occur during transport of pre-wrapped drains, or during handling of fabric on site. Abrasion damage is expressed as a loss in tensile or breaking strength or the development of holes. As function of 250 - 1000 cycles of abrasion, losses in strength were 60 - 90% (Koerner 1994). Government agencies at present do not seem to prescribe limits in their standard specifications for subsurface drainage applications.

There are a number of processes that will deteriorate geotextiles. Degradation occurs as a result of temperature, oxidation, hydrolysis, chemical, biological, and sunlight (Ultra Violet rays). These processes will cause brittleness of the material, loss of strength, affect stiffness, and may attack the polymer of the material itself. The most important processes that affect the use of geotextiles for agricultural drains are: temperature and sunlight degradation during storage and handling, high alkalinity, and high and low pH values. The last two (alkalinity and pH) primarily affect polyesters (PET) and polyamides (PA) neither of which are used in large quantities in geotextiles (Figure 14). However, polyamide is the base polymer of what commonly is referred to as nylon, and knitted socks of nylon are common in drainage application in

Box 13 Summary of desirable properties of synthetic materials for drain envelopes.

This summary presents the state of affairs for synthetic materials based on literature of the early 1990s (up to 1995) and will have to be adjusted when new information becomes available.

1.	Material	*Polypropylene (PP) polymers are preferred, but nylon (polyamide, PA) and polyester (PET) have been used successfully*
2a.	Web formation and bonding (Figure 14)	*non-woven needle-punched (in order of preference for agricultural drain uses)* *non-woven heat-bonded* *knitted* *woven*
2b.	Loose fibres in netting	*Pre-wrapped loose material (PLM, same preference as non-woven needle- punched)*
3.	Permeability normal	*> 50 l/m^2s at 2 kN/m^2 or permittivity*[15] *> 0.1 - 0.7 s^{-1}, or K_e > K_s (Section 5.6.10)* *> 15 l/m^2s at 200 kN/m^2, or typical values of permittivity are 0.01 - 3.0 s^{-1} (Koerner 1994)*
	in plane	*> 100 m/d at 2 kN/m^2 (Typical values for transmissivity*[16] *are 0.01 - 2.0 x 10^{-3} $m^3/min/m$, after Koerner 1994)* *> 10 m/d at 200 kN/m^2*
4.	Method of determination of COS	*wet sieving recommended (O_{90}). When other methods are used COS should be converted to standard opening size (O_{90}) according to Table 31 or Table 27*
5.	Opening size	*retention O_{95} < 297 - 595 μm (AASHTO M288-90, Section 5.6.9c) permeability and anti-clogging O_{90} > 100 - 200 μm (Sections 3.5.4 and 5)*
6.	Soil retention/clogging	*$1 \leq O_{90}/d_{90} \leq 5$ (details in section 3.5.2)*
7.	Thickness	*minimum 1 mm preferred, but dependent on minimum thickness requirement (Section 3.1) and standards (Table 5)*
8.	Mass per unit area	*> 200 g/m^2. Commonly available 130-700 g/m^2.*
9.	Tensile strength	*> 6 kN/m*[17] *(wide width tensile strength, Section 5.6.4)*
10.	Tensile strength of joints	*> 6 kN/m (seam strength usually 50 - 100% of tensile strength)*
11.	Sewn-seam strength	*> 310 - 360 N per m width (for Class B*[18]*, AASHTO M288-90 which class seems most appropriate for agricultural drains).*
12.	Trapezoid tearing strength	*> 110 N (for Class B, AASHTO M288-90)*
13.	Static puncture strength	*> 110 - 160 N (for Class B, AASHTO M288-90)*
14.	Grab strength	*> 360 - 400 N (for Class B, AASHTO M288-90)*
15.	Joint overlap	*> 150 mm joining two pipe sections, joined by tape or sewing (stitching according to Figure 22)* *> 20 mm when wrapping around pipe, joined by sewing (Figure 22, p 79), or overlap when pre-wrapped with netting similar to PLM (Figure 13).*
16.	Weathering (ageing)	*total UV radiation should be < 800 MJ/m^2 during period of exposure*[19]*.* *low temperature can cause brittleness during handling ➡ damage.* *PET and PA polymers are more affected by alkalinity and pH*

[15] permeability is permittivity multiplied by material thickness T_g.

[16] transmissivity is permeability multiplied by material thickness T_g.

[17] 6 kN/m is a minimum value for drain envelope application. Note: for other applications of synthetic materials generally higher values are required (i.e. > 9 kN/m, ASTM D1682)

Canada. To date no deterioration of the nylon socks as a result of low pH values has been reported. Low temperature causes brittleness of plastics, but no problems with drain envelope materials have been reported in the literature. Standard tests for testing the effects of high and low temperatures exist (Section 5.6.14). Only UV radiation has been reported to cause problems with synthetic drain envelope materials (and plastic drainpipe) when materials had not been covered properly during storage. Envelope materials deteriorate beyond acceptable standards when exposed to a total radiation of 3.5 GJ/m^2. In temperate climates it may take one year of exposure, while in tropical climates this limit might be reached in three to six months. The effect of temperature on the tensile strength of different polymers is shown in Figure 17.

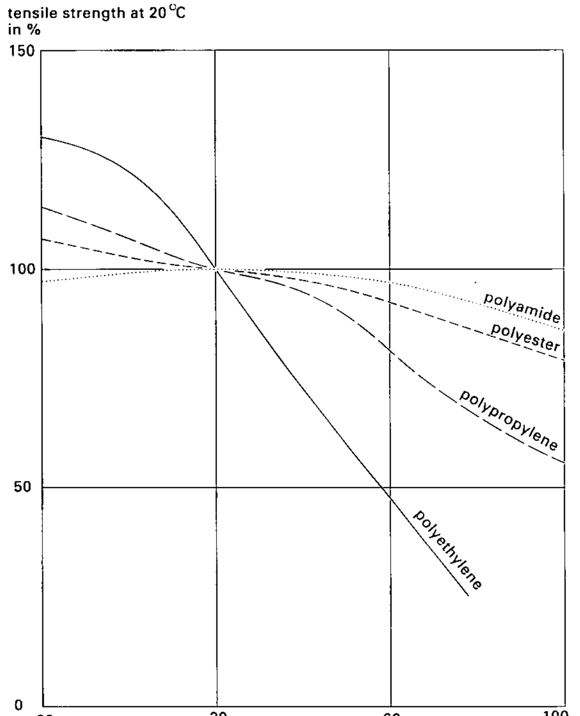


Figure 17 The influence of temperature on tensile strength. CUR/NGO (1995a)

[18] Class A geotextiles are used where installation stresses are higher than Class B applications (i.e. where very coarse sharp angular aggregate is used, a heavy degree of compaction [> 90% of AASHTO T99] is specified, or the depth of the trench is greater than 3 m [10 ft]).

[19] Based on information provided by W. Dierickx. This amount of radiation will be reached in approx. 3 months in an average humid temperate climate (it may be 1 month in the summer and 5 month in the winter. N.B. a draft European Standard for testing of pipes prescribes that exposure should be continued until the pipe has received a total solar radiation of not less than 3.5 GJ/m^2.

3.6 Concluding remarks on envelope design and ranking of selections

After all considerations have been taken into account, there are still many options open for selection of materials. Although, the present criteria are set up to eliminate - step by step - the options of granular, synthetic, and thin or voluminous envelopes, one might wish to use the following ranking method (after Williams and Luna, 1987) to approach the selection is a systematic way. This method could be biased towards preferences of the user, because the user is expected to assign weight to certain aspects of the ranking process.

The first step in decision analysis is to rank the envelope materials according to each of the categories listed below:

1. meeting the required thickness;
2. meeting retention criteria;
3. meeting hydraulic criteria;

4a. satisfying segregation criteria (granular envelopes only);
4b. meeting strength and other mechanical criteria (synthetic fabric envelopes only);

5. chemical compatibility;
6. durability; and
7. cost.

Each envelope material considered is assigned an Envelope Number (EN) for each of the seven performance criteria. The EN varies from one to the number of envelopes x: one representing the poorest performance and x the highest. When two or more envelopes perform equally well the same EN value should be assigned to each of the envelopes. For example, if four envelopes, A, B, C and D, are being ranked for hydraulic criteria and B is the best, A and D are approximately equal (say within an order of magnitude difference) and slightly better than C, and C has the lowest transmissivity, then the EN numbers would be: B = 4, A = D = 2, and C = 1. The ranking of granular envelopes should be based on sieve results, permeameter results, and published characteristics, while geotextile data may be derived from published data such as given in the annual Geotechnical Fabrics Report[20], as well as from indicator test results and previous experience with the material.

The next step is the evaluation of the Performance Criterion Number (PCN). The PCN varies between 1 and 7 with 1 being the performance criterion with the lowest value and 7 the most important criterion for the particular design

[20] Geotechnical Fabrics Report. 345 Cedar St., Suite 800, St. Paul. Minnesota 55101, USA. Phn. +1-612-222 2508, Fax.: +1-612-22 8215.

situation. The number 7 is equal to the number of categories on which the material is judged. Where the performance criterion is rated equally, both should be assigned the same PCN. For example, in unstable soils the retention criteria could be ranked highest, while in heavy soils the permeability could be given more importance. In other cases both might be rated equally. The ranking of the performance criteria is based on the particular conditions and the experience of the designer, whose judgement might be biased towards or against this evaluation methodology. The suitability number, N_s for each envelope is determined from the following equation:

$$N_s = \sum_{i=1}^{7} [1 + 0.5\,(PCN)]\,(EN) \qquad \text{Eq. 10}$$

The envelope with the highest suitability number is the one that is best suited for the particular application. If so desired, the designer can add more performance criteria than the seven given here.

4 Costs, implementation, maintenance and evaluation

This section describes various aspects of drain envelope installation that come into play after the design of the envelope has been completed, with the exception of the costs, which should have been considered during the design. Preparations of the envelope to go around the pipe, the method of wrapping and the handling of pre-wrapped pipes in the field and during transport are essential to the successful application of the drain envelope. Quality control during the construction is therefore essential. Operation and maintenance of the drainage system can have an effect on the life of the drain-envelope combination and the operation in particular on the frequency of flushing the pipes. Monitoring and evaluation of the satisfactory performance of the drain envelope system is briefly described. Monitoring and evaluation for envelopes alone might not be a viable option and the system described herein is used to judge the hydraulic and overall system performance of the drainage system as well.

4.1 Cost indications

Costs of drain envelopes are very transient. Based on several case studies, the relative cost aspects of granular or synthetic, pre-wrapped and wrapped on site are shown. The purpose is to give the reader a sense of the costs of different envelopes with respect to other components of the system. Synthetic envelopes were found to be the cheapest solution. Yet, the authorities did not always decide to go for the cheaper solution because synthetic materials had not yet been proven in the field under local conditions. Section 6.3 describes some field experiences with synthetic envelopes which could help towards assessing the acceptable risk of using untried materials based on design criteria and assessment of envelope need, as presented in the preceding sections. Costs vary greatly dependent on local conditions. To be able to compare costs, Table 7 shows exchange rates for various currencies with based on the US dollar.

4.1.1 Costs of various drain envelopes in Pakistan in 1992

Detailed cost calculations were made for various drainage projects in Pakistan in 1992 (Vlotman *et al.* 1992, Vlotman *et al.* 1993b). For granular material, a 100 mm gravel envelope, measured from the valley of the corrugation, is prescribed for all pipe diameters (100, 150, 200, 254, 300 and 380 mm). Using the total length of all pipes installed the quantity of required gravel per project was calculated including wastage of 25%. Figure 18 shows a comparison between project rates and the average cost of gravel at the quarry and transport costs according to the Pakistan government transport rates (Water and Power Development Authority, WAPDA) and market rates. With the exception of the East Khairpur Tile Drainage Project (EKTD) and the estimates for Khushab SCARP, the costs of all existing projects were below the average government rates (WAPDA).

Table 7 Exchange rates for various currencies to convert costs shown to desired currency. (Source: Rabobank, The Netherlands, and IMF International Financial Statistics).

Year	US dollar	Netherlands guilder			Pakistan rupee	Egyptian pound
		Low	High	Avg.	Avg.	Avg.
1985	1	–	–	2	15.98	0.70
1988	1	1.83	1.95	1.91	18.65	0.70
1991	1	1.64	2.07	1.71	24.72	3.33
1992	1	1.57	1.89	1.81	25.70	3.34
1994	1	1.67	1.97	1.73	30.80	3.39
1995	1	1.53	1.75	1.60	34.25	3.39
1996	1	–	–	1.74	40.12	3.39
1997	1	–	–	1.96	41.41	3.39

Costs of synthetic fabrics were determined at PRs 17/m^2 c.i.f.(cost, insurance, freight) at Karachi, excluding import duties. Total costs of material and transport for synthetic fabrics remained below the gravel costs for all the projects (Figure 19). Transport for synthetic material were based on full containers and for gravel on the 1990 WAPDA transport rates. All costs were converted to the 1992 price level using an escalation correction of 7.5% per annum (this rate is used by the World Bank for project cost purposes, but actual rates are closer to 13% per annum).

The main difference in the gravel costs is the distance from the quarries to the various projects. Transportation costs could probably be minimised by using the railroad system. Trucks could be used to load and unload bogies at the nearest railway stations. Most of the bogies return empty from the northern part of the country to the Karachi seaport and low rates should be possible. This could also result in less damage to the road system and certainly less wear.

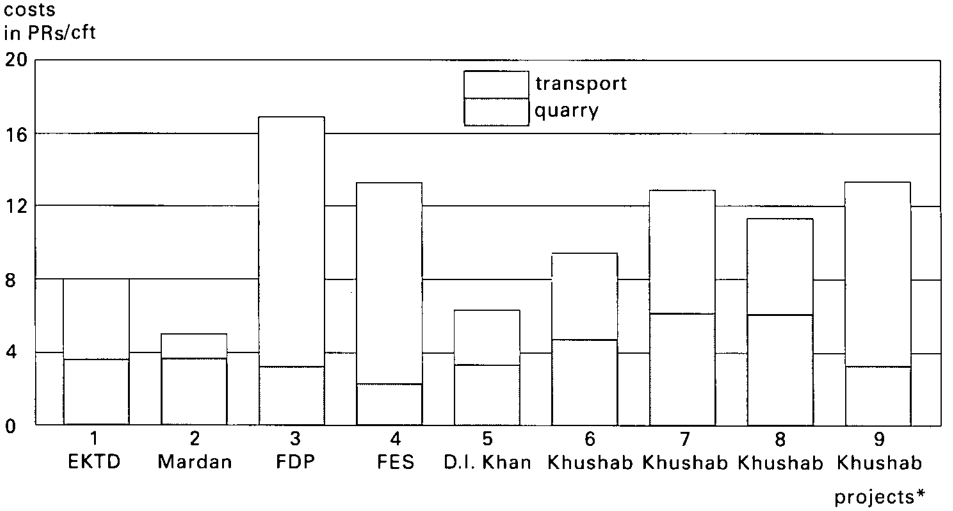


Figure 18 Comparison of drain envelope costs.

Remark: * Khushab 6, estimate by project staff and 10% profit added; Khushab 7, estimate by project director FDP; Khushab 8, estimate by D.I. Khan project; Khushab 9, estimate by Nespak, Mr S.M. Zubair of flood control project.

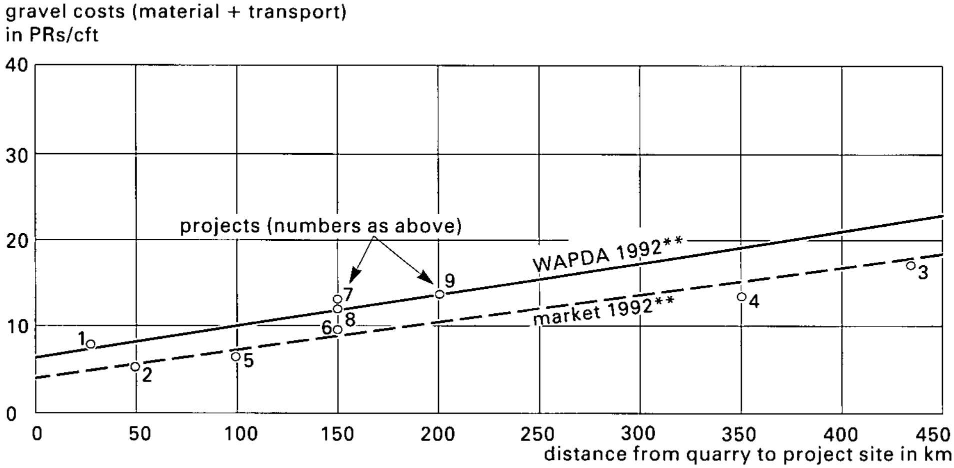


Figure 18 Cont.

Remark: ** The base costs of gravel for WAPDA 1992 and market 1992. Transportation cost is average of cost at quarry including handling at quarry for projects shown.

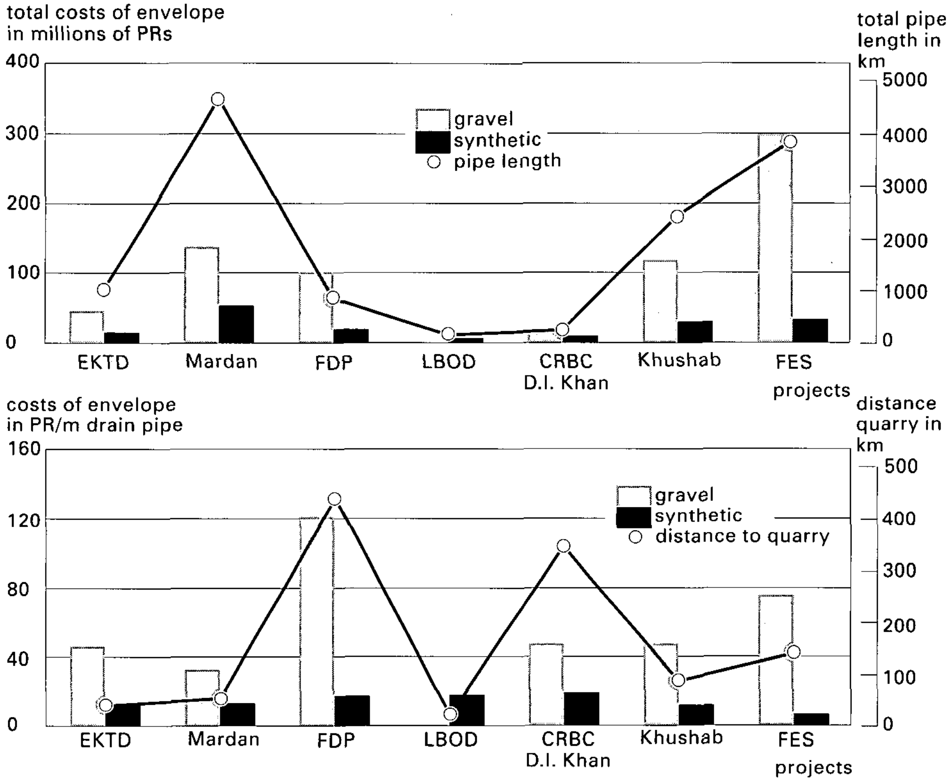


Figure 19 Total estimated costs per project of material and transport for synthetic and gravel drain envelopes at 1992 price levels (Pakistan).

4.1.2 Costs of drain envelopes in the Netherlands in 1991

Assessment of the use of drain envelopes in the Netherlands between 1985 and 1990 showed that voluminous synthetic envelopes were used the most (Figure 20), closely followed by voluminous organic envelopes. Voluminous envelopes comprise materials such as coconut fibre, peat fibre or a mixture of the two. Coconut fibre envelopes made up a major portion of this but the increase of the price of coconut fibres in 1995, which are imported primarily from Sri Lanka, started a move away from organic envelopes. Small quantities of voluminous envelopes consist of mixtures of coconut and synthetic fibres. Synthetic voluminous envelopes are made of polypropylene fibres or polystyrene pellets. The pellets are retained around the pipe by netting and are only applied pre-wrapped. The gravel group includes materials such as granular slag, baked clay pellets and broken seashells, although seldom used on agricultural soils. The thin geotextiles group consists of various spun-bonded polypropylene or polyamide fibres, woven polyester fibres or glass-fibre sheets. The costs of the various envelopes including installation and contractor overhead/profit are shown in Figure 20. All drains were installed with either trenchers or trenchless machines (Figure 23).

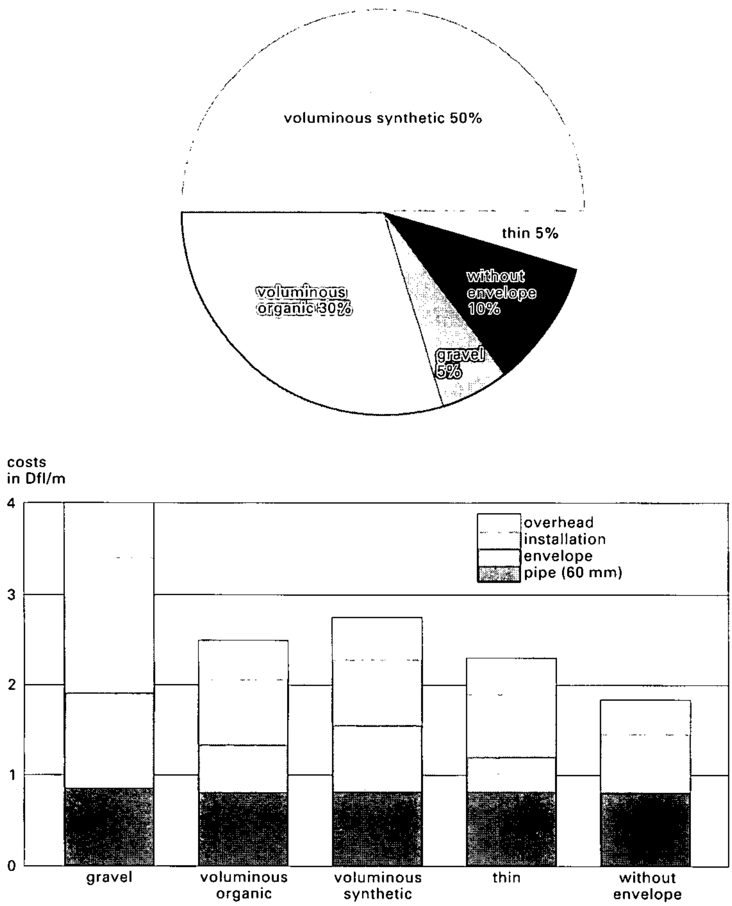


Figure 20 Envelope use and costs in the Netherlands (after van Zeijts 1992)

4.1.3 Cost of drain envelopes in the USA in 1995

In southern California where suitable pit run gravel envelope material is inexpensive and readily available, the cost of an installed 100 mm diameter drain, installed with a trencher, is US$ 3.02 per metre[21]. The cost of a 100 mm diameter drain, installed with a synthetic fabric envelope is US$ 2.85 per

[21] At 1998 price level it would be US$ 3.44/m and 75 mm diameter pipe with a gravel envelope installed with a vertical plough is about US$ 2.79/m.

metre. These drains are installed at a depth of approximately 1.8 metres. In the central part of the United States, where drains are normally installed at a depth of 1.2 metres, the cost of installation of a 100 mm diameter drain with a common synthetic envelope material is US$ 2.62 per metre. The cost of installation may vary from US$ 2.46 to US$2.66 per metre, depending on the quality of the synthetic fabric used as a drain envelope[22]. Installation costs in both areas include the cost of the pipe and the cost of backfilling the trenches. All prices are at 1995 price levels.

4.1.4 Costs of drain envelopes in Egypt in the early nineties

Metzger *et al.* (1992) investigated drain envelope costs for the Integrated Soil and Water Improvement Project (ISAWIP) in Egypt and estimated that local gravel envelope would be four times as expensive as thin synthetic fabric envelopes imported from Canada (duty free). The quality control of graded gravel envelopes was unsatisfactory during installation and were therefore rarely installed in accordance with design requirements.

4.2 Wrapping and connecting synthetic and organic envelope materials

After selecting the material for the drain envelope there is one more very important step to successful application of the drain envelope and that is the way in which the envelope is wrapped around the pipe. Typical questions include: has it been pre-wrapped in the factory or in the field; what overlap should be used; what are the best ways to sew geotextiles together; and what quality of yarn should be used?

Some envelope materials are produced in the form of socks that can be pulled over the pipe, in which case seams will only be needed to join two socks together (one roll is approximately 100 m). With adequate overlap, tightly wrapped yarn can ensure sand-tight connections.

Obtaining soil-tight seams when sheet (envelope) material needs to be wrapped around the pipe is a lot more challenging as may be clear from some of the photos in Figure 21 which are examples of inadequate seams and wrapping of the sheet around the pipe!

[22] In Ohio, at 1998 prices, trencher installed 100 mm diameter drain at 1.2 m depth would cost US$ 2.30/m with gravel and US$2.46/m with a synthetic envelope. Similar drains installed with a plough would be US$2.13/m or less, depending on depth and soil conditions.

Figure 21 Improper wrapping, stitching and handling of envelopes.
A inadequately pre-wrapped fabric.
B repairing exposed pipe with masking tape.
C broken wrapping yarn.
D hole in fabric.
E improper connection.
F poor stitching.

For factory pre-wrapping of pipes, special (band) wrapping machines that can handle pipes up to a diameter of 200 mm are available commercially. If loose fibres are used, the wrapping production line will have a blending machine first before the pipe and fibres are fed into the yarn wrapping part of the machinery. Speed and pressure with which yarn is applied can be adjusted. With the right speed and tension of the wrapping yarn an overlap of 2 cm will be sufficient, but if not, situations as shown in Figure 21 will occur in the field. It takes only one hole, or separated section, to cause the entire drain line fail as a result of soil entry!

For larger diameter pipes, i.e. collector pipes, the envelope will generally be wrapped on site with special, readily available, sewing machines. The strength of the seam is influenced by the following (Santvoort 1994):
- the geotextile material tensile strength and surface roughness;
- the type of seam (overlap);
- the type of sewing machine and stitch type;
- the type of yarn used;
- the number of stitch lines; and,
- the length of the stitch.

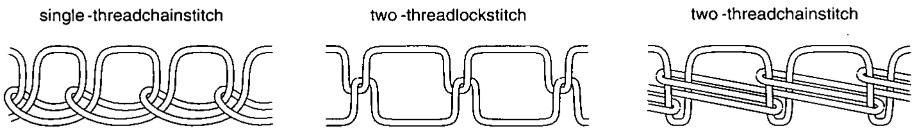
Geotextile material. The weave, the number of yarns, their surface roughness and the thickness of the geotextile all influence the sliding behaviour of the geotextile during installation. The stress with which the two ends are pressed during sewing is less important than the type of yarn. A thick geotextile is less affected (damaged) by stitching than a thin one.

Seam type. Figure 22 shows the various possible seams (overlaps) and their efficiency with respect to soil tightness in fine-grained soils. Except for the double stapled overlap seam, overlap seams can only be produced in the factory and are not appropriate for drain envelope applications in the field. The wrapped staple seam with four layers or three layers of material will guarantee soil tightness when properly executed. The two-layer material and the butterfly staple seams are not recommended. A further disadvantage of the last two is that the strength and quality of the seam is dependent on the quality of the selvage (without a proper selvage the stitch could easily move to the edge). Because drain envelopes are made of lengthways-cut strips they generally do not have a selvage.

Sewing machine and stitch type. The most important stitch types for use with geotextiles are the lock, chain and double chain stitch (Figure 22). The lock stitch is always laid with two yarns, but because it needs a more complex sewing machine it should be done in a workshop. The chain stitch uses only one yarn and if it breaks, the whole seam can work loose. The double chain stitching does not have this disadvantage and is recommended for use with drain envelopes. Portable sewing machines with double stitching can operate with two relative large yarn packages (i.e. 0.5 kg each) which do not have to be changed so often.

The type of yarn. The eye of the needle determines the maximum thickness of the yarn. Thick needles to more damage to geotextiles than thin ones. The sewing yarn must be able to run fast and flexibly through sharp curves and must run through the geotextile without too much friction (high friction can cause melting). Synthetic sewing yarns are made of polyamide (PA), polyester (PET), or aramide (AR): the strength ratio of the three yarns of similar thickness is 1:2:3 respectively (Santvoort 1994). Other materials can be Kevlar, nylon, polyester and polypropylene (in order of decreasing strength and decreasing costs, Koerner 1994).

A



B

description	STAPLESEAMS single	STAPLESEAMS wrapped	OVERLAPSEAMS single	OVERLAPSEAMS wrapped	DOUBLESTAPLE OVERLAPSEAM
shape	1,2or3 stitch	Jseam butterfly seam		capseam	
strengthoftheseamin%of strengthofthewovenfabric	25-50	30-60	60-80	60-80	60-80
soiltightnessfine -gradedsoils	doubtful	guaranteed	doubtful	guaranteed	guaranteed
locationofsewing	onsite	onsite	factory	factory	–

Figure 22 Stitching and different types of seams. (CUR/NGO 1995a, Koerner 1994)
A Stitches.
B Seams.

Number of stitches and length. The efficiency of the seam (soil tightness, safety when yarn breaks, etc.) will generally increase if there are two or three lines of stitching (Figure 22), although the influence of the second and third line is minimal. A stitch density of 1-2 per cm (2-4 per inch) is common. Longer stitches allow faster work, but bind the seam less tightly, which means a lower filtering efficiency. Either a high density of stitches per cm should be used, or more than one line of stitching.

The strength of the seam is generally expressed as a percentage of the tensile strength of the material used (80-90%) or may be expressed as sewn-seam strength. Sewn-seam strength > 12-16 kN is recommended by most States in the US (see Section 5.6.4).

In the US it was found that (at 1994 price levels) renting a sewing machine with thread cost approximately US$150 per day, plus US$100 per day per labourer (three). The team of three labourers easily sewed 1000 m/d (Koerner 1994). Because of the smaller dimensions of overlaps with drain envelopes (5 -10 cm maximum, otherwise feeding problems through the trencher box or plough tunnel could occur), and the higher accuracy of stitching required, lower production rates than those common for road stabilisation work could be expected. At 1996 price levels, the cost of a sewing machine was approximately US$ 1000.

4.3 Installation of drain envelopes

Installation of drains and drain envelopes can be done by hand (not often done, and if so, only on a small scale) or by machines (backhoe). There are two types of machines that can lay the drain and envelope simultaneously: trenchers and trenchless machines. Trenchless machines can be a vertical plough or a V-plough (Figure 23). A comparison between trencher and trenchless construction (van Zeijts and Naarding 1990) resulted in the following observations with respect to drain envelopes:

1. Normal trenchless methods for drain construction are not recommended if gravel is to be used for envelope material. First, the maximum diameter pipe that can be laid by the V-plough and the vertical plough is approximately 125 mm, which does not allow placement of a gravel envelope with a minimum thickness of 75 mm. Second, the flowability of gravel envelope material in the funnel is lower than with trenchers (see Section 5.1) which will limit the chance of successful placement of gravel envelopes with trenchless installation methods. Poor flowability could cause stagnation of the gravel supply;
2. Synthetic envelopes can be laid conveniently using both trenched and trenchless methods.

Laying PVC drains with a 100 mm gravel envelope using the trencher box requires some special measures (Bhatti and Vlotman 1990):

1. All restrictions in the trencher box from the gravel feed tunnel have to be removed and the cross-sectional area of the tunnel needs to increase as the gravel travels downward. This will prevent the gravel from bridging and blocking in the tunnel.

2. Spacers should be placed at the end of the gravel tunnel to maintain the integrity of the trench wall until the gravel is in place.

During construction it is important that the speed of the trenching machine is kept as high as possible. In Pakistan it was found that when (perforated) drainpipes were laid below the water table the installation speed of the trencher laying the collector pipes (10", 12" and 15" diameter) needed to be increased from 2 to 5 metres per minute. This reduced the time from excavating the trench to complete placement of the pipe and gravel, and reduced the effect of upward hydraulic pressure on the placement of the pipe in unstable soil conditions.

The above measures improved gravel placement in all cases (in Pakistan) except when drains were laid 60 - 90 cm below the water table with a high sub-soil permeability. Here, the flowability of the gravel was affected and gravel flow stopped as a result of upward forces (upward hydrostatic pressure and friction in the gravel tunnel) being greater than the downward forces (weight of the gravel).

Some additional construction procedures were found to enhance the placement of drain and envelope:
1. Not pumping the drain empty during construction will reduce the buoyancy effects on pipe placement and prevent the initial high gradients that would otherwise result across the drain envelope (Willardson 1992). The initial inflow of sediment immediately following construction will be reduced. By slowly starting up the pumping of the new system during construction (i.e. reducing the water table to drain level over a period of, say, one week), high gradients near the drain and envelope can be prevented and the trench backfill can consolidate gradually, preventing influx of fines into the drainpipe and substantially reducing the formation of sinkholes.
2. Introducing the power auger at the end of the trencher box (Figure 24) significantly reduces the problems with gravel stoppage even under very adverse conditions. The auger did not function properly in (semi-) saturated conditions, when the collapse of the trench behind the auger tended to stop the outflow of gravel from the auger. Pumping from the sumps during installation of pipes was stopped to keep the water table high along the drain alignment.

Guidelines on installation of drainpipes may be found in ASAE Engineering Practice ASAE EP260.4 (ASAE 1997), in SCS 1988 and other publications. As these provide many details beyond the scope of this book they are merely referenced.

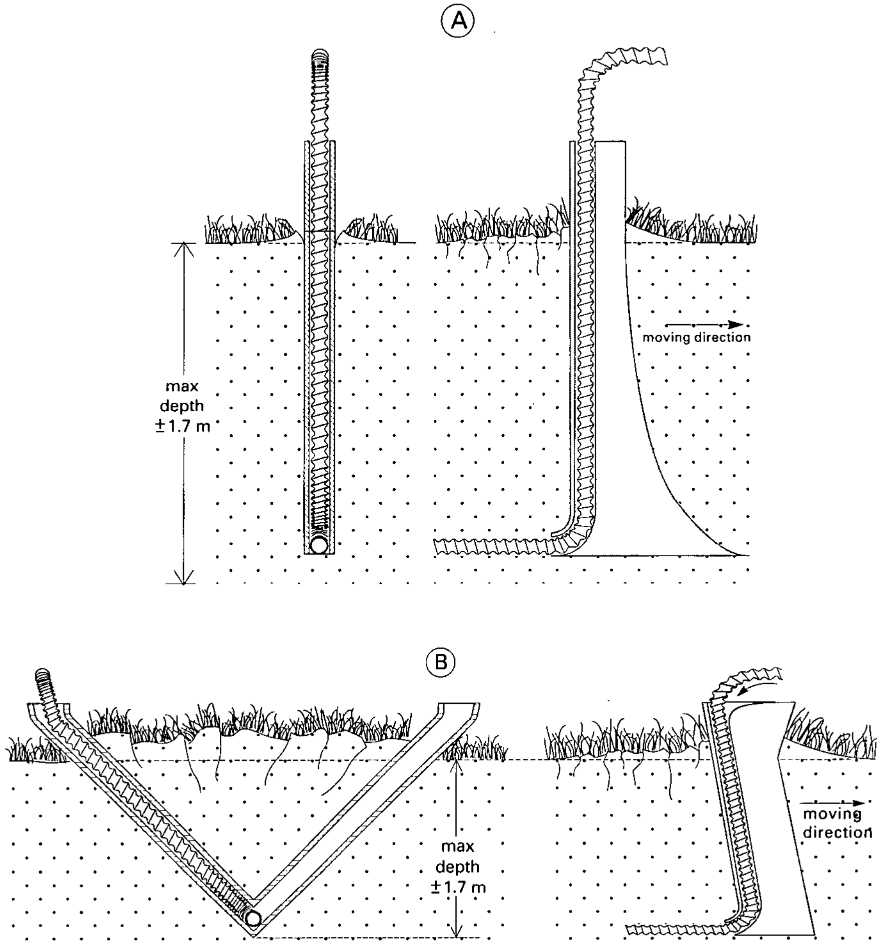


Figure 23 Trenchless pipe laying.
A vertical plough.
B V-plough.

4.3.1 Gravel flow and pipe stretch

With gravel envelopes in particular, problems may occur during construction when the gravel cannot flow unimpeded through the trencher box. This could result in unacceptable pipe stretch. In unstable saturated soils, the trencher box plates (gravel guidance plates, Figure 24A) at the outlet can be squeezed inwards if they are not stiffened properly. This will hamper the outflow of gravel and the section already laid will exert extra pull on the pipe in the box (to get it out of the trencher box). The stretch in such situations measured 10 - 15% against the specified limit of 5%. Stretch was also caused if the gravel placement took place while the pipe was still radially flexed in the trencher box (NB correct way shown in Figure 24 the gravel does not touch the pipe until it is horizontal again). The gravel particles thus forced into the expanded corrugation prevented the corrugation from relaxing back to its natural dimension.

Flow of gravel in the trencher box was also irregular when pipes were laid below the water table (Pakistan), and in some cases stopped altogether. Here, the upward hydraulic force and the friction in the trencher box exerted by the gravel on the trencher box walls was equalled or exceeded the gravity force of the gravel.

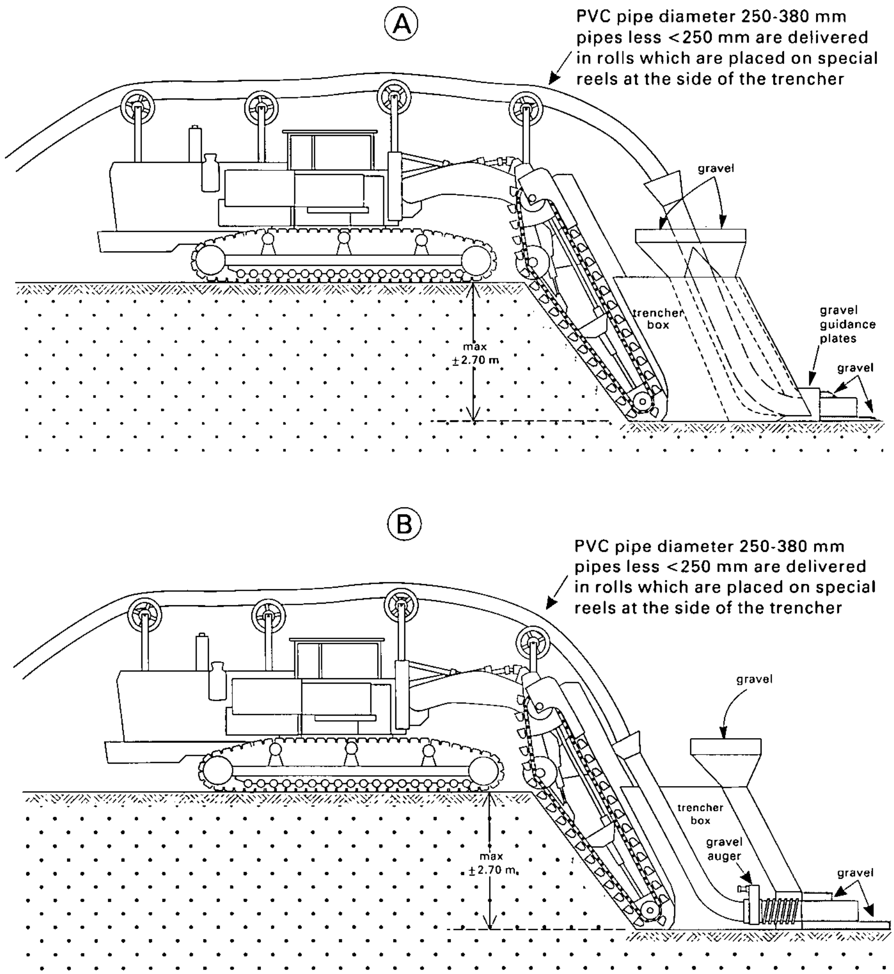


Figure 24 Trencher with and without the gravel auger.
A without gravel auger.
B with hydraulic gravel auger.

Experience in the Western USA (USBR reviewer remark) is that problems with gravel flow through the trencher box are much more prevalent with crushed rock than with rounded gravel.

Both problems were resolved (in Pakistan) with the introduction of the gravel auger at the end of the trencher box (Figure 24). The problem of excessive pipe stretch was totally eliminated, with stretch not exceeding 2 - 3%. Gravel feeding problems still occurred under certain other conditions but reasons were not clear (Personal communication with FDP staff).

4.3.2 Trench backfill and high water tables

Two other factors that can have a major effect on the functioning of the drain envelope are the trench backfill and soil consolidation procedures and the laying of pipes with envelopes below the water table in soils that smeared easily when in contact with water.

Under ideal construction conditions, i.e. pipe above the water table and with stable trenches, the backfilling can be executed as prescribed by the specifications, which generally assume these ideal conditions. Typical specifications will require that a blinding of 15 - 20 cm thickness of well compacted stable-structured soil be made on top of the pipe-envelope, before backfilling takes place. The purpose of the blinding is to protect the pipe and envelopes from disturbance during further backfill procedures (ASAE 1997) and thus assure proper alignment. This is sometimes possible and sometimes it is not (Figure 25). Consolidation of the backfill above the pipe (and blinding) in semi-saturated conditions is generally not possible as no compaction equipment could go deeper than 1-1.5 metres (as shown in Figure 26). The trench wall might collapse in big lumps leaving large voids. The consolidation effort from the surface (Figure 26, method 3) improved the top layers but not the lower sections in the trench immediately above the pipe. After irrigation and rainfall piping resulted in leaving sinkholes that were visible at the surface. Sinkholes will also appear when pipe couplers are pulled or where the (gravel) envelope is not functioning.

High water tables during construction can cause several problems:
1. stoppage of gravel flow as mentioned above;
2. smearing on the outside of the envelope (synthetic fabrics and PLM envelopes primarily) in soils with clay and silt, unfavourably affecting the hydraulics of the envelope; and
3. temporary excessive high gradients immediately following the trencher when the pipe is still empty and water starts flowing into the pipe.

Ninety percent of the first problem was resolved by adding a gravel auger to the trencher box. No problems of a similar nature have been reported with synthetic envelopes.

The second problem is more a theoretical problem than a serious practical one, except when a vertical plough (trenchless construction method) is used (Cavelaars *et al.* 1994). Smearing and compaction of the soil below certain depths (critical depth) were observed when vertical ploughs were used in the Netherlands and the United Kingdom. There have been no reports of recently of smearing of envelopes of drainpipes constructed with modern trenchers and trenchless machinery (V-plough). Wilson-Fahmy *et al.* (1996) reported clogging of drain filters when soil was not backfilled against the filter fabric and fines collected in the voids and through puddling, which eventually clogged the fabric substantially. This was, however, with vertical drainage applications such as shown in Figure 87 (p 306). Some problem soils are whipped into a virtually impermeable slurry by the trenching machine (USBR reviewer remark and personal experience in Pakistan and Egypt by the authors). This happens, in particular, when the speed of the trencher is too slow and/or sticky clay does not separate from the digging chain: the chain thrashes the same soil around and around. For this reason, USBR construction contracts require shielded excavation so that the gravel envelope will be in contact with undisturbed soil and will not be sealed off by the slurry mixture.

The third problem, that of excessive gradients, can be resolved in two ways. First, a properly designed and placed envelope will prevent massive flow of particles under high gradients, but not completely stop it. Some sedimentation is acceptable, but should stop within hours after construction. Second, to avoid excessive gradients one must ensure that the pipes are full of water during construction by not pumping from the lower end, or allowing water to evacuate from the lower end. This will also prevent flotation of the pipe before the backfill is in place. Pumping can start after the backfilling has been completed, and even then, the withdrawal of water from the pipe system should be very gradual so that the system remains flowing under submerged conditions until the water table drops below the top of the pipe in lowest section of the system. With singular drainage systems, where water discharges directly in an open drain, the temporary blockage at the end of the pipe should be removed gradually until the water table drops below the top of the pipe. After this has been done, it is assumed that consolidation around the envelope has taken place and a natural filter build-up has started. Then, any future high gradients should cause no problems. Destruction of the natural filter will take place with flow reversal, in which case the first solution is the only answer: a well-designed and correctly placed filtering envelope.

Under irrigated conditions in dry climates without monsoon rainfall, sink holes may appear even two or three years after construction along certain sections of the drains, when the trench backfill has not been exposed to irrigation and/or a large enough rainfall to consolidate the trench properly.

To prevent delayed sinkhole formation, and possible envelope failure, backfill and consolidation procedures should be prescribed in the specifications, using one of the various methods available for trench backfill and consolidation as shown in Figure 26. Sinkhole development should be stopped immediately. Methods 3, 4, 5, and 6, or combinations of them were deemed to have most chance of success. Method 8 is the one recommended by Willardson (1992) and was the one successfully applied in the Fourth Drainage Project (FDP) in Pakistan (Vlotman *et al.* 1992). Method 3B was used successfully in the Mardan project in Pakistan, possibly because trench collapse due to unstable soil conditions was generally not a problem. In the FDP the method would most likely not have worked because the problematic macro-pores and unconsolidated backfill would still be below the reach of the roller with drains being laid at depths of 1.8 – 3.6 m depth. Controlled backfilling is not possible when trenches collapse (Figure 25B).

Frogge and Sanders (1977) report success in prevention of sinkholes in heavy soils by controlled puddling of the backfill.

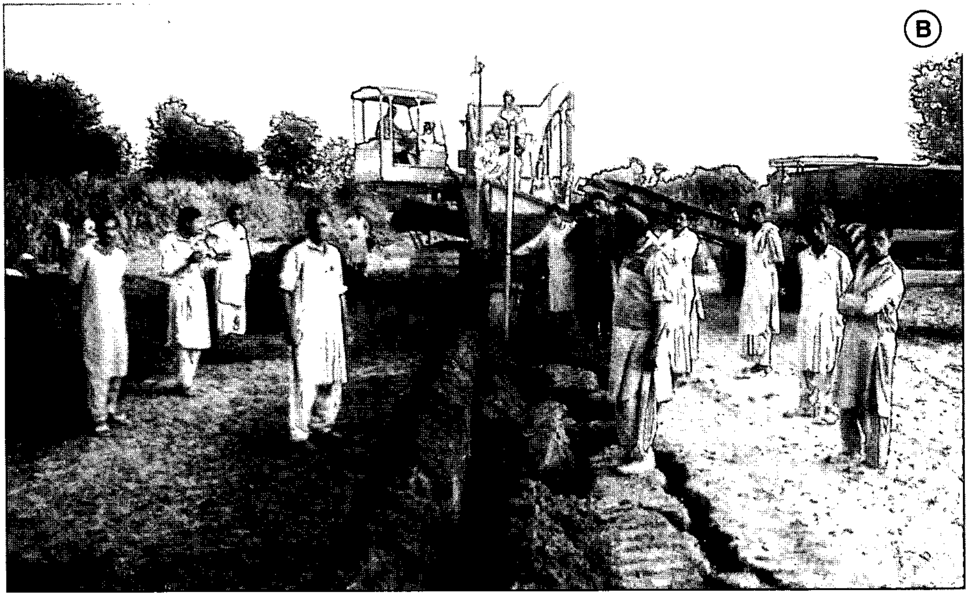


Figure 25 Trench backfill in practice.
A Stable trench allows application of compacted blinding above drainpipe with gravel envelope, using a long pole (Mardan SCARP, Pakistan).
B Caving-in of trench wall in unstable soils at the Fourth Drainage Project, Pakistan.

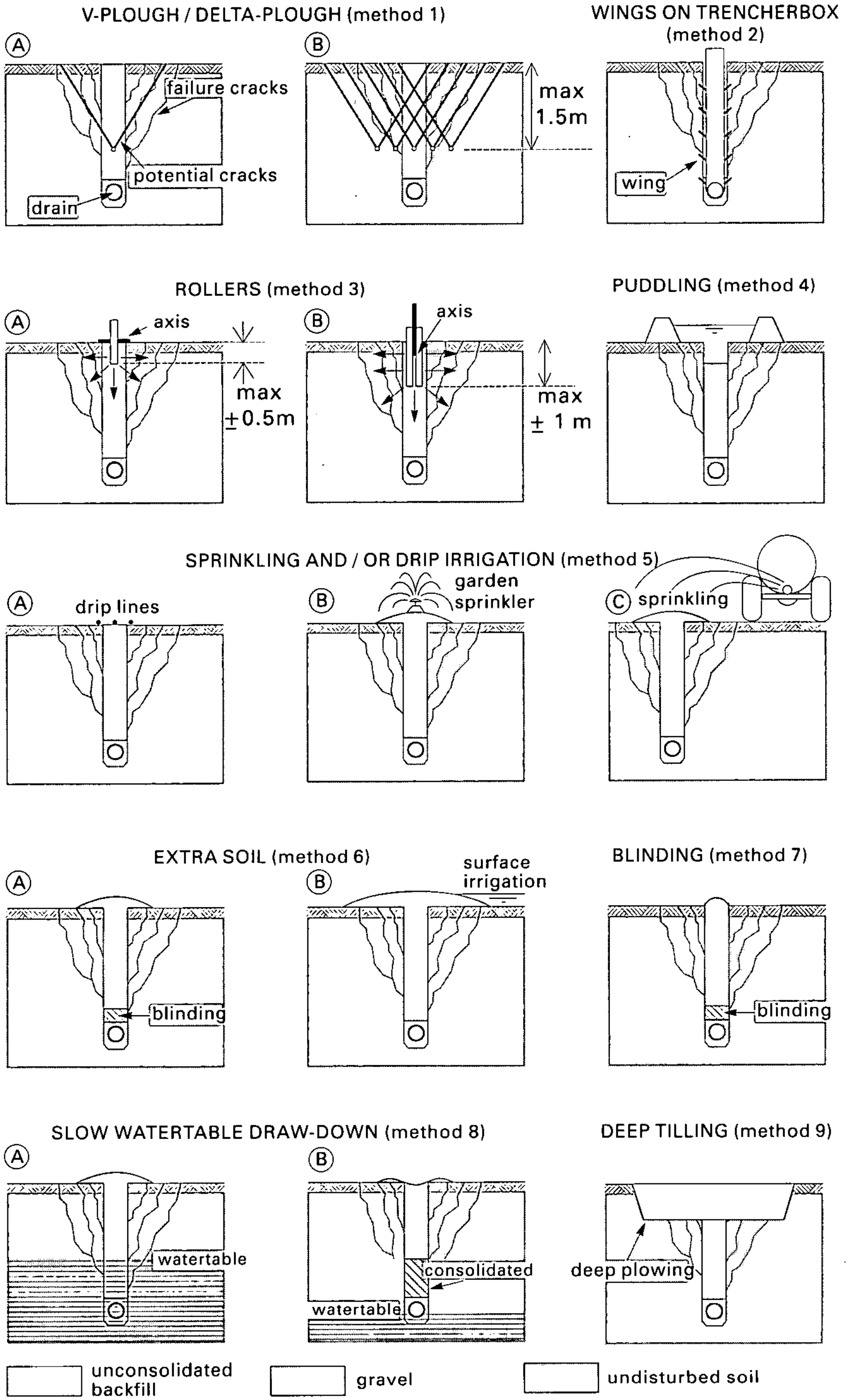


Figure 26 Trench backfill methodologies.

4.4 Quality control during construction

Construction quality control for drain envelopes starts at the quarry for gravel/sand envelopes and in the factory for synthetic envelopes. Quality control of gravel envelopes include checking for missing particle ranges, while the aspect of potential segregation during transport also needs to be taken into account. Screening out large particle sizes, and avoiding oblong particles as prescribed in Section 3.3 will reduce the risk of segregation during transport and re-handling of the envelope. Gravel flowability characteristics should be considered carefully, when laying drains below the water table and without a gravel auger at the outlet of the trencher box. When drains with gravel envelopes are planned in areas where high water tables are likely during construction, a power or gravel auger at the outlet of the trencher box is recommended.

When constructing synthetic envelopes, quality control of the O_{90} by taking random samples according to standard procedures (see Section 5.6.1) is essential for fabrics and for pre-wrapped loose materials (PLM). Apart from a good and appropriate design, wrapping, overlap and stitching/sewing quality is the next quality control aspect. Random sampling will be required to assure the adequacy of the material. Poor quality is usually easily visible (Figure 21), but more elaborate testing may be necessary when the strength of the overlap plays a role (for suggestions concerning actual strength required see Box 8 and Section 5.6.4). Proper handling in the field is essential to avoid damaging the fabric (Figure 21D), for instance, pipes or rolls of pipes should never be lifted by grabbing the fabric; it is likely to tear.

4.5 Operation and maintenance

Operation and maintenance of envelopes involves the proper operation and maintenance of subsurface drains as well. In most cases when the drain discharges freely into open drains there is little to speak of by way of operation of a drainage system. With pumped outlet systems, cycle times and sump capacity need to be determined (or are set on automatic operation depending on water levels). Operation affects the envelopes primarily when surging and reversal of flow takes place. Maintenance of envelopes is mostly not possible, although flushing methodologies can affect the envelopes through the perforations and gaps in the pipe system. Brief descriptions of operation and maintenance practices that may affect the envelopes and envelope functioning are described in the following sections.

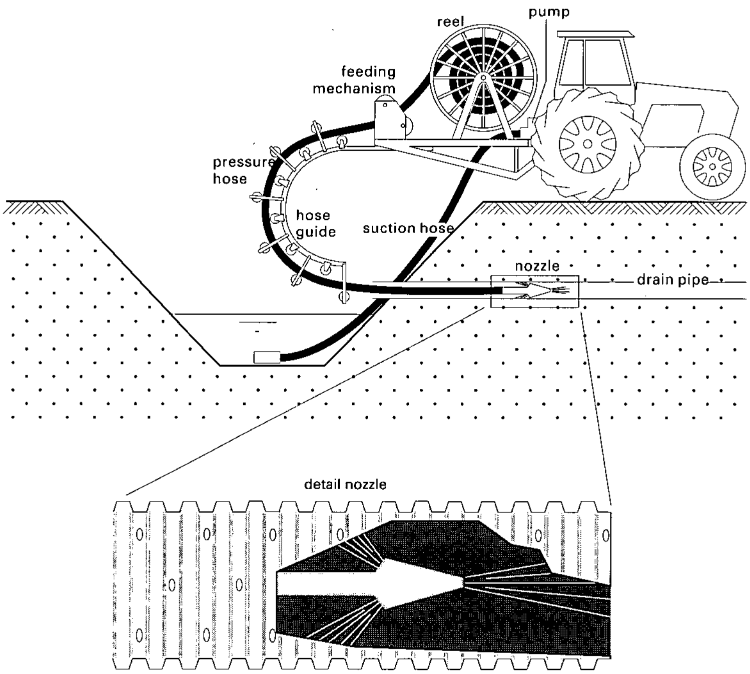


Figure 27 Drain maintenance: tractor-mounted flushing machine.

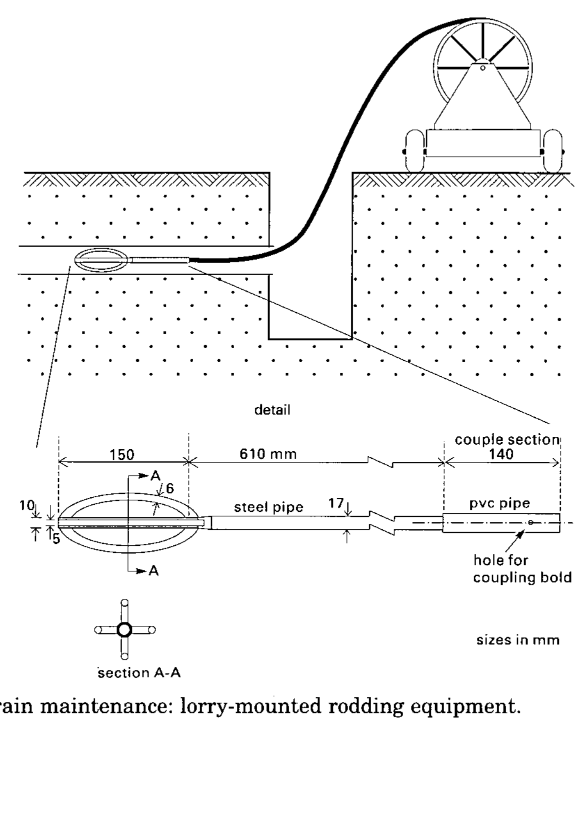


Figure 28 Drain maintenance: lorry-mounted rodding equipment.

4.5.1 Operation

Controlled drainage and sub-irrigation via subsurface drainage systems have recently become the standard mode of operation of drainage systems. This is the result of the increased focus by water management agencies on the reduction of impacts of (poor) water quality on downstream areas. These impacts can be reduced by controlling the discharge from the system during certain periods, to minimise water losses (i.e. from rice fields) and to prevent flushing of agricultural chemicals (fertilisers and pesticides). This means that water levels are allowed to build up in certain periods, and are drained subsequently. There is a danger of surging in the pipe system and potentially temporary high gradients at pipe-envelope and envelope-soil interfaces. These high gradients need to be prevented as much as possible by gradually lowering the water table (over a period of a week or more). When subsurface drainage systems are used for sub-irrigation, reversal of flow will occur that might destroy natural filters established during the drainage cycle because of high reverse gradients. As bridging may or may not be re-established, special attention needs to be given to designing proper filters when reversal of flow is anticipated. Where this is not done, reversed flows might not be sustainable and increased flushing may be necessary to keep the drains free of sediment.

4.5.2 Maintenance

Regular inspection and evaluation (Section 4.6) of the performance of the drains and the implementation of a regular inspection and maintenance programme will protect the drainage investment. A number of tools (Figure 27, 28 and 29) and a number of structures built in the system can assist with proper inspection and maintenance (Cavelaars *et al.* 1994).

Inspection tools include: video inspection equipment, rodding devices and flushing equipment (using the last-mentioned as rodding device without the flushing turned on), which can enter the system at outlets, manholes or special access constructions. Naturally, when problems are suspected, another option of inspection is to excavate the drains, but only as a last resort. Installation of special observation wells could assist in locating the problems, but are generally only used for long-term monitoring or research purposes (Section 4.6).

In a field with a well-designed drainage system, two indications of developing problems are: poor crop growth in certain parts of the field, and localised wet areas when the rest of the field is dry. Local wet spots however can also be caused by deteriorated soil conditions (e.g. slaking of topsoil, sodicity). When drains do not function properly in arid areas, the ground becomes covered

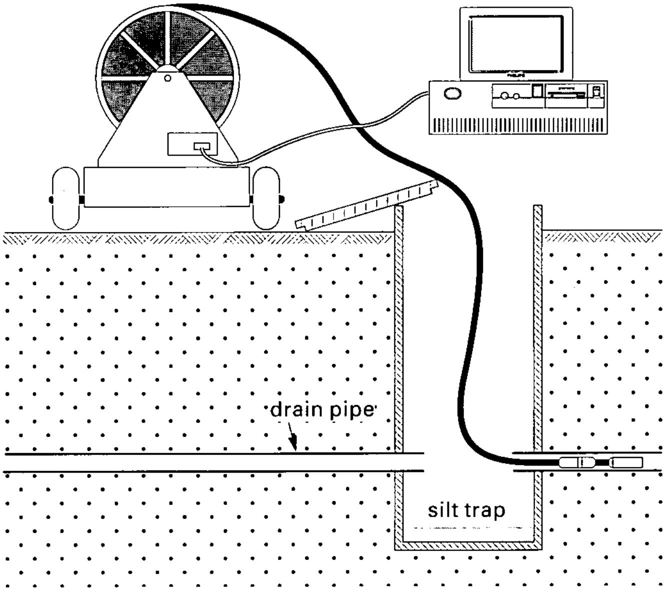


Figure 29 Drain inspection with video equipment.

with a white crust of salt and only salt-tolerant plants will grow. Diagnosis of drain failure at an early stage is extremely important. Problems should be corrected before excessive damage occurs to the crops and the soil. Flushing of drains is an economical way of restoring drain performance.

4.5.3 Flushing

Flushing of drains is one of the more popular methods for the maintenance of drains. The frequency of flushing is highly dependent on local conditions, ranging from several times per year (other measures may have to be considered in such a case) to once every 3 to 5 years (van Zeijts and Bons 1993). Flushing is done: to remove sediment in the pipe (particle sizes less than 0.05 - 0.1 mm); to remove roots (young roots with high pressure only); to remove local blockages caused by sediment (when gaps between clay/concrete pipes are too wide); to remove chemical deposits (iron deposits and slime); or to reopen blocked perforations if possible.

Flushing is done from the downstream end of the drain by pumping water into the drain through a hose that is inserted in the drain by either pushing, or with the help of the high-pressure jet (Figure 27). When the deposits become suspended they are carried out of the drain provided the flow velocity remains sufficiently high. Discharges of 3 - 4 l/s are commonly used (Cavelaars *et al.* 1994).

Flushing units are commonly categorised as follows (Van Zeijts and Bons 1993):

1. high-pressure equipment: pressure at the pump > 60 bar (6000 kPa);
2. medium-pressure equipment: pressure at the pump 20 - 35 bar (up to 50 bar); and
3. low-pressure equipment: pressure at the pump < 20 bar.

The pressure at the nozzle is approximately 50% of that of the pump due to friction losses in the hose. Hoses are either reinforced rubber, which is flexible and can withstand pressures of up to 100 bar, or polyethylene, which can handle pressures of up to 35-50 bar.

High-pressure flushing is a proven method of flushing soil and plant roots from drains. The high-pressure hose and jet nozzle is introduced into the lower end of a section drain. The jet nozzle has some backward jets that pull the hose into the pipe and then sweep out the loosened material when the hose is pulled from the pipe. Sections of drainpipe up to 200 m long can be cleaned effectively and conveniently. Longer lengths (up to 700 m) are possible but at the expense of the nozzle pressure and effectiveness. Some chemical deposits and most roots, as well as deposited soil, can also be removed by jet cleaning. Jet cleaning might also improve the functioning of some drain envelopes as the jets force water out through the drain perforations. The action is similar to 'developing' a well. Care should be taken to keep the jet moving in the pipe at all times to avoid damage to the pipe and envelope materials, as the high-pressure streams of water issuing from the jet nozzle can destroy the pipe if left stationary.

Medium- and low-pressure flushing has increasingly become the preferred method of drain cleaning in Europe, as high-pressure jets tended to enhance sedimentation under certain conditions (Van Zeijts and Bons 1993). These types of equipment do not have enough pressure at the nozzle to move the hose into the drain and special driving mechanisms that push or pull the hose with nozzle have been developed. This limits the maximum flushing length to about 150 - 200 m.

If drains are known to be susceptible to sedimentation, regular flushing (annually or more frequently) is recommended. Flushing should be done before the sediments cake together after successive drying and wetting. High-pressure flushing (50 bar at the machine and ± 30 at the nozzle) is not recommended for drains with large perforations or where there are large gaps between tiles (5 mm or more) as it may stir up well-established natural granular filters. In particular, with older systems that use concrete or clay tiles, high pressure could cause additional sedimentation at the joints. Experience in the Netherlands has also shown that high-pressure flushing of plastic perforated drainpipes did not result in cleaning the PLM envelope (Scholten 1987). Flushing the pipe by simply introducing a large flow of water at the upper end of the drain will not be successful. Especially in arid regions, plant roots might enter and clog up the drains. These can also be removed by flushing with a high-pressure water jet, provided it is done annually before the roots have had the chance to become woody.

Removal of iron ochre by jetting is possible, though the success depends on the type of ochre deposits and the pressure of the jet. Ochre that contains considerable organic carbon is the stickiest. Yet, ochre will not necessarily stick to plastic, glass or other artificial surfaces, except in certain Fe complexes when the bacteria are dead.

4.5.4 Chemical cleaning

In areas where the water has a high iron content, biological clogging of drains and drain openings iron ochre deposition could occur. Iron ochre is precipitated iron oxide and is easily recognisable by its red to yellowish colour and can be removed by jet cleaning or by chemical treatment. Grass *et al.* (1975) describe the removal of oxides of iron and manganese (black deposits in drains) using sulphurous acid. Sulphurous acid is a strong reducing acid made by combining sulphur dioxide gas and water inside a drain. The outlet of the drain is closed and the drain is filled from its upper end with sulphurous acid. The acid remains in the drain for approximately two days and is then released, after which time the acid has become neutralised and has therefore a minimum environmental effect on the water quality in the collector drain.

A recurring iron ochre problem might require periodical treatment. If the iron ochre problem is by poor drainage conditions in a soil profile because of a temporarily clogged drain, the problem can be eliminated by restoring the function of the drain by cleaning. If the source of the iron ochre is the groundwater, then periodic cleaning will be necessary. From experience gained in the Netherlands, annual flushing has been shown to be necessary when severe iron ochre deposits occurred, while for minor problems flushing once every six years was sufficient (Scholten 1989).

To combat iron ochre formation the use of copper slag around drains, if available, can be used to serve as a bactericide. Alternatively, copper solutions can also keep the insides of drains free of ochre, but this could have an undesirable environmental impact (McKyes *et al.* 1992).

Incorporating organic materials with a high tannic acid content in the envelope material has also been tried in an attempt to control iron ochre formation

(Scholten 1989, Ford 1975). To decompose the tannic acid a lot of oxygen is required which will then prevent iron (Fe^{2+}) from oxidising (see Section 5.5.5). The iron then remains in soluble form and can be discharged through the system. This solution was found to work when a temporary iron problem arose, i.e. during the first couple of years after installation. The tannic acid remained effective for two to four years. The disadvantage of this method is its temporary nature and the possible oxygen deprivation in the disposal (surface) drains, as well as blue colouring of the water.

4.6 Monitoring and evaluation of envelope functioning

Monitoring and evaluation of the functioning of the drain envelope is closely related to judgement of the functioning of the drainage system as a whole. The very first indicator that something is not right is a persistent high water table. Hence regular monitoring of water tables over time and over the drained area are essential. To judge the performance of envelopes, drain discharges and hydraulic conductivity measurements are needed (Section 2.4.1). The frequency of measurement depends on the purpose:

- for long-term monitoring (3 - 5 years or more) monthly, or fortnightly observations might be sufficient;
- for detailed observations of draw-down and for measuring a range of entrance resistance values, daily or even continuous measurements during several peak flow periods will be required.

Grids of observation wells (Section 2.4.1), typical of pre-drainage investigations, are not usually used to judge envelope performance. Instead, nested sets of observation wells and/or wells installed around the drain as shown in Figure 30 are common.

Experience (by the authors of this book) has shown that regular measurements at set intervals, combined with continuous observations (i.e. with automatic water level recorders) at some representative locations, provide the best results. Variable intervals of measurements, which may be selected to save on labour and time, invariably lead to missing the first event and require skilled field labourers. Of course field staff can be trained, but this generally requires a long-term commitment of all parties involved which is often not the case and hence the recommendation for simple fixed-interval measurements (including daily or hourly if so desired).

Monitoring and evaluation is seen as a continuous activity. Questions that arise about the functioning of certain components or particular systems a short-term (maximum six months) can be answered by a performance assessment. However, the purpose of the performance assessment and the indicators that will be used must be clearly specified beforehand. As Smedema and Vlotman (1996) describe: "Drainage performance assessment is done to determine the functioning of the drainage system compared with established design criteria, and to identify the cause of malfunctioning". Here, this definition can be further detailed by specifying that the performance assessment be executed to determine that the drain envelope functions according to the design criteria. The performance assessment parameters need to stay within the established indicator ranges such as shown in Box 14 and Table 8 p 128), when conditions are similar to those indicated in the box. Sediment deposits should remain below the values indicated further down in this section.

Regularly quoted tables with indicator values for entrance resistance are those in Dieleman and Trafford (1976), Box 14a, and b. However, the entrance resistance values given in their table are subject to the soil hydraulic conductivity as well as the irrigated conditions (Box 14b). Moreover, the values in Box 14a, are only valid for conditions close to the design discharge, whereas Box 14c, takes out all but one site-specific factor when the indicator that includes K is used. The only factor not taken care of is the distance from the drain centre at which h_e is measured which is also elaborated on below. The relative entrance head (h_e/h_t) was considered for conditions where the design depth of the drains is between 0.5 and 1.4 m, with corresponding midway design water table depth of 0.20 m for pastures and 1.0 m for irrigated conditions such as the Nile Delta. When drain depths are 1.5 - 3 m, the design midway water table is 1.2 m with drain spacing ranging between 150 and 300 m (such as the irrigated conditions prevalent in Pakistan, where salinity control is the prime design criteria), caution in the use of he/ht as given in Box 14c, should be exercised.

When reporting average values of h_e, they should always be presented with the total number of observations and the Coefficient of Variation (CV= standard deviation/average). The coefficient of variation in particular will give an impression of how well the average represents the data set (NB the use of CV applies to all kinds of variables for which reporting of average values is useful). The following ranges are recommended:

CV < 0.25	average is representative;
CV = 0.25 - 0.5	average is moderately representative;
CV = 0.5 - 0.75	average is poorly representing the data set;
CV > 0.75	average is meaningless for reporting purpose.

To compare results from various projects, it is essential that the location of the wells from which the h_e is determined be described in some detail. For instance the distance from the outside of the pipe, and the distance from the outside of the envelope, as well as the distance of the trench wall from the outside of the drainpipe, if applicable, are important parameters to judge the relative magnitude of h_e and ratios that use h_e (Figure 30). This is particularly important when the gradient of the water table is steep at the point of observation; a small difference in distance from the drain could make a large difference in the measured h_e value.

Apart from entrance resistance, sedimentation in manholes and drain lines are also indicators of poor envelope performance. In the Netherlands a layer of 15 mm sediment in 60 mm diameter drainpipes is not acceptable. For the Rajad Project (Rajad Project staff 1995) in India, envelopes were recommended where drainpipes without envelopes contained more than 30 mm of sediment (nominal diameter of laterals: 80 and 100 mm; criterion not differentiated between two diameters). Broughton *et al.* (1987) mention that the functional life of a drain could be considered to be over when the sediment depth averages half a diameter along the drain length. Others have expressed acceptance of sediment in a pipe indirectly in the form of the safety factor used to determine the dimensions of the drainpipe. Typical safety coefficients are 40% for laterals and 25% for collector drains, although considerably higher values of 75% and 60%, respectively, have been reported as well (Cavelaars *et al.* 1994).

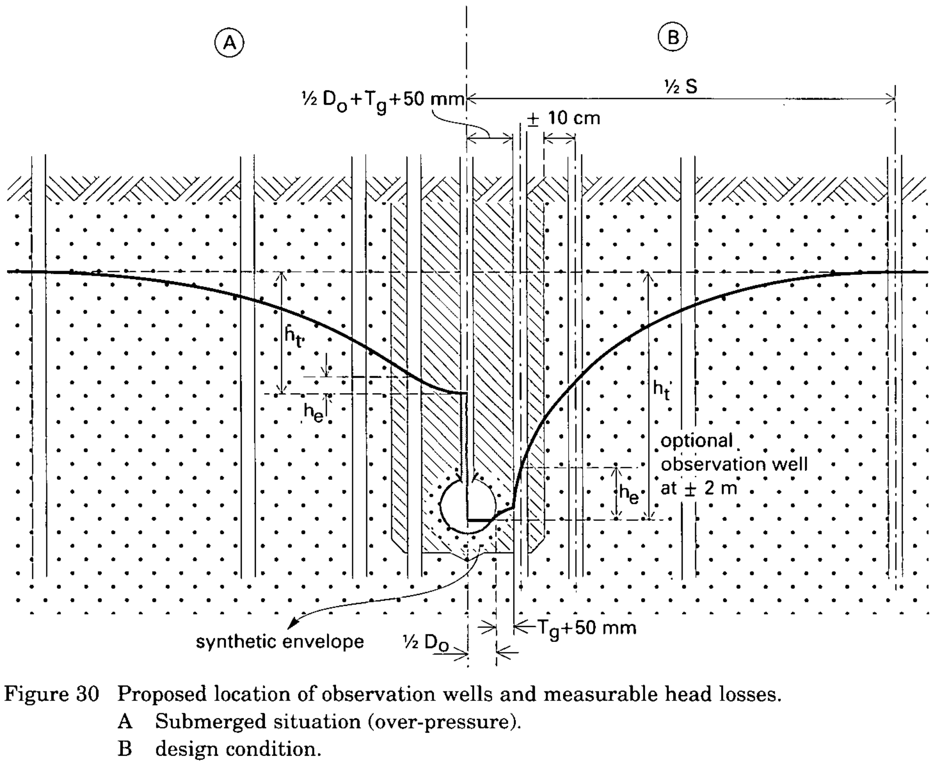


Figure 30 Proposed location of observation wells and measurable head losses.
A Submerged situation (over-pressure).
B design condition.

The effect of sediment on the hydraulic performance of the drain for a full-flowing pipe is shown in Figure 31. It is up to the individual designer, and later, the person who assesses the performance, to select the appropriate safety factor and the acceptable amount of sediment in view of flushing intensities. Flushing intensities can be obtained from local experience. Some reported flushing intensities have been described in Section 4.5.3.

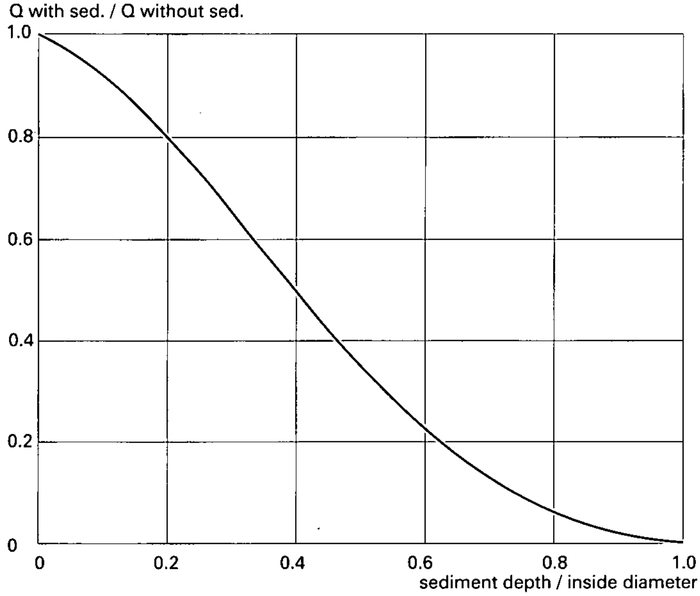


Figure 31 Reduction of discharge as a function of the sediment in the pipe. (Cavelaars *et al.* 1994)

4.7 Field trials

Field trials or pilot area testing are regarded by some as the only way to adequately assess the functioning of drain envelope material. However, there will always be room for laboratory experiments: to select the material-soil combinations that are best tested in the field and to limit the number different envelope combinations to be tried under the much more expensive field testing. In addition, methodologies to assess the need for envelopes will be mostly based on laboratory-measured indicators. Laboratory experiments are both less costly and less time-consuming. Yet, it might be hard to relate the laboratory results to field conditions, and hence pilot area testing is often desirable. For large projects in particular, field tests can be helpful in the selection of materials, design and construction procedures, provided they are completed before final designs are made.

4.7.1 Methodology of field testing

The selection and design of the drain envelope are but two aspects of a successful drain envelope installation: proper handling and construction techniques are also essential for the ultimate successful installation. For this reason field trials should comprise the following distinctly different assessments:

1. Pre-drainage soil and water investigations. All factors that affect the performance of drain envelopes and that determine the need for a drain envelope must be determined and properly documented in a way that allows valid judgements to be made by scientists that are neither directly involved nor familiar with detailed local conditions (see Section 2.4).
2. Construction quality control monitoring. Improper handling of envelope material may lead to segregation of gravel particles or damage to organic and fabric envelopes. Use of improper machinery, construction under unfavourable soil-water conditions (smearing), inadequately protected joints and couplers, and improper backfill procedures can lead to high entrance resistance and/or sediment in the pipe. Hence, it is essential that detailed monitoring of what actually happens during the construction should take place, and this needs to be documented in published reports (see also Section 4.4).
3. Post-construction quality control investigations. The very first investigation after construction is to verify grade, elevation and integrity control of the constructed drain lines. This may be done by: rodding or pulling an inspection cage through the pipe; video inspection; and using elevation grade control equipment. When this type of equipment is not available, excavations must be done either randomly, or at locations where construction observations make one suspect poor quality construction. Post construction maps (as built) should be prepared.
4. Performance assessment to determine entrance resistance and sediment occurrence. Entrance resistance can be measured by installing observation wells at appropriate locations and measuring drain discharge at the same time as the other observations. The observations wells should be installed as follows (Figure 30 and Figure 33):
   - One with access into the pipe (Figure 32).
   - One just outside the envelope but not against it (i.e. 50 mm from it). Make sure not to touch the envelope, this observation well will be in the trench where K_s is expected to be higher than outside in the undisturbed soil. If there is no trench this observation well and the one outside the trench wall are one and the same.
   - One just outside the trench wall (i.e. 10 cm).
   - One midway between parallel drains.
   - Optionally: one observation well could be installed two metres away from the drain. This would serve to assess whether there is a gradual

Box 14 Indicators to evaluate entrance resistance

a *- For drainage conditions close to the design discharge (after Dieleman and Trafford 1976).*

Head loss fraction h_e / h_t	*Drain performance*
<0.20	*good*
0.20 - 0.40	*moderate*
0.40 - 0.60	*poor*
> 0.60	*very poor*

b *- For irrigation conditions in arid zones with drain depth of 1.8 m, spacing = 50 m, water table depth two days after irrigation 1,0 m, design discharge = 4 mm/d. (after Dieleman and Trafford 1976).*

Entrance resistance $r_e = h_e / q_1$ *in d/m*	*Entrance loss* h_e *in m*	*Drain performance*
< 0.75	*< 0.15*	*good*
0.75 - 1.50	*0.15 - 0.30*	*moderate*
1.50 - 2.25	*0.30 - 0.45*	*poor*
> 2.25	*> 0.45*	*very poor*

c *- Based on theoretical assumptions used in traditional drainage design* a_e *is expected to be approx. 0.4 (after Cavelaars et al. 1944).*

Evaluation parameters (Section 5.1.1)		*Enterance Resistance*	*Drain (lateral) Performance*
$a_e = K_s w_e / S$ *(Eq. 10)*	h_e / h_t		
< 0.4	*< 0.2 - 0.3*	*Normal*	*good*
0.4 - 1.5	*0.3 - 0.6*	*High*	*moderate to poor*
> 1.5	*> 0.6*	*Excessive*	*very poor*

The above is based on conditions where the design depth of the drains is between 0.5 and 1.4 m, with corresponding midway design water table depth of 0.20 m and 1.0 m respectively.

Note:

- r_e *is the entrance resistance per unit discharge in d/m;*
- a_e *is the total entrance resistance contraction constant, dimensionless;*
- h_e *the head loss determines as difference between observation well closest to the drain (near the envelope-soil interface, or just outside the trench boundary) and the water level in the drainpipe in m (Eq. 12 and Figure 30);*
- h_t *head midway between drains in m with respect to selected reference level (usually centre line of the drain, but better is to take the invert of the drain when to be used with entrance loss calculations);*
- K_s *hydraulic conductivity of the undisturbed soil adjecent the observation wells in m/d;*
- S *the drain spacing in m; and*
- w_e *the total entrance resistance (*$w_c + w_r$*, see Section 5.1.1) in d (Eq.9).*

or sudden drop near the drain, indicating possible considerable difference in vertical and horizontal hydraulic conductivity.
- Finally, if there are distinct soil layers, nested sets of piezometers, rather than observation wells might have to be installed (Figure 33).

Sedimentation can be observed at manholes, or using video inspection equipment. Hydraulic conductivity measurements during the pre-drainage investigation with the auger hole method are acceptable to determine site independent entrance resistance coefficients (Section 5.1), provided the trouble spots (if any) are covered by the standard grid of the soils investigation (Section 2.4.1). If not, additional hydraulic conductivity measurements will have to be performed.

Some essential characteristics of good field trials are:
1. Limit the variable investigated to one only. For instance, when different envelopes are being tested, spacing and depth investigations should not be done in the same plots without proper statistical design. Preferably all but one variable should be constant in the various replications. Random block designs may be used.
2. Install several test lines that are likely to fail. Only when failure occurs can the boundaries at which they are likely to occur be established. This is true for both field and laboratory tests.
3. Be prepared to replace drain lines that are expected to fail, after they have failed.
4. Monitor outside influences, i.e. place sufficient water table observation wells outside the immediate experimental area, to assess normal undrained water table contour (elevation) and depth lines.

Because of the high costs of conducting field experiments on drainage technology, such experiments should be carefully planned and fully instrumented. Replication of the drainage treatments is very important and the installation conditions should be standardised to eliminate them as a cause of differences in drain performance. Measurements should be made with sufficient frequency, over a sufficiently long period of time, to allow meaningful interpretation of the results.

The data collection intensity should be high in the beginning when the drains first begin to function. As time progresses, the frequency of measurement can be reduced. Measure drain discharge and the position of the water table at and between the drains. Estimate the recharge rate and compare with the discharge from the drains to evaluate the response of the drains and to determine whether the water table drawdown rate is acceptable.

Theoretically, when drains are functioning properly, i.e. they are not operat-

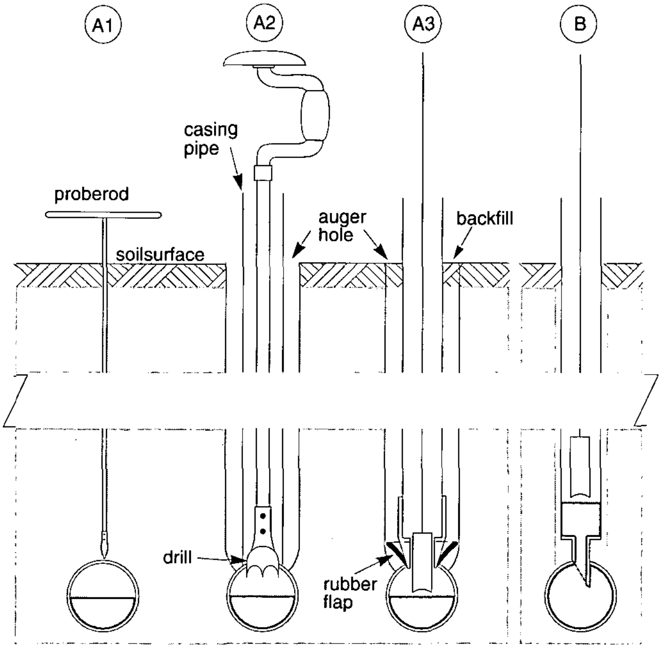


Figure 32 Installation of piezometer to measure overpressure or water level in drainpipe.

A1 The drainpipe has to be located with a probe rod.

A2 An auger hole is made to the drainpipe. The hole is freed from soil and mud by using a small diameter pipe like a pipette. A circular hole is drilled into the drainpipe. The drill is kept into place by an appropriate casing pipe.

A3 The piezometer pipe with a diameter matching the drill, or with a tapered end such that the pipe does not penetrate too far into the drainpipe, is placed into the hole. The rubber flap is to tightly seal off the piezometer and the drainpipe. The augerhole is backfilled with soil. If the pipe is to be removed later, care should be taken to properly seal the hole.

B If the only purpose is to measure over pressure (and not the water level inside the pipe), this construction will work (Karaman et al. 1997). The sharp hollow point at the bottom of the piezometer is punched through the pipe after a matching augerhole has been drilled.

ing with overpressure and the outlet is not submerged, there should never be water above the drains. If the drainpipe is submerged, the envelope or entrance resistance is too high, indicating a clogged envelope, clogged drain openings, or an envelope that has inadequate permeability. Some examples of the situations that one might encounter are given in Figure 33. Progressive clogging can be detected by periodic measurements of drain performance and water table response under similar recharge conditions. Water standing above a drain might also be due to too wide a spacing, and the diameter of the lateral drainpipe too small to convey the full flow under non-pressurised conditions (Gammal *et al.* 1995).

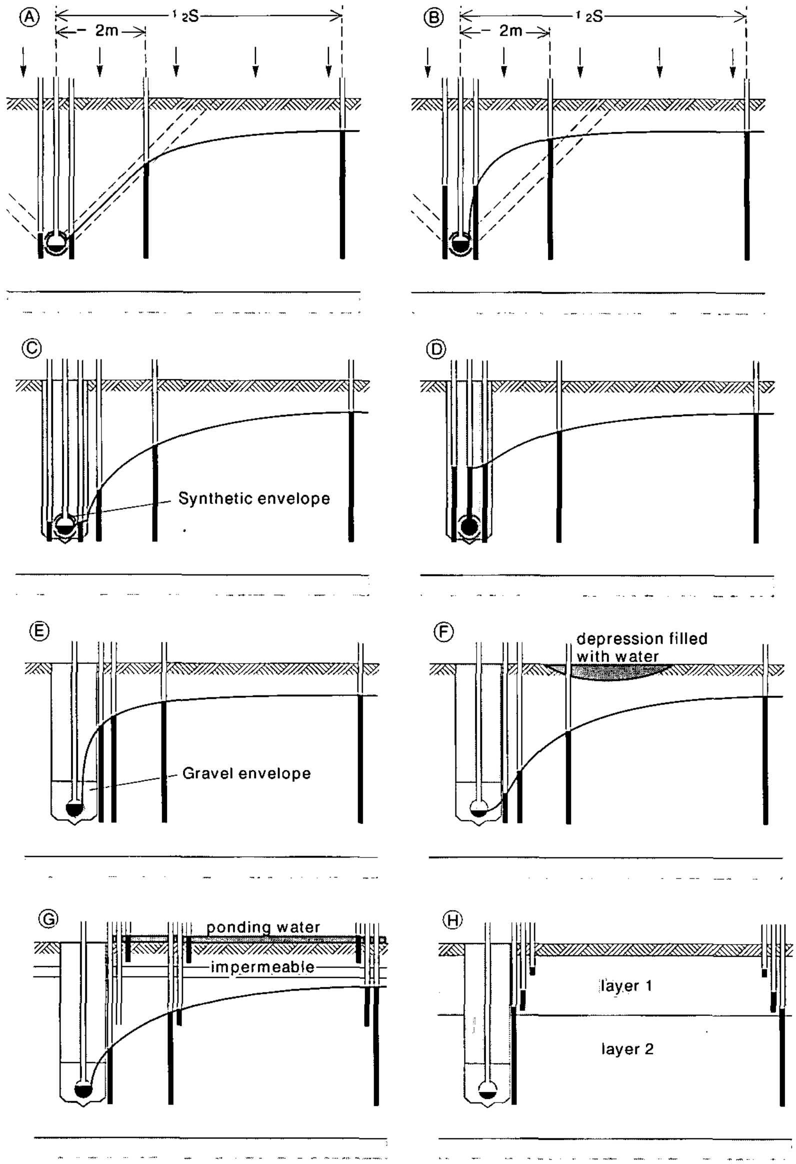


Figure 33 Examples of the (mal-) functioning drains and drain envelopes.
A V-plough, no trench, no envelope, crack causes sudden drop.
B V-plough, no trench, synthetic envelope, high entrance resistance.
C Trencher, synthetic envelope, design situation.
D Trencher, synthetic envelope, overpressure.
E Trencher, gravel envelope, low vertical K_s, high K_s in trench.
F Trencher, gravel envelope, normal functioning, low infiltration.
G Trencher, gravel envelope, waterlogging at land surface, perched water on plough pan.
H Trencher, gravel envelope, low K_s top layer, high permeable layer underneath.

When no field trials have been executed observations of existing drains can be useful provided that the necessary soil and envelope information of those drains can be obtained (either already exists or can be collected from the field). There is no replication and the installation conditions are not controllable, but useful information can be gathered.

4.7.2 Historical layout of field trials

Some traditional layouts of experimental fields and observation points are shown in Figure 34. To some extent they are characterised by a random block design with 3 - 5 replications, but others have non-traditional designs with an apparent lack of statistical relevance. The latter may be only apparent because along the drain line more than one observation can be made, which may count as a replication.

Figure 34A shows the traditional layouts recommended by Dieleman and Trafford (1976), which highlight the layout of the observation wells and the use of units and buffer zones. Three rows of observation wells at 0.25, 0.5 and 0.75 length of the drain line are shown (Figure 34C), but two rows at 1/3 and 2/3 of the length are used too (Figure 34F). Little attention has been paid to

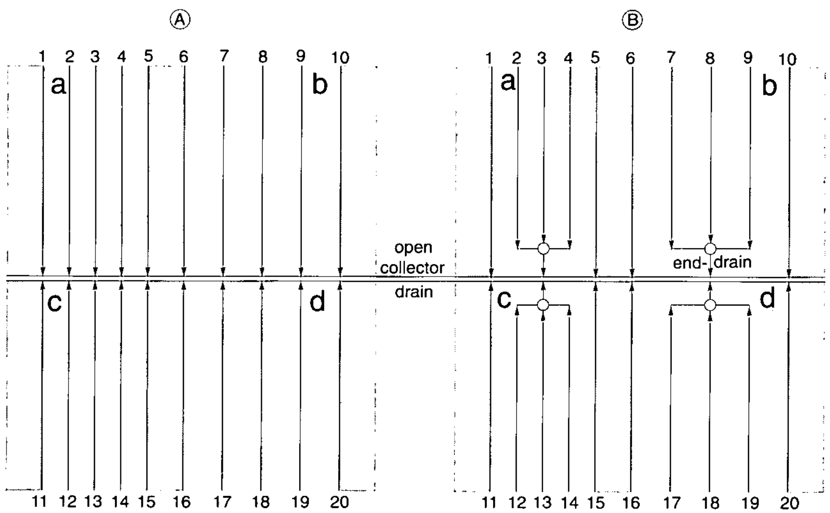


Figure 34 Field layouts used in the past.
A Experimental setup consisting of four units a,b,c and d with drains flowing individually into a collector drain (Dieleman and Trafford 1976).
B Experimental setup consisting of four units a,b,c and d with drains of each unit flowing into a manhole first and then via an end drain into the collector drain (Dieleman and Trafford 1976).

field divisions and irregular irrigation patterns along a lateral drain. Note also that different spacings are indicated: it is not advisable to mix spacing trials with envelope trials! Recommendations made by Dieleman and Trafford are primarily based on experiences in temperate climates with ample rainfall (and hence applicable to certain monsoon climates) which assure more or less uniformly distributed recharge design rates over the catchment of the particular drain.

Examples of near ideal pilot area conditions are shown in Figure 34F and 34G, except for the two units with different spacing in the Dutch experimental site. Regular field layout is not often possible in many pilot areas in Pakistan and India due to large spacing between laterals and the irregular nature of irrigated conditions (Figure 34 E, 34I and 34H). The pilot areas in India show a possible combination of spacing trials and envelope trials; the envelope trials are all within the 30 m spacing replications. Recommended layout features and observations are suggested in the next two sections.

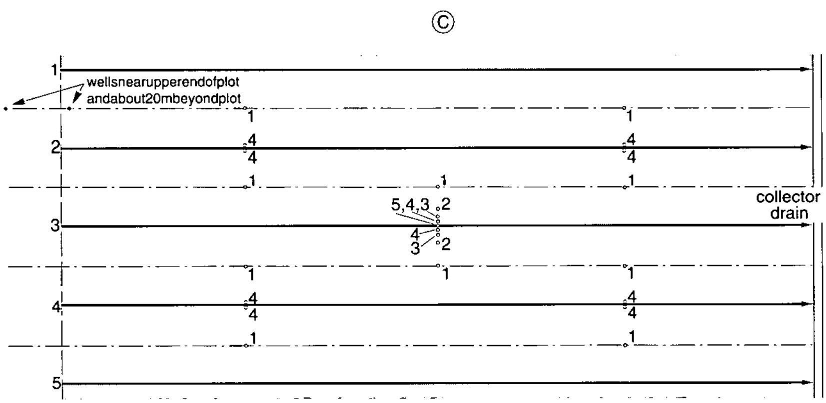
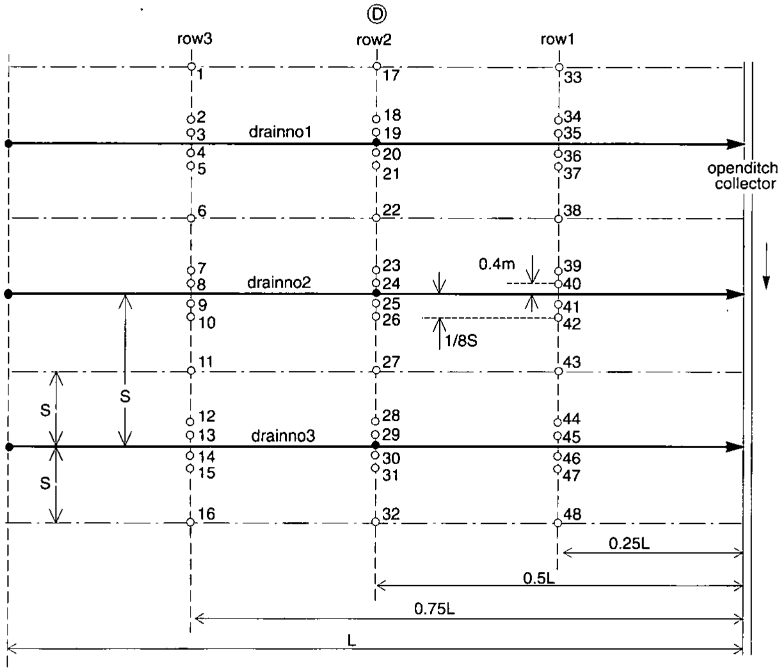


Figure 34 Cont. Field layouts used in the past.
C Detail of A and B, showing placement of observation wells. Drain numbers are same as in A and B. Placement of wells: 1) midway between drains; 2) 5 m from drain, 3) 1.5 m from drain; 4) 0.4 to 0.5 m from drain; 5) on top of drain.



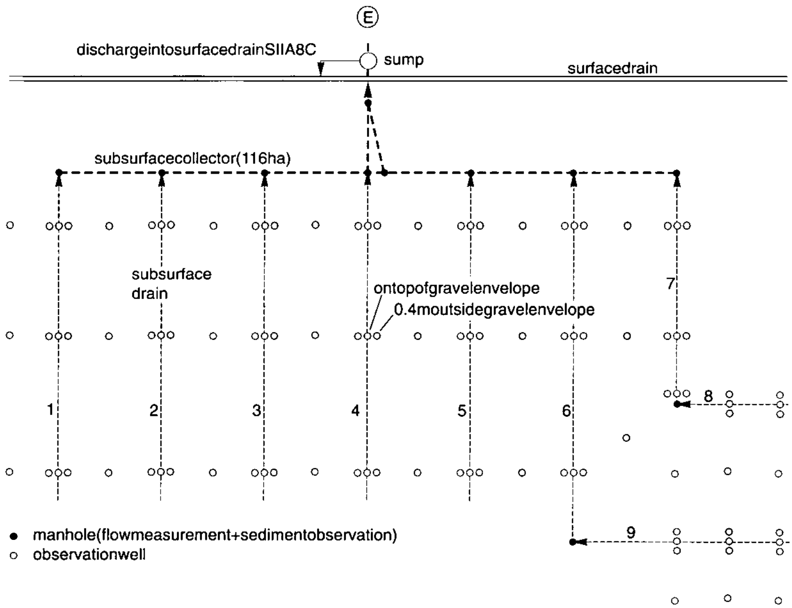
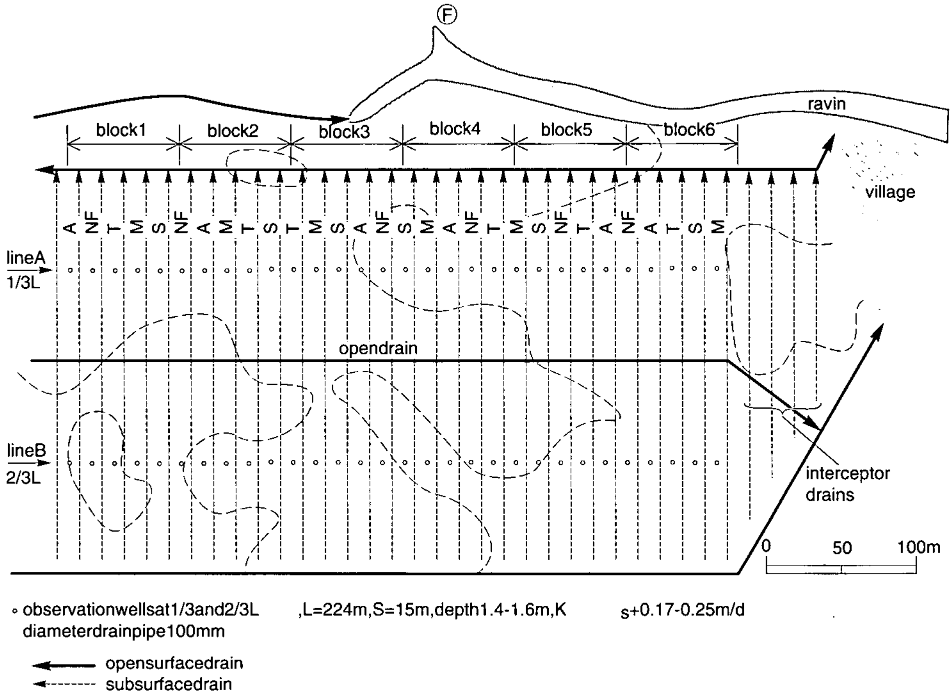


Figure 34 Cont. Field layouts used in the past.

D Typical layout showing observation well numbering system (Dieleman and Trafford 1976).

E Test layout at SIIA8C in slightly adjusted drain system. Note replication was not sought, rather, testing of as many envelopes as possible in widely spaced system of Fourth Drainage Project. Pakistan (Bhutta 1995).



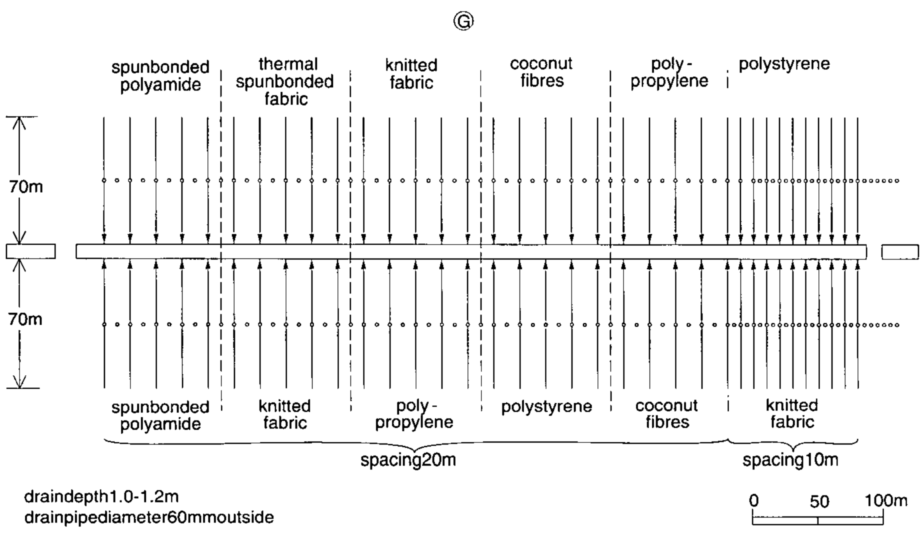


Figure 34 Cont. Field layouts used in the past.

F Subsurface drainage plan at Ormstown Field Experiment, Canada (Bolduc *et al.* 1987). Randomized complete block design. Six blocks each containing 5 lines: M – Mirafi synthetic envelope, S – Sock (Big 'O' Sock), T – Texel non-woven fabric, A = Alidrain a non-woven fabric, NF – no filter.

G Layout of drainage/infiltration pilot area Valthermond in peat area in the north-east of the Netherlands (Stuyt and Oosten 1987 and Stuyt 1992).

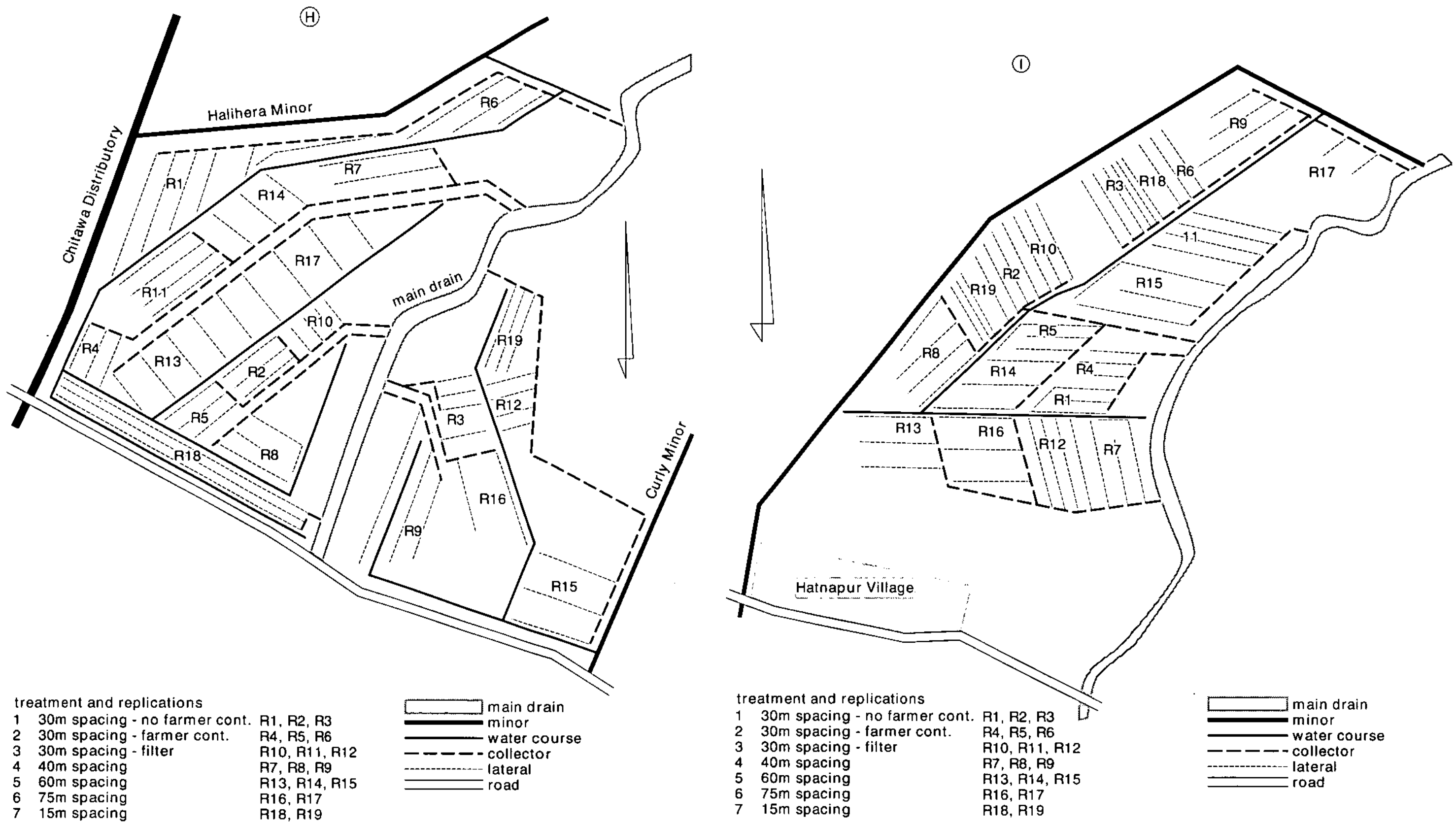


Figure 34 Cont. Field layouts used in the past.

H Ranpuria test site, India (Rajad Project staff 1995). Possible mixture of spacing and envelope trials, and because of possible setup comparison with other site(s).

I Hatnapur test site, India (Rajad Project staff 1995). Possible mixture of spacing and envelope trials.

4.7.3 Field layout for envelope testing

As mentioned earlier, of the characteristics of a good test field, the most important is trying to limit the variable investigated to one only. Essentially, there are two types of field experiments: pilot areas and research areas. The difference between the two is that in pilot areas solutions that are expected to work are tested without going to extremes; the only purpose generally being to find out which soil-envelope combination works best. Research areas are test areas where the full range of possibilities from failure to over-design are tested with the purpose of finding out the upper and lower boundaries of failure and over-design. Nevertheless, it is possible to include failure in pilot areas if proper arrangements are made to safeguard the farmers against economic losses during the experiment, and at the end of the test period, provisions are made to replace the failed tests with the best solution from the experiment.

Figure 35 gives a sampling of layout features. It is not an ideal field layout because it contains different drain spacings, and contains two methods of drain construction (V-plough and trencher). Instead, the figure is intended to show certain features possible in test sites. The following characteristics and options are shown:

1. The area selected for the experiment has been assessed for uniformly distributed soil, drainage and irrigation characteristics. It is located in a low lying area so that when the drainage system has been installed, steady recharge to the area either from irrigation or from elsewhere will assure more or less continuous exposure of the envelopes to water flow.
2. The spacing of the laterals is based on blocks with the same hydraulic conductivity (geometric mean with low coefficient of variation) and all drain depths are the same. If water contour lines are available from the pre-drainage investigation, the design of the depth and slope of the lines must be such that the observations along the drain line all yield more or less the same undrained water table height above the drain invert (e.g. the ideal hydraulic grade line in Figure 36). Collector drains should not be perforated.
3. Soils are relatively uniform (the two drain spacings shown are not significantly different); the lines without envelope are placed in the set with the wider spacing, implying lighter soils and, therefore, the likelihood of particle movement.
4. Nine envelopes are investigated and one set of drains without envelopes is installed. The lines without envelopes are expected to fail. Hence, as close as possible a drain line with an envelope known to function well is constructed ('Es' in Figure 35). Both lines are to be kept draining at the same time (i.e. do not block off the backup drain with the envelope). Both lines are of equal length and field divisions (and hence irrigation applications) are the same for both lines. If one line is longer and irrigation takes

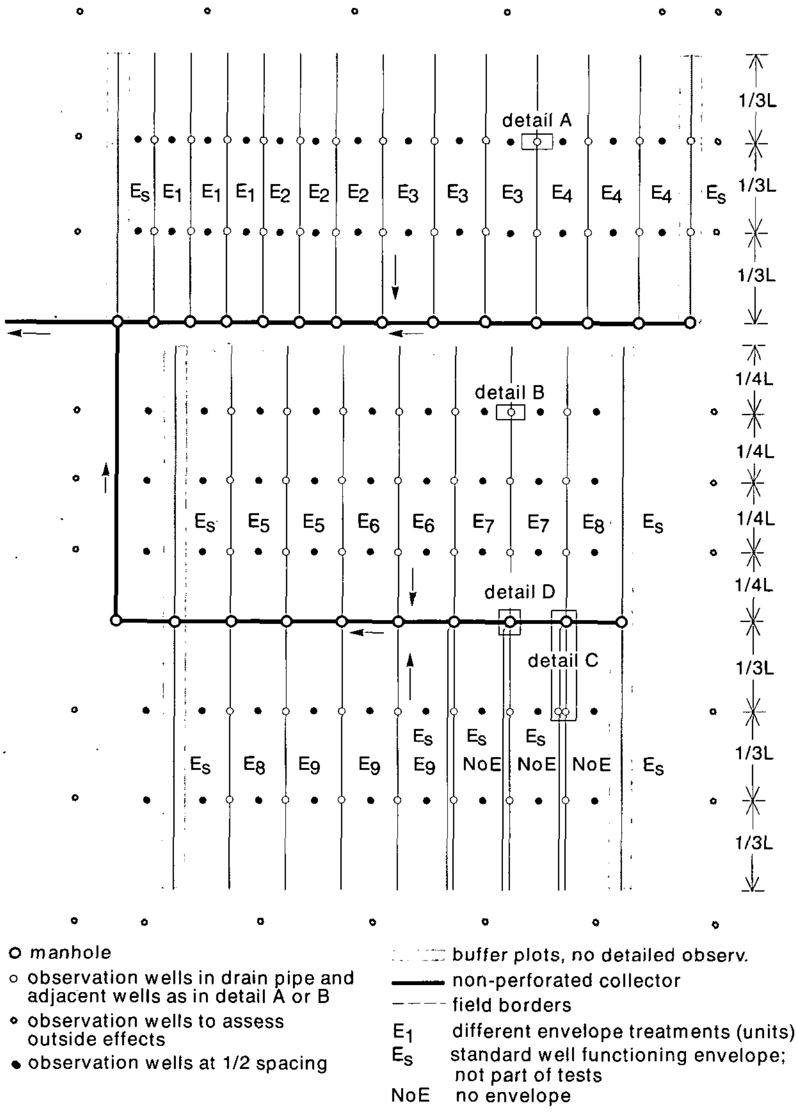


Figure 35 Recommended field layout for drain envelope research.

place in the longer section, the simultaneous discharge data of the two lines are difficult to interpret.

5. Each treatment has at least six observations (replicates) of water table levels. If more observations per treatment are feasible, more should be constructed. Experience elsewhere has shown that it is not uncommon that more than half of the observations are rendered unusable for a variety of reasons (see next section).
6. Uniform observation rows at either ⅓L and ⅔L, or ¼L, ½L, and ¾L are indicated but this division has no particular significance other than

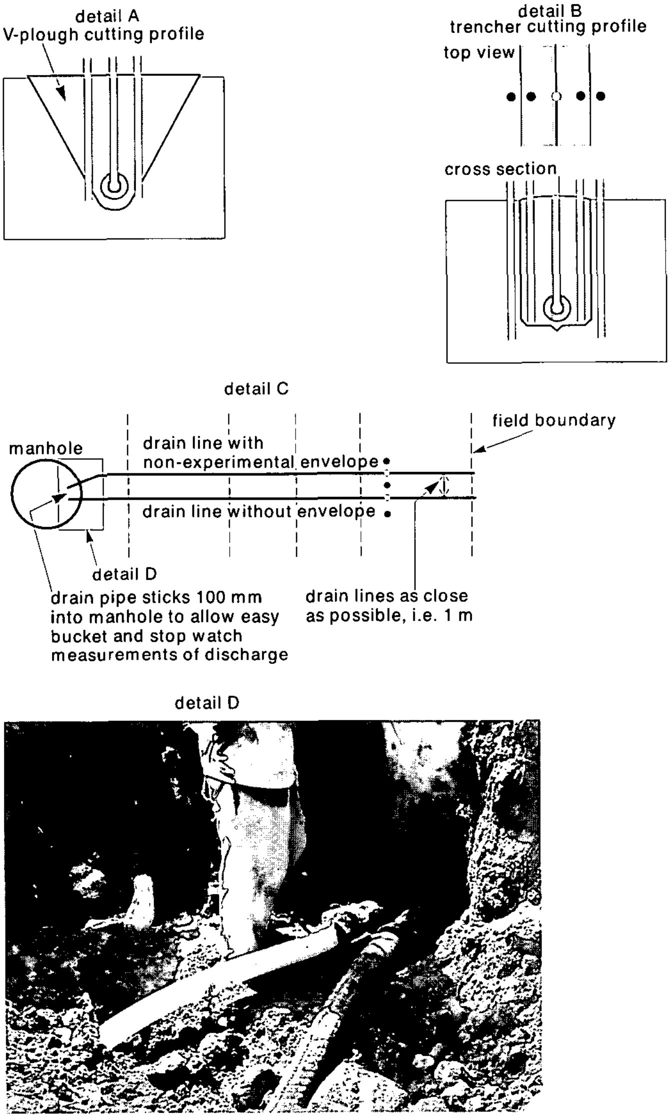


Figure 35 Cont. Recommended field layout for drain envelope research.

equally spaced observations if the system is designed according to the foregoing points and functions as expected. In other words, observed water heights along a drain with a similar envelope are expected to be more or less the same at ⅓, ⅔, ¼, ½, ¾ lengths (according to the above-mentioned ideal hydraulic grade line in Figure 36). If this can be assured from the beginning then instead of having two or three observations along the drain line one could consider all six replications to be along the same drain. This is only advisable in relatively flat areas with widely spaced water table contours (i.e. also small hydraulic grade lines in the direction of ground water flow). The significance of spacing the observation points equally along the line (for both sloping and flat lines) is that the heads along the line can be averaged and correspond to the total drain discharge measured in the manhole at the end of the line. However this is only true if the irrigation is uniformly applied at the same time along the drain. If irrigation is irregular then one should screen the data as outlined in the next section and/or install observation wells in each field section that is likely to receive separate irrigation.

7. Although not indicated in Figure 35, there is nothing against having observation of water levels in the pipe, and adjacent to the envelope and/or trench, at intervals spaced 3 - 4 metres apart along the drain line. All six replications can then be implemented along a 20 m section of the drain, with the least amount of variation in heads at a given time, and a discharge that can be considered representative for all six. If one installs extra manholes along the drain line to measure discharges, a single lateral of 100 m could have at least three sets of observations. This last idea is somewhat similar to the set-up at SIIA8C for the line with envelopes nos. 6 and 9 which are in series, separated by a manhole to measure discharge (Figure 34E).
8. Make sure to include one or more treatments with envelopes that do not comply with the design criteria of Chapter 3: at least one treatment which would fail the retention criteria, and one treatment that would fail the hydraulic and/or clogging criteria. Unless the research is specifically geared towards finding the exact point of failure, the envelopes that are expected to fail should not be close to the value that represents the boundary value, but well above or below that value, because the boundary values are generally set conservatively.
9. Include plans for replacement of failed lines in the experiment, and compensation funds for the farmers that are affected by the failure.
10. Make sure that water levels in the drainpipe can be measured to determine the true he values (Figure 32). In particular, when entrance heads in the observation well adjacent to the envelope are low (100 - 200 mm above the top of the pipe) errors become relatively large when the precise detail of water level in the pipe is not known.

4.7.4 Essential data to be reported and screening of the data

Although results of many field studies are reported (Stuyt and Oosten 1987, Bolduc *et al.* 1987, Gallichand *et al.* 1992, Bhutta 1995, Faure *et al.* 1995), it is difficult to compare the results of the various experiments because details are lacking. <u>No detail should be left out; better too much detail reported than something missed!</u> A similar appeal could be made for the measurements taken in the field; <u>better too much data than one critical data point or set</u>

missing. However, at no time can quantity compensate for quality! A well thought out experiment involves collection of all data needed. When in doubt whether a certain parameter will be needed later: measure it!

Reported results are clear indicators of what might or might not work. Many valuable insights are shared in published reports and it will be primarily the scientist in charge of similar studies who will be looking for the details and may find them missing. Even source reports often lacked the original data collected; the data often remaining in personal files. It is highly recommended to include all original data in appendices or on a CD-ROM. Data should be screened but not processed. Do not present data that has already undergone a calculation such as conversion to desirable units: report in the units measured! The only reason for recommending CDs is the fact that they are thin, fit easily in the back of a book or report and can contain 600+ Mb of data. CD-ROM readers are widely available.

Often certain conclusions can only be made when limiting qualifiers are made clear as well. A case in point is the table with entrance resistance in FAO 28 (Table 10a and b in FAO 28 and reproduced here in Box 14) which can only be used for the conditions for which it was derived. The method of presentation of the data is also very important: the qualifying statements for Table 10a and b were 'hidden' in the original text and often these tables have been presented in other publications without the qualifying text! Therefore, qualifying information should be an integral part of the table or figure (See Box 14)!

The following are some checklists for each of the four major assessments required for a complete field investigation of drain envelope performance.

1. Pre-drainage soil and water investigation checklist
   - Location and conditions of soil data collection points (see Section 2.4). All details on laboratory procedures and equipment (see Section 5.7.4).
   - Good maps giving full details of the irrigation and drainage system as well as field boundaries and/or irrigation units. Knowing the boundaries of water user associations will be helpful in obtaining their support. Collect information adequate for final presentation of results in modern GIS systems, or other computer applications helpful in presenting results such as contour maps.
   - All details of the design procedures used, including assumptions made.
   - Long-term water table records of the area.
   - Typical irrigation schedules, with timings, quantities, and a water balance assessment of the area.
2. Construction quality control monitoring checklist
   - Details of the envelope material to be used from the quarry/factory until transport to the site.

- Conditions during construction: was the envelope/drain constructed below water table or above water table (smearing, puddling, proper backfill, blinding achieved or not).
- Check integrity of laid pipes by pulling a plug or cage through the drain.
- No damage to fabric, proper connectors, adequate sewing quality.
- Measurement of pipe elevations if trenching techniques are used.

3. <u>Post-construction quality control investigation checklist</u>
   - Preparation of as-constructed maps and profiles (elevations).
   - Checking of elevations by probing for the drain with rods from the surface (not the same rodding that goes in the drain from the manhole or outlet), or by using grade control equipment.
   - Video inspection when available, and check manholes for water and sediment levels.
4. <u>Performance assessment to assess entrance resistance and sediment occurrence checklist</u>
   - Measure water depths in observation wells and discharge from the drains (Figure 37) at the same time (within one hour of each other).
   - If the layout is according to traditional patterns in irrigated areas, perform the following screening procedure before comparing results of various treatments.
     - The values of h_e from the rows along the drain (¼-, ½-, ¾-, or ⅓- and ⅔-L) should be all positive. Reverse flow conditions should not be considered for entrance resistance analysis (Figure 36A, B).
     - The coefficient of variation amongst the values of h_e and h_t along the drain line should preferably be less than 0.25 (Figure 36C). If higher, be careful with calculating averages to be used with h_e/q_l and a_e determinations.
     - There should be measurable discharge from the drain line at the time of h_e and h_t observations. Moreover, if bucket and stopwatch measurements are made, all data sets with observation times less than 10 seconds should be discarded as measurement errors are relatively too large for comparison amongst different treatments. Field staff should be instructed to repeat flow measurements with a larger container when the time of observation is around 10 seconds or less. When the diameter of the drainpipe is large enough measuring devices such as shown in Figure 37 are recommended.
     - The hydraulic gradient of the water table observation rows should increase in the direction of the flow. If by accident the time of measurement selected is inappropriate, one of the intermediate row observations could be temporarily higher due to localised irrigation application.
     - Plot h_e/h_t against time and check for outlying values (often they are based on either h_e or h_t being inappropriately high or low compared to the other (dependent) variable. Take out these values.

    - Plot q in units that are familiar to you (i.e. mm/d) in time, or perform statistical analysis with min, max, avg., CV, etc. to screen out exceptionally high values (i.e. q_d = 2 mm/d, while several values are 19 - 22 mm/d and full flow capacity of the drain is 5-6 mm/d. For example, a high q is possible when pipes operate under pressure but this needs to be checked!).
    - The h_e/h_t ratios are in theory a function of the discharge as well. The statistical analysis of q performed under the previous point may serve to select discharge intervals for grouping. In that case, analysis of discharges close to the design can be grouped and results compared with, for instance the indicator values in Box 14 provided conditions are similar.
- If the layout is not according to traditional patterns, but adjusted for irrigated conditions, (i.e. observation wells along the lateral are not at regular intervals but adjusted to match field boundaries to assess the weight of each observation with respect to the discharge of the lateral), the screening as described in the previous point should still be carried out. Eissa *et al.* (1996) described a comparable situation in the Fayoum, except that the difficulty in interpreting results collected from drain lines with non-uniform surface recharge (irregular irrigation patterns) was encountered when establishing the Q-h relationships.
- Statistical analyses must be done on the data screened as above to assess significant differences between treatments.
- Inspect drain lines with video equipment to assess any damage and sediment deposits. If video equipment is not available, inspect from manholes and perform excavations where deemed necessary after other tests have been completed.
- If results contain unexplainable differences, additional soil survey analyses may have to be performed (including texture analysis, PI and K determination). In any case, it is advisable to perform additional soil surveys after hydraulic performance measurements have been completed, as local variability in the soil might help explain the differences. Then, excavated envelope material could also be tested in the laboratory. No excavations should be done if further observation of hydraulic performance is still intended at the same location.

A drain less than 150 mm flow is usually measured with bucket and stopwatch method. What is shown is a broad-crested weir in circular cross-section constructed from plastic (for more details on broad crested weirs see Bos *et al.* 1996).

If not enough room is available for a broad-crested weir a sharp-crested weir may be used. The weir is glued or welded in pipe section of similar diameter and material as the drainpipe, and attached with a coupler at the end of the pipe.

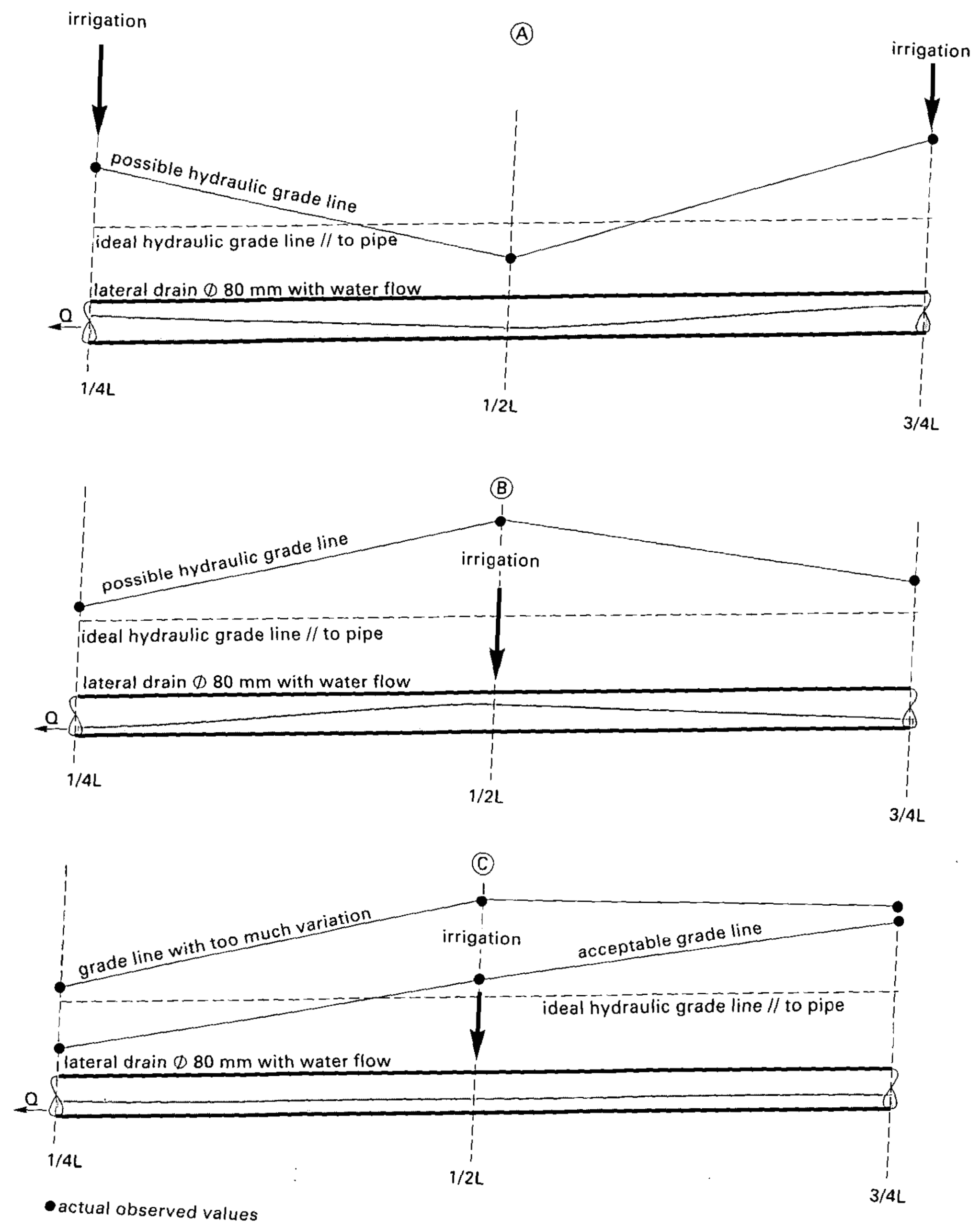


Figure 36 Example of water level observations along the drain line and resulting hydraulic grade-line.

A Irregular irrigation pattern causes temporary reverse flow.

B Observation at 3/4L should not be considered.

C Normal flows, except coefficient of variation amongst observations may be higher than recommend CV of less than 0.25 or between 0.25 and 0.50.

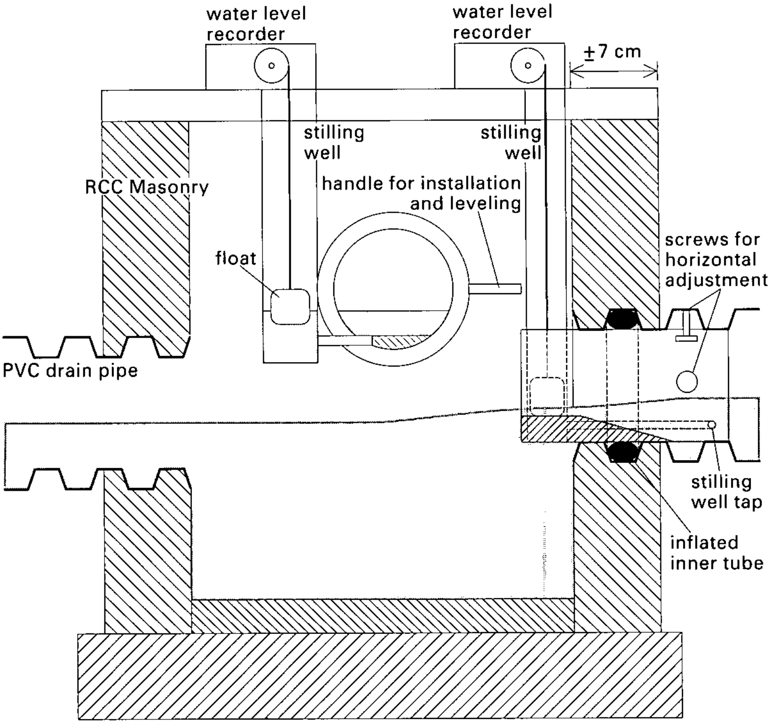


Figure 37 Flow measurement for drains of 150 mm and larger in manhole with broad-crested weir.

Part 2 Resource materials

5. Design of drain envelopes: theory and testing

This Chapter aims at achieving one of the goals outlined in the introduction, namely, to present the backgrounds of the various drain envelope design criteria developed by researchers worldwide, including those of on-going investigations. It strives to elucidate the conditions, recommendations and criteria for drain envelope design. To fully appreciate the complexities of drain envelope design, the theory of water flow towards the drain and the latest information on particle movement will be described first. Then, descriptions will follow of standard tests that need to be performed to provide the necessary indicators for comparison between materials and performance of drain envelopes in the field.

As mentioned earlier, the standard reference to particle size of base or soil material and envelope material of d_{xx} and D_{xx} is used, where lower case d refers to the soil material, either granular, organic or synthetic envelope material, and capital D denotes the particle sizes of granular envelope material. The opening size of synthetic and organic envelopes is denoted by O_{xx}. The number xx following each letter is the percentage - by weight - of the sample that is finer than the size indicated (cumulative percentage passing) as determined by a sieving test (Section 5.5.1 and 5.6.8). The size will be given in microns (µm, = 10^{-6} m) or in mm, with xx denoting the percentage of the particles or pores with a diameter smaller than that size.

Common terms used in describing properties of granular envelopes and base soil, such as Coefficient of Uniformity (C_u), Coefficient of Curvature (C_c), and Plasticity Index (PI) are described in the Laboratory Tests section (Section 5.5). Terms for organic and synthetic materials such as Characteristic opening, FOS, EOS, and AOS are described in Section 5.6.9. Chemical properties affecting the functioning of drain envelopes are dealt with in Section 5.5.5.

The design criteria for drain envelopes are governed by two conflicting objectives: 1) prevention of excessive amount of particles passing through the envelope; and 2) unimpeded flow of water. Classical criteria established for granular filters based on laboratory findings and theoretical considerations were justified after many years of successful field experiences. It would seem that the development of criteria for synthetic filters is progressing in a similar way, except that the many years of experience have not yet been reached, while the spate of criteria presented (Chapter 6, and Section 5.9), has prevented a singular approach to design and assessment of the function of the envelope (filter). This is not surprising considering the very many factors that affect the functioning of filters: two-directional flow; chemical and electrical interactions between particles and fabrics; geometrical conditions (shape of particles, distribution of particle sizes, location of fluid, direction of flow); mechanical conditions (stress and gravity); material properties (composition, density, viscosity, opening sizes and distribution, polymers, manufacturing processes and compressibility); and conditions during storage, transport and construction. In this chapter the properties that most affect the function of the filter, i.e. the retention, hydraulic and mechanical criteria, will be described.

5.1 Flow towards and into the drain

To understand flow towards and into drains, an overview of some of the most common layouts of subsurface drainage systems and their operational conditions are described. Then the entrance resistance is described, followed by a detailed analysis of particle movement and expected exit gradients near the drain. All this background information is needed to be able to judge the conditions encountered in the field and to develop a sound basis for design of a drain envelope.

The following hydraulic conditions are most commonly encountered at the base soil - envelope - pipe interfaces:

- Laterals draining naturally into an open (surface) drainage system, with free outflow conditions (Figure 38A). Fairly steady flow conditions exist at the interfaces. Gradients build up slowly and reduce again in a matter of days.
- Laterals draining naturally into an open (surface) drainage system, which frequently experience backwater effects causing the drainpipe outlet to be submerged (Figure 38B). More or less stationary flow with a chance that reverse flow could take place in the pipes. Changes are gradual and no excessive hydraulic gradients occur in the soil (i.e. $i < 2$).
- Pumped subsurface drainage systems (Figure 38C). Unless the drainage surplus is extreme (or the pumping facility is under-designed), these systems will always operate under cyclic flow conditions. Often the collector drains discharge into a sump from which water is pumped. As water from the collector(s) does not fill the sump as quickly as it discharges the water, the pumps have an on-off cycle, which is controlled either manually or automatically (with float-switches). The automatic system may not always work due to power failure or due to maintenance. Water levels might therefore rise substantially in the sump filling the whole pipe system and causing water to stand above the pipes in the soil. When the pump is switched on after such a period, the pipes will be emptied rather rapidly (within the hour) and the situation shown in Figure 38C will develop. High flow gradients that will stress the filtering function of the envelope material

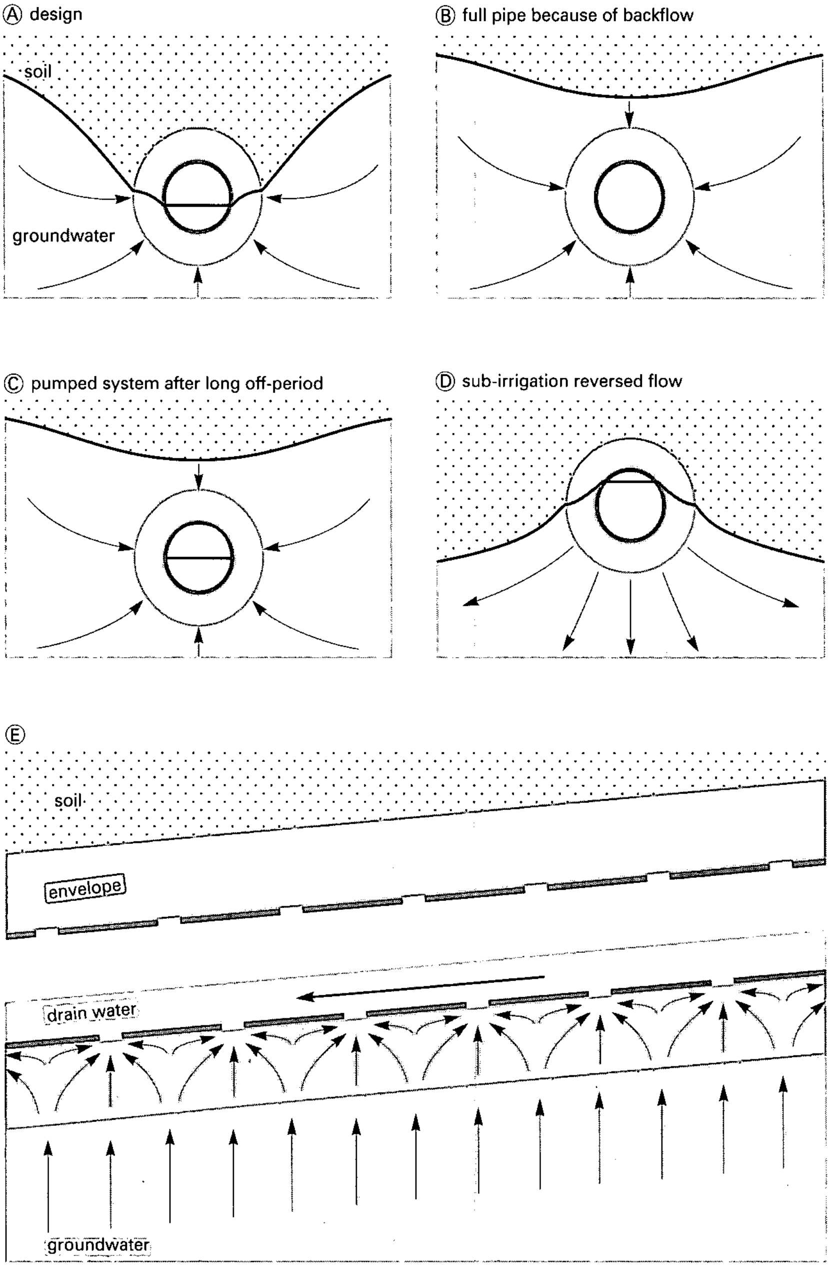


Figure 38 Possible flow conditions near the subsurface pipe drain.
- A Preferred design situation.
- B Full pipe caused by downstream constriction or backflow.
- C Pumped system just after pump was started after long off period.
- D Sub-irrigation; reversal of flow though envelope material.
- E Flow pattern and convergence in the envelope material along the perforated drain.

could occur. The water level in the sump should be kept below the drains discharging into the sump.

- Drainage systems that are also used for sub-irrigation(Figure 38D). Sub-irrigation is generally not effective in heavier soils, hence, reverse flow gradients are likely to occur in lighter soils, when there is also need for a filtering envelope. If gradients are rather moderate, flow through the envelope will likely take place safely in both directions. With high gradients, any natural filter, or stabilised soil structural condition that would have developed during the draining cycle is likely to be destroyed by a reversed flow during the irrigation cycle. Flow (i.e. gradients) into and out of the pipes should also be kept low to avoid clogging the envelope material.

Apart from the above four typical drainage system designs and modes of operation there are two additional situations which will also play a major role in the success or failure of drain envelopes:

- The period immediately following construction. With trenching techniques, the backfilled soil will generally be less dense than the undisturbed soil outside the trench and may thus have a considerable number of preferential flow paths with potential high flows. High flows with corresponding high flow forces may cause particles to move which would not have moved otherwise with water percolating through the soil.
- Controlled drainage. Here, the drainage system is temporary blocked to prevent outflow. Depending on local conditions, high gradients could develop if the pipes are suddenly emptied while the water table in the soil is still above the drain. This situation is very much like the one described in pumped drainage systems.

Drainage situations not explicitly considered are road, railway and construction site applications and vertical strip drainage.

5.1.1 Entrance resistance

Water flowing into a drain radially converges in the surrounding soil with a secondary convergence at the drain openings. The typical flow patterns around a drain with widely spaced openings as occurs with clay and concrete drains is shown in Figure 39. The flow pattern for evenly distributed openings, such as in corrugated plastic drains with geotextile envelopes, is shown in Figure 40. For calculation purposes, even a drain with a finite number of openings is often considered to be a hydraulically-ideal drainpipe that allows water to enter uniformly over its full surface. A hydraulically-ideal drain is essentially a completely permeable drain without any appreciable entrance head loss or secondary convergence. In gravel envelopes, any secondary convergence takes place in the high permeability gravel and the pipe-envelope

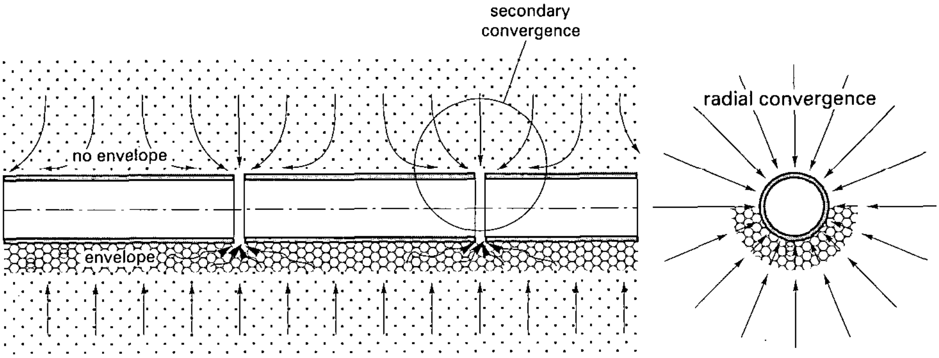


Figure 39 Flow pattern towards clay and concrete tiles.

system approximates a hydraulically-ideal drain. With synthetic envelopes some secondary convergence also takes place in the envelope, but it depends on the thickness of the material used, whether or not this results in a hydraulically ideal pipe-envelope system.

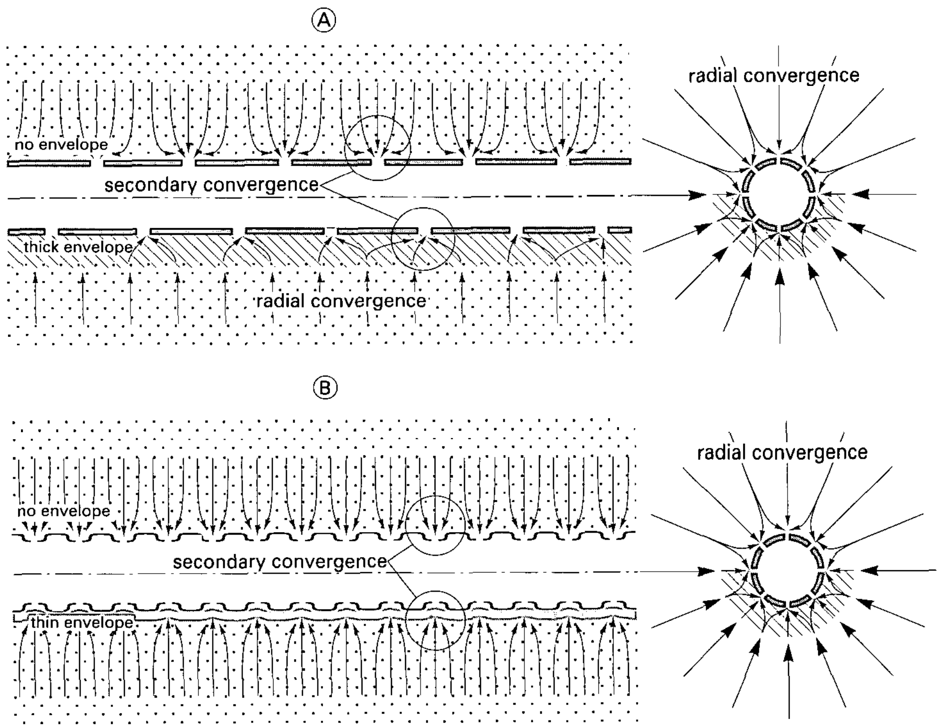


Figure 40 Flow pattern towards perforated plastic pipe drains.
A perforated smooth plastic pipe.
B corrugated plastic pipe.

The soil condition around a hydraulically ideal drain is assumed uniform. In reality, there is normally a heterogeneous, non-uniform situation caused by: (1) natural variability of the soil conditions adjacent to the pipe; (2) discontinuous soil conditions caused by trench excavation and backfilling; (3) application of an envelope; and (4) converging stream lines towards the drain perforations or joints (secondary convergence). Entrance head loss (h_e) is the sum of (secondary) convergence head loss (h_c) and the combined radial head loss in the soil, trench and envelope (h_r) as given in the equation below.

$$h_e = h_c + h_r \qquad \text{Eq. 11}$$

In the field, it is impossible to measure the convergence or radial head loss separately, and entrance losses reported refer to the typical total entrance loss measured as shown in Figure 30. Therefore, entrance resistance reported from field experiments is the total entrance head loss h_e. The term entrance resistance is used to express the head loss independent of the discharge in the equation:

$$w_e = \frac{h_e}{q} \qquad \text{Eq. 12}$$

where,

- h_e is the head loss that is determined as the difference between the water level in the observation well closest to the drain (at envelope-soil interface, or just outside the trench boundary) and in the drainpipe in m (Figure 30);
- q the actual drainage coefficient in m/d, which can be either the design drainage coefficient, or actual discharge (Q) divided by drain length (L) and spacing (S); and
- w_e the total entrance resistance ($w_c + w_r$) in d.

A dimensionless value of entrance resistance, the contraction constant (sometimes referred to as resistance factor) for total entrance resistance, may be obtained from:

$$a_e = \frac{w_e K}{S} = \frac{h_e K}{qS} \qquad \text{Eq. 13}$$

where,

- a_e represents the total entrance resistance contraction constant;
- S the drain spacing in m; and
- K the hydraulic conductivity in m/d.

Another entrance resistance has often been reported (Dieleman and Trafford 1976):

$$r_e = \frac{h_e}{q_1} \qquad \text{Eq. 14}$$

where,

r_e represents the entrance resistance in[23] d/m; and

q_1 the drainage discharge per unit drain length in $m^3s^{-1}m^{-1}$ (m^2/s).

This is not good practice, because the drain spacing and hence implicitly the hydraulic conductivity (K) is still included in r_e, and therefore comparison between drainage systems and different design features is made difficult. Hence, instead of using r_e values and interpretation of drain line performance as suggested in FAO 28 (Dieleman and Trafford 1976), the criteria described in Section 4.6 and 4.7 should be verified (Box 14).

The hydraulic conductivity of the soil (K-value) in the vicinity of the drain often deviates (considerably) from that of the adjacent undisturbed soil. If the K-value near the drain is higher than the K-value of the undisturbed soil, the corresponding reduction in flow resistance might compensate the convergence resistance and justify the assumption of a hydraulically ideal drain. If the K-value of the material in the trench is ten times or more than the K-value of the undisturbed soil, the extra head loss due to entrance resistance can be ignored in drain spacing calculations (Smedema and Rycroft 1983, Cavelaars *et al.* 1994, Ritzema 1994). K-values in the excavated trench are initially higher than that of the surrounding undisturbed soil, but will decrease over time. This assumes that construction takes place under normal to ideal conditions and that excavated material is not whipped into impermeable slurry. Bentley and Skaggs (1993) observed this effect in tank and permeameter experiments in the laboratory: K-values dropped during the first 20 - 60 days and then stabilised (experiments were run for 70 - 160 days).

Water enters a drain through the gaps between the ends of clay and concrete pipes or through the perforations of plastic pipes. The inlet area normally comprises only 1 to 2% of the wall area. The convergence of streamlines towards those openings is inherently related to the type, spacing, area of

[23] This value is different from h_t/q, which is part of the methodology to determine K from field measurements (Oosterbaan and Nijland, 1994). It should also not be confused with its reciprocal q/h_t, which is known as the steady state criterion; a measure of the responsiveness of a drainage system (Ritzema, 1994).

drainpipe openings, and the hydraulic conductivity of the material in which the convergence takes place. The secondary head loss (h_c) is given by:

$$h_c = q\, w_c = \frac{q \cdot S}{K}\, a_c \qquad \text{Eq. 15}$$

where,

h_c is the head loss as a result of the convergence (contraction) resistance in m;

w_c the contraction entrance resistance in d; and

a_c a dimensionless contraction coefficient (Table 8).

The contraction constant of the entrance resistance of clay and concrete pipes is higher than that of smooth plastic pipes, which, in turn is higher than that of corrugated plastic pipes (Table 8). Laboratory research has revealed (Cavelaars, 1967, Dierickx, 1980) that not only are area and distribution of the inlet openings important, but the shape, dimensions and width of the openings also influence the entrance resistance.

Table 8 Dimensionless contraction constant, a_c, and resistance factor, a_e, for different drain tubes.

Type of pipe	a_c Dierickx (1980, 1982)	a_e Smedema and Rycroft (1983)
Clay and concrete	1.0 - 3.0	0.4 - 2.0
Smooth plastic	0.6 - 1.0	0.4 - 0.6
Corrugated plastic	0.3 - 0.6	0.05 - 0.1

Remarks: a_c is based on modelling work by Dierickx, while a_e is cross-referenced to Wesseling 1978, but details are not given. Both values are for pipes without envelope. Bentley and Skaggs (1993) refer to contraction constant of Dierickx as the entrance-resistance constant.

Bentley and Skaggs (1993) compared theoretical values of Dierickx with those determined from laboratory measurement in soil tank and permeameter tests. They found that for corrugated pipes a_c levelled off at 0.25, while for pipes with synthetic envelopes the experimental values were a magnitude higher than those calculated from theory according to Dierickx (1980). For instance, measured head losses are higher than those computed.

The different contraction constants in Table 8 are valid for drains without an envelope. Good envelopes substantially reduce the entrance resistance of drainpipes and render differences in entrance resistance between different types of drainpipes negligible. Flow through drain perforations is generally considered to enter the drainpipe uniformly over the drain surface, while it actually enters through the perforations in the bottom part of the drain circumference that are under water (wetted perimeter, u). Different situations may occur in the field with respect to the position of the water table and the drain and whether the pipes flow full (with or without back-pressure) or partially full (Figure 38 and Figure 30). Contraction constants are subject to these flow conditions and the same constant may not be applicable for different flow conditions.

The radial entrance head loss for a full flowing ideal drain in homogeneous soil may be determined from (Cavelaars *et al.* 1994):

$$h_r = q\,w_r = q\,\frac{S}{2\,\pi\,K_r}\ln\frac{R}{r_0} \qquad \text{Eq. 16}$$

where,

- h_r is the head loss due to radial flow towards the drain in m;
- w_r the radial entrance resistance in d;
- K_r the radial hydraulic conductivity in m/d, in practice one uses the K value measured with the auger hole method or determined otherwise for the surrounding soil;
- R the radius of influence of radial flow in m, the distance of the nearest observation well from the centre line of the drain (but not further away than 0.2 - 0.3 m, see Figure 30); and
- r_0 the (ideal) drain radius in m.

When the drain is half full w_r will be twice as large because 2π reduces to π in Eq. 16. For partially full drains the factor 2π can be reduced according to the actual wetted perimeter, u, such that w_r becomes:

$$w_r = \frac{S\,r_0}{K_r\,u}\ln\frac{R}{r_0} \qquad \text{Eq. 17}$$

where,

- u represents the actual wetted perimeter inside the drainpipe in m.

Ziegler (1978) found that a substantial head loss occurred in the last 25 mm adjacent to the drainpipe because of the large converging flow velocities near the drain openings.

Dierickx (1980) found from laboratory experiments that the entrance head loss of drainpipes (h_e) decreased up to an envelope thickness of 5 mm. Envelopes thicker than 5 mm did not reduce the entrance head loss significantly but did help reduce the exit gradient of water from the soil since a thicker envelope has a larger radius and circumference and consequently a lower exit gradient at the soil-envelope interface. The 5 mm is a theoretical value and in the field, it could be quite different because of a range of factors (e.g. material properties such as listed in Figure 14 and soil pressure).

5.1.2 Sediment transport in granular non-cohesive soils and filters

Water passing through saturated soil exerts a frictional drag force on the soil particles. As the water velocity and hydraulic gradient of the flow increases, the additional force exerted by the water on the soil particles increases until it reaches a limit at which the soil particles can no longer resist the drag force of the water. The soil particles then begin to lose contact with each other in the case of non-cohesive soil and where the direction of flow is against gravity. Soil particle movement will also take place in cohesive soils if the frictional drag force starts to destroy the cohesive bonds between particles.

Research conducted for granular filter construction requirements of the Eastern Scheldt Surge Protection Dam in the Netherlands (Graauw *et al.* 1983, Adel 1992a) showed that the transport of particles through various layers of a filter can be individual or collective. The mode of transport (collective or individual) was found to depend on the flow direction of the interface and could be perpendicular, either with or against gravity, or parallel. Perpendicular flow is generally not turbulent and particles move collectively. Parallel flow is more turbulent and individual particle movement takes place. With angular flow approaching a soil-envelope interface it was observed that when the perpendicular component is dominant, the flow type is collective. Flow perpendicular to the interface (and upward against gravity) revealed that erosion of single particles (i.e. movement of one particle) almost never occurred, but that erosion was characteristically a mixture of water and particles: i.e. collective particle movement (Adel 1992a).

What actually happens at the interface of the filter and the base material is best illustrated by considering a fine and a coarse filter overlying the same base material. In Figure 41a and b, the case of parallel flow to the interface is depicted. Friction on top of the particles of the base material is proportional to the flow velocity of the water in the filter. If the drag force exceeds the friction force it will cause the individual particles to move. Since the flow conditions around the particle remain the same when it moves along the filter-base material interface, the particle keeps on moving, and they can and do move individually, and not collectively. Because of the larger pore spaces in the filter, the flow velocity in the filter will be greater than that in the base material at the same gradient. Similarly, the flow velocity in the fine filter will be less than that in the coarse filter at the same gradient. Since the critical flow velocity of the base material will be the same in both cases, the critical gradient at which particle movement starts taking place will be higher for the finer filter than for the coarse filter. This critical gradient will be referred to as the Hydraulic Failure Gradient, which is not only a function of the characteristic particle diameter and porosity, but is also a function of cohesion, and filter or drain perforation opening size, as will be explained later.

For flow perpendicular to the interface between the filter and the base material, the process of the start of particle movement is slightly different (Figure 41c and d). In all cases it is assumed that the water will flow first through the base material before it enters the filter. When water moves from the base to the filter it will move to an area with larger pore spaces. Therefore, at the same discharge, flow velocities in the filter will be lower than in the base material. Contrary to parallel flow, failure of perpendicular flow is caused by conditions in the base material, which assumes that the water will flow upward against gravity from the base material into the filter. The mechanics involved with flow in the direction of gravity are similar except that the forces are different in magnitude, while bridging will also play a role (Section 5.2).

With perpendicular flow it may be noted that if the porosity of the two materials is the same, the Darcy velocity will also be the same ($V = Q/A$), but the gradient in the finer material will be greater.

Particle transport in the perpendicular flow case will occur if quicksand conditions occur in the base material at the interface, where the gradient in the base material is the deciding factor governing soil movement. Quick-sand movement conditions are collective. The reason for collective transport of particles rather than individual movement of particles is the loss of drag-force of the single particle the moment it arrives in a larger pore. When more particles come into the larger pore in the filter drag forces increase again and soil particle movement continues. In real time this becomes a massive movement of particles. Nevertheless, it has been observed that hydraulic failure gradients in such cases are also considerably higher than one, explained by the fact that the filter material will mechanically restrict movement of the particles into the filter. Obstruction in a finer filter will be higher than in a coarse filter. It would therefore appear that the critical gradient at which soil particles begin to move is higher in a fine filter (Figure 41c) than in a coarse filter (Figure 41d).

Even though the process of particle movement with perpendicular and parallel flow is completely different, in qualitative terms, the influence of particle size ratios of certain filter-base material on the critical gradients at which particles start to move is the same. With a small ratio of D_{xx}/d_{xx} the hydraulic failure gradient will be higher than with larger ratios.

In a subsurface drainage system, as water approaches the drain, the hydraulic gradient increases because of flow convergence. The gradient reaches its highest value at the soil-drain interface of the drain openings. Envelope materials placed around a drain will enlarge the effective circumference of the drain and move the soil interface away from the pipe perforations, thereby reducing the gradient of the water exiting the soil. The gradient at the soil-envelope interface then becomes the critical design factor.

Sherard *et al.* (1984a and 1984b) tested filter construction for application with downstream protection of dams and found that a protective layer of granular material assumes the function of a filter and it will do so over a very short distance into the surface of the filter, close to the soil. In laboratory tests they

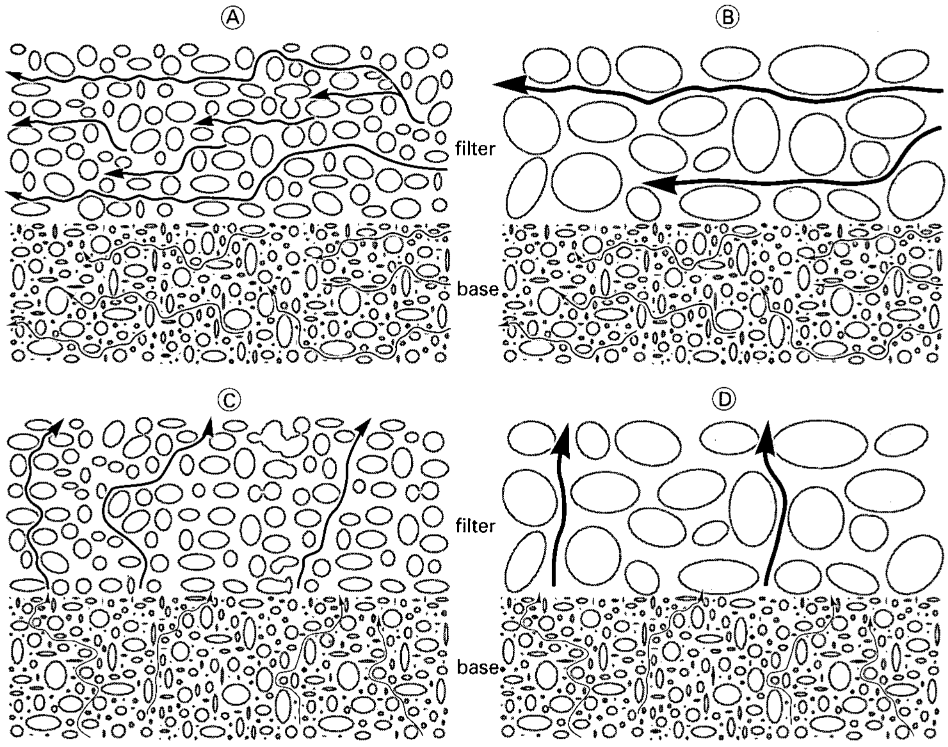


Figure 41 Particle movement with flow perpendicular and parallel to the filter-soil interface.
A fine filter with parallel flow.
B coarse filter with parallel flow.
C fine filter with upwards perpendicular flow.
D coarse filter with upward perpendicular flow.

found that the filtering action took place over the first 2 mm and the rest of the filter did not contribute to the filtering action. It is however known that the thicker the envelope the coarser it can be. If the sheet envelopes are thin, soil particles have to bridge the openings otherwise they will pass through. In the case of thick envelopes bridging of soil particles will not necessarily occur at the surface because they can also bridge within the envelope itself. Geotextile fabrics can be made thin because openings can be made very fine. Construction procedures for granular envelopes, on the other hand, require a thickness of 75 mm or more.

5.1.3 Darcy and Reynolds

Reference to the Darcy equation occurs throughout the book: with subsurface flow into the drain and its relationship with the exit gradient; with characterising flow through gravel and geotextile envelopes; and with calculations in the laboratory when envelope-soil combinations are tested in permeameters. A generalised form of flow through a soil matrix (Muskat 1946), which includes Darcy's version for laminar flow is:

$$\frac{Q}{A} = V = K\left(\frac{H}{L}\right)^{m} = Ki^{m} \qquad \text{Eq. 18}$$

where,

- Q is the rate of flow, or discharge in m^3/s (or m^3/d);
- A the cross-sectional area of flow in m^2;
- v the average flow velocity in the direction of decreasing H/L in m/s (or m/d);
- K saturated hydraulic conductivity in m/s (or m/d);
- H head loss in m;
- L length of the linearised flow path (actual flow path is tortuous) in m;
- m exponent, m = 1 for Darcy's Law and laminar flow, when $m \neq 1$ flow is non-laminar and Darcy's Law doesn't hold (see also section 5.6.10, p 210); and
- i the hydraulic gradient.

The average flow velocity is not the actual flow velocity in the soil pores but rather an apparent velocity, which is also described as discharge per unit area, or as specific discharge. A velocity closer to the actual velocity will be found if the porosity of the media is taken into account:

$$v_{\varepsilon} = \frac{Q}{\varepsilon A} = \frac{K}{\varepsilon}\left(\frac{H}{L}\right)^{m} \qquad \text{Eq. 19}$$

where,

v_ε average linear velocity of pore flow in m/s (or m/d);

ε porosity of the medium under consideration, dimensionless.

Still, the linear flow velocity is not the actual flow velocity as may be clear from Figure 42. Flow through the tortuous path will take longer, and to arrive at the same point the velocity needs to be higher.

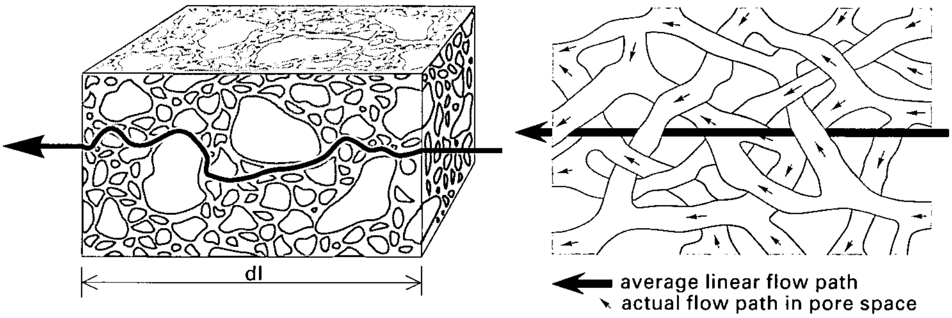


Figure 42 Porosity and average linear velocity.

Darcy's law is not valid for all porous media. Flow should be laminar. As a practical matter, such a condition will prevail in silts and finer materials for any commonly occurring hydraulic gradients found in nature. To apply Darcy's law[24] to sands, and especially coarser sands, it will be necessary to restrict the hydraulic gradient to values less than about 0.5 to 1. The range of validity of Darcy's law can be demonstrated by measuring the water flux in a soil sample for a series of hydraulic gradients. The result should be a linear relation between the flux and the hydraulic gradient.

Darcy's law is extended for use in unsaturated soils by assuming that the hydraulic conductivity is a function of the degree of saturation, i.e. the volumetric water content.

To determine the applicability of the Darcy equation, the Reynolds number (R_e), a dimensionless number that expresses the ratio of inertial to viscous forces during flow, is widely used in fluid mechanics to distinguish between laminar and turbulent flow. Reynolds number can be determined from:

$$R_e = p_e \, v \, \rho_w / \eta_T \qquad \text{Eq. 20}$$

[24] If we accept this then one might conclude that under general drainage conditions Darcy's law rarely applies, as gradients close to the drain will tend to be higher than one.

where,

R_e represents Reynolds number (–);
p_e the effective pore diameter (m);
ρ_w mass density of water (kg/m^3) adjusted for temperature (Table 9);
v flow velocity (m/s) which can be either the average flow velocity of Eq 18, or the average linear velocity of Eq 19; and
η_T dynamic viscosity of water (kg/m s) adjusted for temperature.

Some values of dynamic viscosity and water density are given in Table 9, based on which the dynamic viscosity can also be obtained from:

$$\eta_T = 1.787 * 10^{-3} - 4.546 * 10^{-5}\,T + 4.409 * 10^{-7}\,T^2 \qquad \text{Eq. 21}$$

where,

T is the temperature in °C

Table 9 Physical Properties of water as a function of temperature. (Hillel 1971).

Temperature T in °C	Density ρ_w in kg/m^3	Dynamic viscosity η_T in kg/m s
0	999.87	1.787×10^{-3}
4	1000.0	1.567×10^{-3}
5	999.99	1.519×10^{-3}
10	999.73	1.307×10^{-3}
15	999.13	1.139×10^{-3}
20	998.23	1.002×10^{-3}
25	997.08	0.890×10^{-3}
30	995.68	0.798×10^{-3}
35	994.06	0.719×10^{-3}
40	992.25	0.653×10^{-3}
45	990.24	0.596×10^{-3}
50	988.07	0.547×10^{-3}

Hydraulic conductivity is usually determined at a standard temperature of 20°C and for the purpose of comparison, hydraulic conductivity under actual conditions should be converted to K at 20°C as follows:

$$K_{20} = K_T * (\eta_T / \eta_{20}) \qquad \text{Eq. 22}$$

Turbulent flow deviates from Darcy's law when R_e is in the order of 1000 - 2000 (Scheidegger 1957, and Childs 1969) in straight tubes (comparable to the average linear flow velocity of Eq 19). When tubes are curved the critical value of R_e is greatly reduced and for porous media it is safe to assume that flow remains linear with hydraulic gradient as long as R_e is smaller than unity ($R_e < 1$, Hillel 1971). Bos (1994a) also suggests that with drainage applications R_e should be less than one to apply Darcy. In porous media, Hazen (in Means and Parcher 1963) found that laminar flow would occur in uniform soils with particles with a diameter of less than 0.5 mm.

To determine the effective pore diameter, also called characteristic pore diameter, or equivalent pore size diameter (Fisher *et al.* 1990), one must assume that the particles are uniform spheres. For the closest packing where one sphere of one layer fits into the space between four spheres in the layer below (hexagonal arrangement) the ratio of the volume of a unit sphere to a unit void is $V_d/V_v = 0.71/0.18$, with a porosity (ε) of 25.95% (Escher 1905, Vlotman *et al.* 1993c). Giroud (1982) found that the ratio between the diameter of the uniform spheres and the diameter of the largest sphere likely to go through this hexagonal arrangement (bridging) is $\sqrt{3}/(2 - \sqrt{3}) = 6.5$[25]. In a loose state (cubic arrangement) this becomes $1/(\sqrt{2} - 1) = 2.4$. An approximate average value is 4. In case of the retention criteria of $D_{15}/d_{85} < 4$ (Terzaghi's original criterion) it can be interpreted that large particles of the soil (d_{85}) must be larger than the openings of the filter $(D_{15}/4)$[26].

Assuming the unit sphere to be d_{50} it follows that with a porosity of 25.95% the unit void or effective pore is $(0.18/0.71)d_{50} = d_{50}/4$. Hence:

$$p_e = 0.25\, d_{50} \qquad \text{Eq. 23}$$

$$p_e = \varepsilon\, d_{50} \qquad \text{Eq. 24}$$

$$\varepsilon = 1 - \rho_b / \rho_s \qquad \text{Eq. 25}$$

where,

p_e is the effective or equivalent pore diameter;

ε the porosity, the ratio volume of voids over total volume. Typical values for granular material are 0.2 - 0.4;

ρ_b bulk density of soil or gravel material (kg/m^3); and

ρ_s mass density of particles (usually 2600 - 2700 kg/m^3).

Fisher et al. (1990) mention work done by Atterberg in 1908 who proposed:

$$O_{50} / d_{10} = 1/5 \qquad \text{Eq. 26}$$

where,

O_{50} the pore size diameter for which 50% of the pores are smaller.

[25] In the original paper this was $3/(2 - \sqrt{3}) = 6.5$. Giroud corrected this in a reprint of the paper in 1984 for the International Fabrics association International, St. Paul, Minnesota, USA.

[26] Fisher *et al.*, 1990 to convert granular envelope (filter) criteria for use with synthetic envelopes (Section 6.2.3) used a similar approach.

The ratio of 1/5 is reasonable in view of the range of 1/2.4 - 1/6.5 reported above. For non-uniform soils the ratio will change with the particle size distribution. For instance, the ratio will be less when using d_{50} sizes rather than d_{15} sizes (see also Section 5.2), because the finer particles reduce the number of larger pore sizes (and hence the characteristic pore size). On combining these various ratios with a straight line extension of the Coefficient of Uniformity, C'_u (Eq. 43), the O_{50}/d_{50} and the O_{50}/d_{15} can be approximated by (Fisher *et al.* 1990):

$$O_{50}/d_{50} = \frac{10^{0.1 * \log (C'_u)^2}}{5C'_u} \qquad \text{Eq. 27}$$

$$O_{50}/d_{15} = \frac{1}{5\ (10^{(0.05 * \log (C'_u)^2})} \qquad \text{Eq. 28}$$

In view of the foregoing reasoning and since the characteristic pore size is closely related to d_{10} or d_{15} (Section 5.4) we propose that the effective pore size can be calculated from:

$$p_e = \varepsilon\ d_{15 \text{ or } 10} \qquad \text{Eq. 29}$$

Note, this is different from the effective (or characteristic) particle diameter De used to calculate saturated hydraulic conductivity in Section 5.4. Here, merely a relation between particle diameter and pore diameter is proposed, which may be helpful for assessing bridging criteria (Section 5.2).

Van der Sluys and Dierickx (1987) investigated the applicability of Darcy's Law to geotextiles. They considered various laminar flow models commonly used with porous media (capillary model, hydraulic radius model, and drag forces model) to test their applicability for use with flow through geotextiles (both woven and non-woven). None of the more theoretically based models showed adequate resemblance to the experimental data. They found that expressing water conductivity characteristics as discharge rate at a certain hydraulic loss was more appropriate and that the type of flow occurring should always be checked (see Section 5.6.10). The basic reason for this was that geotextiles are rather different from the classical description of porous media. Moreover, the porosity of non-woven geotextiles was significantly higher than that of porous media (ε = 0.7 - 0.95 in unconfined conditions, and ε = 0.6 - 0.8 in confined conditions, whereas for porous media we mentioned that ε = 0.2 - 0.4).

Darcy's Law and the test for laminar flow - using Reynolds number - are also used in the interpretation of laboratory tests (Sections 5.5.4 and 5.7).

5.1.4 Expected gradients

Important in assessing the situation that may occur at drain level is an estimate of the hydraulic gradient that might be expected to occur at the various interfaces: drainpipe-envelope, envelope-soil, and drainpipe-soil. The gradient is also referred to as the exit gradient, and is different from the critical and hydraulic failure gradients described in Section 5.3. To distinguish between the various options the following gradients and symbols will be used:

i_c Critical gradient (Section 5.3, Eq. 31).
HFG Hydraulic Failure Gradient (Section 5.3).
i_x Exit gradient as calculated from the Darcy equation (Eq. 5 or Eq. 30) at the perforations of the drainpipe (without drain envelope!), or at the envelope openings of the envelope-soil interface. This value is theoretical and does not consider effects of cohesion or surcharge.

The exit gradient can have many different values not only depending on the hydraulic head, which is relatively easy to estimate from design features or values measured in the field, but more importantly, as function of the area of flow! The areas of flow are as follows (Figure 8):

A_p Area of the perforations or gaps per metre pipe length in m^2 for the pipe flowing full.
A_{pu} Actual area of flow per unit length when the pipe is not flowing full (the water table intersects the drainpipe at some point, but the pipe is not submerged). If no envelope is used, the ratio (R_u) of the wetted perimeter in the soil to the total perimeter of the pipe will give the area of flow at the drainpipe-soil interface when the pipe is not flowing full ($A_{pu} = (u/2\pi r_o) * A_p = R_u * A_p$). This assumes that the perforations are evenly distributed around the drain perimeter.
A_{pe} Area of actual flow into the drain envelope per unit length which is a function of wetted perimeter ratio (R_u), the radius of drainpipe plus envelope ($r_o + T_g$) or ($r_o + T_d$), and the exposed area of the corrugations as ratio of total unit area (a_e in Figure 8). This assumes no or negligible parallel flow in the envelope fabric; fabric directly in contact with the drainpipe does not contribute significantly to the flow into the pipe. This could be conservative for voluminous synthetic envelopes. Finally, A_{pe} depends on the porosity of the non-woven geotextile or the percent open area (POA) of the woven geotextile (Section 5.6.8). In case of granular envelopes the porosity, as determined from Eq. 25 or Eq. 33,

should be used as follows:

$$A_{pe} = 2\pi\,(r_o + T_g) * R_u * a_e * POA, \text{ or}$$
$$A_{pe} = 2\pi\,(r_o + T_d) * R_u * a_e * \varepsilon$$

where,

R_u is the ratio of wetted perimeter over total perimeter of the drainpipe;

u the wetted perimeter in m;

a_e the ratio of the area of synthetic envelope exposed to perforations over the total area per metre pipe length. The area exposed is generally the area spanning the valley of the corrugation (Figure 8). This reduction is more critical with woven than non-woven fabrics. The latter may have some in-plane flow contributing to water entry, which we will ignore in our calculation;

ε porosity of the gravel envelope, the nonwoven synthetic envelope or an organic envelope (Sections 5.1.3, 5.3 and 5.6.8);

POA Percent Open Area of a thin woven synthetic envelope (Section 5.6.8);

T_g thickness of geotextile under standard pressure of 2 kPa in m; and

T_d thickness of granular envelope in m.

The simplest way to calculate the exit gradient is by assuming that Darcy's equation can be used. This is most likely not true as we might expect turbulent flow conditions at the perforations of the drainpipe, and also at the higher flow rates at the soil-envelope interface. However, this will do for starters. Assuming equal intensity of flow around the drainpipe, Darcy's equation (Eq 18) can be written as follows:

$$i_x = \frac{q_1}{KA} \qquad \text{Eq. 30}$$

where,

i_x is the exit gradient at one of the interfaces;

q_1 the flow into drain per unit length of drain in $m^3d^{-1}m^{-1}$ (m^2/d);

K hydraulic conductivity in m/d of the surrounding base soil or of the envelope material; and

A the area open to inflow (perforation area) per unit length in m^2/m.

Few researchers report measured values of the exit gradient. Luthin *et al.* (1968) reported exit gradients as function of drain depth, depth to the impermeable layer, and pipe diameter, that range between 1 and 12. Rollin *et al.* (1987) reports that Lagacé measured values as high as 200 but did not report exit gradients, although in the laboratory they created gradients from 17.5 to 35. The French committee on geotextiles (CFGG 1986) recognises the possibility of gradients between 20 and 40 (Table 35).

5.2 Bridging

The term bridging is related to the similarity between the arch construction of a masonry bridge or arch, and the arrangement of particles at a drain opening (Box 15). The ability to form this arch has been translated as bridging criteria. Bridging criteria are similar to retention criteria (such as O_{90}/d_{90}) in purpose. The difference is that in bridging experiments the openings are usually of one size only, whereas with other experiments to derive retention criteria, such as described in Sections 5.7 and 6.2, openings have a range of dimensions that are characterised by the O_{90} value.

As will be clear later in this section, to derive O_{90}/d_{90} criteria, researchers have made direct use of the bridging criteria (opening size/particle size ratio). The work is reported following the original text closely as possible, while a uniform interpretation is given in Table 11.

If particles of smaller diameter than the drain opening form a structural arch over the opening, they will not pass through the opening. Particles larger than the opening obviously cannot pass through the opening and particles as small as one third the diameter of the opening will always form an arch (Willardson 1979). Particles one fourth the size of the opening will usually form an arch, and particles one fifth the size of an opening or smaller will mostly not form an arch. In the case of well-graded soils and gravel, few finer particles will pass through the drain opening; the remaining larger particles will form an arch. When the arch is formed and held in place by the forces of gravity and friction of the flowing water, no further material will enter the drain unless the forces that constitute the stable arching are reduced so that the arches can be destroyed. The last-mentioned will happen when reverse flow takes place such as when the system is used for sub-irrigation (Section 5.1).

Willardson performed the bridging study using dry sand that was poured from a height of 15 cm into a cylinder with a plate with circular holes at its base. Different size holes were tested: 1.6, 3.2, 5.8, 6.4 and 9.5 mm diameter. A slotted hole with dimensions 3.2 x 32 mm was also tested. It was found that the effective size (narrow dimension) for bridging of the slotted hole was three times that of the circular holes ($O_{circ} = 3 * O_{slotted}$).

For firm mechanical support, Nelson (1960) suggested a conservative criterion for glass fibre filter of $O_{50}/d_{50} \leq 1$, while Davies *et al.* (1987) recommend $O_{50}/d_{50} \leq 5$ derived from their research work on mechanical support of geotextiles for drainage wrapping. Davies et al. (1987) used a permeameter set-up (Section 5.7) to test the following combinations: mesh/glass spheres ($O_{50}/d_{50} \leq 2.4$); filter/glass spheres ($O_{50}/d_{50} \leq 3.3$); mesh/soil ($O_{50}/d_{50} \leq 4.5$); and filter/soil (never failed but theoretical extrapolation resulted in $O_{50}/d_{50} \leq 6.2$). Hence they

Box 15 Typical and standard pipe drain opening sizes (gaps and perforations).

General remarks:	*minimum opening area for subsurface drains 800 mm^2/m (Smedema and Rycroft 1983, with reference to ISO 1979).* *Typical openings between clay tiles of 250 - 300 mm long were assumed to be 1/16" = 1.6 mm. With a 130 mm diameter tile this would result in 2 * 3.14 * 6.5 * 1.6 * 3.3 = 215 mm^2/m which is far less than the desired minimum of 800 mm^2/m.* *Openings of plastic perforated pipes are typically 0.5 - 2 mm width and 5 mm long in the valleys of the corrugations. Wave length of corrugations are typically 5 - 15 mm for diameters up to 100 mm, 15 - 30 mm for diameters up to 200 mm, and as much as 50 - 75 mm for pipe diameters of 300 - 750 mm (Cavelaars et al. 1994).* *Typical opening area would be 1 - 2% of the pipe surface area (Bentley and Skaggs 1993, Rollin et al. 1987)*
USA	*Tiles: recommended crack widths 3- 6 mm in stable soils; unstable soils as close as possible. Crack widths of 0.8 - 3 mm have been observed (Schwab 1957). To assure crack widths USBR (1978 - 1993) recommended using spacer lugs of 3 mm as standard.* *Corrugated plastic pipe (USBR 1993): a minimum of 2120 mm^2/m (1 sq. inch per foot). To meet this criterion manufacturers provide pipes up to 250 mm diameter with 5 mm round holes and pipes > 250 mm with 10 mm round holes!* *SCS 1994 gives a design example mentioning that standard perforations are ¼ inch plus-or-minus 1/16 inch: max. = 5/16 in = 8 mm.* *Dimensions allowed in the US are seemingly excessively wide!*
Canada	*Corrugated pipes should have at least 1% open area. Rectangular slots vary in shape from 0.5 mm x 5 mm to 2 mm x 15 mm (Rollin et al. 1987)*
Pakistan	*Corrugated pipes: minimum entrance area 2115 mm^2/m (1 in^2/ft), circular holes max. dia. 4.75 mm (3/16"), slotted holes max. width 3.2 mm(1/8") and length 31 - 44 mm. (FDP 1987).*
Egypt	*Standard used by Aga factory for PVC corrugated pipes external diameter 80.7 mm, inside diameter 73 mm. 640 notches/m, area of perforation 1.3 mm x 3.0 mm, water entry area should be at least 2400 mm^2/m.*
Netherlands	*Corrugated pipes: circular holes diameter 1.5 mm and approx. 1000 mm^2/m; slotted holes length < 5 mm, class I width 0.8 - 1.2 mm, class II width 1.2 - 2.0 mm, and entrance area 2000 - 5000 mm^2/m (Someren 1970). According to NEN 7036 (1976) width of Type A: 0.9 - 1.4 mm, Type B: 1.4 - 2.0 mm, length of perforation equal or greater than width, perforations in at least 3 rows equally spaced, min. 1000 mm^2/m.*
France	*Corrugated pipes: perforations of 1.5x5 and 1.5x1.5 mm^2, entrance area of 3000 mm^2/m. Smooth pipes width 0.5 - 0.8 mm and length 20 - 40 mm with entrance area of 1000 mm^2/m (Someren 1970)*
Germany	*Corrugated pipes: perforation width 1 - 2 mm, entrance area 830 mm^2/m for pipes with diameter < 65 mm and 2200 - 3260 mm^2/m for pipes with diameter 65 - 160 mm (Someren 1970). In Framji et al. (1987): three classes 0.6 - 0.9, 1.1 - 1.5, and 1.7 - 2 mm slot width for pipes 50 - 200 mm, number of slots 560 - 715 per m, entrance area: 2300 - 9000 mm^2/m.*
Europe	*Arranged in any pattern in not less than 4 rows in the valley of the corrugation, with at least 2 perforations per 100 mm of each single row. Nominal perforation width 1 – 2.3 mm by increment of 0.1 mm, average perforation width shall not deviate more than 0.2 mm, single perforation more than 0.4 mm. Entrance area > 1200 mm^2/m. Nominal diameters 50 - 630 mm. (CEN/TC 155 N 1259 E, 1995).*

concluded that a ratio of 5 was safe for UK soils. Graauw *et al.* (1983) reported bridging ratios of D_{50}/d_{50} of 3 - 5 for granular filters based on laboratory research for the Storm Surge Barrier of the Oosterschelde in the Netherlands. With cyclic flow through the filter the value of 3 was recommended, while the value of 5 was for stationary flow conditions.

The US Army COE (1978) does not specifically refer to bridging but gives the ratio of D_{50} and the perforation size as criteria to prevent infiltration of filter (envelope) material into perforated pipes as used with filter designs for levees:

circular openings: D_{50}/hole diameter ≥ 1
slotted openings: D_{50}/slot width ≥ 1.2

Trials in the laboratory to use readily available sand as envelope material in Pakistan, showed that sand with $D_{15} = 0.3$ mm, $D_{50} = 0.52$ mm, $D_{85} = 0.76$ mm and $C_u = 3.3$ would not bridge over the slotted perforations with a width of 3.2 mm until the slot width was reduced to 0.5 mm (Vlotman *et al.* 1992). This matches closely the US Army COE guideline (Table 16).

Schwab *et al.* (1986) describe tests and results of the determination of bridging factors for synthetic woven fabrics and pin-holes in a plate surface simulating perforations in plastic tubing. They used permeameters with downward flow at increasing gradients and observed failure (soils passage through the fabric or pin-holes) of the bridging. Willardson (1979) used dry soil for the test. In both cases the opening size was related to the corresponding D_{60}. The synthetic nylon fabrics specially woven for precision and uniformity, had square openings of 0.25, 0.30, 0.425, 0.50, 0.60, 0.71, 0.85 and 1 mm. Bridging factors (O/D_{60}) ranged between 1.5 and 3.5. A bridging factor of 2.0 is recommended for commercially produced fabrics that do not have the uniformity of opening sizes as used in the tests. Note, as the openings were very uniform they could be considered to represent the O_{90} of the fabrics.

Soil gradation curves used with the fabric tests were also given and from these it was determined that the O/d_{90} ratio could have varied between 0.6 and 4.3, with the combinations of most soil-fabric being $O/d_{90} > 1$. Schwab *et al.* report that the bridging factor for wet sand was lower than that of the dry sand used by Willardson (1979). Willardson (1979) found a bridging factor of 3. Schwab *et al.* (1986) further observed that at higher gradients the envelope became more stable (with downward flow), while most particle passage occurred at gradients of 1 or less. For the pin-hole tests (openings varying between 0.4 and 2.5 mm) it was found that a bridging factor (O/D_{60}) ranged between 4.9 and 9.9. Bridging factors of 6 were observed with commercially available plastic tubing (Box 15 and Box 16) with average hole size of 0.635 mm. However, initial sediment inflow in the pipe was observed and hence a bridging factor of 4 was recommended. The bridging factor was 13 to 83% higher with D_{60} than using D_{85}. The average D_{85} was 41% greater than D_{60}. USBR (1973, design of filters for small dams) and US Army Corps of Engineers (1977, "plastic filter fabrics") both used d_{85} and came up with bridging factors of 0.5 and 1 respectively. Canadian agencies prescribe $O/d_{85} < 2$ (Lagacé and Skaggs 1982). A potentially interesting study on bridging criteria is reported by Broughton *et al.* (1987) who compared drainpipes with regular slotted holes and with very small pinholes, to investigate whether pipes without envelopes but with appropriately small perforations could be used on typical Canadian soils. Unfortunately, to derive the bridging ratios, not enough details are reported in the article. The article hints at more information forthcoming or available elsewhere.

Box 16 Standards for pipe qualities.

From: ASAE Engineering Practice ASAE EP260.4 (ASAE 1997), USBR 1993, and Framji et al. (1987). Use of latest revisions is required.	
ASTM C4, C13, C200	*Specifications for Clay Drain Tile.*
ASTM C118, C444	*Specifications for Concrete Drain Tile.*
ASTM C14, C412	*Specifications for Concrete Drain Tile.*
ASTM C498	*Specification for Perforated Clay Drain Tile.*
ASTM F405	*Specification for Corrugated Polyethylene (PE) Tubing and Fittings.*
ASTM F667	*Specification for 8, 10, 12, and 15 in. Corrugated Polyethylene Tubing. From: CV (1988) and ASAE 1997.*
NEN 7036	*Geribbelde draineerbuizen van ongeplastificeerd PVC. (Dutch; Corrugated unplasticised PVC pipes for sub-soil drainage, 1976).*
NEN 7080	*Cilindrische moffen van kunststof voorzien van klikverbindingen voor geribbelde draineerbuizen (Dutch; Synthetic cylindrical click-connectors for corrugated drainpipes).*
DIN 1180	*Standardised dimensions, weight and properties of clay tiles. Germany.*
DIN 1187	*Standardised dimensions, weight and properties of unplasticised PVC pipes (PVC-U). Germany.*
CEN/TC 155 N 1259 E	*Plastics piping systems for agricultural land drainage (PVC-U), Part 2: Pipes without envelope.*

John and Watson (1994) investigated bridging using two- and three-dimensional physical models (theoretical approach in a sense). To simulate the openings they used acrylic sheets to vary the O_{90} between 85 and 222 mm, and metal washers to simulate the soil in the 2D model and mixtures of differently sized spheres for the 3D model (d_{90} varying between 36.7 and 100 mm). The more realistic 3D model showed results that were within 1.4% - 3.9% of the 2D model. By gradually increasing the O_{90} of acrylic sheets, and using 'soils' with coefficients of uniformity (C_u) between 1 and 8.35 they determined the O_{90}/d_{90} ratio at which bridging would no longer occur. They observed that well before the collapse, smaller particle size not essential to the bridging flowed through the filter. It was determined that soil particles smaller than 0:228 $* \; O_{90}$ were of no significant use in bridge formation. Hence they established an upper retention limit as function of the C_u and a corresponding lower limit, which they refer to as positive wash through for non-compressible geotextiles with unidirectional flow conditions. Table 10 gives their findings. They also made a comparison with criteria given by Giroud (1982) and Schober and Teindl (1979), both given in Table 35, using the concept of the linear coefficient of uniformity (C'_u) as described in Section 5.5.1 (see Figure 52) to convert various O/D ratios to O_{90}/d_{90}. The new limits developed by John and Watson span basically the values obtained by Giroud and Schober/Teindl (Table 10).

Table 10 The O_{90}/d_{90} boundary values as derived from various sources.

C'_u	John and Watson (1994)		Giroud (1982)*		Schober and Teindl (1979)
	Upper limit** 'Retention' (λ_{r1})	Lower limit** 'Permeability' (λ_{p1})	Dense soil	Loose soil	
1	4.21	0.96	2.47	1.05	2.41
2	2.41	0.55	2.65	1.21	2.16
3	2.37	0.54	2.61	1.28	1.85
4	2.31	0.53	1.53	0.76	1.46
5	2.28	0.52	1.04	0.53	1.15
8	2.23	0.52	0.44	0.22	0.63
10	2.19	0.50	0.30	0.15	0.43

* Giroud reported O_{95}/d_{85} in his Fig 7 which was converted to O_{90}/d_{90} using the linear PSD curve to convert d_{85} to d_{90} and something similar as the ratios in Table 27 to convert O_{95} to O_{90}

** Retention criteria $O_{90} \leq \lambda_{r1} * d_{90}$. Permeability criteria $O_{90} \geq \lambda_{r1} * d_{90}$

Additional description of the application of the bridging factor is in Section 6.2.1.

Concluding remarks

Bridging factors have been reported in the literature since the early days of filter designs (1950s) until recently (1994). Bridging factors are different from retention criteria of envelope design in that they relate a uniform opening to particle size distribution, while retention criteria consider the distribution of openings against the distribution of particles. Several authors theorised the transfer from opening size to a d_{50} or d_{90} size, while others actually reported distribution details in sufficient detail to include ratios with d_{50} d_{85} and d_{90}. These finding have been summarised in Table 11. From the table it might be clear that it is hardly possible to distil a uniform criterion, even if granular and fabric criteria were to be considered separately. Conversion from an single opening size or from D_{50} values to O_{90} or D_{90} should clearly reveal that some inflow of sediment is expected of particles that are not meeting for instance the bridging criteria of Willardson (1979). Except for two reported values (US Army COE 1978 and Vlotman *et al.* 1992) all criteria seem to allow openings that are either equal to or 2 -3 times the characterising soil particle size. For values below 1 it can only be hypothesised that the soil samples used were gap-graded, with predominant fines in the sample (see Section 5.5.1, Figure 56).

Table 11 Summary of Bridging Criteria.

Bridging criteria	Converted to O_{90}/d_{90} or D_{90}/d_{90} *	Remarks
Willardson (1979)		Dry soil poured onto plate with perforations
$O_p/d < 3$	$D_{90}/d_{90} < 3$	Always bridged
$O_p/d = 4$	$D_{90}/d_{90} = 4$	Usually bridged
$O_p/d > 5$	$D_{90}/d_{90} > 5$	Never bridged
Pakistan (Vlotman *et al.* 1992)		Fine non-cohesive sand with $C_u = 3.3$
$O_p/d_{15} = 1.7$		Bridging ratios for same soil.
$O_p/d_{50} = 0.95$	$D_{90}/d_{90} < 0.6$	Ratio of d_{85} / d_{50} for the investigated soil was 1.5
$O_p/d_{85} = 0.67$		
US Army COE (1978)		
$O_p/d_{50} < 1$	$D_{90}/d_{90} < 0.5$	For circular holes
$O_p/d_{50} < 0.83$		For slotted holes (originally: $D_{50}/O > 1.2$)
Schwab et al. 1986		Wet sand in permeameter
$O_p/d_{60} = 1.5$ - 3		Uniformly holed synthetic fabrics => $O_p = O_{90}$
$O_p/d_{60} < 2$	$O_{90}/d_{90} < 1$	Recommended for commercial fabrics
$O_p/d_{60} = 4.9$ - 9.9		For pinholes as used in tests
$O_p/d_{60} < 4$	$D_{90}/d_{90} < 2$	Recommended for commercial perforation sizes
Graauw et al. 1983		Granular filters
$D_{50}/d_{50} < 3$	$D_{90}/d_{90} < 1.5$	Cyclic flow
$D_{50}/d_{50} < 5$	$D_{90}/d_{90} < 2.5$	Stationary flow conditions
Davies et al. 1978		
$O_{50}/d_{50} < 5$	$D_{90}/d_{90} < 2.5$	Pipe field drains. O_{50} reported but probably is O_p
John and Watson (1994)		Physical model study; fabrics were not used but
$O_{90}/d_{90} < 2$ - 4	$O_{90}/d_{90} < 2$ - 4	the authors converted their data already to O_{90}

* - using Schwab's observations for conversions that the bridging factor with D_{60} are 13 - 83% higher than with D_{85}, where applicable 50% was used for the conversion. Also, O_{90}/d_{90} is given when fabrics were involved and D_{90}/d_{90} is given when granular material was used.

5.3 Hydraulic failure gradient of the base soil

The success of a drain envelope material is largely dependent on the relative stability of soil particles at the soil-envelope interface or of the soil aggregates (Box 17).

Bridging of soil particles over drain perforations or external to the openings in a drain envelope are essential. It is best if the base soil has the capacity to bridge over the drain openings without an envelope material. This depends very much on the hydraulic gradients that are to be expected at the drain openings, the size of the drain openings, and the stability of the soil particle aggregates in the base material.

Box 17 Aggregate stability from intrinsic soil components.

A study to investigate the intrinsic factors that control water stability of soil aggregates offers some interesting viewpoints worth considering in future work with drain envelope design research (Bazoffi et al. 1995). The abstract of the article best explains the results of the study:

"The objective of this study was to evaluate the nature of the relationship between the water-stability of soil aggregates and some physical, chemical and mineralogical properties of surface (0-20 cm) soils from central Italy. The index of stability used is the mean-weight diameter of water-stable aggregates (MWD). The ratio of total sand to clay which correlated negatively with MWD ($r = -0.638$) is the physical property which explained most of the variability in aggregate stability. The Chemical properties which correlated best with aggregate stability are FeO ($r = 0.671$), CaO ($r = 0.635$), $CaCO_3$ ($r = 0.651$) and SiO_2 ($r = 0.649$). Feldspar, chlorite and calcite are the minerals with the most controlling influence on MWD with respective r values of -0.627, 0.588 and 0.550. The best-fit model developed from soil physical properties explained 59% of the variation in MWD with a standard error of 0.432. The best-fit model developed from chemical properties explained 97% of the variation in MWD with a standard error of 0.316 and that developed from mineralogical properties explained 78% of the variation in MWD with a standard error of 0.222. Also the closest relationship between measured and model predicted MWD was obtained with the chemical properties based model ($r = 0.985$), followed by the mineralogical property based model ($r = 0.884$) and then the physical properties based model ($r = 0.656$). This indicates that the most reliable inference on the stability of the soils in water can be made from a knowledge of the amount and composition of their chemical constituents".

The hydraulic gradient at which a confined cohesive soil cannot resist the drag force of water on soil particles is defined as the Hydraulic Failure Gradient (HFG), (Samani and Willardson 1981). The HFG is different from the "critical gradient" used in soil mechanics. The "critical gradient" is when the upward drag force on the soil particles equals the buoyant weight of the particles for unconfined conditions and the soil loses its structural integrity (Taylor 1948, Irwin and Hore, 1979, Dunn et al. 1980).

Non-cohesive soils develop a quicksand condition and become unstable at the critical gradient. This is also called fluidisation (Adel 1992a, b, and c) or liquefaction (USBR 1974). The critical gradient for non-cohesive soils may be determined from:

$$i_c = \frac{G-1}{1+e_v} \qquad \text{Eq. 31}$$

where,

i_c is the critical gradient (generally close to 1 for non-cohesive soils);

G the specific mass of the individual soil particles (± 2.65 g/cm^3, see also Box 17 , p 146);

e_v the void ratio of the soil (when $e_v = 0.65$, $i_c = 1$)

and

$$e_v = \frac{\varepsilon}{1 - \varepsilon} \qquad \text{Eq. 32}$$

or

$$\varepsilon = \frac{e_v}{1 + e_v} \qquad \text{Eq. 33}$$

where,

e_v the void ratio is the division of the volume of pore space by the volume of solids;

ε is porosity: the ratio of volume of pore space to total bulk volume.

Instability at a soil-envelope interface occurs at the critical gradient, plus the gradients necessary to overcome inter-granular stresses due to soil surcharge and soil cohesion (Luthin *et al.* 1968). This total gradient required is then called the Hydraulic Failure Gradient (HFG). The HFG of a particular soil can be determined in the laboratory by means of permeameter tests (Section 5.3.1) and can be related to basic soil properties such as Plasticity Index (PI), (Section 5.5.2) and the saturated hydraulic conductivity of the soil. Figure 43 shows a number of soils for which HFGs have been determined. A positive correlation was found between PI and HFG but this relationship was not transferable between arid and humid regions. However, the HFG could be predicted for soils from both humid and arid regions in the USA, when hydraulic conductivity of the soil as indirect measure of soil compaction, and the PI as a direct measure of the cohesiveness and stability of a soil, were both included in the assessment. Using the data given by Samani and Willardson (1981) the following relationship was derived ($r^2 = 0.94$):

$$HFG = e^{0.332 - 0.132K + 1.07 \ln PI} \qquad \text{Eq. 34}$$

where,

HFG is the Hydraulic Failure Gradient;

K the hydraulic conductivity in m/d;

PI Plasticity Index; and

e the base of natural logarithm (2.7183).

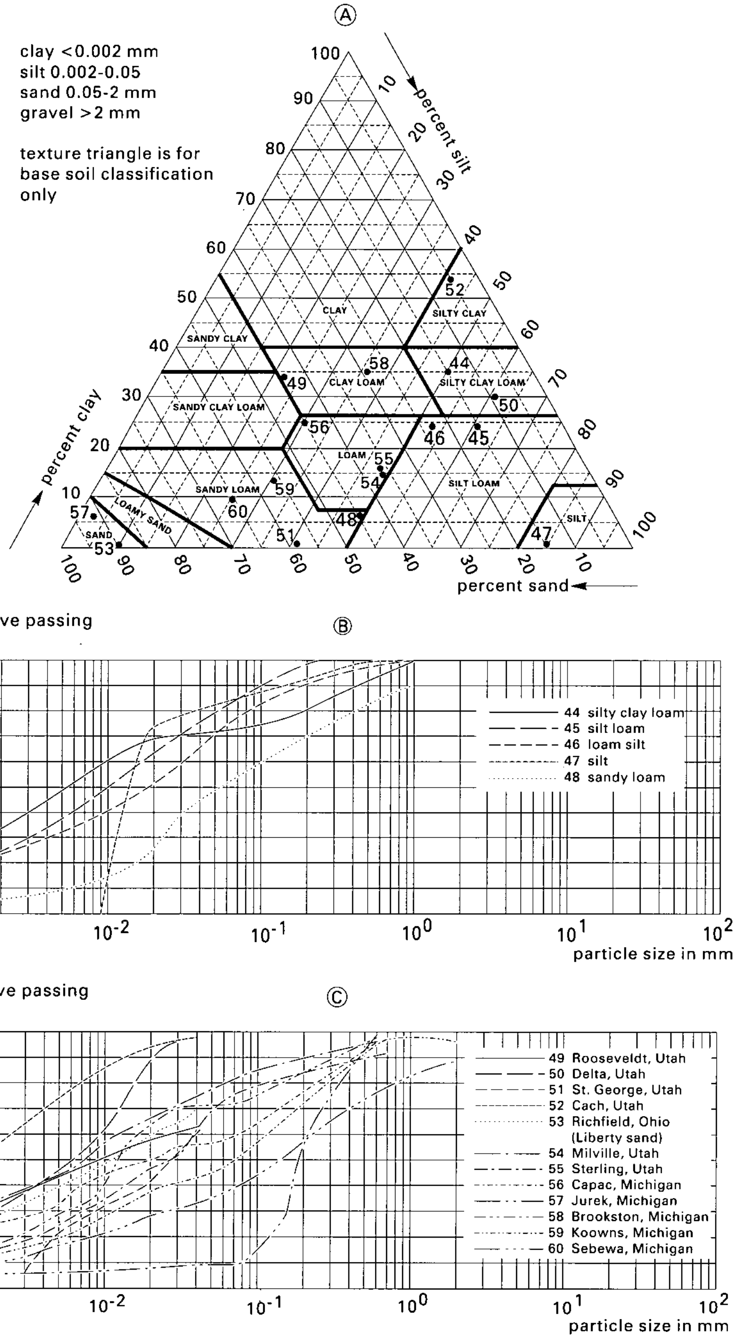


Figure 43 Soils for which Hydraulic Failure Gradients have been determined.
- A Texture triangle with soils indicated.
- B Cache Valley soils (Willardson and Ahmed 1988).
- C After Batista 1987 and Samani 1979.

Note, Eq. 34 is different from the one presented by Samani and Willardson (1981) because different units are used. Also, may be observed (Table 12) that neither Eq. 34 nor the original formula in the paper by Samani and Willardson reproduces all values as mentioned in the original paper. No explanation for this could be traced. In a later reference (Stuyt and Willardson, in 1999) the formula was adjusted to:

$$HFG = e^{0.102 - 0.108K + 1.09 \ln PI} \qquad \text{Eq. 35}$$

On comparing both equations with the original data in Table 12 it appears that Eq. 34 gives a slightly better fit than Eq. 35. No regression coefficients were reported for Eq. 35. It is recommended that Eq. 34 be used unless data that show different relationships become available for an independent correlation. Figure 44 shows that with K ≤ 1 m/d the K-value is not important anymore and HFG becomes a function of PI only with a constant intercept determined by K = 1 m/d.

Table 12 Original data to derive HFG formula and HFG calculated. (after Samani and Willardson 1981).

Soil	PI	K measured from original tests			HFG measured	HFG calculated		
		cm/min	m/d			paper*	Eq. 34	Eq. 35
Roosevelt	19	3	43.2	Samani	0.5	0.11	0.11	0.26
Roosevelt	19	2.3	33.12	(1979)	0.6	0.4*	0.41	0.77
Roosevelt	19	0.24	3.46		11	22.21*	20.63	18.88
Roosevelt	19	0.18	2.59		28	23.07*	23.12	20.73
Roosevelt	19	1.1	15.84		4	3.98*	4.03	4.96
Roosevelt	19	0.9	12.96		2.4	3.89*	5.89	6.76
Delta	16	0.05	0.72		3.5	24.61*	24.62	21.04
St. George	3	0.28	4.03		0.7	2.65	2.65	2.37
Cache	26	0.05	0.72		40	41.37*	41.40	35.71
Liberty sand	1	0.4	5.76		0.5	0.65	0.65	0.59
Liberty sand	1	0.37	5.33		0.45	0.69	0.69	0.62
Liberty sand	1	0.5	7.2		0.45	0.54	0.54	0.51
Millville	3.8	0.28	4.03	Batista	2.2	3.42	3.42	3.07
Millville	3.8	0.05	0.72	(1978)	5	5.29	5.29	4.39
Millville	3.8	0.08	1.15		4.5	5.00	5.00	4.19
Sterling	1.7	0.8	11.52		0.8	0.54	0.54	0.57
CAPAC	15	0.001	0.014		29	25.22	25.22	21.16
Jurek	1.5	0.7	10.08		0.8	0.57	0.57	0.58
Brookston	25	0.00042	0.00605		40	43.61	43.61	36.96
Keowns	5	0.0006	0.00864		19	7.79	7.79	6.39
Sebewa	4.7	0.00098	0.0141		14	7.29	7.29	5.97

* Difference between values reported by Samani and Willardson (1981) and Eq. 34 could not be reconciled.

The HFG for a soil was essentially not affected by the envelope material against the soil (Table 13) in the permeameter tests, although Willardson and Ahmed (1988) observed that HFG decreased for various soils when the coarseness of the envelope material increased. This, however, was more a function of bridging over openings (see Section 5.2 and Box 18) than of the envelope material.

Table 13 HFG of five soils supported by envelopes or screens and one test without screen. (Willardson and Walker 1979).

Material or screen number	Openings in mm	Hydraulic Failure Gradient per soil type				
		St. George *	Richfield	Roosevelt **	Delta	Cache
none	–	–	–	0.8	–	–
8	2.36	–	–	**6.9**	–	–
screen	1.6	**2.4**	–	6.4	–	–
30	0.6	2	7	**6.1**	3.5	3
60	0.25	2.6	–	**6.8**	–	–
120	0.125	2.1	–	–	–	–
230	0.063	1.9	–	–	–	–
Drainguard®	–	2.2	–	7.1	–	–
Typar®	–	**2.1**	–	6.2	–	–
Mirafi®	–	2.7	–	–	–	–

* - max. particle size 0.124 mm
** - max. particle size 0.1 mm
bold HFG values: test period longer than 24 hrs.

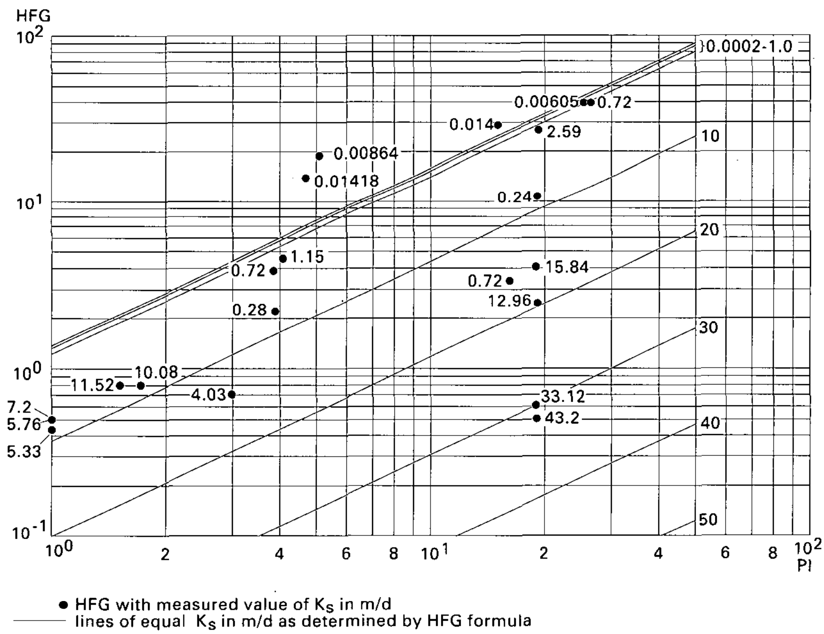


Figure 44 Fit of original HFG formula.

The critical gradient is approximately 1.0 but depending on the porosity of unsupported (non-cohesive) soil, it might be higher or lower than 1.0. As indicated above, cohesive soils fail at higher gradients. The Roosevelt soil failed in tests by Samani and Willardson (1981) at a gradient of 0.8 when it was unsupported by an envelope material. Here, the individual particles were not attached to each other or held together by inter-granular stresses. When supported or restrained by a drain envelope the Roosevelt soil failed at a gradient of 6.1 - 6.9. The soil had been dried, crushed, and sieved before it was re-wetted to field capacity and compacted in the permeameter. Apparently compaction alone did not make the soil reach its potential cohesive mass again but support was necessary as well (see also Box 18).

Box 18 What affects the HFG?

Why does the HFG formula only have PI and K_s as parameters if surcharge and envelope material seem to have an effect as well? Answer: surcharge is responsible for developing the inter-granular stresses that hold the soil together. The HFG in the unsupported Roosevelt soil was 0.8. When it was supported, i.e. the inter-granular stresses came into play, the HFG went up. When the gradients get high enough, the soil structure (cohesion and inter-granular stresses in action) cannot withstand the forces and that is what constitutes the supported potential HFG, which seemed to be most accurately modelled by using PI and K_s. It should not be a surprise that the opening size (i.e. the material) has an effect on HFG. If the openings in the drain or the envelope material are too large, there is no bridging and the inter-granular stresses cannot develop. If the openings are too small, then clogging will occur. Some gravel envelopes in Pakistan were too coarse to allow the maximum HFG to develop and the soil failed as though unsupported by an envelope.

Does the drain envelope have a major effect on the HFG? Answer: when we choose an envelope material with the correct O_{90} for a soil, we are choosing an opening size that develops the full bridging potential of the soil. If you choose an envelope with openings that are too large, the soil may fail at the low critical gradient of the soil because the soil is effectively unsupported and cannot develop bridging strength over the opening. The particle size distribution of the soil determines what will bridge and what size opening is required to allow bridging, before too much soil has gone through the opening and before some particles large enough to bridge the openings show up at the same time. Fortunately, soils bridge well.

When Willardson and others (Section 5.3.1) were testing screens, they obtained good bridging for most soils with window screen size openings. These tests started out using actual envelope materials, but it was soon clear that it was the opening size that was important, and they switched to sieve screens to find out what the true HFG of the soils were. When the screens were too large, the soils failed at low gradients. When they reached the minimum size for bridging for a particular soil, then the HFG did not get any higher for screens with smaller openings. Because the permeameter used was 10 cm across, they had to support the envelope materials with some hardware cloth (heavy, coarse, galvanised screen that would prevent the deformation of the envelope material and the movement of the soil). The openings were relatively large so that they would not offer any hydraulic resistance or opportunity for the soil particles to bridge against them. If they had tried to stretch a piece of envelope across that opening, it would have moved and the soil would have failed at the critical gradient because the restraining support for the soil would have been lost. They very carefully maintained straight parallel flow through the system so that the gradients would be true gradients and would not be affected by secondary convergence.

From the foregoing it may be clear that based on the literature review it cannot be conclusively determined that the HFG method is fully applicable in all circumstances. There also seems to be some conflicting reporting on whether the envelope opening size (i.e. the envelope material) affects the results or not. Hence further study on the HFG method is needed. The next sections give additional details on the methods and materials used by the original researchers.

5.3.1 Determination of HFG in laboratory

HFG values were determined and reported on four occasions: Willardson and Walker (1979), Batista (1978), Samani (1979), and Willardson and Ahmed (1988). The same permeameter with upward flow was used in all cases (Figure 45).

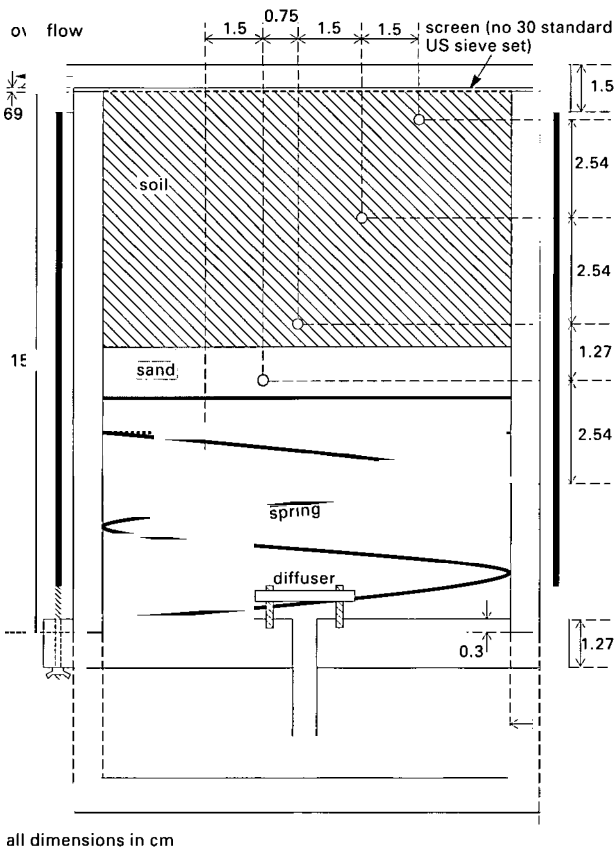


Figure 45 Cross-section of permeameter used for HFG determinations (Samani and Willardson 1981).

All tests were done using a no. 30 screen (0.059 mm), which had a porosity of 34%, to confine the soil in the permeameter, except during the testing of different screens (Table 13, and Box 18). The soils used in the permeameter were compacted using a load of 29.54 kN/m^2, simulating the soil pressure expected at 1.5 m depth. The configuration of the permeameter is the same as the one described later in Section 5.7.4.

5.4 Calculating saturated hydraulic conductivity

The saturated hydraulic conductivity (K) of the envelope material, the trench backfill material and the undisturbed soil outside the trench and/or pipe (in case of trenchless construction), plays a major role in the ultimate successful operation of the drain and the drainage system. From field investigations the first observations of the hydraulic conductivity at drain depth are obtained. Table 14 gives the range of K-values one might expect in different soils. Often K-measurements are not available but if the gradation curves of the soils at drain depth are known, the various percentages of certain size material passing can be derived (i.e. d_{15}, d_{50}, etc.). Note, hydraulic conductivity 'D' or 'd' are used interchangeably.

For sandy, uniform soils the soil permeability might be related to the grain-size distribution using the specific surface ratio (U) of the various grain size classes. This U-ratio is defined as the total surface area of the soil particles per unit mass of soil, divided by the total surface area of a unit soil mass consisting of spherical particles of one cm diameter (De Ridder and Wit, 1965). The U-ratio of a certain sub-fraction may be determined from:

$$(U_{cm})_i = \frac{0.4343}{\log d_1 + \log d_2}\left(\frac{1}{d_2} - \frac{1}{d_1}\right) \qquad \text{Eq. 36}$$

where,

$(U_{cm})_i$ is the U-ratio of the sub fraction; specific surface of the soil particles, dimensionless. The U ratio of each sub-fraction is multiplied by the weight of the fraction, and the sum of all these products is divided by the total weight of all the sub-fractions in grams in order to find the U-ratio of the sample as a whole;

d_1 the grain size (diameter) of soil particle in cm of the upper boundary of the sub-fraction of interest (i.e. the sieve size of the sieve above the one from which the sample was taken); and

d_2 the grain size (diameter) of soil particle in cm of the lower boundary of the sub-fraction of interest (i.e. the sieve size from which the sample was taken).

and,

$$U_{cm} = \frac{\sum_{i=1}^{n} W_i \, (U_{cm})_i}{\sum_{i=1}^{n} W_i} \qquad \text{Eq. 37}$$

where,

U_{cm} is the U-ratio of the soil sample; specific surface of the soil particles, dimensionless;

W_i weight of sub fraction i in g; and,

n the number of sub fractions.

De Ridder and Wit (1965) reported good results for calculating the K from disturbed soil samples using:

$$K = \frac{864 f_s}{\eta} \frac{V_v^3}{(1 - V_t)^2} \frac{1}{U_{cm}^2} \qquad \text{Eq. 38}$$

where,

f_s represents a factor accounting for the influence of the shape and structure of the soil particles on the actual length of the flow lines. No values are suggested however;

V_v volume of voids through which flow takes place;

V_t total volume of voids (pores); and

η the dynamic viscosity.

The formula by De Ridder and Wit is merely presented as one of the methods found in the literature for calculating hydraulic conductivity from particle size analysis. The method was not used because most of the data available did not have enough detail, such as the upper and lower boundaries of sub-fractions and a set of shape factors (f_s).

Other researchers presented their methods in more detail and these are described below. The hydraulic conductivity of granular material with a uniform particle size distribution is proportional to the square of the diameter of its particles. Classically, when the particle size distribution is not uniform, the hydraulic conductivity is proportional to either d_{15}^2 or d_{10}^2. Hence Terzaghi's original criteria that $D_{15}/d_{15} > 4$ implies that the hydraulic conductivity of the filter is 16 times that of the base soil, although it is usually said to be approximately 10 times. The difference is because of the difference in theory and actuality, which is quantified, in yet another shape factor (sf, see below).

Table 14 Range of K-values for various texture classes. (Vlotman *et al.* 1992, Smedema and Rycroft 1983)

Texture	K in m/d
Gravel, crushed rock	1500 - 3500
Gravel, Natural river run	100 - 1500
Sand and gravel mixtures	5 - 100
Gravely coarse sand	10 - 50
Medium sand	1 - 5
Sandy loam, fine sand	1 - 3
Loam, clay loam, clay (well structured)	0.5 - 2
Very fine sandy loam	0.2 - 0.5
Clay loam, clay (poorly structured)	0.002 - 0.2
Dense clay (no cracks, pores)	< 0.002

Work done at the Pennsylvania State University by Hazen indicated that flow was laminar in all soils of uniform grain size diameter equal to or smaller than about 0.5 mm, and showed that the flow may be turbulent for granular materials coarser than 0.5 mm diameter (Means and Parcher, 1963). Sherard *et al.* (1984a) investigated their laboratory results along similar lines of thinking. They found that for uniform and well graded filters with C_u up to 10, which do not have a significant quantity of material finer than no. 200 sieve[27] (0.074 mm), the hydraulic conductivity for dense filters (well compacted) is primarily dependent on the D_{15} size.

The relationship between hydraulic conductivity and the particle size that represents best pore size is given by:

$$K_c = sf_{xx} * D_{xx}^2 \qquad \text{Eq. 39}$$

where,

K_c is the average (calculated) hydraulic conductivity in m/d (for $D_{15} > 0.2$ mm, Figure 49);

sf_{xx} a shape factor related to the characteristic particle diameter D_{xx}: $sf_{10} = 864$ derived from Hazen (originally $sf_{10} = 100$ cm^{-1} sec^{-1} with K_c in cm/s and D_{xx} in cm). For clean coarse sands and gravels; $sf_{15} = 302.4$ derived from Sherard et al. (1984a). Originally $sf_{15} = 0.35$ with K_c in cm/d and D_{xx} in mm. Valid for uniform filters and well graded filters with $C_u<10$, $D_5 > 0.074$ mm, and $D_{100} < 4.76$ mm (Sieve No. 4, Table 15); and

D_{xx} the characteristic or effective diameter in mm. Hazen found the D_{10} to be the best representative particle size, while Sherard and co-authors settled on the D_{15} as the most representative.

27 The plots in the two articles suggest that there was essentially no material finer than No. 200 sieve, and even a 5% passing could dramatically alter the relationship between K and D_{15}.

Hazen defined the effective grain size as the diameter of the particle size of a soil of uniform particles that would have the same permeability as a graded soil consisting of a range of particle sizes. Hazen's value for the shape factor gives K-values approximately three times higher than Sherard's.

Sherard *et al.* (1984a) found equally good correlation with D_{10} and D_{20}, but plots of the measured hydraulic conductivity versus D_{25} and coarser sizes did not show a good correlation. Contrary to what is mentioned in the literature they found that D_{50} is not a good measure for pore channel sizes. A further limitation is that instead of using the D_{15} of the whole sand-gravel filter but one should take the value that results from plotting the material that is smaller than no. 4 sieve (4.76 mm).

Figure 46 shows the gradation curves of the soils used to derive the formula presented by Sherard *et al.* (1984a) and those reported by Santvoort (1994), Muth (1994), and Rehman and Vlotman (1993). Sherard's data presented in their Table 2 are shown in Figure 47. From these figures and the text of the article it would seem that the data fitting was set with a fixed power of 2 (to match theoretical derivations), while the figures show a power relation closer to 1.7. Different relationships were found for the data presented by Santvoort (1994) and Muth (1994), and the measured data from Rehman and Vlotman (1993, Figure 48). However, it was confirmed that selection between either D_{15} or D_{10} as characteristic particle size for hydraulic conductivity calculation did not alter the relationship substantially. Only Santvoort had some data in the K-range tested by Sherard; the others reported soils with lower K values. Although not much data is available in the range of K < 0.002 m/d, it would appear that Eq. 39 overestimates the hydraulic conductivity and a better fit would be obtained for clay soils using:

$$K_{cc} = cf_{xx} * D_{xx}^{3.6} \qquad \text{Eq. 40}$$

where,

K_{cc} represents the average (calculated) hydraulic conductivity in m/d or cm/s for when $D_{15} < 0.2$ mm;

cf_{xx} a shape factor for clayey soils (similar to sf_{xx} but not the same);
6270 when K in m/d and D_{15} (Figure 49);
7.3 when K in cm/s and D_{15}; and

D_{xx} the characteristic particle size diameter in mm at either 10% or 15% passing. A slight better fit was obtained using D_{10}.

Some caution should be used in the use of Eq 40, because there are only three data points that basically determine the exponent of D to be in the order of 3.6. These data points came from Santvoort (1994). Without these points the exponent would be close to between 1.7 and 2.

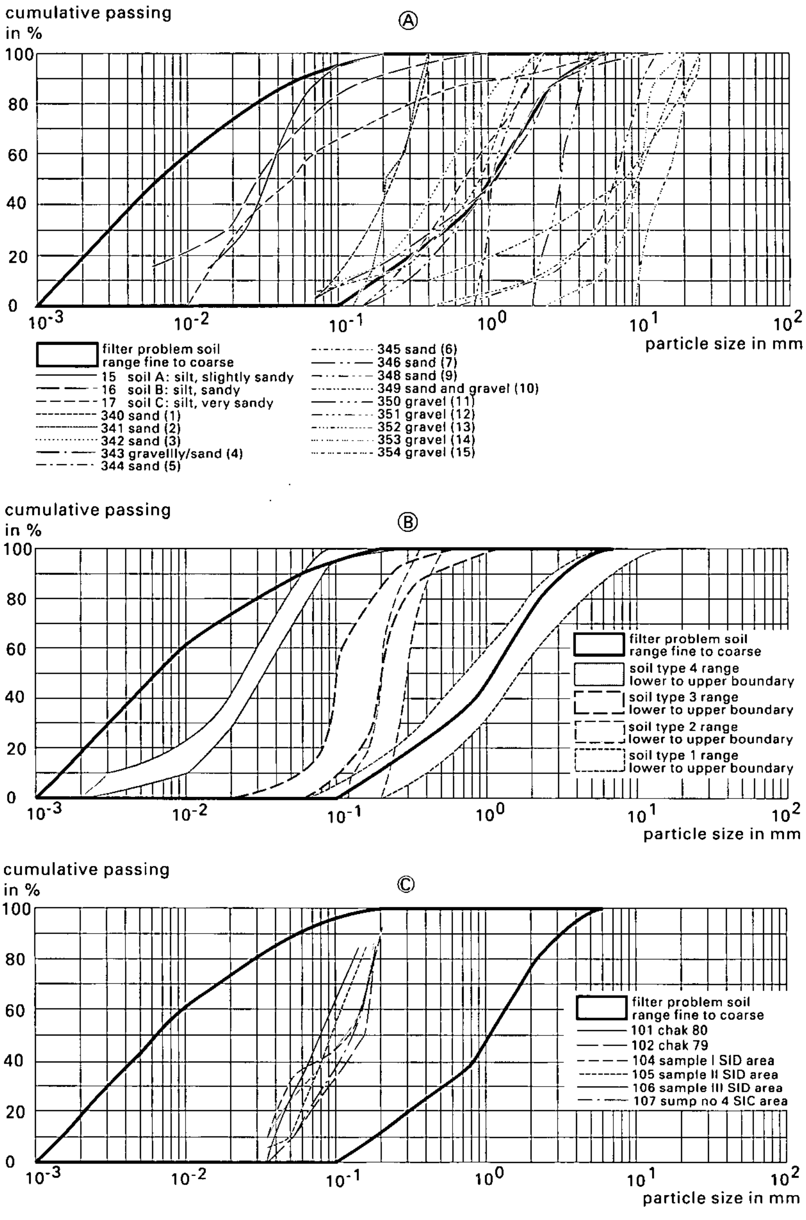


Figure 46 Gradation curves for which hydraulic conductivity was also reported.

A Regions of certain soil/geotextile characteristics according to German FSV (1987) and soils used with measured K-value after Muth (1994), soil nos. 15 – 17, and Sherard *et al.* (1984a), soil and gravel nos. 340 – 354.

B Regions of certain soil/geotextile characteristics according to German FSV (1987) and BAW soils with reported K-value (Santvoort 1994), soil nos. 93 – 100.

C Regions of certain soil/geotextile characteristics according to German FSV (1987) and soils used in the laboratory in Pakistan with measured K- value (Rehman and Vlotman 1993), soil nos. 101 - 107.

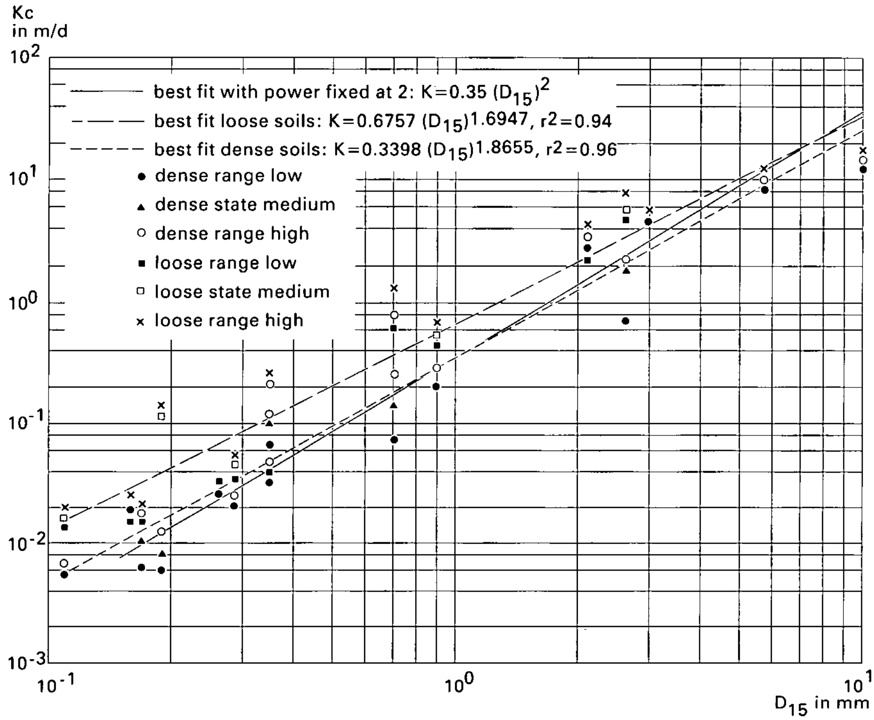


Figure 47 Hydraulic conductivity as function of particle size diameter. (after Sherard *et al.* 1984a).

Remark: Calculated hydraulic conductivity, K_c, in m/d using the data presented by Sherard et al. 1984a. Note that curve parameters are different from the ones presented by Sherard because of changes in units of K.

It is clear that calculating the hydraulic conductivity from a characteristic particle size remains questionable and is open to much discussion. However, when no other data are available, and if one needs to make hydraulic conductivity estimates for drain envelope design, then for D_{15} < 0.2 mm Eq. 40 is appropriate and for 0.2 mm < D_{15} < 10 mm Eq. 39. For gravel envelopes no satisfactorily relationship was found between characteristic particle size and measured K-value (Figure 48 where r^2 = 0.60 with D_{15} r^2 was only 0.36 when D_{10} was used). It was clear in this case, however, that both Eq. 39 and 40 underestimate the K_c. For more information on the determination of the permeability from grain size distribution see also Vukovic and Soro (1992) who reviewed ten methodologies to determine the hydraulic conductivity, and provide a disk with a computer program that could calculate the K_c from a particle size distribution curve (PSD curve).

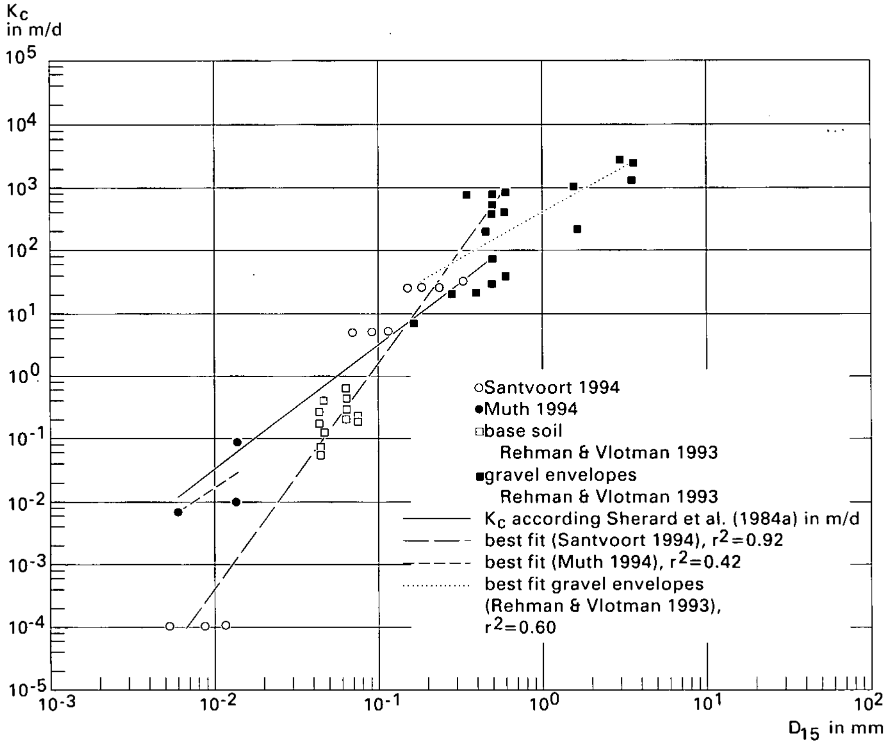


Figure 48 Hydraulic conductivity as function of characteristic particle sizes.

Remarks: Calculated hydraulic conductivity, K_c, in m/d using all the data available and D_{15} or d_{15} as characteristic particle size:

1. Sherard *et al.* 1984a;
2. Santvoort (1994): 4 soil ranges each having low, medium and high D_{xx} with same K: 12 data points;
3. Muth (1994): 3 data points;
4. Rehman and Vlotman (1993): 14 permeameter tests each replicated three times, K_{avg} of 3 reps. (NB. tests G46 – 51 of the original set were not considered in this analysis);
5. Rehman and Vlotman (1993): 23 permeameter tests each replicated three times, K_{avg} of 3 reps. (K7-9), blended and non-blended gravel (both river run and crushed rock material).

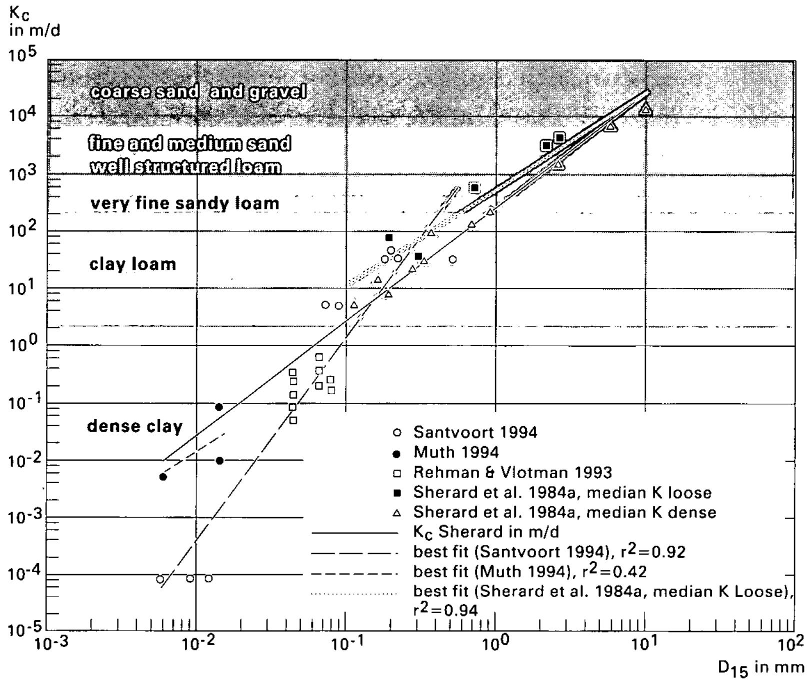


Figure 49 Calculated hydraulic conductivity (K_{cc}) as function of characteristic particle size.

Remarks: Calculated hydraulic conductivity, K_{cc}, in m/d using all the data available and D_{15} or d_{15} as characteristic particle size:

1. Sherard *et al.* 1984a: 15 soils replicated 6 times each;
2. Santvoort (1994): 4 soil ranges each having low, medium and high D_{xx} with same K: 12 data points;
3. Muth (1994): 3 data points;
4. Rehman and Vlotman (1993): 14 permeameter tests each replicated three times, K_{avg} of 3 reps. (tests G46 – 51 of the original set were not considered in this analysis).

5.5 Laboratory and field tests for granular material and ground water

Assessment of granular material, whether base soil or envelope material, is essential for the determination of the need for a drain envelope, and a proper drain envelope design. A number of standard tests need to be performed. These are the particle size distribution of the base soil and granular envelope, and tests to determine the density, porosity, liquid limit, plasticity index, hydraulic conductivity, electrical conductivity (salinity) and other chemical indices. Plasticity Index (PI), for instance, is used for assessing the need for an envelope. The density is required to help explain differences amongst replications and to compare field densities with laboratory densities. Chemical analysis of the soils and granular materials are important in the laboratory for tests (sodic soils and fresh water could cause the sample to become impermeable) and in the field to assess dangers of biological, iron and other forms of clogging. Moreover, some specifications put restrictions on calcium content (USBR 1989) in the envelope material and it is good to know, whether materials considered for drain envelope contain these compounds (N.B. see also Box 16).

5.5.1 Gradation analysis and classification; particle size distribution

Gradation analysis or particle size analysis is a measurement of the size distribution, or particle size distribution (PSD) of individual particles of a soil or gravel sample. This requires:
- destruction or dispersion of soil aggregates into discrete units by chemical (with sodium hexametaphosphate for instance);
- mechanical (washing and pounding), or ultrasonic means and the separation of particles according to size limits by sieving (up to US Standard sieve no. 200 or 400, particle sizes up to 0.074 - 0.038 mm, Table 15); and
- sedimentation tests of the materials passing standard sieve no. 200 or 400 (hydrometer or pipette method for silt and clay particles).

Detailed procedures for analyses can be found in standard soil analysis texts (i.e. Gee and Bauder 1986, or USBR 1974) or laboratory manuals (Dierickx and Vlotman, 1995). Depending on the fineness of the material the sedimentation analysis may be done first or last; in either case the same result will be achieved when proper procedures are followed. Sieving first has the advantage that most of the indicator particle sizes at 90, 85, 60, 50, 30, 15, 10 and 5 percent material passing become quickly available. For the design of synthetic drain envelope materials (the methods suggested in, Section 3.5.2 - 4) only the d_{90} is required. Vlotman and Omara (1996) showed that it is not necessary to perform sedimentation analysis, since acceptable synthetic envelope materials should not have openings less than 100 µm. Therefore the d_{90} values of interest are equal to or greater than 100 µm. By using the standard sieve set with the smallest sieve size of 0.074 mm, you will get all the data needed.

Mechanical analysis is the standard test for determining the texture of a particular soil or other granular material. A dry weighed sample of the material is placed in a set of precision-screened sieves with successively finer opening sizes. The sieves are placed in a standard shaking machine until the material finer than the screen size has passed through the individual screens (shake for 5 min at amplitude of 0.75 mm, with a frequency 50-60 Hz). A sieve analysis using one of the standard sieve sets (Table 15) will yield the primary information needed for the design of drain envelopes. The data thus obtained should be displayed in three types of graphs:

1. the percentage sand, clay and silt for evaluation in a soil texture triangle (Figure 50A);
2. the cumulative percentage passing certain sieves in a semi-logarithmic gradation curve (or PSD curve, Figure 50B); and
3. the weight of the material on each sieve in a histogram (Figure 51).

The purpose of the soil texture triangle is to give a uniform name to the soil type for ease of comparison. The triangle can also be used for the first assessment of whether an envelope material is needed using the percentage of clay as indicator.

The semi-logarithmic plot (particle size distribution curve, or PSD curve) is the most common graph for displaying the soil characteristics and is used to determine certain characteristics that describe the soil. It is used in the determination of the drain envelope specification (both for granular and fabric envelopes). The curve enables certain particles to be chosen to represent significant values, such as particles just larger than one-fourth of the distribution (d_{25}) representing the first quartile, and particles just larger than the three-fourths of the distribution (d_{75}) for the third quartile. Quartile measures are confined to the central part of the PSD curve, and the values obtained are not influenced by extreme particles, either very large or very small. Quartile measures (d_{25}, d_{50}, d_{75}) are readily determined from the PSD curve. A very steep curve indicates that the material is poorly or uniformly graded; one particular particle range is dominant. This should not be confused with a uniform distribution of particles over the various sieving ranges ('same weight on each sieve' in Figure 50), which generally indicates a well-graded soil. A soil uniformly distributed over the sieves does not show up as a straight line because the opening size interval between sieves is not the same (Table 17). When the particle weights are proportionally distributed the curve becomes a straight line ('weight proportionally' in the figures, i.e. Figure 51). If the curve has a smooth S-shape, it is called well graded or naturally graded (river run material in Figure 50B), provided it has some material of all of the various particle size ranges.

Another form of a poorly graded soil is the gap-graded soil that lacks certain particle ranges, which in theory will show up on the graph as a horizontal or near horizontal section of the PSD curve. This, however, will not be the case if successive sieves in the set chosen are too widely spaced. Also, the logarithmic scale reduces the length of horizontal sections, making them less obvious. The lack of certain particle sizes, which remained hidden in the cumulative curves, was one of the main reasons that caused failure of crushed rock envelope material in Pakistan (Bhatti and Vlotman 1990), although the successive sieves selected were not too widely spaced. (More will be mentioned about this later.) Depending on its origin, the classification of soils as gravel, sand, silt and clay is slightly different (USDA, USC, AASHTO, SRWSC).

The histogram was found useful in assessing missing particle sizes in both the soil and, more importantly, in the granular envelope material, provided the full set of sieves was used (Table 15). The histogram, or frequency pyramid, is a graphical presentation of the number, weight, volume or percentage of items in given class intervals, usually in geometric progression, along the x-axis. The y-axis represents either actual concentration or percentage (by number, volume or weight) of the total sample contained in each class interval (particle size range). It is important to realise that the class intervals are not equally spaced and that values of some of the typical statistical and probability theory parameters may not seem (and are not!) quite right.

It is critical that during the initial project stages, when base soil analyses are performed and materials for use as drain envelope are selected, all sieves indicated in a particular set are used rather than just seven or nine sieves deemed necessary to produce a semi-logarithmic gradation curve. For quality control during construction, which comes after more intensive investigation and testing of potential envelope materials, one can resort to using 7 to 9 sieves only.

To qualify certain characteristics of the particle size distribution it is common practice to express the particle size for which a certain percentage is smaller, i.e. d_{10}, d_{85}, d_{60}, d_{xx}, etc. The value represented by d_{xx} is the diameter of the particles for which xx %, on dry weight basis, has a smaller diameter (N.B. 'd' denotes the particle size of the base soil; for granular envelope material 'D' is used). For instance, notation d_{15} means that 15 % of the soil particles, by weight, are smaller than that particular diameter. The individual passing value percentages used in evaluations are:

- 5% (d_5) to limit the number of fines in granular envelope materials;
- 10% (d_{10}) for determination of the coefficients of uniformity and curvature, and as indicator of pore size in an empirical formula for the hydraulic conductivity;
- 15% (d_{15}) as an indicator of the fineness of the material and for empirical formulas determining the saturated hydraulic conductivity of the soil;
- 25% (d_{25}) a quartile measure of the fine particles. It excludes extremely small values. Generally not used with envelope design;
- 30% (d_{30}) in formula to calculate the coefficient of curvature;
- 50% (d_{50}) as an indicator of the median particle size. Also one of the quartile measures;
- 60% (d_{60}) for determination of the coefficients of uniformity and curvature;
- 75% (d_{75}) a quartile measure of the fine particles. It excludes extremely large values. Generally not used with envelope design;
- 85% (d_{85}) for an indication of coarseness of the material;
- 90% and 95% (d_{90} and d_{95}) are generally used in relation to the characteristic opening size of geotextiles.

Table 15 Overview of various sieve sets used for gradation analysis.

British Standard 410-1962		US Standard (1924) and ASTM (E11-61) designation			US Tyler (1910)		IMM (1907)		German Standard (DIN 1171-1926)		
Mesh No.	Sieve aperture (mm)	US Standard Mesh	ASTM designation (microns)	Sieve aperture (mm)	Mesh No.	Sieve aperture (mm)	Mesh No.	Sieve aperture (mm)	Mesh (per cm)	Mesh (per cm^2)	Sieve aperture (mm)
					2.5	7.925					
					3.0	6.680					
		3.5	5,660	5.66	3.5	5.613					
		4.0	4,760	4.76	4.0	4.699					
		5.0	4,000	4.00	5.0	3.962					
5	3.35	6.0	3,360	3.36	6.0	3.327					
6	2.80	7.0	2,830	2.83	7.0	2.794					
							5	2.540			
7	2.40	8.0	2,380	2.38	8.0	2.362					
8	2.00	**10.0**	2,000	2.00	9.0	1.981					
10	1.68	**12.0**	1,680	1.68	10.0	1.651					
							8	1.574			
									4	16	1.50
12	1.40	**14**	1,410	1.41	12	1.397					
							10	1.270			
14	1.20	**16**	1,190	1.19	14	1.168			5	25	1.20
							12	1.056	6	36	1.04
16	1.00	**18**	1,000	1.00	16	0.991					
18	0.850	**20**	841	0.841	20	0.833					
							16	0.792	8	64	0.75
22	0.710	**25**	707	0.707	24	0.701					
							20	0.635			
25	0.600	**30**	595	0.595	28	0.589			10	100	0.60
									11	121	0.54
30	0.500	**35**	500	0.500	32	0.495			12	144	0.50
36	0.420	**40**	420	0.420	35	0.417	30	0.421	14	196	0.43
									16	256	0.38
44	0.355	**45**	354	0.354	42	0.351					
							40	0.317			
52	0.300	**50**	297	0.297	48	0.295			20	400	0.30
60	0.250	**60**	250	0.250	60	0.246	50	0.254	24	576	0.25
72	0.210	**70**	210	0.210	65	0.208	60	0.211			
					70	0.210			30	900	0.20
85	0.180	**80**	177	0.177	80	0.175	70	0.180			
100	0.150	**100**	149	0.149	100	0.147	80	0.157	40	1,600	0.15
							90	0.139			
120	0.125	**120**	125	0.125	115	0.124	100	0.127			
									50	2,500	0.12
150	0.105	**140**	105	0.105	150	0.104	120	0.107			
									60	3,600	0.102
170	0.090	**170**	88	0.088	170	0.089	150	0.084	70	4,900	0.088
200	0.075	**200**	74	0.074	200	0.074			80	6,400	0.075
240	0.063	230	63	0.063	250	0.061	200	0.063	100	10,000	0.060
300	0.053	270	53	0.053	270	0.053					
350	0.045	325	44	0.044	325	0.043					
		400	37	0.037	400	0.038					

Source: Veldhuijzen van Zanten 1986/Santvoort 1994, References of Standards are not given in original source.
Bold numbers are sieve designations used in ASTM D4751-95.

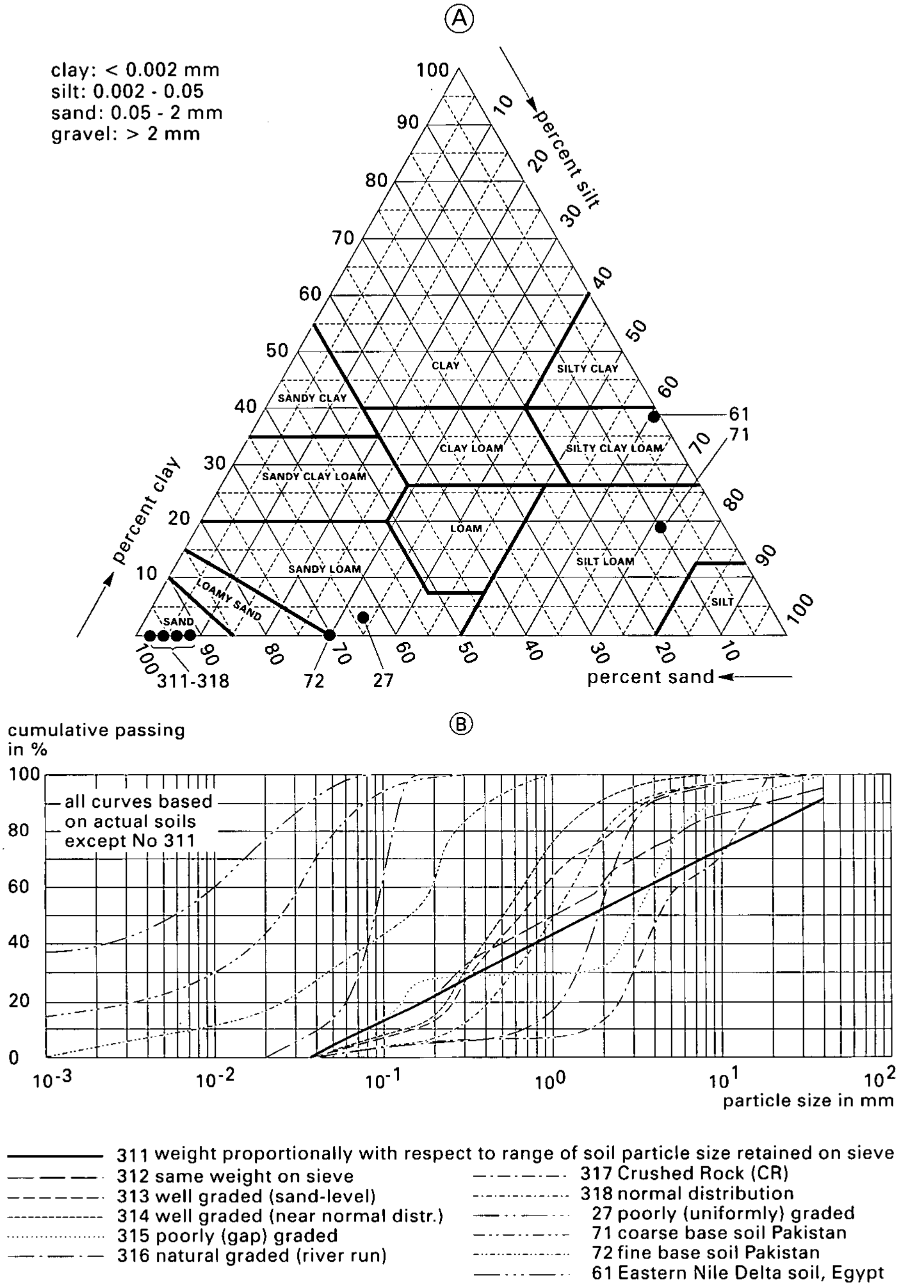


Figure 50 Presentation of selected base soils and gravel in texture triangle and the semi-logarithmic gradation plot.

A Texture triangle to classify soils.

B Semi-logarithmic particle size distribution plot.

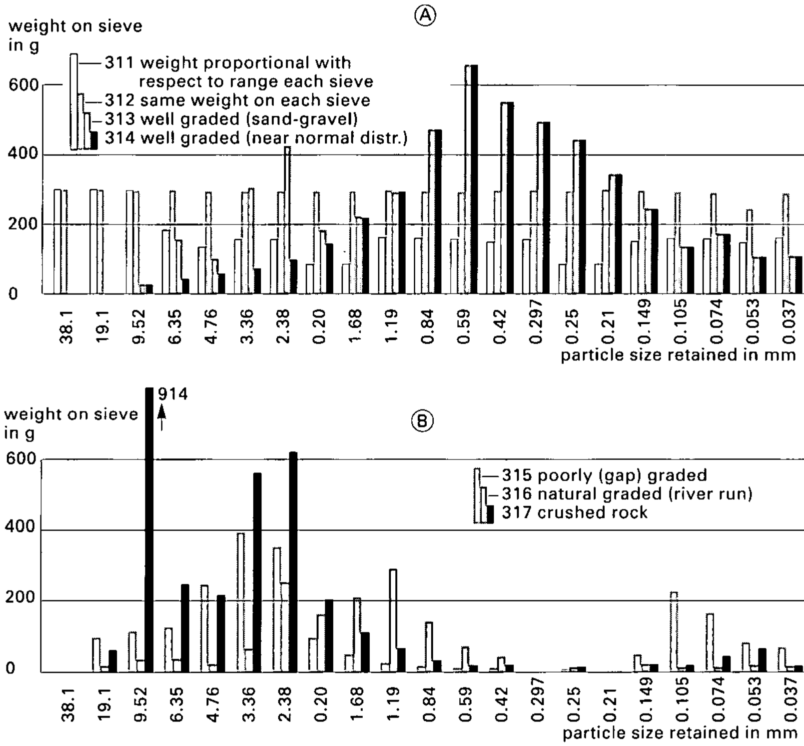


Figure 51 Example histograms of particle size distributions.
A Theoretical distributions.
B Poorly graded gravel samples.

The coefficient of uniformity (C_u) is an expression of the steepness of the middle portion of the semi-logarithmic gradation curve. Means and Parcher (1963) mention that Allan Hazen (during studies on sand filters during the 1890s at Pennsylvania State University) used two widely separated points on the gradation curve to represent the general slope of the grain size distribution. For this he used the 60 percent finer size and an 'effective grain size' that he determined was best represented by the 10% finer size.

Hence the coefficient of uniformity is:

$$C_u = \frac{d_{60}}{d_{10}} \qquad \text{Eq. 41}$$

The terms 'well-graded' and 'uniform' are frequently used in the literature to describe a certain desirable soil or gravel envelope property. A low uniformity coefficient indicates uniform (poorly-graded) material. It also means a relatively steep line when the gradation curve is plotted on standard semi-log paper (i.e. soil type 27 in Figure 50). Kruse (1962, in Willardson 1974) defines a soil as uniform when the coefficient of uniformity, C_u, is 1.78. A well-graded soil or envelope would have a less steep curve on the semi-log paper and this is usually taken to indicate that all particle sizes from coarse sand to clay and silt are present in the sample. Well-graded natural soils and envelope materials are highly stable and do not separate when handled in the field.

Giroud (1982) expressed some reservation as to the use of C_u to classify soils. To express the filter criteria, he believed that it is more appropriate to use only the slope of the central portion of the gradation curve, thus eliminating the coarsest and finest particles that have a negligible influence on stability of the soil structure in the filtration process. He drew a straight line as close as possible to the central portion of the curve and defined the linear coefficient of uniformity C'_u as:

$$C'_u = \frac{d'_{60}}{d'_{10}} \qquad \text{Eq. 42}$$

where, d'_{60} and d'_{10} are read from the graph where the straight line crosses the particle sizes which are 60% and 10% smaller, respectively. Alternatively, because of the logarithmic scale the C'_u may be determined from:

$$C'_u = \frac{d'_{60}}{d'_{10}} = \frac{d'_{50}}{d'_{0}} = \frac{d'_{90}}{d'_{40}} = \frac{d'_{100}}{d'_{10}} = \sqrt{\frac{d'_{100}}{d'_{0}}} \qquad \text{Eq. 43}$$

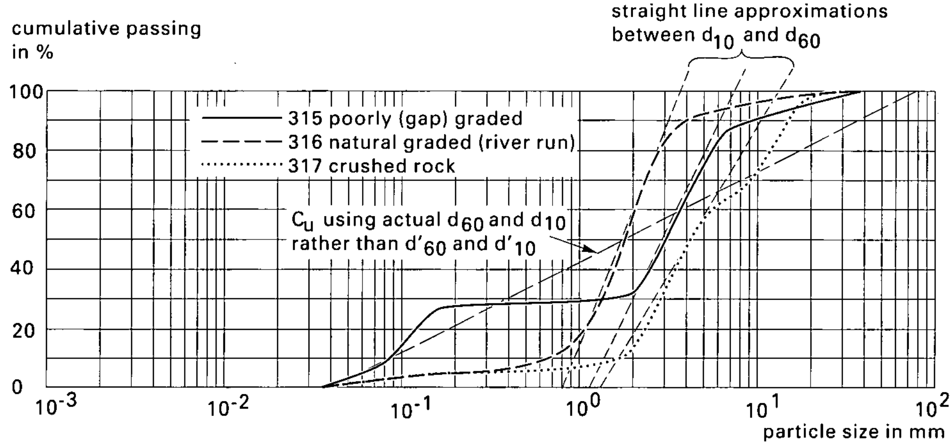
It is easier to read d'_{100} and d'_0 than a value somewhere in the middle of the curve. In the case of gap-graded soils (Figure 52), i.e. soils lacking certain particle sizes, it is not possible to draw a meaningful straight line. In this case one should consider only the finer portion of the particle size distribution curves of the soil and envelope materials. A similar observation was also made by Sherard *et al.* (1984a, 1984b). They suggested using d_{15} for the determination of the saturated hydraulic conductivity; only the material smaller than 4.76 mm (i.e. < US sieve no. 4) should be considered in determining a d_{15}. Continuing the analysis of C_u Giroud observed that at $C_u = 1$ the soil has uniform particle size distribution and the void space is large even after compaction. At $C_u = 3$ it was shown theoretically and experimentally (Horsfield 1934 in Giroud 1982), that the highest density will be obtained if appropriate compaction is applied (i.e. at optimum soil moisture content according to road construction procedures). The stability of the soil or gravel structure can be related to its density index I_D (relative density):

$$I_D = \frac{e_{max} - e_v}{e_{max} - e_{min}} * 100\% \qquad \text{Eq. 44}$$

where,

e_v is the void ratio of the soil in place (see Eq. 33 Section 5.3);
e_{max} is void ratio in its loosest; and
e_{min} densest state.

C'_u and I_D govern the interlocking of soil particles and Giroud used these two parameters for drawing up his design criteria for geotextiles[28].



PARTICLE SIZE IN MM

	d_{60} in mm	d_{10} in mm	C_u	d'_{60} in mm	d'_{10} in mm	C'_u
gap graded	3.821	0.084	45.5	3.821	1.4	3.8
river run	2.041	0.67	5.5	2.041	1	1.5
crushed rock	5.753	1.709	3.4	5.2	1.8	3.1

Figure 52 Linear coefficient of uniformity.

In order to further define the shape of the gradation curve, the USBR (1978) also uses a coefficient of curvature (which Means and Parcher (1963) called the coefficient of gradation):

$$C_c = \frac{d_{30}^2}{d_{10} * d_{60}} \qquad \text{Eq. 45}$$

[28] (see Section 6.2.4, N.B. Eq. 44 is used to define the loose, medium dense and dense soil in Table 35).

The origin of the coefficient of curvature is not clear as all references consulted refer to it as a given[29]. However, perhaps the clearest hint to its origin is given by Richardson (date unknown) who remarks that $C_c = 1$ for a normally distributed soil. This was investigated by synthesising a number of soils that could be referred to as normally distributed (Table 16). Because of a number of reasons that will be explained later after describing the sieve sets commonly used in detail, this could not be confirmed very conclusively.

Table 16 Weight and characteristics of material on each sieve of soils shown in Figure 50.

Description	Weight proportionally with respect to range on each sieve	Same weight on each sieve	Well graded (sand-gravel)	Well graded (near normal	Poorly (gap) graded	Natural graded (river run)	Crushed rock	Normal distr.	Poorly (uniformly graded	Coarse base soil Pakistan	Fine base soil Pakistan	Eastern Nile Delta soil,
Soil ID no:	311	312	313	314	315	316	317	318	27	71	72	61
						Particle size in mm						
d_5	0.033	0.033	0.085	0.079	0.040	0.188	0.189	0.176	0.031	-	0.004	-
d_{10}	0.080	0.077	0.156	0.133	0.084	0.370	1.709	0.260	0.045	0.001	0.008	-
d_{15}	0.118	0.112	0.216	0.194	0.114	0.867	2.114	0.288	0.053	0.001	0.018	-
d_{30}	0.370	0.264	0.315	0.280	1.446	1.337	2.926	0.599	0.072	0.010	0.074	0.001
d_{50}	1.677	1.015	0.632	0.506	3.138	1.809	4.285	1.015	0.090	0.025	0.140	0.006
d_{60}	3.632	1.872	0.836	0.663	3.821	2.041	5.753	1.290	0.100	0.032	0.186	0.010
d_{85}	25.619	9.044	2.809	1.571	6.809	3.213	14.608	2.299	0.120	0.069	0.310	0.028
d_{90}	36.113	17.200	3.499	1.926	9.482	4.107	16.299	3.151	0.125	0.088	0.540	0.036
d_{95}	46.606	37.150	4.738	2.711	18.426	7.853	17.990	5.646	0.130	0.300	1.000	0.085
C_u	45.31	24.28	5.37	5.00	45.54	5.52	3.37	4.96	2.22	44.55	23.25	37.00
C_c	0.47	0.48	0.76	0.89	6.52	2.37	0.87	1.07	1.15	4.40	3.69	0.24
Stdev95 (Eq. 46)	27.40	22.03	2.50	1.34	9.32	3.69	8.36	2.82	0.02	0.17	0.52	0.05
Stdev5	1.00	0.60	0.33	0.26	1.89	0.99	2.50	0.51	0.04	0.01	0.07	0.003
G (Eq. 47)	205.61	170.90	13.20	8.53	81.93	5.42	5.96	10.93	1.89	106.38	28.88	107.18

USBR (1974) specifies for a soil to be well graded, the coefficient of uniformity must be greater than 4 for gravel and greater than 6 for sand. At the same time, the coefficient of curvature must be between 1 and 3 for both gravel and sand to be used as an envelope material. Soils will be graded poorly, if either the C_u or the C_c does not fall within these two ranges.

Basing gradation assessment on the assumption that soil particles are normally distributed, Pillsbury (1967) used what he termed the standard deviation of the envelope material as an index for design of a drain envelope:

[29] The oldest reference consulted was Winger and Ryan (1970a and b) who refer to the Earth Manual of USBR p 26 as the source of C_u and C_c. The first, tentative edition of the Earth Manual was in 1951. The first edition, rev. second printing was in 1968.

$$\sigma_e = \frac{D_{95} - D_{50}}{1.64} \qquad \text{Eq. 46}$$

where,

σ_e represents the standard deviation for envelope material; and
1.64 the value based on the principle of a normal distribution (Figure 53).

If the particles of soils were indeed normally distributed, then (D_{50}-D_5) could also be used in Eq. 46, and either standard deviation could also be read from a frequency distribution plot on log probability paper. However, as may be seen from the examples in Figure 51 and Table 16 that is rarely the case. Richardson (date unknown) remarked that both geologists and hydraulic engineers believe that the size distribution of natural sediments will plot as a straight line on log probability paper. If this is so, then the soil can be completely described by the median diameter (d_{50} or D_{50}) and the slope of the line on log probability paper. This linearity on log probability paper could not be confirmed using some of the specifically synthesized soils herein (Figure 55, i.e. soil numbers 311 – 318 and actual soils numbers 319 – 324). The slope of the line is proportional to the standard deviation. The gradation coefficient is:

$$G = 0.5\left(\frac{d_{95}}{d_{16}} + \frac{d_{84}}{d_{50}}\right) \qquad \text{Eq. 47}$$

where,

d_{16} represents the particle size at one standard deviation below the mean; and,
d_{84} the same above the mean.

Both the d_{16} and the d_{84} can be replaced without error, by the more commonly reported d_{15} and d_{85} respectively. This coefficient is the average slope of approximately the central portion of the PSD curve, assuming that the first and third quartiles have slightly different slopes which need to be combined to describe the average slope of the curve.

To further explain and understand the relationships between frequency and probability distribution requires some explanation of the terms used. When the number of sieves is sufficiently large the cumulative relative frequency of the distribution may be approximated by the cumulative probability (Ilaco staff 1981) function. The most frequently used probability distribution is the normal or Gaussian distribution. When the frequency distribution such as shown in Figure 53A is summarised cumulatively, the cumulative probability distribution or duration curve is obtained (Figure 53B). The probability is usually determined from tables that are based on:

$$G(x) = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{y} e^{-1/2\, y^2}\, d\, y \qquad \text{Eq. 48}$$

where,

$G(x)$ is the distribution or cumulative probability function, with $0 \le G(x) \le 1$ and $-\infty < x < \infty$;

x the particle size in mm;

y a transform determined from $y = (x - x_{avg}) / \sigma$;

σ standard deviation; and

x_{avg} D_{50} or d_{50}.

If the scale of the axis with the probability G(x) is calibrated according to Eq. 48 a straight line will result (Figure 53C) instead of the curved line of Figure 53B. This plot is the normal probability plot often used in hydrology. When the x values are then plotted logarithmically as originally introduced by Hazen, the log normal presentation of the probability is obtained. Some selected cumulative soil distribution curves are plotted on a log-normal paper

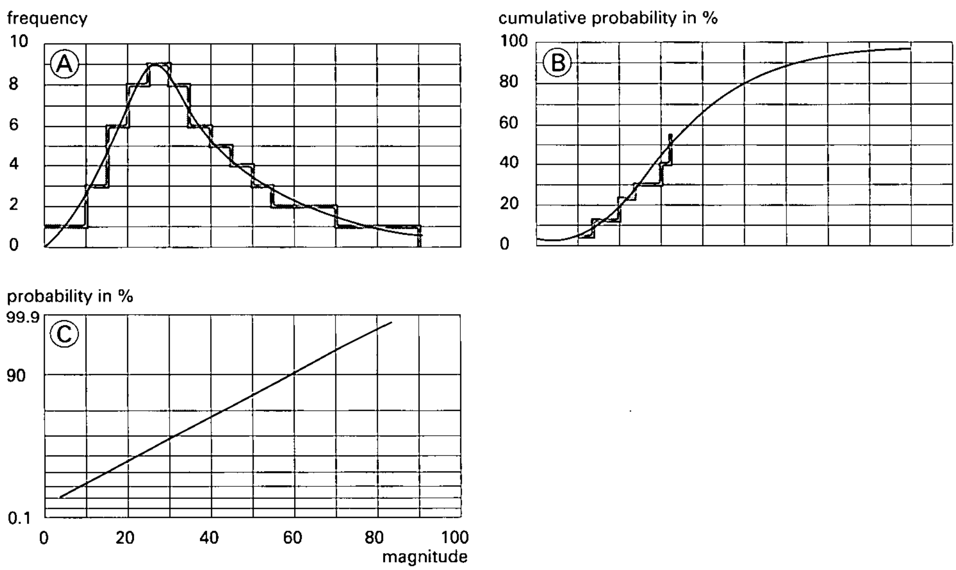


Figure 53 Normal distribution and probability.
A Example of frequency distribution.
B Same data as A displayed in cumulative probability curve.
C Probability curve of normally distributed data set.
D1 Normal distribution and relationship with particle size distribution.

designed by Hazen, Whipple and Fuller[30] (Figure 55). It was assumed that they used the regular probability scale paper based on Eq. 48. However the lack of straight lines, can either be caused by a slight difference in scale, or it may be because of the irregular sieve intervals (frequency analysis with a limited number of intervals requires equal interval size for results that comply with probability theory).

The C_c of a normally distributed sample is 1. The coefficient of curvature is therefore a measure of asymmetry of the sample. To determine normality and degree of deviation from the normal distribution the skew or the C_c can be used.

A sufficient large number of samples need to be used in order to improve the similarity between cumulative frequencies and probability curves. Furthermore, frequency intervals are generally required to be equally spaced. Both of these requirements or assumptions are somewhat questionable as may be clear from Table 16 and Figure 54.

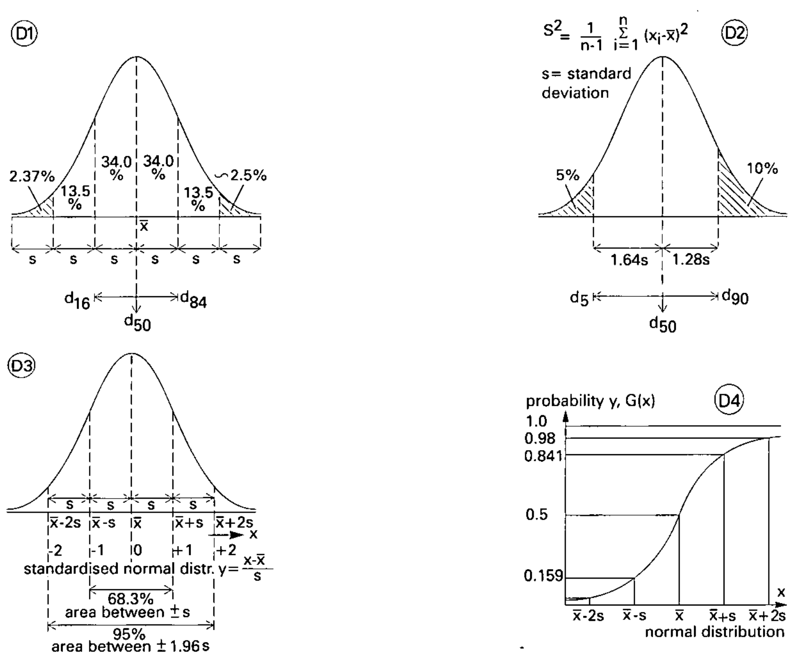


Figure 53 Cont. D1 Normal distribution and relationship with particle size distribution.
D2 Same as D1, but other PSD parameters identified.
D3 normal frequency.
D4 Probability curve of nornal distribution.

[30] The paper used was from Codex Book Co. Norwood, Mass. 02062, USA. No. 3128. Logarithmic Probability, designed by Hazen, Whipple and Fuller.

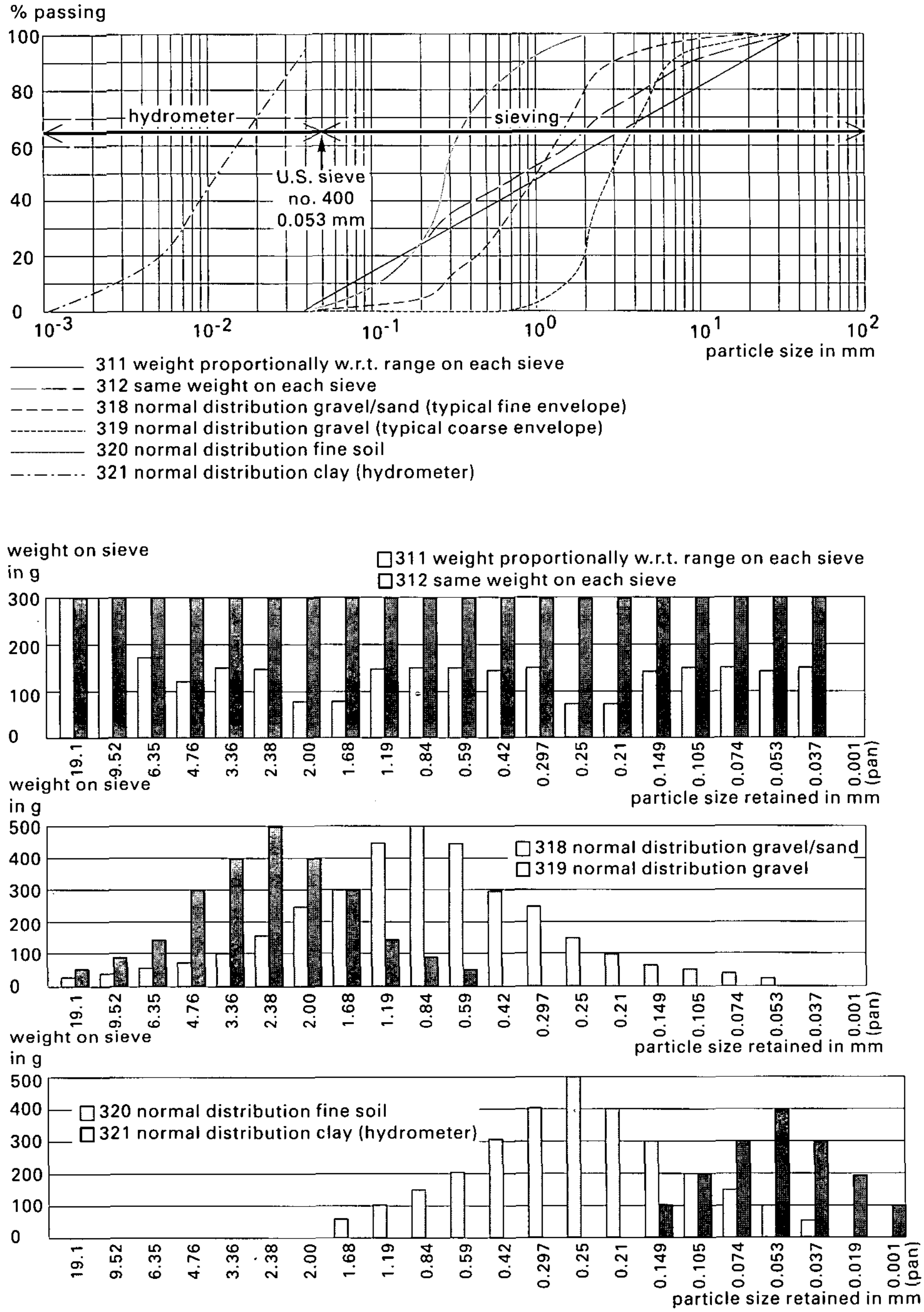


Figure 54 Synthesised normal and uniform soils.

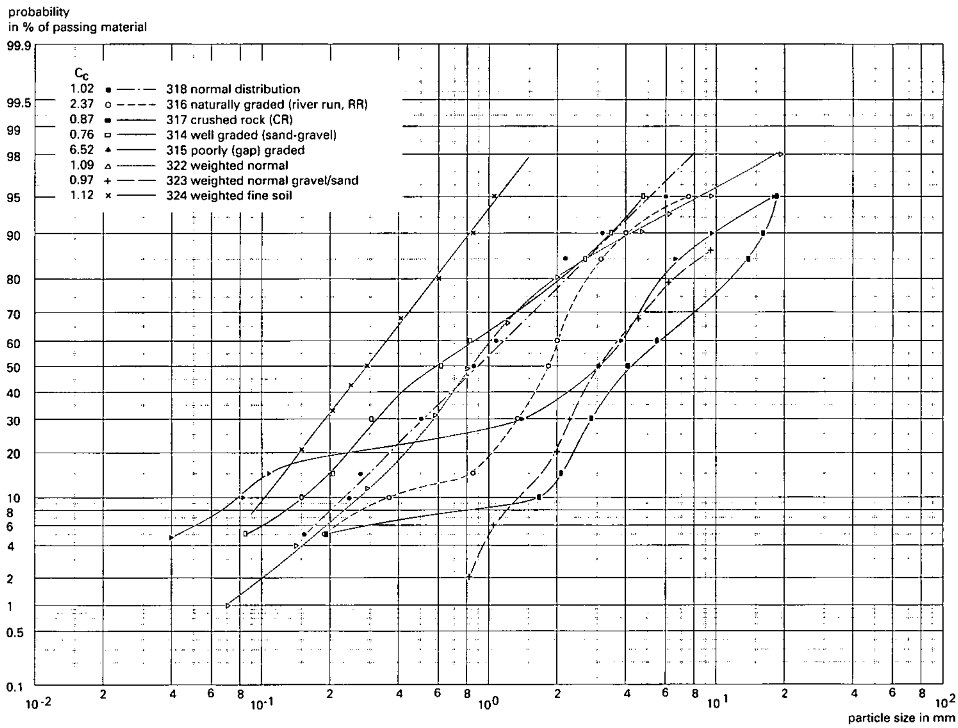


Figure 55 Examples of actual and generated soil PSD curves on log-probability paper.

In the laboratory one should always use the full set of sieves (21 sieves and the pan) for the initial assessment of the PSD. The first reason is to have the option to use formulae based on the normal distribution principle, and the second reason is to identify missing particle sizes. For instance, one of the reasons for failing granular envelope material at the Fourth Drainage Project in Pakistan was missing particles sizes in the crushed rock material used (on sieves no. 10 - 100, 2.0 - 0.149 mm). This was not identified by using the standard set of 7 sieves (Figure 56) when crushed rock and river run material for drain envelopes were compared (Vlotman *et al.,* 1990). Crushed rock material (blocky structure and no rounded particles) had a very high hydraulic conductivity and failed in the field to keep sediment out of the drainpipes. River run material, on the other hand, had much a lower hydraulic conductivity and worked well in the field. With the 7-sieve analysis there was no obvious difference in the two particle size histograms (Figure 56A), while more particle size ranges are identified with the 21-sieve analysis (Figure 56B). Histograms as in Figure 56 are essential in the assessment of potential envelope material, since a semi-log gradation curve does not reveal the missing particles size because the data is presented cumulatively.

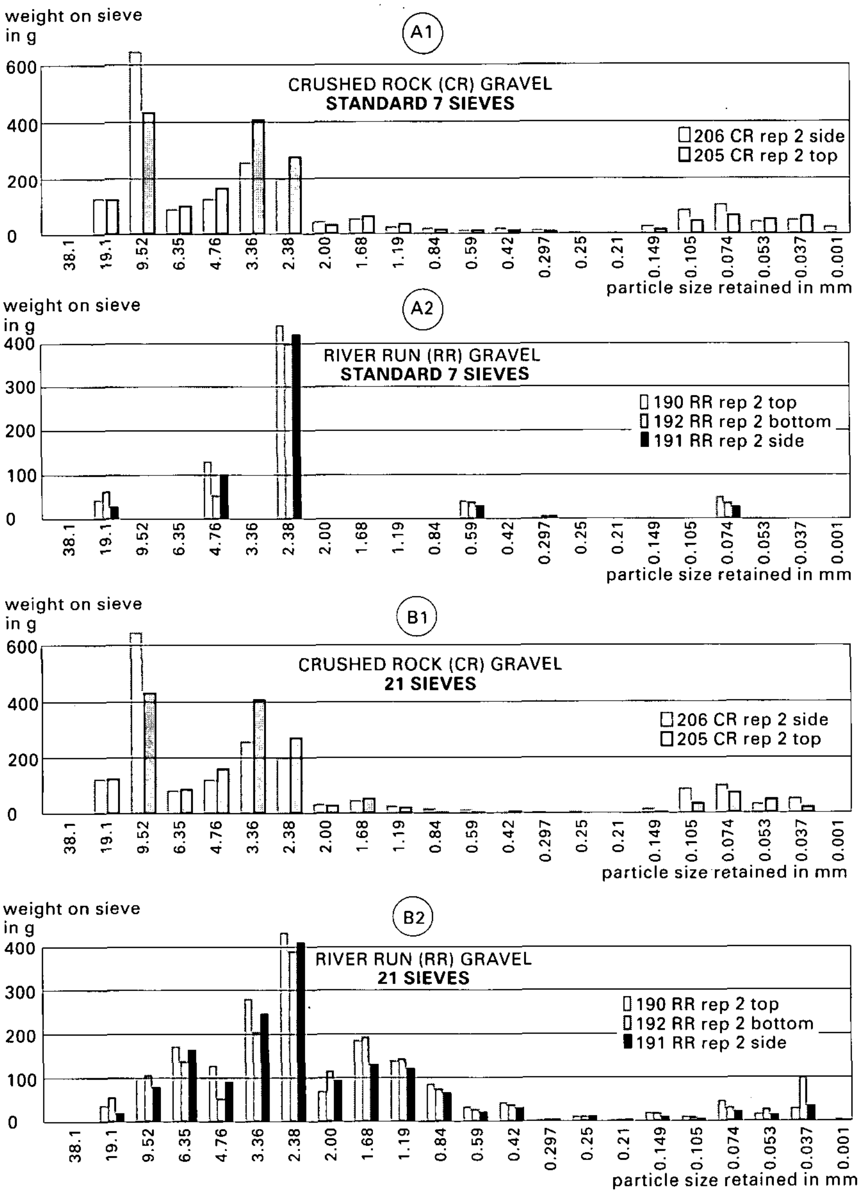


Figure 56 Crushed rock (CR, A1 and B1) and river run (RR, A2 and B2) material gradation histograms with 7 (A) and 21 sieves (B).

Remarks: Histograms based on gravel samples taken from the top, the side, and the bottom of the pipe at test site SIB6 (Fourth Drainage Project, Pakistan), where the two test lines were constructed at the same depth and 3 metres parallel to each other. Although some invasion of fines had taken place into both the river run (RR) and crushed rock (CR) samples, both were considered to be samples of actual gravel envelope in-situ. No bottom samples from CR could be obtained. For PSD see Figure 83.

Table 17 7 and 21 sieves used as well as particle ranges for sieve and sedimentation (hydrometer) analysis.

Serial No.	Sieve Size/No.	Sieve mm	Log difference between successive sieves	
			21 sieves	7 sieves
1	**1.5"**	**38.1**	-	-
2	**.75"**	**19.1**	0.300	**0.300**
3	3/8"	9.52	0.302	
4	3	6.35	0.176	
5	**4**	**4.76**	0.125	**0.603**
6	6	3.36	0.151	
7	**8**	**2.38**	0.150	**0.301**
8	10	2	0.076	
9	12	1.68	0.076	
10	16	1.19	0.150	
11	20	0.84	0.151	
12	**30**	**0.59**	0.153	**0.606**
13	40	0.42	0.148	
14	**50**	**0.297**	0.150	**0.298**
15	60	0.25	0.075	
16	70	0.21	0.076	
17	100	0.149	0.149	
18	140	0.105	0.152	
19	**200**	**0.074**	0.152	**0.604**
20	270	0.053	0.145	
21	400	0.037	0.156	0.301
22	**Pan**	**0.001**	1.568	1.568

Hydrometer

Time of sedimentation		Approx. mm	Log diff.	
1	1 min	0.0267	0.142	Log difference of total hydrometer range is 1.568
4	4 min	0.0154	0.239	
19	19 min	0.0081	0.279	
60	1 hr	0.0048	0.227	
435	7 hrs 15 min	0.0019	0.402	
1545	25 hrs 45 min	0.001	0.279	

Not all 21 sieves will fit at the same time on a sieving apparatus and a soil/gravel sample of approximately 500 grams should be weighed successively in three sets (Serial nos. 1 – 7, 8 – 14, 15 –21 in Table 17). The material left in the pan is sieved in the next set, until all sieves have been used. If the sieving apparatus has adjustable vertical amplitude, it should be set at 0.75 mm, with a frequency of 50 Hz. Depending on the weight of the sample being sieved the amplitude needs to be adjusted as outlined in the manual with the apparatus. Sieve the material for 5 minutes (different times are allowed, but make sure that always the same time is used, so results of different sieving analyses can be compared).

Conversion between particle sizes

On many occasions only a few of the required percentages of particle size passing are reported and it is desirable to convert a particular size to another. Recent examples of this can be found in the SCS 1994 guidelines, where d_{15}/d_{10} = 1.2 and C_u are used, and in a landmark study for the Federal Highway Administration (FHwA, Wilson-Fahmy *et al.* 1996). The study investigated 91 sites where synthetic materials were used with highway drainage. When an insufficient amount of soil was available d_{15}, d_{50}, d_{85}, and d_{90} were obtained from d_{10} and C_u that were always directly measured. The practice of size conversion is also used to convert between opening sizes of synthetic fabrics (Table 27). It will certainly be helpful when comparing various drain envelope retention and permeability criteria, and it could even be one of the steps to obtain the characteristic d_{10} or d_{15} for calculation of the hydraulic conductivity (Wilson-Fahmy *et al.* 1996). Giroud (1982) and Fisher *et al.* (1990) to convert typical granular filter criteria to synthetic envelope criteria also extensively used this methodology.

Reported ratios are given as well as ratios determined from the data presented on actual PSD curves in this book in Table 18. The C_u or C'_u can be used for sizes between the 10% and 80% passing but for ratios between 5 and 15% and between 80 and 100% the ratios given below are generally recommended. To be safe one can compare the ratios with some available PDS curves of the area and match the ratios determined from those with the one given in Table 18.

Table 18 Average ratios of selected particle sizes combinations.

	Soil type 1: actual soils					Soil type 2: actual soil boundaries				
Ratio	Average	Sdev	CV (%)	Median	Count	Average	Sdev	CV (%)	Median	Count
d_5/d_0	6.65	12.54	188	2.59	12	1.66	0.68	41	1.50	7
d_{10}/d_5	1.78	0.55	31	1.58	13	1.60	0.73	46	1.26	8
d_{15}/d_{10}	1.52	0.64	42	1.22	20	1.41	0.42	30	1.18	12
d_{30}/d_{15}	1.98	1.26	64	1.47	21	2.25	2.37	105	1.46	13
d_{50}/d_{30}	4.30	9.07	211	1.46	20	1.65	0.70	43	1.43	14
d_{50}/d_{10}	11.35	23.76	209	2.40	17	4.83	5.17	107	2.29	12
d_{60}/d_{10}	20.89	51.42	246	2.76	20	6.13	6.74	110	2.49	12
d_{60}/d_{50}	1.32	0.50	38	1.21	20	1.20	0.20	17	1.18	15
d_{85}/d_{60}	5.44	12.46	229	1.69	23	1.85	0.83	45	1.67	15
d_{90}/d_{85}	1.67	1.16	69	1.24	23	1.19	0.39	33	1.17	15
d_{95}/d_{90}	2.50	3.13	125	1.32	22	1.37	0.37	27	1.26	14
d_{100}/d_{90}	2.41	1.93	80	1.80	21	1.67	0.62	37	1.50	15

Based on actual data of soil presented throughout the book.
Soil types selected were: 1 - actual soil, 2 - actual soil boundary (not shown but also checked were: 3 - actual gravel, 4 - actual gravel boundary, 5 – synthesised soil, 6 - laboratory soil).

Concluding remarks

The foregoing made it perfectly clear that there is a lot more to gradation analysis than simply calculating the C_u or the C_c and reporting on the particle sizes with certain percentage passing. Several conclusions can be drawn as follows:

1. To truly represent the particles sizes it is essential that all sieves of the selected set (Table 15) are used during first assessment of base soil and potential granular material for the envelope. This will help identify missing particle ranges, and it will accommodate using assessments based on frequency/probability analysis.
2. Histograms showing the weight (or percentage) of the material retained on certain sieve sizes should be produced as a matter of course. These are useful to identify missing particle size ranges and the degree of normality of the particle size distribution, only when a sufficient number of sieves are used (full set preferably, no minimum number is recommended).
3. The linear coefficient of uniformity best describes the quality of gradation, i.e., uniform, well graded, etc.
4. Sub-sets of PSD analysis need to be determined if D_{xx} or d_{xx} values are used for calculating hydraulic conductivity. Use results of sieves with particle size less than 4.76 mm only. Report values using symbols such as D_{sxx} and d_{sxx} to distinguish from the results obtained from full sets.
5. There is not enough evidence based on the soil PSD available to the authors of this book to support the notion that naturally-graded soils have normal particle size distributions, and that formulas derived from probability theory are useful in describing soils distinctly and uniquely.

5.5.2 Plasticity index

The Plasticity Index (PI) is a measure of the plasticity of a soil. It is defined as the difference in water content expressed as a percentage of the weight of oven-dried soil, of a soil at liquid limit and at plastic limit.

$$PI = \theta_{LL} - \theta_{PL} \qquad \text{Eq. 49}$$

where,

PI represents the Plasticity Index (%);

θ_{LL} the water content of the soil at the liquid limit (%); and

θ_{PL} water content of the soil at the plastic limit (%).

The Plasticity Index requires the determination of the liquid limit and the plastic limit. The liquid limit of a soil is the water content, expressed as a percentage of the weight of oven-dried soil, at the boundary between the liquid and plastic states. The liquid limit of a soil is the water content at which two halves of a soil cake will flow together for a distance of 12.5 mm along the bottom of the groove separating the two halves, when the cup is dropped 25 times for a distance of 1 cm at a rate of two drops per second:

$$\Theta_{LL} = 100 \frac{W_{LL} - W_{DS}}{W_{DS}} \qquad \text{Eq. 50}$$

where,
W_{LL} soil mass at liquid limit in g;
W_{DS} mass of the oven-dried soil in g.

The plastic limit of a soil is the water content at the boundary between the plastic and semi-solid states, expressed as a percentage of the weight of oven-dried soil. The water content at this boundary is arbitrarily defined as the lowest water content at which the soil can be rolled into a thread of 3.2 mm (1/8 in) diameter without the thread breaking into pieces.

$$\Theta_{PL} = 100 \frac{W_{PL} - W_{DS}}{W_{DS}} \qquad \text{Eq. 51}$$

where,
W_{PL} soil mass at plastic limit in g.

The Plasticity Index (PI) represents the range of moisture content within which the soil exhibits the properties of a plastic solid. Also, it is a measure of the cohesive properties of the soil and indicates the degree of surface chemical activity and the bonding properties of the fine clay and colloidal fraction of the material. Hence soils can be characterised by their Plasticity Index (Dierickx and Vlotman, 1995):

For sandy soils PI will always be < 15
For silty soils PI will be between 5 and 25
For clayey soils PI will always be > 15

Some typical values of PI reported in the literature of soils used in the laboratory, or encountered in the field where drain envelopes have been used, are given in Table 19.

If the numerical values of the plastic and liquid limit are the same, then the Plasticity Index equals zero. Such a soil is definitely plastic in nature, although the range of moisture content within which it exhibits the properties of a plastic solid is so small that this range cannot be measured. Soils with a zero Plasticity Index should not be confused with non-plastic soils.

Non-cohesive soils, such as sand, which are relatively free from clayey material change rather abruptly from the viscous liquid state to a dry incoherent granular material which does not form into clods. It is not possible to roll a material of this kind into a thread as small as 3 mm in diameter. Therefore, a plastic limit cannot be determined. Such a soil is said to be non-plastic and in test reports it is designated as NP (Figure 74).

Table 19 Plasticity index of selected soils and other soil characteristics.

Soils no. and description of soil	PI (%)	C_u	C_c	K_s cm/s	Clay (%)	Silt (%)	Sand (%)	Reference
22 and 23, Marine clay, Malaysia, soils tested with geotextiles; no change in O_{90} over time.	60-80			$1*10^{-7}$	40-50	36-60	5-10	Loke *et al.* (1994)
15, Lab test soil A, silt, slightly sandy	5	4	2.25	$1.1*10^{-5}$	2.5	80	17.5	Muth (1994)
16, Lab test soil B, silt, sandy	7	5.4	2.6	$8.0*10^{-6}$	5	68	27	Muth (1994)
17, Lab test soil C, silt, very sandy	2	10	1	$1.0*10^{-4}$	0	55	45	Muth (1994)
no ID, Lab soil	3	85	10	-	12	32	56	Dierickx (1996)
61, Eastern Nile Delta soils	36.5±8.5	75	0.8					Metzger *et al.* (1992)
62, Nile Delta soils in need of envelope	27.9±10.8	75	0.8					Metzger *et al.* (1992)

5.5.3 Hydrometer analysis

The hydrometer analysis relies on the relationship between the settling velocity and particle diameter. When a soil sample is prepared according to Section 5.5.1 (p 161), mixed with water and poured into the hydrometer cylinder, readings at certain time intervals are read from the stem of the hydrometer, according to standard procedures. The settling velocity is related to the diameter of a spherical particle. The force acting downward on each particle due to its weight in water is:

$$F_{down} = \frac{4}{3\pi \dfrac{d^3}{8} (\rho_p - \rho_l)\, g} \qquad \text{Eq. 52}$$

where,

F_{down} represents the downward force on particle in N;
d the particle diameter in m;
ρ_p particle density in kg/m^3;
ρ_l liquid density in kg/m^3; and
g acceleration due to gravity (9.81 m/s^2).

Because of the viscous resistance of the water, the opposing upward force is:

$$F_{up} = 3 \pi d \eta \nu \qquad \text{Eq. 53}$$

where,
F_{up} is the upward force in N;
η dynamic fluid viscosity in kg/m s; and
ν velocity of fall in m/s.

The resisting force is zero where velocity, v, is zero at time t = 0, and it increases with increasing v until it is equal to the downward force. For particles that are settling in a dilute dispersing solution, it can be shown that the terminal velocity for silt and clay-size particles is reached in a relatively short time (a few seconds). Equating F_{down} and F_{up} relates the terminal velocity to the particle diameter as follows:

$$\nu = \frac{g(\rho_p - \rho_l) d^2}{18\eta} \qquad \text{Eq. 54}$$

A form of this relationship was first developed by Stokes (1851) and is now known as Stokes' Law. Basic assumptions used in applying Stokes' Law to soil suspensions in which soil particles are settling are as follows:

1. Terminal velocity is attained as soon as settling begins.
2. Settling and resistance are entirely due to the viscosity of the fluid.
3. Particles are smooth and spherical.
4. There is no interaction between individual particles in the solution.

In mineralogical analysis there is often a need to separate various clay fractions for specific analysis. The removal of the clay fraction by sedimentation can be accomplished by homogenising a soil suspension and decanting all that remains above a distance H from the top after time t, where,

$$t = \frac{18\eta H}{g(\rho_p - \rho_l) d^2} \qquad \text{Eq. 55}$$

Quantitative separation by decanting requires that the residue be re-suspended and decanted repeatedly to salvage those particles that were not previously at the top of the suspension at the start of the sedimentation period. The hydrometer method, depends fundamentally upon Stokes' Law, which for the hydrometer may be written as:

$$d = \left(\frac{\theta}{t}\right)^{1/2} \qquad \text{Eq. 56}$$

where,

θ is the sedimentation parameter

From Eq. 55 it follows that:

$$d = \left(\frac{18\eta H}{g\,(\rho_p - \rho_l)}\right)^{1/2} t^{-1/2} \qquad \text{Eq. 57}$$

Hence, the sedimentation parameter is a function of the hydrometer settling depth, solution viscosity, and particle and solution density:

$$\theta = \frac{18\eta H}{g\,(\rho_p - \rho_l)} \qquad \text{Eq. 58}$$

where,

H represents the hydrometer settling depth.

The hydrometer settling depth H is a measure of the effective depth of settlement for particles with diameter d. It can be related to the hydrometer stem reading L_1 by considering the specific design and shape of the hydrometer (Kaddah, 1974; ASTM D 422-63). The relationship of the settling depth to the hydrometer dimensions can be approximated by:

$$H = L_1 + 0.5\left(L_2 - \frac{V_B}{A}\right) \qquad \text{Eq. 59}$$

where,

L_1 is the distance along the stem of the hydrometer from the top of the bulb to the mark for a hydrometer reading in cm;

L_2 the overall length of the hydrometer bulb in cm;

V_B volume of hydrometer bulb in cm^3; and

A cross sectional area of the sedimentation cylinder in cm^2.

This stem reading L_1 is a measure of the soil solution concentration R. For the ASTM 152H hydrometer and a standard sedimentation cylinder, L_1 = 10.5 cm for a soil solution concentration R = 0 g/l and 2.3 cm for R = 50 g/l; L_2 = 14.0 cm; V_B = 67.0 cm^3; and A = 27.8 cm^2. Substitution of these values into Eq. 59 and solving in terms of R yields:

$$R = \frac{16.3 - H}{0.164} \qquad \text{Eq. 60}$$

where R is the uncorrected soil solution concentration. ASTM 152H hydrometers are calibrated at 20º C directly in terms of soil solution concentration, expressed as grams of soil per litre of solution (ASTM D 422-63). Correction of hydrometer readings for other temperatures and for solution viscosity and density effects is made by taking a hydrometer reading, R_L, in a blank (no soil) solution. This reading should be taken immediately after the uncorrected reading, R, is taken. The corrected concentration of soil, C, in suspension at any given time is:

$$C = R - R_L \qquad \text{Eq. 61}$$

where,
C is the corrected concentration in g/l.

If the test is started with a uniformly dispersed suspension of fine grains in a liquid, in time t any grain of a certain diameter d will have settled a distance H below the original position. At a distance H from the top, the suspension will consist of grains smaller than d only, and each size that is present will occur in the same degree of concentration as in the original mixture. If the viscosity and the specific mass of the suspending fluid and the specific mass of the solid particles are known, the diameter of the largest grain in suspension at depth H after time t can be determined from Eq. 57.

A difference in particle density for different soils affect particle settlement times and hence requires the correction of hydrometer readings and sedimentation parameter values. However, Gee and Bauder (1979) and ASTM D 422-63) show that moderate changes in particle density have only small effects on a given size determination. For example, errors in particle density of approximately 0.1 g/cm^3 result in errors of less than about 0.5 weight % clay for soils with clay contents up to 50 weight %.

Flocculation of clay by soluble salt or gypsum during sedimentation may cause significant errors in the hydrometer method, since no pre-treatment is used. Kaddah (1975) recommends increasing the concentration of Hexametaphosphate (HMP) to levels high enough to maintain dispersion. If higher concentrations are used, the blank solution must contain the same concentration of HMP as that used in the soil solution, so that the blank reading R_L, corrects for the increased solution viscosity and density. If soil is high in soluble salts or gypsum, pre-treatment procedures, removal techniques (Rivers *et al.*, 1982), or chemical treatment (Hesse, 1976) may be needed.

Gee and Bauder (1986) made a detailed error analysis for the hydrometer. They indicate that the major source of error is in the hydrometer reading. An error of about 1 g/l hydrometer reading results in an error of about 2 % weight for clay-size particles.

5.5.4 Saturated hydraulic conductivity

The hydraulic conductivity of a soil is a measure of its ability to transmit water. The hydraulic conductivity depends on the size and geometry of the pores. The texture and structure of the soil are the principal determinants of the hydraulic conductivity. For saturated soil the hydraulic conductivity is constant while for unsaturated soil the conductivity decreases with decreasing water content.

Constant-head method for determination of hydraulic conductivity
The hydraulic conductivity can be carried out in situ or in the laboratory. In the laboratory hydraulic conductivity measurements can be done on disturbed or undisturbed soil samples. Measurements of the hydraulic conductivity of saturated soils in the laboratory are based on the direct application of Darcy's law (see Eq. 18, Section 5.1.3) to a saturated soil column of uniform cross-sectional area. Imposing a hydraulic head difference on a soil column and measuring the resulting water flux, the hydraulic conductivity can be calculated:

$$K_s = \frac{V}{tA}\frac{L}{H} = \frac{Q}{A}\frac{L}{H} = \frac{Q}{Ai} = \frac{v}{i} \qquad \text{Eq. 62}$$

where,
- v is the filter velocity in m/s;
- Q the water flux in m^3/s;
- A cross-sectional area in m^2;
- V volume of water in m^3;
- t time during which V is measured in s;
- K_s hydraulic conductivity of the soil in m/s;
- H hydraulic head difference in m;
- L height of the soil column in m; and
- i hydraulic gradient (dimensionless).

An idea about the range of saturated hydraulic conductivity values that will be encountered in soils according to their textural class, is given in Table 14.

Falling-head method for determination of hydraulic conductivity
With the falling-head method a hydraulic head difference H across the

sample, the volume of water dV that passes through the sample in time dt, is determined from:

$$v = \frac{dV}{Adt} = -K_s \frac{H}{L} \qquad \text{Eq. 63}$$

The differential volume of water dV may be replaced by adH, where a is the internal cross-sectional area of the stand pipe. Integrating between limits t_1, H_1 and t_2, H_2 and solving the conductivity yields the following result:

$$K_s = \frac{aL}{At} \ln\left(\frac{H_1}{H_2}\right) \qquad \text{Eq. 64}$$

in which A is the cross-sectional area of the sample.

The actual form of the apparatus may be quite varied. The support for the sample should have a high conductivity relative to that of the soil. A suitable screen gauze or cloth barrier may be fastened to the bottom of the sample cylinder, or a very high conductance porous stone may be used. The diameter of the standpipe should be chosen so that easily measured changes in head will occur in a reasonable time, say between 1 and 100 min. The required tube diameter can be estimated from:

$$d = \left[\frac{K_s\, t\, D^2}{L \ln(H_1/H_2)}\right]^{1/2} \qquad \text{Eq. 65}$$

using appropriate values for the soil sample length L, the diameter of the sample D, the time t, the hydraulic head ratio H_1/H_2 and the conductivity K_s. With a standpipe 2 cm in diameter and a soil sample with a diameter of 7.5 cm, a length of 5 cm and a head ratio of 1.2, the largest conductivity that can be measured using a fall time of 1 min, is about 1×10^{-3} cm/s. The practical range of diameter for the standpipe is 0.2 to 2 cm. If the fall time is limited to 100 min or less and a standpipe with a diameter of 0.2 cm is used, the smallest Ks that can be measured is about 1×10^{-7} cm/s. This corresponds approximately to the lower limit of conductivity of silt and coarse clays.

Concerning the applicability of Darcy's law for these calculations see Section 5.1.3.

5.5.5 Chemical analysis of granular materials, base soil and ground water

Iron ochre

Soluble iron (Fe^{2+}, as iron sulphide or iron hydroxide) will deposit in soils and drainpipes when oxidation takes place (Fe^{3+}). This process might occur in subsurface drains. Micro-organisms (bacteria) play a key role in this process, and can also be active under anaerobic conditions. Hence keeping the subsurface drainage system submerged may not always be effective in controlling iron ochre deposits. This was experienced at a farm in Canada, when 50 - 75% of a controlled drainage system that was kept submerged most of the time and was still found to filled with an ochre gel (McKyes *et al.* 1992). The resulting slime can clog envelope material, block perforations of the pipe, or form a gelatinous mass inside the pipe itself. Colours range from red to yellowish or tan, and when sulphides are involved black deposits may be found as well (Lauwerszee in the Netherlands, Scholten 1989). Iron ochre problems have also been reported from the Netherlands (Knops and Dierickx 1979, Stuyt and Oosten 1987), the USA (Ford 1982), Germany (Kuntze and Eggelsmann 1974), Canada (McKyes *et al.* 1992) and France (Houot *et al.* 1984). Iron precipitation from ground water in conjunction with bacterial activity is common throughout the world (Cullimore and McCann 1977, surveyed 150 countries).

Ford *et al.* (1968) were among the first to report iron ochre problems in the subsurface drainage systems of citrus groves in Florida. In the Netherlands iron ochre problems primarily occurred in the newly reclaimed areas. Since then, considerable work has been done to understand the nature of the problem (Ford 1982, Scholten 1989). Iron is present either in soluble form in the ground water (Fe^{2+}) or in oxidised, non-soluble, form (Fe^{3+}) in the soil. In both cases bacteria are necessary to move from one phase to the other. When iron (Fe^{3+}) is in the soil, iron reducing bacteria (Gallionella and Leptothrix) can transform it to the soluble form (Fe^{2+}) if enough energy providing material (organic material such as root and plant residue, but also certain acids like malic, citric, tannic and lactic acids) are available. The conversion process requires more energy in clay soils than sandy soils, hence the problem can be more serious in lighter soils, or put differently, is less likely to occur in heavy soils at the same Fe^{2+} concentrations. Even if iron is not present in the soil it could reach the drainage system in soluble form with ground water (seepage) flows from elsewhere. The problem for the drainage system occurs during a subsequent step when the water carries the soluble iron to the drain. There, in an oxygen environment, the Fe^{2+} will be oxidised by bacteria in cracks, root channels, and drain perforations, or remain in slime deposits produced by the bacteria. The slime deposits also tend to trap the insoluble Fe^{3+}. Slime formation and oxidation will even occur under submerged conditions as was observed in Canada and the Netherlands (Scholten 1989). Apparently enough oxygen is dissolved, or suspended in the water for the bacteria to function.

Only the soluble iron forms a risk for ochre formation. Iron concentrations attached to soil particles were 100 - 300 times higher than the concentration in the water at the same location. There are four ways to assess the danger of iron ochre formation:

1. The simplest method to determine the presence of soluble Fe^{2+} in ground (drain) water is the use of indicator strips. A strip is immersed in the water sample for 1 second. If Fe^{2+} is present the paper will turn blue and after 20 s the strip can be compared with a colour chart that gives the concentration in certain classes of mg Fe^{2+}/l (i.e. 2, 5, 10, 25, 50 and 100). Soon after use the colour will fade.
2. Kuntze and Eggelsmann (1974) used a method where they added certain chemicals to a recent water sample (a buffered solution of ammonium-acetate and 0.5% solution of O-phenanthrolene-hydrochloride, [a slightly different method is mentioned in Dierickx and Vlotman 1995]). After 10-30 min the Fe^{2+} can be determined from yet another colour chart which gives the concentration in 0, 0.5, 1, 2 or 4 ppm Fe^{2+}/l. The pH is determined using indicator strips. The risk of iron ochre clogging is then determined from Table 20.

Table 20 Risk of iron ochre clogging based on soluble iron and pH. (Kuntze and Eggelsmann 1974).

Fe^{2+} in mg/l at pH = 7	pH	Risk
0.5	1.0	None
0.5 - 1.0	1.0 - 3.0	Small
1.0 - 3.0	3.0 - 6.0	Moderate
3.0 - 6.0	6.0 - 9.0	High
6.0	9.0	very high

3. The Ford method is independent of pH and the soil type. It assesses the Fe^{2+} in solution available for ochre formation in both the groundwater and the soil and will therefore give information on the source of the iron as well. Ten ml of on-site ground water should be collected in 10 ml syringes and pressure filtered through 0.45 m membranes into sulfamic acid in a completely closed system. The sulfamic acid contains phenanthrolene and formalin so that the Fe^{2+} concentration in the filtrate, as indicated by a red colour, is stable for up to 3 days. The colour can be interpreted after 3 - 5 minutes of application of the sample to the filter. The amount of Fe^{2+}/l is read from a colour chart. Soil samples, which will yield results similar to groundwater, are incubated for 13 days at 29-30°C in test tubes containing water with and without organic carbon source. After incubation, the tubes are shaken and placed upright for 1 day to allow a supernatant to form. The supernatant is removed with a syringe and collected in sulfamic acid-

phenanthrolene via pressure filtration through a 0.45 m filter. The colour is an indication of the concentration of Fe^{2+}. If the soil and water samples are stored in airtight containers with temperatures between 10 - 20° C they will be useable for several weeks. In low permeable soils difficulties with collecting recent water samples were experienced in the Netherlands: Fe^{2+} oxidised before the concentration could be determined.

4. A potential problem with iron ochre may be assessed using the guidelines given in Table 21 without laboratory assessment. Once positive identification of a potential problem has been made, further laboratory analysis (such as the Ford method described above) is necessary.

Scholten and Ven (1987) compared the indicator strip method and the Ford method and came up with tentative guidelines for determination of the risk of iron ochre clogging (Table 22). They warned that the 60-odd samples of 23 test sites in the Netherlands are barely enough to present definite guidelines. It was also found that the amount of deposit did not always match the concentration of soluble iron, which could be because deposits took place when different concentrations were present in the water.

Table 21 Identification of danger of iron ochre problems based on visual observations. (after Kuntze and Eggelsmann 1974).

Iron content of ground water	Visual signs in open drains	Visual sign on ditch bottom	Visual signs in soil profile
low	Clear	dark grey/black	homogeneous colour, no rust marks
moderate	clear red flocculation, oil type spots on surface	red brown	clay: gelatine deposits sand: iron concretions
high	murky water, brown oil-type spots on the surface	clearly visible red gelatine depositions	not only gelatine deposits but also iron concretions

Table 22 Estimate of iron deposit in subsurface drains by Ford and indicator strip method. (Scholten and Ven 1987).

	Fe^{2+} in mg/l	
iron deposit	Ford method	Indicator strips
none	0.5	1
little	0.5 - 2	1 - 5
moderate	2 - 5	5 - 10
high	5 - 10	10 - 25
very high	10	25

Remarks: Comparison based on 60 samples from 23 sites with various degrees of ochre problems in the Netherlands. The Machery-Nagel firm of Düren, Germany, supplied indicator strips used but no further details were provided. There is a correlation of r = 0.98 between the two methods. Indicator strips gave results 3 - 4 times higher than the Ford method.

The risk of iron ochre clogging depends on:
1. the amount of soluble iron available near the drains;
2. the amount of organic matter near the drains to provide an energy source for the bacteria needed for conversion of Fe^{2+} to Fe^{3+} and for the conversion of Fe^{3+} to soluble (and therefore mobile) Fe^{2+};
3. the soil type: clay binds Fe^{2+} more tightly than sandy soils;
4. the temperature; and
5. the cycle of dry and submerged conditions of the drainage system.

It is very difficult therefore to give precise estimates of the risk of iron ochre clogging, and the duration of it. When iron is present in the soil, it may eventually be removed totally when frequent flushing of the drains takes place. In severe cases flushing once a year was necessary in the Netherlands, while once in six years was adequate in other cases. Some places where samples with iron ochre had been obtained had not been flushed for ten years.

Calcium (carbonates) and pH

Soluble chalk or limestone (Calcite = $CaCO_3$) in gravel envelopes or other backfill materials may cause problems when the pH becomes less than 5.8 (Dennis 1982). Although not mentioned it is assumed that the problem refers to sediment deposits of calcareous nature in the drains. Calcite is commonly found in most arid soils. In Belgium it was found that $CaCO_3$ caused the gravel around the drain to kit together and form an impermeable block.

USBR (1989) mentions potential problems with carbonates and has restricted carbonates in the gravel to less than 5%. No specific mention is made of calcium carbonates. Carbonates come in many forms associated with cations of salt-affected soils such as Na^+, Ca^{2+}, Mg^{2+}, and to a lesser extent, K^+. The major anions involved are Cl^-, SO_4^{2-}, HCO^{3-}, NO^{3-} and at high pH, CO_3^{2-}. Carbonate alkalinity is reported in the literature to describe both HCO^{3-} and CO_3^{2-} ions (Jurinak and Suarez 1990).

The presence of inorganic carbonates in the soil or the envelope material can be investigated by applying some drops of a HCl solution of 10% concentration; when no visible or audible reaction takes place the sample has little or no calcium (carbonate).

Gypsum is another common mineral in arid soils, and a form of calcium ($CaSO_4.2H_2O$) which can precipitate under certain conditions. It is also commonly used to improve hydraulic conductivity of soils that have alkalinity problems. Gypsum solubility is not a function of pH (Jurinak and Suarez 1990).

There are numerous (laboratory) methods to determine inorganic carbonates (Dierickx and Vlotman, 1995, Nelson *et al.* 1978) which are beyond the scope of this book. There is very little evidence of reported problems due to the calcium and/or carbonates (in its various forms).

Organic matter
The decay of organic envelope materials is enhanced by the presence of organic matter in the soil, in particular for clay and loamy soils with a high pH (although not reported this could happen in sandy soils too). The USBR (1978/1993) mentions the undesirability of vegetable matter and other deleterious substances in the gravel envelope which, in time, could alter the hydraulic conductivity. In the same sentence clay is also mentioned as undesirable. However, clay would reduce hydraulic conductivity, while deleterious materials (materials that 'harm' the gravel envelope) over time could leave gaps after decomposition, and hence reduce the filter action.

The amounts of organic matter are also important when there are substantial quantities of soluble iron (Fe^{2+}) due to the danger of iron ochre, sulphur and manganese deposits. There are no actual measurements of the amount of organic matter content required for drain envelope design, and judgement of the presence of organic matter, particularly in granular envelopes, has been purely subjective. Geotextiles seem to be more susceptible to clogging by organic matter and iron ochre.

Sulphur and manganese
Biological activity in subsurface drains may also result in manganese and sulphur deposits as reported from the Netherlands (Scholten 1989) and the USA (Grass *et al.* 1975, Ford 1975). Typically, sulphur deposits are encountered in acid sulphate soils encountered in heavy clayey, coastal, areas. When a full soil chemical analysis is performed the presence of sulphur and manganese will be known, but at present no guidelines are available as to which concentration will cause trouble.

As mentioned before in the drain maintenance Section (4.5.4), Grass *et al.* (1975) describe the removal of oxides of iron and manganese (black deposits in drains) using sulphurous acid. Sulphurous acid is a strong reducing acid made by combining sulphur dioxide gas and water inside a drain.

5.6 Organic and synthetic envelope laboratory analysis

Many standard tests have been developed for specifying the properties of geotextiles few of which apply to organic envelopes as well (primarily the prewrapped loose material such as the coconut fibres). Well-known standards on geotextiles are those of the American Society of Testing and Materials (ASTM), the European Committee for Standardisation (CEN) and the

International Organisation for Standardisation (ISO). These tests are primarily designed to meet the requirements for civil engineering applications of geotextiles and are largely concerned with mechanical properties. However, in the following sections where applicable reference will be made to the various guidelines and standards, no attempt has been made to compare or judge them. They are not specifications!

Box 19 explains some of the abbreviations used in the coding system of the European (CEN) and International Standards (SI). The preferred method of reporting values is in ISO units, most US-based guidelines are based on the English units. Guiding values of both systems are therefore mentioned in the text so that those familiar with the English units might recognise the original values. Other organisations such as AASHTO and Task Force 25 try to formulate unified specifications. AASHTO stands for the American Association of State Highway and Transportation Officials and Task Force 25 is a joint committee of various US construction associations (Koerner 1994). Values recommended by these two organisations will be given where appropriate and available.

Box 19 Abbreviations and explanation of letter codes used with European standards.

WI	*Work item*
WG	*Working group*
Mandate	*Order voucher leading to financing of standardisation work; not the mandate containing the essential characteristics*
Stage	*Milestone in the standardisation process:* *stage 11: discussion in WG going on* *stage 32: first draft circulated to Technical Committee* *stage 40: CEN enquiry (6 months period for comments on a draft standard)* *stage 49: formal vote (final stage before publication)*
EN	*European standard (to be reviewed every 5 years)*
ISO	*International standard (to be reviewed every 5 years)*
EN ISO	*Standard that is both EN and ISO*
WD	*Working draft (stage in ISO corresponding to stage 11 in CEN)*
ENV	*European pre-standard (validity period of 2 years)*
prEN	*European draft standard*
prENV	*Draft European pre-standard*

Mechanical strength, abrasion resistance and degradation properties of geotextiles are important in drain envelope applications because of the handling requirements of the covered pipes. In particular, for the larger pipe diameters that come in lengths of approximately 6 metres, it is important that the material has adequate grab/tear strength. Field staff tend to lift the pipes by grabbing the envelope material rather than embracing the pipe and envelope during the lifting (embracing is a difficult manoeuvre with pipe diameters over 300 mm). The envelope materials must be able to withstand normal handling and passage through the drainage machine without developing tears or holes through which soil materials can enter the pipe. The materials must not stretch excessively or they will not offer the desired mechanical support to the soil by bridging over the tops of the corrugations. The corrugations of the pipe must remain open to serve as an entrance area and passage for water. The most important physical characteristics of a drain envelope material, after it is shown to have adequate physical strength, are the size of the openings and the permeability perpendicular to and in the plane.

5.6.1 Taking samples for testing

There are essentially two procedures for sampling (ASTM D 4354-96): procedure A is for specification conformance testing, and procedure B is for quality assurance testing. Both procedures require samples that include all possible sources of variation. EN 963 (1995) also gives guidelines for sampling and preparation of test specimens. The following steps in sampling can be applied to all the tests described:

1. Decide on the division of lots of any shipment or production quantity that differs from other portions in specifications, style or physical characteristics.
2. Then, from each lot randomly select a number of sample (production) units (rolls of fabric, bales of fibre, cases of yarn, or rolls of pre-wrapped drainpipes) for testing. The (production) unit is a quantity of geotextile agreed upon by purchaser and seller for sampling. Where there is no agreement a production unit quantity of 500 m^2 (600 yd^2) is suggested. For specification conformance testing, take from each lot the equivalent to the cube root of the total number of units included in the lot, but not less than one. If the cube root is a fractional number, take the next whole number. For quality assurance testing select 1 sample unit for a total of 200 units in the lot, 2 units for 500, 3 units for 1000 and 4 units for 1001 or more units. An exception may be made for time intensive quality tests such as ultraviolet degradation (Section 5.6.14); no more than 2 sample units should be selected per lot.
3. Select laboratory test samples from each sample unit as suggested for individual tests in the specifications or standard test descriptions.
4. Finally, from each laboratory sample select or prepare a number of test specimens depending on the type of test. In some cases laboratory samples and tests specimens can be the same.

5.6.2 Thickness

The method used to classify envelope material characteristics depends on the nature of the envelope. Thickness of loose fibre wrappings can be measured directly around the drain with a circometer or measuring tape (CEN/TC 155/WG 18). Thickness determination of geotextiles can be carried out on the geotextile itself, either prior to wrapping or after being removed from the drains, according to ISO 9863 (1990) or EN 964-1 (1995). Koerner (1994) observes that thickness is more a descriptive property rather than a design-oriented property. However, as was shown in Section 5.1.1, the thickness of the envelope has a major effect on the entrance resistance to a drain. Bhattia *et al.* 1994 indicated that there is a linear relationship between mass per unit area and thickness for non-woven materials (Box 20). ASTM D1599-88 stipulates that thickness should be measured to an accuracy of at least 0.02 mm (or 0.001 in) under a pressure of 2.0 kPa (= 0.29 lb/in^2 = 42 lb/ ft^2 = 20 gf/ cm^2). Thickness of commonly used geotextiles range between 0.25 and 7.5 mm (10 - 300 mils, where 1 mil = 0.001 inch = 0.0254 mm).

5.6.3 Mass per unit area

Mass determination of loose fibre wrappings can be carried out according to the method discussed by CEN/TC 155/WG 18 (see Box 20). For geotextiles ISO 9864 (1990), EN 965 (1995), or ASTM D5261-92 are applicable. The mass per unit area is directly related to material cost and to some mechanical properties and hence is of great importance. Mass per unit area available on the market ranges between 130 and 700 g/m^2. The selection for drain envelopes depends on a range of other requirements such as thickness in view of entrance resistance, strength in view of handling, characteristic opening size in view of filtering requirements etc. In most cases the mass per unit area will be the independent variable amongst the variables. When two materials are near equivalent in all other properties, the one with a lower mass per unit area is likely to be cheaper.

5.6.4 Tensile strength, grab strength, tear strength, elongation, and seam strength

Tensile strength for geotextiles is the maximum resistance to deformation and can be characterised by force elongation, the breaking force, or the breaking elongation (ASTM D 4439-95, and D 4595-86, EN ISO 10319 (1996), BS 6906 part 1). The grab strength of a fabric is a measure of resistance to deformation but only part of the width of the specimen is gripped in the clamps (ASTM D 4439-95, and D 1682). Seam strength is a measure that is either reported in absolute units (i.e. kN/m) or as a percentage of the strength of the sheet (Koerner 1994, ASTM D 4884-90, EN ISO 10321 (1996)).

Box 20 Relationship thickness and mass of synthetic envelope material.

Bhatia et al. 1994 reported linear relationships between the mass per unit area and the thickness of the 22 non-woven polypropylene geotextiles (needle-punched and heat-bonded), but these relationships were not the same for geotextiles manufactured by different processes and fibre types (Smith 1993). Indeed, data found in articles published in the proceeding of the Fifth Int. conference on Geotextiles, Geomembranes and Related Products (Figure 57) seem to indicate a linear relationship, but care should be taken when using it.

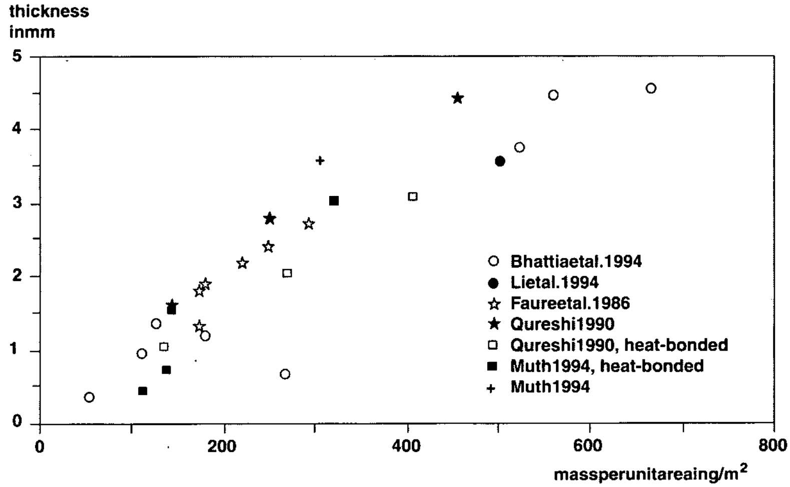


Figure 57 Relationship between mass per unit area and thickness of non-wovens.

Santvoort (1994) describes a number of tensile strength tests: strip tensile strength; grab tensile strength; manchet tensile test; plane-strain tensile test; wide-width tensile test; and the biaxial tensile test. Description of all of these is beyond the scope of this book and further details will be limited to the tests that have been accepted by ASTM and/or ISO.

Tensile strength is perhaps one of the more important mechanical properties of geotextiles. Geotextile applications rely on this property as the primary function in reinforcement applications, or as secondary function in filtration, separation and drainage. The basic idea is to test the geotextile between a set of clamps or jaws, and stretch the geotextile in tension until failure of the material or the seam occurs. The behaviour of the characteristic opening size (COS) under stretch is of importance in drainage applications when the material is stored on roles wrapped around the pipe, or when tension is applied during the construction when the material around the pipe moves through the trencher box or trenchless drainage plough conduit.

The grab strength is important for field handling of wrapped drainpipes, in particular those with larger diameters, when field staff grab the envelope to lift the pipe. Equally important during field handling and construction is the tear strength of the material. It is defined as the force required to either start or propagate a tear in fabric under specified conditions (ASTM D 4439-95). The three commonly used methods to determine the tearing strength are: the Trapezoid Tearing test, the Tongue Tear test and the Elmendorf Tear test. The first is the one most commonly used in the USA (ASTM D 4533-95).

To determine the tensile strength of the material, the laboratory sample should consist of a full width swath of approximately 1 m in the machine direction from each roll of the lot sample, and preferably not from the outer wrapping of the roll. Elaborate instructions for the number of test specimens based on statistical significance at the 95% probability level are given in ASTM D 4596-93. For test specimens of the seam test, 6 samples are suggested per lot sample. Detailed instructions on how to cut and sew the samples are given in ASTM D 4884-90. Grab strength determination is given in ASTM D 1682.

Typical tensile strength values of available geotextiles are (Koerner 1994):
- wide width tensile strength 9 - 180 kN/m (50 - 1000 lb/in)
- seam strength 50 - 100% of tensile strength.

Interestingly, Koerner (1994), describes the tensile strength tests in some detail, but does not describe the grab test or grab strength. Yet, in a listing of requirements of government/state agencies of the US, only grab strength and grab elongation are reported not tensile strength. For road and construction drainage requirements the following values were prescribed (values in N are rounded to the nearest 10). Class B requirements seem closest to what may be expected with agricultural drainage.
- Grab strength Class A[31] 800 N (180 lb), one state with 890 N (200 lb). AASHTO M288-90 and Task Force 25 require 400 N (80 lb).
- Grab strength Class B 360 - 400 N (80 - 90 lb). AASHTO M288-90 and Task Force 25 require 360 N.
- Grab elongation 15 - 20 %, some States as high as 50, 80 and 100%. AASHTO M288-90 and Task Force 25 do not give a required value.

[31] Class A geotextiles are used where installation stresses are higher than Class B applications (i.e. very coarse, sharp, angular aggregate is used, a heavy degree of compaction [>95% of AASHTO T99] is specified, or the depth of the trench is greater than 3 m [10 ft].).

Class B geotextiles are used with smooth graded surfaces having no sharp, angular projections and sharp, angular aggregate is not used. Compaction requirements are light (<95% AASHTO T99) and trenches are less than 3 m in depth.

For Trapezoid Tearing strength the following values are required in various US States:

- Trapezoid tearing strength Class A: 220 N (50 lb).
- Trapezoid tearing strength Class B: 110 N (25 lb).
- When Class A or B is not specified prescribed values are 110 - 220 N (25 – 50 lb).

For sewn-seam strength the following are required in various US States:

- Sewn-seam strength Class A: 310 - 710 N (70 -160 lb). AASHTO M288-90 and Task Force 25 require 710 N.
- Sewn-seam strength Class B: 310 - 360 N (70 - 80 lb). AASHTO M288-90 and Task Force 25 require 310 N.

5.6.5 Static puncture test

The static puncture test determines the force to push a flat plunger through a geotextile and is described in EN ISO 12236 (1996) and ASTM D4833-88. The test measures the resistance of the geotextile against punctures by rocks, gravel, and other sharp objects. The damage might occur during transport or during installation of the drainpipe.

For the laboratory sample take a full width swath of sufficient length along the selvage from each sample roll so that the requirements for test specimens based on statistical considerations (details see ASTM D 4833) can be met. The selvage is the edge of a fabric woven so that it will not unravel.

For road and construction drainage requirements the following values are prescribed (Koerner 1994) by various states in the US and AASHTO and Task Force 25:

- Puncture strength Class A: 110 - 360 N (25 - 80 lb). AASHTO M288-90 and Task Force 25 require 360 N.
- Puncture strength Class B: 110 - 160 N (25 - 35 lb). AASHTO M288-90 and Task Force 25 require 110 N.

5.6.6 Compressibility

Compressibility of both pre-wrapped loose material and needle punched non-woven material will have a major effect on the transmissivity of the material, and also on the permeability perpendicular to the plane, as well as on the characteristic opening size. Compressibility as a function of applied stress or load is given in Figure 58 for different geotextiles.

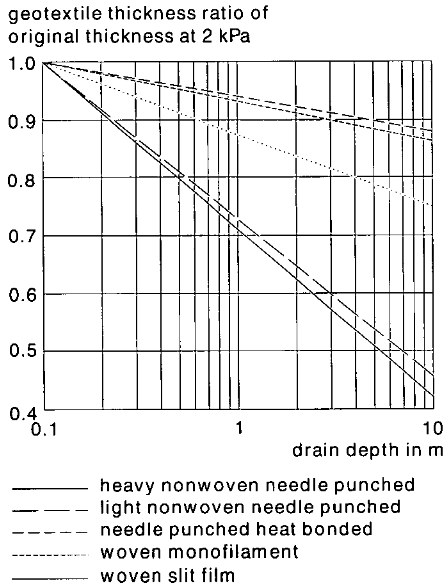


Figure 58 Compressibility of geotextiles as function of load.

Based on the figure and using the standard testing pressure of 2 kPa, which is equivalent to approximately 0.1 m of saturated soil (specific mass of saturated soil taken as 2000 kg/m^3), it can be determined what the reduction in thickness will be as function of drain depth (Figure 58).

No range or limits of compressibility are given in the US State specifications, instead, the requirements are implicit in permeability and permittivity requirements (Sections 5.6.10 and 11).

5.6.7 Abrasion damage

Abrasion is defined as the wearing away of any part of a material by rubbing against another surface. Devices for testing are available commercially and most of them are used to test textiles. The standard test procedures available for geotextiles are: EN ISO 13427 (1998), ASTM D 4886-88 and DIN 5385. Results are reported as a loss in tensile or breaking strength of the material, as percentage of the unabated strengths. Koerner (1994) reported losses in strength of 60-90% as function of 250 - 1000 cycles of abrasion. Not reported is whether certain types of geotextiles (woven, non-woven, filaments, etc.) are more susceptible to abrasion damage than others. Abrasion damage with drainage applications can occur during transport, and in the trencher box or plough conduits. No abrasion damage has been reported for subsurface drainage applications and no percentages are given in the overview of US

State agency requirements (Koerner 1994). Santvoort (1994) does not address the issue of abrasion.

5.6.8 Porosity, percent open area

Porosity of geotextiles is a property that is used with non-woven fabrics. Percent Open Area (POA) is primarily applicable for woven, mono-filament fabrics (Koerner 1994). POA varies from 1 - 36% (essentially closed - extremely open) for woven mono-filament geotextiles, with commercial geotextiles often with a POA = 6 -12%. There are no standards for measurement of POA and planimetry is the best way to determine it. Project light through the specimen onto a large poster size piece of cardboard, which may or may not be covered with millimetre paper. Then use either a planimeter or count squares where light is projected. The total area (yarns plus voids) must be measured at the same magnification as the voids measured. The technique is not applicable for non-wovens.

Van der Sluys and Dierickx (1987) found that the porosity of non-woven geotextiles they used in permeability experiments had $\varepsilon = 0.81 - 0.87$. The porosity increased with increase in mass. Several other geotextiles (mono- and multi-filaments) had POA values between 0.65 and 0.77, however there were not enough samples reported to determine a relationship with mass. The range of typical values for porosity was between 50-95% (Koerner 1994).

Porosity of geotextiles can be determined from:

$$\varepsilon = 1 - \frac{\mu}{\rho_f T_g} \qquad \text{Eq. 66}$$

where,

μ mass per unit surface area in g/m^2 ;
ρ_f fibre density in g/m^3 (Box 21); and
T_g mean geotextile thickness in m.

Box 21 Specific gravity of various materials related with envelope design.

Specific gravity is defined as the ratio of the unit volume weight of the substance to that of distilled water at 4°C (1000 kg/m^3 or 1 g/cm^3).

Steel	*7.87*	*Polyvinyl chloride (PVC)*	*1.69*
Rock/soil	*2.4 - 2.9*	*Polyester (PET)*	*1.22 - 1.38*
Sand	*2.68*	*Polyethylene (PE)*	*0.90 - 0.96*
Glass	*2.54*	*Polypropylene (PP)*	*0.91*
Cotton	*1.55*	*Nylon, polyamide (PA)*	*1.05 - 1.14*

Porosity of geotextiles is occasionally measured directly as described in Section 5.6.9. Porosity though highly dependent on the pressure under which the fabric is applied, does not change as dramatically as the thickness of non-wovens (Figure 58). Giroud (1996) demonstrated that a 50% decrease in thickness reduced the initial porosity of 0.91 of a typical non-woven, needle-punched geotextile by 10% to 0.82. POA and ε are ratios of open area to total area or volume of pore space to total volume. The actual size (distribution) of the pores in geotextiles are characterised in the next section. Just like compressibility, POA and porosity are generally not required in specifications, instead, these characteristics are implicit in the requirements for transmissivity and permittivity (Sections 5.6.10 and 11).

5.6.9 Opening size distribution and characteristic opening size

The main function of wrapping geotextile around drain tubes is to enhance the conveyance of water towards the openings of the drainage system while restraining soil particles. To assure soil particle retention, the soil gradation curve must be known as well as the opening size of the geotextile. Therefore, it is essential that at least one characteristic opening size (COS) of the geotextile is known besides the particle size distribution or a characteristic particle size of the soil.

There are different definitions of COS and values often used are O_{85}, O_{90}, O_{95} and O_{98} which can be defined as the diameter of the opening of which 85, 90, 95 or 98 percent of the openings have a smaller diameter (this corresponds with the 85, 90, 95, or 98 percent of the material retained when using sieving techniques to determine COS). It should be noted that although COS is an opening size, the definition is often given in terms of an equivalent grain size (since it is usually determined by using the material as a sieve). The designer should be aware of these differences and consider the opening characteristics in relation to the determination method. Depending on the method of determination reported COS values can differ by a factor of 2 - 4.

There are a number of direct and indirect methods for the determination of the COS as well as the opening size distribution of geotextiles. The indirect methods comprise three basic sieving methods (Van der Sluys and Dierickx 1990, Gerry and Raymond 1983, ASTMD 4751, Mlynarek *et al.* 1993): dry sieving, wet sieving, and the hydrodynamic sieving. A fourth method, a combination of the dry and wet sieving (modified wet sieving), is also given as a variant. Direct methods are the Bubble Point Method (Hoffman 1983, ASTM 1991), the Mercury Intrusion Method (Prapaharan, *et al.* 1989, ASTM D-4404); and Image Analysis (Masounave *et al.* 1980, Lombard and Rollin 1987, Faure *et al.* 1990, Bhatia *et al.* 1993). Many manufacturers according to Rollin *et al.*

(1990) use the bubble point method. Finally, another method just mentioned for completeness sake but will not be described in detail, is the moisture desorption method (Dennis and Davies 1984).

Box 22 Standards for opening size determination of fabric envelope materials.

ASTM D 4751-95	*Dry sieving using glass beads, USA, 1996*
ASTM F 316-86	*Bubble point and mean flow pore test, USA, 1991*
BS-6906-89	*Dry sieving using soil particles, UK*
SW-640550-83	*Effective opening size, Switzerland*
NF G38-017-89	*Hydrodynamic sieving with sand particles, France*
SM-G-8.1	*Hydrodynamic sieving with sand particles (RILEM, France; The International Union of Testing and Research Laboratories for Materials and Structures).*
ISO/TC38/SC21/N-9	*Hydrodynamic sieving with soils particles*
ISO 2591-1	*Test Sieving, Part 1, Methods using test sieves of wire cloth and perforated metal plate.*
CAN/CGSB-148.1-10	*Hydrodynamic sieving with glass beads, Canada fifth draft (1991).*
NEN 5168	*Characteristic opening size dry sieving with sand fractions, the Netherlands.*
EN ISO 12956	*Geotextiles and geotextile related products - Determination of the Europe*
AS 3706.7	*Dry sieving method, Australia, 1990 (Li et al. 1994)*

The sieving technique is an indirect method that uses the geotextile as a sieve. It is popular and therefore widely used in many research laboratories. It is based on the passage of known sizes of sand particles or glass beads and the opening size is related to the observed size of the granular material that passes the geotextile. The most familiar sieving methods are dry and wet vibration and hydrodynamic sieving, and many countries have developed standards for these (see Box 22).

The three sieving methods result in different COS values (Fayoux *et al.* 1984). The hydrodynamic sieving gives the filtration opening size (FOS, O_{95}) is obtained, the wet sieving the equivalent opening size (EOS, O_{90}) and the dry sieving the apparent opening size (AOS, O_{95}). Different results are attributed to differences in the characteristics of the particles used (glass beads, sand, silica) and in data processing. The latter problem for instance is because it is easy to overlook that one method results in O_{95} while the other gives O_{90}. In addition AOS does not necessarily give the actual value of the opening size (depending on the preference of the reporter) but reports the mean size of the particle range (Table 23). ASTM D 4751-95 allows the AOS to be reported in terms of a sieve number; the one having the nominal openings, in millimetres, next larger than the actual opening size determined from graphing the results. If when the sieve number is translated in mm a (small) deviation has been introduced, this could account for some of the reported differences.

When the same base material was used and data processing was done in the same way, no significant differences were found between the three sieving methods, even when either sand or glass beads were used (Van der Sluys and Dierickx, 1990). With dry sieving static electricity and loss of fines becoming airborne due to the vibrations affected sieving results with the very fine particle sizes (Dierickx 1993b). Rollin *et al.* (1990) suggested that glass beads should be used when FOS lower than 75 µm is expected to be determined for selecting envelope materials to be used in silty uniform soils.

Calculating the COS using textile properties such as the Denier number and mass of the geotextile have met with mixed success. The pore size distribution of two non-woven geotextiles with the same weight but one manufactured with a denier fibre of 3 and the other of 9 will be completely different. The size and type of needles used for needle punched non-woven geotextiles, the specific polymer used (knowing that it is polypropylene is not enough!), the stabilisers and the sizing or finishing, all have different hydrophobic and hydrophilic effects: they all affect and determine the characteristic opening size. All these parameters cannot be examined without an unreasonable amount of work, and it is therefore necessary to have an index or performance test (Floss *et al.* 1990). The COS determination methods most used will be briefly described, followed by a conclusion and comparison between the methods.

Indirect Methods: sieving methods

Dry Sieving. In the dry sieving method, the geotextile is used as a sieve and the COS or opening size distribution results from the sieving of well-defined dry sand fractions or dry spherical solid bead fractions. Table 23 gives an example of the fractions that can be used. Other intermediate fractions can be selected according to existing sieves, if required.

Dry sieving requires commercially available sieving equipment with a vertical vibration of 0.75 mm amplitude (total swing height of 1.5 mm) at a frequency of 50 Hz. The test piece is clamped between two rings with a minimum net diameter of 130 mm and placed on the pan of the sieving apparatus that receives the passed granular material. To avoid excessive deformation under the weight of the granular material, a grid with a 1 mm diameter wire and a mesh size of 10 mm supports the test specimen. The whole is then fixed on the sieve apparatus (Figure 59). A certain amount (e.g. 50 g) of a given granular fraction is poured on to the geotextile and sieved for 5 min. The granular material remaining on and in the geotextile is determined as the difference between the total amount and the passed material collected in the pan. Other fractions up to that fraction of which only 1 % passes and down to that fraction of which 99 % passes, can be used to select - by interpolation - the diameter of which 90 % of the particles are retained. The test should at least be carried out in triplicate, although five replicates are recommended.

Dry sieving is a quick and simple method which can be done with commercially available sieving equipment. Its disadvantage, however, is that it is difficult to apply to fine structured geotextiles with a COS smaller than 100 µm because of electrostatic forces which make the fine granular particles adhere to the geotextile, giving false results. The method also requires the availability of granular fractions. Dry sieving has been accepted to determine the opening size of loose fibre wrappings (CEN/TC 155N 1261E 1994).

Table 23 Sand fractions to determine the characteristic opening size of geotextiles. (after Santvoort (1994).

Range of particle size, µm	Mean particle size used for COS, µm	Range of particle size, µm	Mean particle size used for COS, µm
37 - 53	45.0	420 - 590	505.0
53 - 74	63.5	590 - 840	715.0
74 - 105	89.5	840 - 1 190	1 015.0
105 - 149	127.0	1 190 - 1 680	1 435.0
149 - 210	179.5	1 680 - 2 000	1 840.0
210 - 250	230.0	2 000 - 2 380	2 190.0
250 - 297	273.5	2 380 - 3 360	2 870.0
297 - 420	358.5	3 360 - 4 760	4 060.0

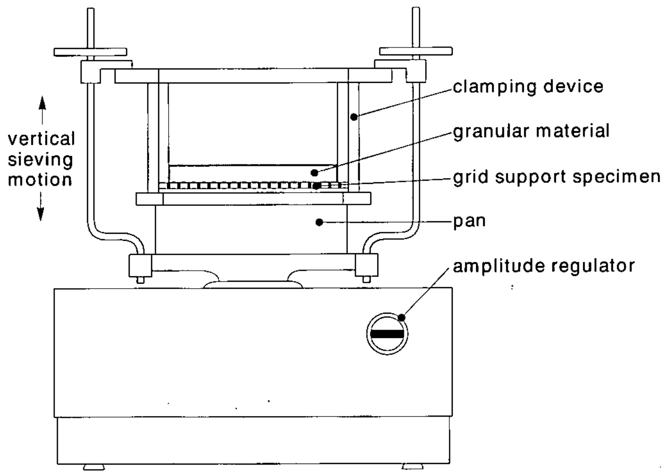


Figure 59 Diagram of the equipment for dry sieving.

Wet Sieving. A sieving apparatus similar to that for the dry sieving is used for wet sieving. The upper clamping ring is a transparent disk to which one or more spray nozzles are attached to enable wetting of the granular material, and a transparent cylinder which encloses the unit. The upper clamping disk with the transparent cylinder acts as a covering cap to avoid soil loss (Figure 60). The water supply must be adjusted with a regulating valve to be compatible with the specimen permeability to ensure that soil particles are completely wetted, but without the water rising above the granular material; in other words, there should not be any excess water left standing on the geotextile specimen. The pan is provided with a connection tube, which leads the water with the passed granular material to the collection device where they are separated by means of a filter. Sieving is also done at a frequency of 50 Hz but here an amplitude of 1.5 mm (3 mm swing height) is applied. The wet sieving makes use of a graded soil of known particle size distribution. A mass equivalent to 7 ± 0.100 kg/m^2 is poured onto the geotextile specimen. Wet sieving makes also use of a supporting screen to prevent excessive deformation of the geotextile specimen. After a sieving time of 10 min the amount of soil passed through the geotextile specimen (done in triplicate) is combined with the other sieved specimens for the further determination of the particle size distribution curve of the passed soil. The characteristic opening size is derived from the corresponding particle size of the passed soil.

Although the sieving process itself is not time-consuming, each soil that has passed the test has to be subjected to a particle size determination. The wet sieving also requires the availability of a well-graded soil with a coefficient of uniformity between 3 and 20. Moreover, the soil must have a COS between d_{20} and d_{80}, being the particle size at which 20 % and 80 % respectively on dry weight basis are all smaller. This sieving method is also applicable for fine-textured geotextiles with a COS smaller than 100 µm, in contrast to the dry sieving. Problems can occur with the discharge water if the collected soil material blinds the filter causing it to become less permeable.

The wet sieving method has been accepted as a European standard (EN ISO 12956, 1999) to determine the COS of geotextiles because very consistent results were obtained from different laboratories (Dierickx and Myles, 1996).

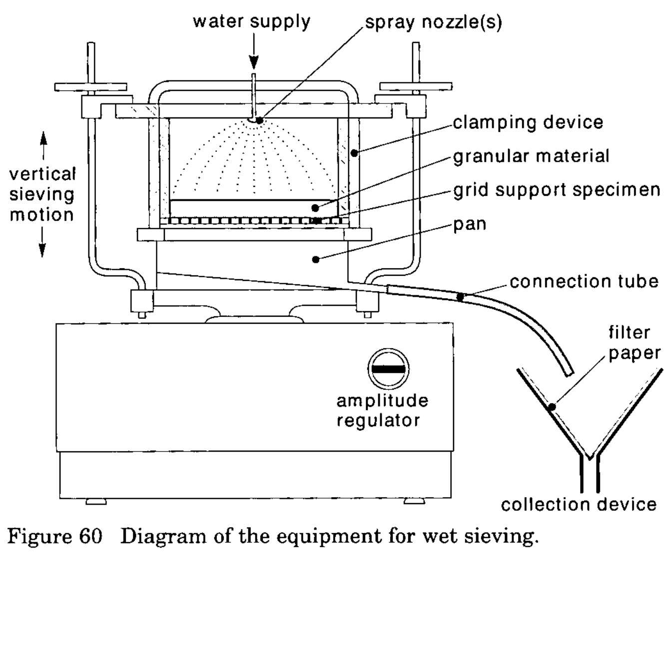


Figure 60 Diagram of the equipment for wet sieving.

Hydrodynamic Sieving. The hydrodynamic sieving method is based on cyclic immersion of a geotextile specimen clamped in a frame and loaded with a soil of known gradation (Figure 61). The graded soil must have a coefficient of uniformity greater than 6 and a d_{98} larger than twice the COS, while its d_{10} must be smaller than one fourth of the COS. Data analysis must be done on the results of at least 3 replicates. The area of each specimen is required to be 250 cm^2 and a soil mass corresponding to 3 to 5 kg/m^2 of geotextile area must be tested. The number of cycles is at least 2,000 which means that the test lasts almost 24 hours if replicates are done simultaneously. The passed soils of each specimen are combined for the further determination of the particle size distribution and the COS is derived from the corresponding particle size of the passed soil.

The main disadvantage of this method is its duration. Another drawback is that the equipment is not standardised or commercially available. Furthermore, for this method, the passed material has to be dried and subjected to a particle size analysis as for the wet sieving. The hydrodynamic sieving method also applies to fine structured geotextiles with a COS smaller than 100 μm.

Modified Wet Sieving. Dry sieving with fractions is a simple and easy method to determine the COS of loose fibre wrappings and geotextiles. The disadvantage, however, is that dry sieving is not applicable to fine geotextiles with O_{90} smaller than 100 μm because electrostatic forces make the finer sand fraction adhere to the geotextile fibres.

To overcome this problem, and to enable the determination of the opening size of fine geotextiles as well, a wet sieving with fractions can be done instead which takes 10 min. After drying the granular material that passed the spec-

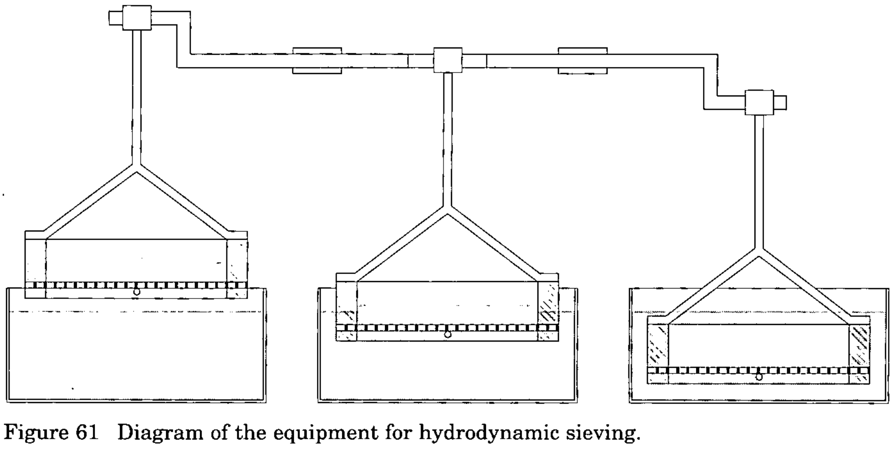


Figure 61 Diagram of the equipment for hydrodynamic sieving.

imen, the data of the modified wet sieving can be analysed in the same way as for dry sieving.

Direct methods

Bubble Method. This method is based on the following two principles: (1) a dry porous material will allow the passage of air through all of its pores when any amount of air pressure is applied to one side of the material; and (2) a saturated porous material will only allow a fluid to pass through when the pressure applied exceeds the capillary attraction of the fluid in the largest pore. Because the smaller pores pass air as the air pressure is increased, the largest opening (O_{95}) and pore size distribution of the geotextile can be evaluated. The main drawback of the method is that the results depend on the interpretation by the observer.

Mercury Intrusion Method. The mercury intrusion porosimetry method is based on the theory (Washburn equation) which relates the pressure required to force a non-wetting fluid (mercury) into pores of a geotextile with the radius of the pores intruded. Pore size distribution can be determined. Although the mercury displacing some of the fibres affects mercury intrusion, the results of replications remain in a very narrow range.

Image Analysis. This is a technique that measures pore spaces directly within a cross-sectional plane of a geotextile. The tests were time-consuming and there was no standard methodology then. The results could be replicated fairly well (Bhatia *et al.* 1994).

Concluding remarks on determination Characteristic Opening Size

Bhatia *et al.* 1994 compared the various methods of determining COS, and Table 24 and Table 25 summarise their findings based on the material presented in their paper. Taking the dry sieving method as the baseline method with an O_{95} as the AOS, Table 24 shows that the AOS determined by the wet sieving, hydrodynamic sieving, and the bubble point method gave 50% smaller AOS values on average than the dry sieving method. The mercury intrusion and image analysis methods produced higher values. Preference is given to the wet and hydrodynamic sieving methods as the best and most practical method for determining the EOS and FOS (O_{90}, O_{95} respectively). However, this conclusion is subject to the type of material tested (Table 25); not a single method performed best for all materials.

Table 24 Pore size distribution methods compared.
(after Bhatia et al. 1994).

Test method	Test mechanism	Standard and/or methodology	Test material	Sample size cm^2	Time for 1 test	Ratio O_{95}method/O_{95} dry sieving*		
						Avg.**	Min.***	Max.***
Dry sieving	Sieving dry	ASTM D-4751	Glass beads fractions	434	2 hrs	1	–	–
Hydrodynamic sieving	Alternating water flow	CAN/CGSB-148.1-10	Glass beads mixture	257	24 hrs	0.5	0.33	1.75
Wet sieving	Sieving wet	Franzius Institut	Glass beads mixture	434	2 hrs	0.5	0.2	1.25
Bubble point	Comparison of air flow dry vs. saturated	ASTM F-316	Porewick	22.9	20 min.	0.5	0.2	1
Mercury intrusion	*Intrusion of a* liquid into a pore	*draft ASTM*	*Mercury*	1.77	35 min.	1.6	1	4.2
Image analysis	Direct measurement of pore spaces in cross-section of geotextile	No standard; Bhatia *et al.* 1993	none	1.5	2 -3 days	2.2	1.14	2.5

* - Values determined from Figures 3 - 7 in Bhatia et al. 1994.

** - *Determined by drawing an eyeballed average line parallel to ratio 1 line.*

*** - Determined by drawing min and max. lines parallel to ratio 1 line, excluding some outlying maximum values.

It is important that manufacturers specify the method used to determine the COS. Wet, dry and hydrodynamic sieving can result in the same COSs provided the procedures of Van der Sluys and Dierickx (1990) are followed. Generally, however, one should expect differences on the order of those indicated in Table 24 and results and criteria reported should be adjusted to a base method; the wet sieving method is suggested as the standard method. Dry sieving is generally applied to loose fibre wrappings around drainpipes and wet sieving to geotextiles. Dry sieving should not be used for geotextiles as the fineness of the geotextiles that are typically used for filtering action in drainage applications is such that electrostatic and scatter effects (of fines) will affect the outcome. The wet sieving and the modified wet sieving are the simplest of the remaining sieve methods. The other methods require more sophisticated equipment, which may not be readily available. It is suggested that the wet sieving method results be used and results of other methods adjusted according to the conversion factors in Table 26.

Table 25 Applicability of pore size determination method and material used. (Bhatia *et al.* 1994).

	Material tested by manufacturer				Can the method be used as standard method for all types of materials?
	B	C	D	E	
Number of geotextiles used	5	6	5	6	
Thickness in mm	0.90 – 4.54	1.32 – 4.44	1.17 – 3.72	0.32 – 0.63	
Mass in g/m^2	115.59 – 669.31	131.11-562.36	184.4 – 524.7	57.74 – 271.04	
Test method:					
Dry sieving	highly variable	highly variable	highly variable	best	Presently no, but possible
Hydrodynamic sieving	–	greatest variation	–	best	no
Wet sieving	–	best	–	greatest variation	no
Bubble point				greatest variation	yes
Mercury intrusion	----------------- narrow range -----------------				yes
Image analysis	---- no standard yet but results are repeatable ----				yes

Manufacturer B and D: nonwoven polypropylene/polyester, staple fibre, needle punched geotextiles.
Manufacturer C: nonwoven polypropylene, continuous filament, needle punched geotextiles.
Manufacturer E: nonwoven polypropylene, continuous filament, heat-bonded geotextiles.

Wet sieving was recommended for the European and ISO standards (Dierickx and Myles 1996) based on comparison of sieve results by laboratories in Europe, China and North America. The inter-laboratory trials showed that dry sieving and hydrodynamic sieving had relative large differences in sieve results of the same materials. Contrastingly, results from wet sieving remained in a narrow band, moreover, the equipment for wet sieving is simple and easy to come by. Also O_{90} rather than O_{95} is recommended as the standard COS. The values of O_{90} are much more consistent than the O_{95}, which might be clear from a typical cumulative particle size passing gradation curve, where O_{90} is mostly in the transition zone between a steep and gentle slope, while the O_{95} values are predominantly in the gentle sloping portions.

Dry sieving results in COSs that are 30 - 50% greater than those of the wet sieving (ASTM D 4751). Consequently, applying the retention criteria, based on wet sieving, with the O_{90} obtained by dry sieving, is certainly on the safe side. The commended procedure is wet sieving. However, criteria developed in the USA are typically based on dry sieving techniques, which should be adjusted with an appropriate ratio if wet sieving is used to determine the O_{90}.

Table 26 Wet Sieving Results as standard and conversion factors for comparison with other methods.

Test method used	Standard and/or methodology according to:	Multiply with factor to convert to method: Dry sieving	Hydro-dynamic sieving	Wet sieving	Bubble point	Mercury intrusion	Image analysis
			from Table 24, using published O_{95} ratios				
Dry sieving	ASTM D-4751	1	0.5	0.5	0.5	1.6	2.2
			derived values with wet sieving as standard				
Hydrodynamic sieving	CAN/CGSB-148.11	2	**1**	1	1	3.2	4.4
Wet sieving	Franzius Institut	2	1	**1**	1	3.2	4.4
Bubble point	ASTM F-316	2	1	1	**1**	3.2	4.4
Mercury intrusion	draft ASTM	0.6	0.3	0.3	0.3	**1**	1.4
Image analysis	No standard; Bhatia *et al.* 1993	0.5	0.2	0.2	0.2	0.7	**1**

Fisher *et al.* 1990 developed relationships between various pore opening sizes in geotextiles for comparison of criteria established by others (Table 27). They based the mean and standard values on pore size distribution curves given by Millar *et al.* (1980), Rankilor (1981) and Prapaharan *et al.* (1989).

Table 27 Relationships between different pore sizes for non-woven geotextiles. (after Fisher *et al.* 1990 and Li *et al.* 1994).

Pore opening ratios	Mean ratio	Standard deviation
O_{95}/O_{15}	2.4	0.9
O_{95}/O_{50}	1.7	0.5
$\mathbf{O_{95}/O_{90}}$	**1.2**	**0.1**
1.05*		
O_{90}/O_{15}	3.1	0.8
O_{90}/O_{50}	1.5	0.3
O_{50}/O_{15}	1.4	0.3

* value for spunbonded needle punched non-woven S1 fabric in Li *et al.* 1994.

From Figure 62 additional ratios can be derived as needed.

Some typical and required AOS values (Koerner 1994) are:

- Typical AOS fell between sieve no. 10 and sieve no. 200 (Table 15);
- AOS requirements of various US states were given to be either sieve no. > 40, 50, 70, or O_{95} < 0.425 mm, 0.296 mm, 0.212 mm. In several states a sieve number range was given such as no. 70 - 100, 70 - 120, 40 - 100, etc.;
- AASHTO M288-90 and Task Force 25 recommend both: Soil with 50% or less particles by weight passing US no. 200 sieve, O_{95} less than 0.595 mm (AOS > no. 30 US standard sieve). Soil with more than 50% particles passing US no. 200 sieve, O_{95} less than 0.296 mm (AOS > No 50 US standard sieve).

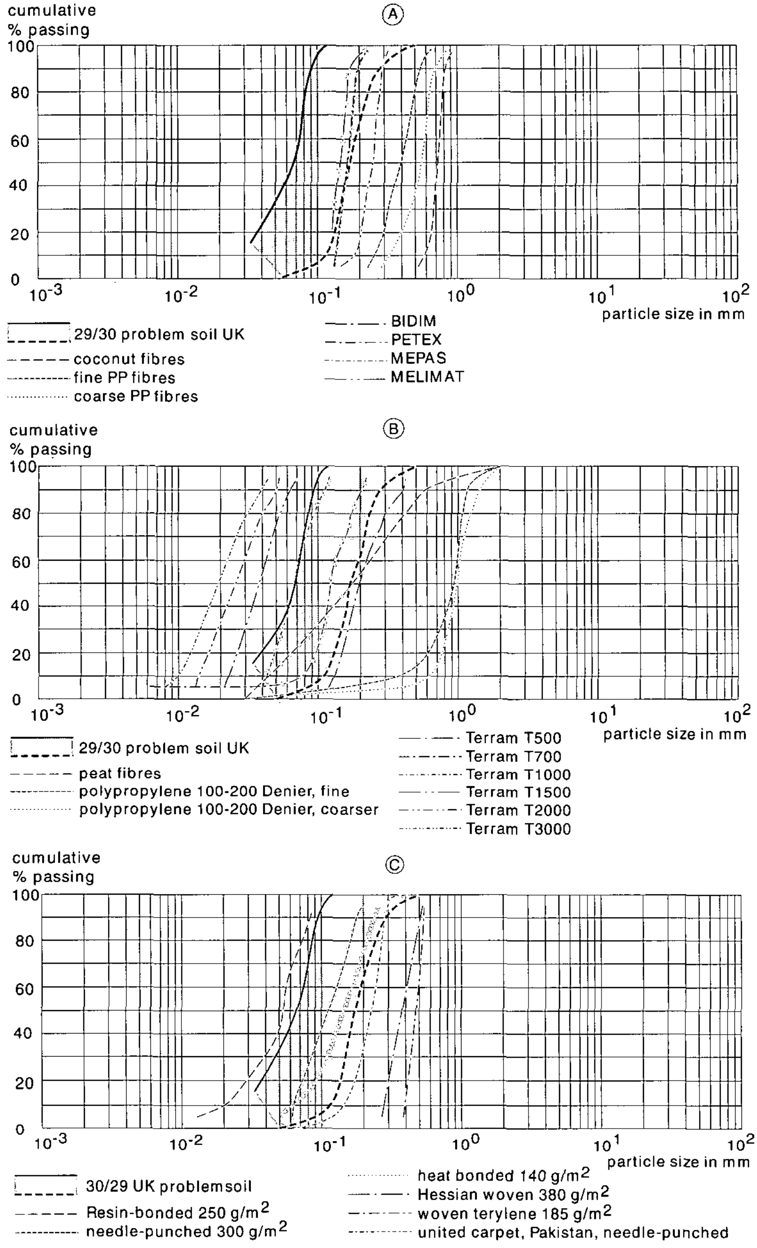


Figure 62 Examples of pore size distributions for various types of geotextiles in relation to typical UK problem soil.

A Organic and synthetic filters tested by Kabina and Dierickx (1986).

B Geotextile and peat envelopes (Stuyt 1992) and ICI Terram geotextiles (Koerner 1994).

C Geotextiles bonded by different methods (McGown 1976) and local Pakistan geotextile.

5.6.10 Water permeability normal to the geotextile plane without load

Permeability through a medium (geotextile is this case) is the amount of (water) flow per time, commonly referred to as Hydraulic Conductivity (Koerner 1994). When permeability is divided by the thickness of the layer through which flow occurs this is called permittivity. Permittivity is a better measure of flow characteristics for comparison amongst different materials than permeability because it expresses flow characteristics per unit of thickness (or head) and per unit area.

Permeability

The pure water permeability of a geotextile cannot always be determined at laminar flow conditions and should therefore be described by the general quadratic flow equation (from the Forchheimer equation):

$$H = av + bv^2 \qquad \text{Eq. 67}$$

where,

H represents the head loss across the geotextile in m;
v the filter velocity in m/s;
a product-depending resistance coefficient in s; and
b product-depending resistance coefficient in s^2/m.

The second term of Eq. 67 is negligible at lower flow velocities and the laminar flow equation results in:

$$H = a\,v \qquad \text{Eq. 68}$$

while the first term is negligible at higher flow velocities resulting in the turbulent flow equation:

$$H = b\,v^2 \qquad \text{Eq. 69}$$

The relationship between the head loss H and the filter velocity v can also be expressed by the equation:

$$H = c\,v^n \qquad \text{Eq. 70}$$

where,

c is the resistance coefficient ($s^n\,m^{1-n}$); and
n the exponent varying between 1 and 2 depending on the flow type:
laminar flow: $n = 1$
turbulent flow: $n = 2$
transition zone: $1 < n < 2$.

Eq. 70 can also be written as:

$$i = c_i \, v^n \qquad \text{Eq. 71}$$

with,

$$i = \frac{H}{T_g} \qquad \text{Eq. 72}$$

where,
i is the hydraulic gradient across the geotextile;
T_g the geotextile thickness in m; and
c_i resistance coefficient in sn m-n.

For laminar flow is n = 1 and Eq. 71 can be written as (see also section 5.1.3):

$$v = K_n \, i \qquad \text{Eq. 73}$$

where,
K_n the permeability coefficient or the filter velocity for a hydraulic gradient of 1.

Permittivity
Also Eq. 68 can be written in that way. Only in the case of laminar flow can the K_n-value be determined and from that the permittivity Ψ, being the permeability per unit of thickness or the filter velocity per unit of hydraulic head loss:

$$\Psi = \frac{K_n}{T_g} = \frac{v}{H} \qquad \text{Eq. 74}$$

where,
Ψ is the permittivity in s^{-1}

For most coarse-textured geotextiles laminar flow is not obtained. In these cases the water permeability of geotextiles can be characterised in two ways:
- the hydraulic head loss at a given filter velocity ; and
- the filter velocity or discharge rate at a given head loss.

The unknown quantities a and b of Eq. 67 and c and n of Eq. 70 can be determined from the best-fit curve through the paired experimental data v and H. Values of H calculated for a given v or inversely of v calculated for a given H according to both equations do not differ significantly.

As the viscosity of a liquid influences its flow, a temperature correction will be necessary if experiments are carried out at temperatures T different from the reference temperature R. For laminar flow a linear relationship exists between the velocity at reference temperature, v_R, and the velocity at the measured temperature, v_T, namely:

$$v_R = R_T\, v_T \qquad \text{Eq. 75}$$

in which $R_T = \eta_T/\eta_R$ is the temperature correction factor given by the ratio of the dynamic viscosity of water at the measured temperature, η_T, and at reference temperature, η_R. The dynamic viscosity η_T of water at a temperature T (°C) is, according to Poiseuille, given by:

$$\eta_T = \frac{1.779}{1 + 0.03368\,T + 0.00022099\,T^2} \qquad \text{Eq. 76}$$

where,

η_T dynamic viscosity at temperature T in mPa s

Not considering the flow regime, the temperature correction for Eq. 67 results in:

$$H = \frac{a}{R_T}\, v_R + b\, v_R^2 \qquad \text{Eq. 77}$$

and for Eq. 70 in

$$H = c\, R_T^{-n}\, v_R^n \qquad \text{Eq. 78}$$

This temperature correction does not apply to Eq. 67 and Eq. 70 (Bezuijen *et al.* 1994, Dierickx and Leyman, 1994), because the viscosity does not exert an important influence on the water flow under turbulent flow conditions and should not be considered for turbulent flow. Since Eq. 67 consists of a fictive laminar and turbulent part, temperature correction of the laminar part may suffice for a correct temperature correction of the water permeability parameter (Dierickx and Leyman, 1994). Hence the quadratic equation, after taking into account a certain reference temperature, can be written as:

$$H = \frac{a}{R_T}\, v_R + \frac{b}{R_T^2}\, v_R^2 \qquad \text{Eq. 79}$$

For Eq. 70 the splitting into a fictive laminar and turbulent part is not so evident but knowing that the temperature correction for laminar flow is R_T and that for turbulent flow no correction is required, the following exponential flow formula is applicable according to Dierickx and Leyman (1994):

$$H = c\, R_T^{n-2}\, v_R^n \qquad \text{Eq. 80}$$

Permeability determination is executed by one of two methods: constant head or falling head. These are briefly described in the next sections.

Constant head method.
A unidirectional flow of water normal to the plane of a single layer of geotextile, without load, is applied under a range of constant heads. A geotextile specimen is installed between two flanges in a tube. A manometer at both sides allows the determination of the head loss across the geotextile at various discharges. The determination of the permeability of geotextiles according to the constant head principle is described in EN ISO 11058 (1999) and ASTM D 4491[32]. The required parameters can be calculated from the head loss, discharge and temperature measurements.

Falling head method.
A unidirectional flow of water normal to the plane of a single layer of geotextile or related product, without load, is applied according to the falling head procedure. The principle, possible registration methods and data analysis are described in EN ISO 11058 (1999) and ASTM D 4491[32].

Concluding remarks
As temperature correction for flow velocity is only applicable at laminar flow, and flow in the transitory zone is found mostly between laminar and turbulent, it is advisable to carry out permeability measurements within a narrow range of 18 - 22° C close to the reference temperature of 20° C, to minimise the error introduced by applying the temperature correction, regardless of the flow regime.

Typical values of permittivity are 0.02 - 2.2 s^{-1} and corresponding permeability values (permittivity multiplied with thickness T_g) K_n = 0.0008 – 0.23 cm/s (0.0016 - 0.46 ft/min). Conventional soil permeameters cannot be used to determine the permittivity, as the flow coming through would be too large (normally the soil will control the flow). Few US states have permittivity requirements (Koerner 1994). Those that do specified values ranging from 0.1 - 0.7 s^{-1}.

[32] If d_{10} > 0.78 mm (US Sieve No 200) it recommended to use ASTM 2434-68 (1994) for tests on granular material.

However, all of them had requirements for the permeability such as: $K_e>K_s$, and $K_e>10K_s$ for critical (flow) conditions where mono-filament fabrics are used (POA > 4, and AOS < No. 100 sieve). Some required absolute values: $K_e>$ 0.01 cm/s, or 0.1.

5.6.11 Water permeability normal to the geotextile plane with load

When a geotextiles is under pressure it is expected that the flow characteristics will change. Determinations of the permittivity or permeability across the plane under load have been developed, but no conclusive results seem to have been reported to date such that definite guidelines can be given. Koerner (1994) reports the following trends:

- for woven mono-filament geotextiles there is no change to a slight increase when under load (it would seem that pressure stretches the weave; POA increases);
- for woven slit film geotextiles there was too much scatter in the data to report trends;
- for non-woven heat bonded geotextiles there is no to slight decrease when under load; and
- for non-woven, needle-punched geotextiles there is a slight to moderate decrease depending on the magnitude of the load.

ASTM D5493 describes testing under load and normal to the plane of the geotextile. ASTM D 4491 only deals with permittivity without load. Similarly EN ISO 11058 (1999) only deals with unloaded geotextiles, however, there is work planned in the CEN/TC 189 programme to test geotextiles under load and flow perpendicular to the plane (Dierickx personal communication). Only French guidelines specifically mention the effect of loading (Santvoort 1994): they suggest that permittivity determined in the laboratory (no load) is at least 3 times higher than with load. It is assumed that the specification values mentioned in the previous section, permittivity without load, are actually valid for conditions under load, although this is not mentioned specifically. The typical value range for permittivity under load (Koerner 1994) is 0.01 - 3.0 s^{-1}, which is rather close to the values without load (see Section 5.6.10).

5.6.12 Water flow capacity in-plane

The water flow capacity in the plane of a geotextile or related product is determined at a range of constant heads. Because the flow of water within the plane will be influenced by the applied load, it is necessary to specify the load at which the water flow capacity is determined (ASTM D 4716 suggests 10, 25, 50, 100, 250 and 500 kPa). Normally the water flow capacity is measured under varying compressive stresses, typical hydraulic gradients and defined contact surfaces. Existing test equipment can be classified in parallel (ASTM) and radial flow models (Figure 63). Parallel models are preferred when the fabrics have preferential flow directions. Radial models give an average as flow occurs in all directions. For both flow models the relation between the head loss H and the filter velocity v is given by Eq. 63 or Eq. 70, the parameters a and b or c and n depend on the flow regime.

Linear regression between H and v exits in the case of laminar flow, hence Darcy's law is applicable.

For parallel flow, taking into account that the flow section A is given by:

$$A = T_g \, w \qquad \text{Eq. 81}$$

where,

w the width; and

T_g the thickness of the geotextile specimen.

The permeability coefficient in the plane K_p can be calculated as follows:

$$K_p = \frac{V}{T_g \, w \, t} \frac{L}{H} R_T = \frac{Q}{T_g \, w} \frac{L}{H} R_T \qquad \text{Eq. 82}$$

The transmissivity, θ, being the discharge rate in the plane of the geotextile per unit hydraulic gradient and per unit width can be obtained from:

$$\theta = K_p \, T_g \qquad \text{Eq. 83}$$

For radial flow, the permeability in the plane is obtained by applying the radial flow formula:

$$H = \frac{Q}{2 \pi K_p T_g} R_T \ln\left(\frac{R}{R_0}\right) \qquad \text{Eq. 84}$$

with R the outer and R_0 the inner radius of the geotextile specimen. Hence,

$$K_p = \frac{Q}{2 \pi H T_g} R_T \ln\left(\frac{R}{R_0}\right) \qquad \text{Eq. 85}$$

from which the transmissivity can be calculated according to Eq. 83.

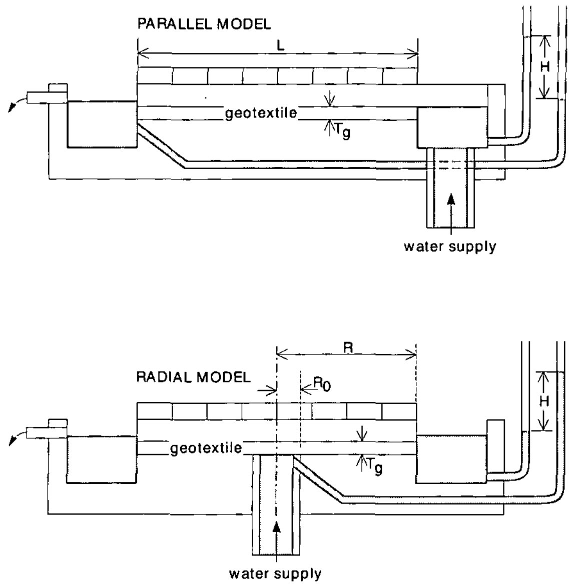


Figure 63 Parallel and radial flow model for permeability determination of a geotextile in its plane.

To determine the flow regime under specific test conditions at least 5 hydraulic gradients are required for each applied normal load. The flow regime is considered laminar if a linear relation between head loss and discharge exists. The best-fit curve passing through the origin:

$$H = a_1 \, Q \, R_T \qquad \text{Eq. 86}$$

allows determination of the regression coefficient a_1 and, consequently, the permeability K_p in the plane and the transmissivity q for each applied normal load.

For parallel flow it follows from Eq. 82 that:

$$a_1 = \frac{L}{K_p \, T_g \, w} \qquad \text{Eq. 87}$$

hence,

$$\theta = K_p \, T_g = \frac{L}{a_1 \, w} \qquad \text{Eq. 88}$$

For radial flow it follows from Eq. 84 that:

$$a_1 = \frac{\ln\left(\frac{R}{R_o}\right)}{2\,\pi\,K_p\,T_g} \qquad \text{Eq. 89}$$

hence,

$$\theta = K_p\,T_g = \frac{\ln\left(\frac{R}{R_o}\right)}{2\,\pi\,a_1} \qquad \text{Eq. 90}$$

Typical examples of the influence of the normal load on the transmissivity of geotextiles are given in Figure 64.

Based on Figure 64B Koerner (1994) concluded that most geotextiles reach a constant value for transmissivity at loads of 24 kPa and greater. The yarn structure is then tight and dense to hold the load and maintain hydraulic properties. However, Figure 64A does not show this phenomenon.

In the European standard (EN ISO 12958, 1999) parallel flow is accepted and from the simple figure in ASTM D 4716 parallel flow is also intimated. The geotextile or related product should be installed between two foam rubbers fixed on parallel plates and loaded with a constant normal stress. The supply side is connected to a constant head reservoir of which the water level can be adjusted; the outlet has a constant overflow level. Two manometers, one at the supply and one at the outlet allow determination of the head loss within the geotextile. To make temperature corrections, the discharge has to be determined and the temperature measured. Water flow capacity should be determined at 4 normal loads (ISO, ASTM recommends 3 loads selected from the values given above) and for each load at two hydraulic gradients. To obtain representative results at least 3 replicates are required. Typical values for transmissivity are (Koerner 1994): $0.01 - 2.0 \times 10^{-3}\ m^3min^{-1}m^{-1}$. None of the US states are reported to require transmissivity values.

Since measurements are not always done under laminar flow conditions, especially for geotextile-related products with a central core, only the in-plane flow capacities per unit width at defined loads and hydraulic gradients are determined. Transmissivity can only be used in case of laminar flow, and since this is not always the case with geotextiles or geotextile related products, the EN ISO standard does not use transmissivity but water flow in the plane.

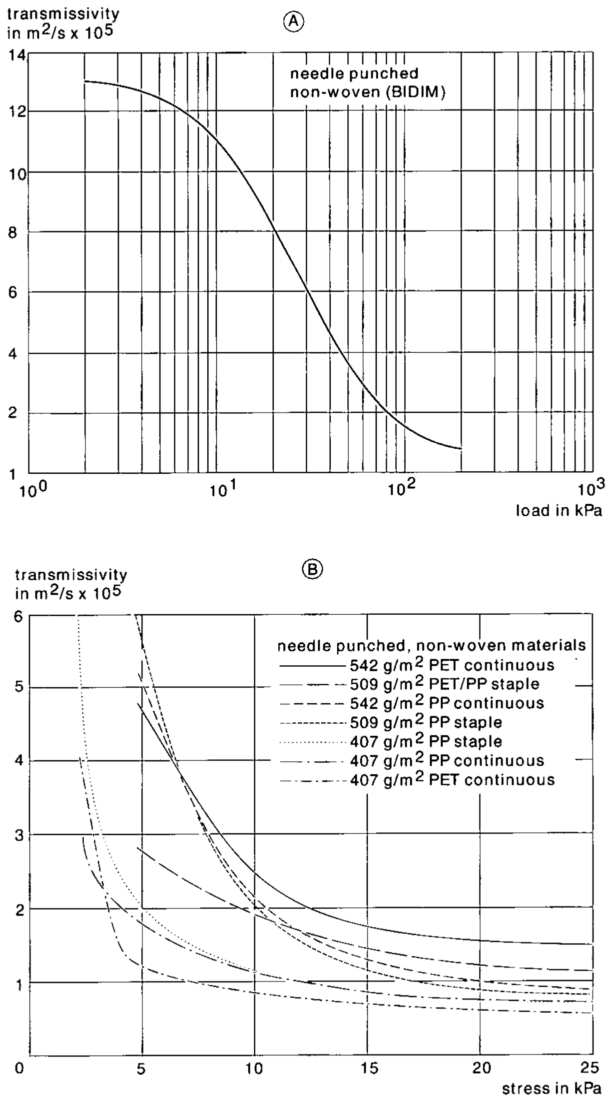


Figure 64 The influence of load on transmissivity.
A Dierickx and Vlotman (1995).
B Transmissivity response versus applied normal stress for various needle punched non-woven polyester (PET) and polypropolene (PP) geotextiles (after Koerner 1994).

5.6.13 Water penetration resistance

In France in 1981, problems that occurred with drainage systems that had geotextile envelopes were attributed to resistance to wetting of geotextiles (Lennoz-Gratin 1987). The phenomena seemed typical for fine textured non-woven geotextiles. Since then tests have been developed to assess the phenomena (Lennoz-Gratin 1992, Dierickx 1994, 1996b). The test on water penetration resistance determines the hydrostatic head supported by a geotextile which is a measure of the opposition to the passage of water through the geotextile. This is done by subjecting a geotextile specimen to a steadily increasing water head on the face - under standard conditions - until water penetrates at three places, and noting the corresponding head. The apparatus for determining the water penetration resistance is shown in Figure 65. The complete test procedure is described in ISO 811 (1981, and equivalent BS 2823-82) and a new EN standard (prEN 13562, 1999), possibly the same with minor modifications, is under preparation.

Laboratory tests on wettability problems showed that the initial heads varied between 5 and 30 mm (Dierickx 1996b), which seems hardly enough to be of practical value.

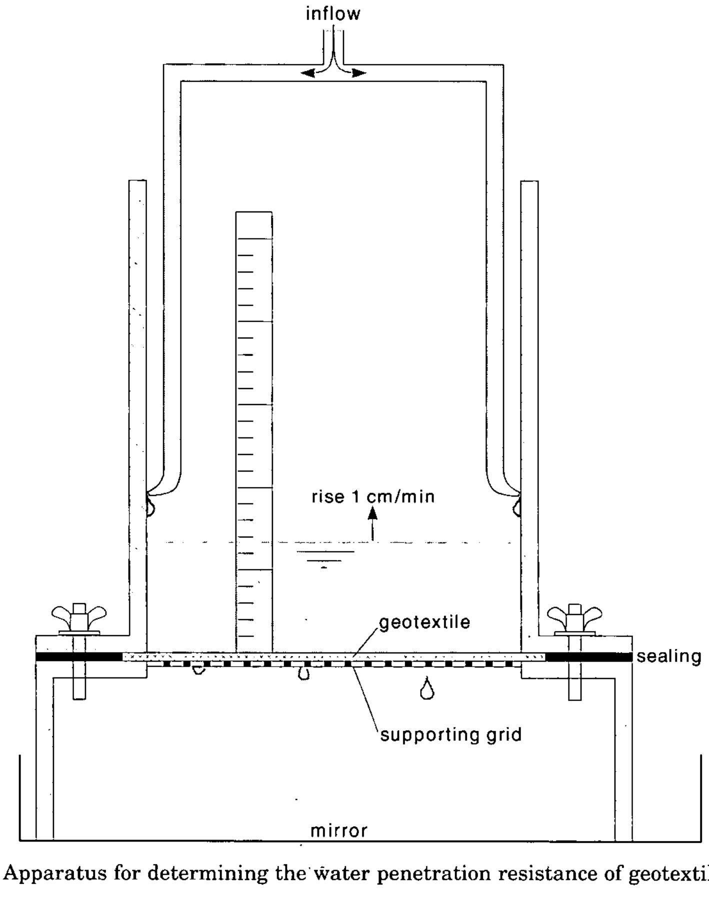


Figure 65 Apparatus for determining the water penetration resistance of geotextiles.

5.6.14 Degradation and durability properties

There are a number of processes that have a bearing on how long a geotextile lasts. They are (Koerner 1994): temperature, oxidation, hydrolysis, chemical, biological, and sunlight (UV) degradation. Ageing, which is the alteration of physical, chemical, and mechanical properties caused by the combined effects of environmental conditions over time, is another term used to describe degradation. Ageing manifests itself in numerous ways (ASTM D 5819): blistering, chalking, changes in chemical resistance, in puncture, burst and tear resistance and other index properties, discoloration, embrittlement, permeability changes, stiffness changes, and tensile or compressive changes.

McKyes *et al.* (1992) describes various geotextiles tested in Canada, mentioning that only the fibreglass fabrics more commonly used in the past for envelope material seemed to become brittle over time and collapsed when exhumed. Several general types of guidelines to assess the degradation and durability of geotextiles have been issued or are under preparation: ASTM D 5819, and ENV 12226 (1996). To obtain exhumed specimens the following guidelines are issued: ASTM D 5818, and EN ISO 13437 (1998).

Temperature. Higher temperatures will cause polymer degradation systems to occur at a higher rate, while low temperatures will cause brittleness which impacts strength on the material. ASTM D 4594 describes a test to determine the effect of temperature on the stability of the geotextile. The stability is expressed as a percentage change in tensile strength or in elongation as measured at specific temperature and compared to values obtained at standard conditions for testing geotextiles (i.e. Section 5.6.4). ASTM D 746-95, which deals with the effects of cold temperatures on plastics, also addresses brittleness.

Oxidation. All types of polymers are susceptible to some form of oxidation, but polypropylene and polyethylene (PP and PE) more so than others (Hsuan *et al.* 1993). The European Committee for Standardisation (CEN) has drafted a pre-standard for testing resistance to oxidation (EN ISO 13438, 1998). ASTM has a recommended practice for high temperature oxidation testing for plastics (ASTM D 794-93). With drain envelope applications little difficulty, if any, is expected of oxidation processes.

Hydrolysis. Hydrolysis is the chemical reaction between a specific chemical group within a polymer and absorbed water causing scission and reduction in molecular weight. This can affect yarns internally or externally (Hsuan *et al.* 1993). Polyester geotextiles are chiefly affected when exposed to high alkalinity values. High pH affects some polyesters, while low pH is more harsh on polyamides. The European Committee for Standardisation (CEN) is working on resistance to liquids. Resistance of geotextiles to hydrolysis can be determined according to ENV 12447 (1997).

Chemical. Manufacturers have tested a wide range of materials such as acetate, nylon and rayon, with chemicals such as sulphuric acid, hydrochloric acid, organic acids, bleaching agents, and salt solutions. ASTM Committee D35 is also working on chemical degradation assessment of geotextiles after either laboratory (D 5322) or field immersion (D 5496) procedures. CEN has drafted a pre-standard: ENV ISO 12960 (1998) to determine the chemical resistance of geotextiles. These types of tests will be particularly useful when the materials are used with landfills to assess the effect of leachates.

Biological. Bacteria and fungi can deteriorate polymers if they can attach themselves to the yarn and use the polymer as feedstock. This however is unlikely for the commonly used polymeric resins. In Europe a pre-standard ENV 12225 (1996) is currently available as a prospective standard for provisional application, which describes measurement of micro-biological resistance by a soil burial test. ASTM prescribes no methods, except the biological clogging (ASTM D 1987) which is more a mechanic problem rather than biological one. Ionescu *et al.* (1982) described test results on four PP, one PE and a composite geotextile when subjected for 5 - 17 months to distilled water (the control test), sea water, compost, a particular soil, iron bacteria, laven-synthesizing bacteria, desulfovibrio bacteria and a liquid mineral. Results showed no measurable effect of permeability (although some roots were growing through the geotextile), only small variations in tensile strength, and no structural changes visible on infrared spectroscopy.

Sunlight (UV). Degradation by sunlight or ultra violet rays (UV) is of importance because of storage of drain envelope material onsite, just before construction, and when improperly stored (e.g. in direct sunlight). The UV region of light is subdivided in UV-A (wavelength 400 - 315 nm), which causes some polymer damage, UV-B (315 - 280 nm), which causes severe damage to polymers (Cazuffi *et al.* 1994), and UV-C (280 - 100 nm). The last-mentioned is only found in outer space. According to Santvoort (1994) PE is most sensitive to 300 nm, PE to 325 nm, and PP to 370 nm UV wavelengths. Obviously geographic location, temperature, cloud cover, hours sunlight per day, wind and moisture content, all play a role in UV degradation. Draft European standards on testing of pipes note that material changes be expected only after a total radiation of 3.5 GJ/m^2 has taken place (document 155N677, or standard ENV 12224-96). In the Netherlands average annual radiation is 3.4 GJ/m^2, while in Egypt it is 6.86 GJ/m^2 (Omara and Abdel-Hady, 1997). ASTM provides guidelines on how to test for UV degradation in G23 (Carbon arc), D 4355 (Xenon arc), G53 (UV Fluorescent), D 5208 (UV Fluorescent), and D 1435 (Outdoor weathering of plastics). A number of authors have recently reported on UV effects: Cazuffi *et al.* (1994), Hsuan *et al.* (1994), McGown and Al-Mudhaf (1994), and Tisinger *et al.* (1994).

Geotextiles exposed to natural weathering are vulnerable to degradation (ageing) and it is therefore useful to test the effect of weathering on the properties of geotextiles. Since natural weathering requires testing over a long duration, information could be obtained more rapidly by accelerated procedures using weathering devices with specific artificial light sources. Specimens of the material to be tested are exposed to a light source for a defined exposure time at recommended temperature and moisture conditions. After exposure the decay performance of the specimens can be determined. The accelerated test procedure and the required equipment is described in ENV 12224 (1996), ASTM D 3083, ASTM D 5397, and ASTM D 4355, to name a few.

5.7 Indicator Tests: Soil-Envelope Laboratory Tests

Tests with soil and envelope materials aim at determining the need for drainpipe envelope materials for particular soils and, subsequently, the selection of the most suitable envelopes. The tests are used to either investigate soil-envelope properties or the results are used directly to select the desired envelope material. The tests allow study of the functional properties of drainage envelopes that include:

- the ability to restrain soil particles and to prevent a massive invasion of soil particles into the drainpipe (for granular, organic and synthetic envelopes);
- the blocking of envelopes, which is the immediate reduction in water permeability of an envelope brought in contact with a soil (generally only with synthetic envelopes); and
- the clogging of envelopes, which is the time dependent decrease in water permeability of an envelope and can result from trapped soil particles in the envelope or from an accumulation of fine particles, which can happen with granular and synthetic filters.

The tests described in these sections are simple tests of the possible ways of testing soil-envelope-drain combinations in the laboratory. They are indicator tests: situations that can be encountered in the field are simulated in the laboratory and results give an indication of what might happen in the field. Results are not necessarily directly transferable (i.e. hydraulic conductivities or head loss measured in the laboratory may not be related to values expected in the field). This is also the main criticism of these types of tests and the reason why various researchers in the past and recently have been using soil (sand) tanks (Figure 66) in the hope of getting results that would be more directly applicable to field conditions (Cavelaars 1965, Knops 1979, Zuidema and Scholten 1979, Lennoz-Gratin 1992, Kumbhare *et al.* 1992, Wenberg and

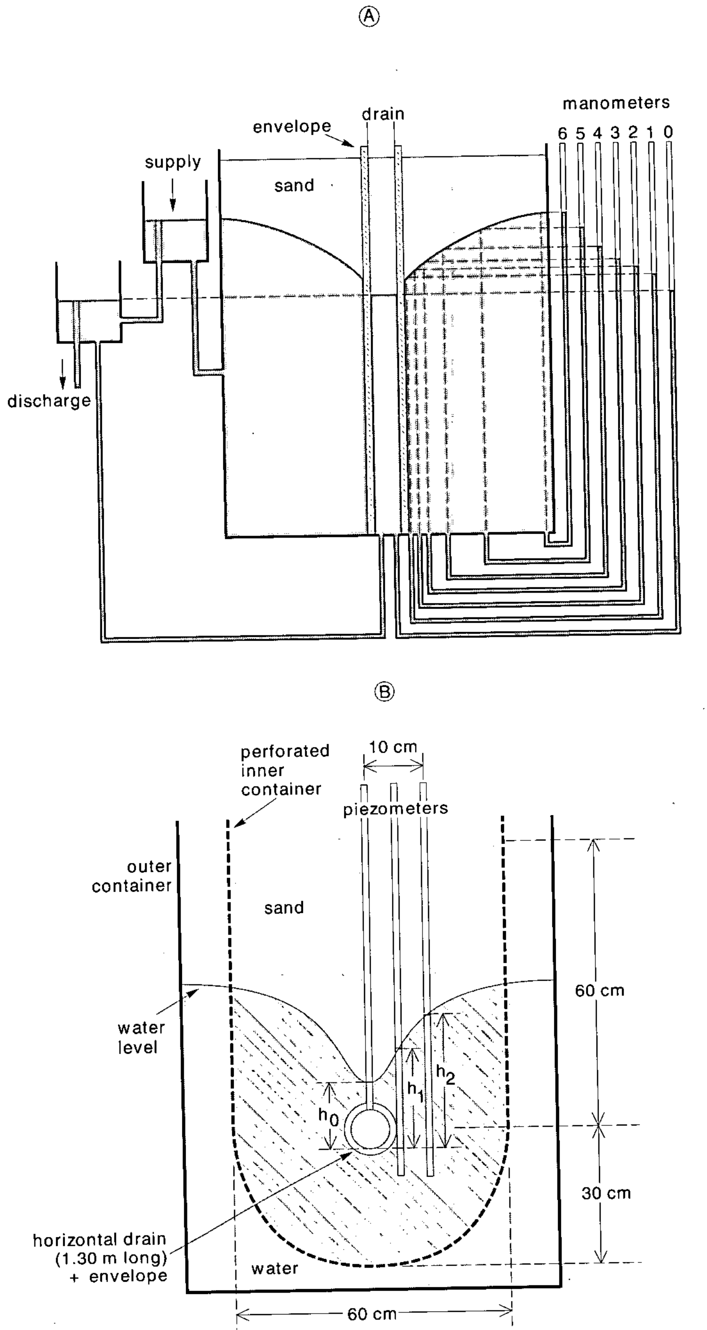


Figure 66 Examples of soil/sand tanks.
A Schematic of cylindrical vertical sand tank (Knops 1979).
B Cross section of horizontal sand tank (Knops 1979, Scholten 1988).
C Sand tank after Eichenauer *et al.* (1994).

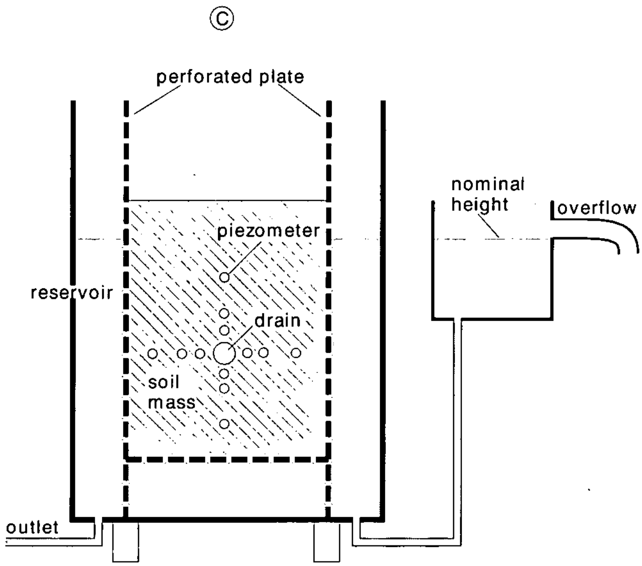


Figure 66 Cont. Examples of soil/sand tanks.
C Sand tank after Eichenauer *et al.* (1994).

Talbot 1992). However, on reviewing the literature (Stuyt 1992, Wenberg and Talbot 1992) most of the tests in soil tanks were shown to have a variety of drawbacks, which made it difficult to relate the test results directly to the field. Moreover, they were not simple to execute. This led Stuyt (1992) to conclude that only field tests can provide the final and definitive answer. Nevertheless, indicator tests using permeameters are useful and may be prescribed (Koerner 1994), while they are essential for research.

In France, AFNOR standards exist for both sand tank filtration tests (AFNOR NF U 51-162, 1990) and for permeameter tests (AFNOR NF U 51-161, 1990 in Lennoz-Gratin 1992). Some of the key results used in granular filter design was based on the work by Sherard *et al.* (1984a and b), which was extended for use with geotextiles by Lafleur *et al.* (1994, Figure 67). Only permeameter indicator tests will be described in the sections following. A major difference between permeameters used by various researchers, or prescribed in the various guidelines, is whether the flow through the permeameter is upward or downward. In Belgium, the Netherlands, France, Pakistan, and Egypt, the upward flow permeameter is generally favoured (Dierickx and Yüncüoglu 1982, Stuyt 1992, Lennoz-Gratin 1987, 1992, Vlotman *et al.* 1990, 1992, DRI staff 1992). The downward flow permeameter is more common in the USA, as well as the Gradient Ratio test (Koerner 1994) which has been used in Australia (Li *et al.* 1994) too. Samani and Willardson 1981 used an upward flow permeameter for HFG determination in the USA.

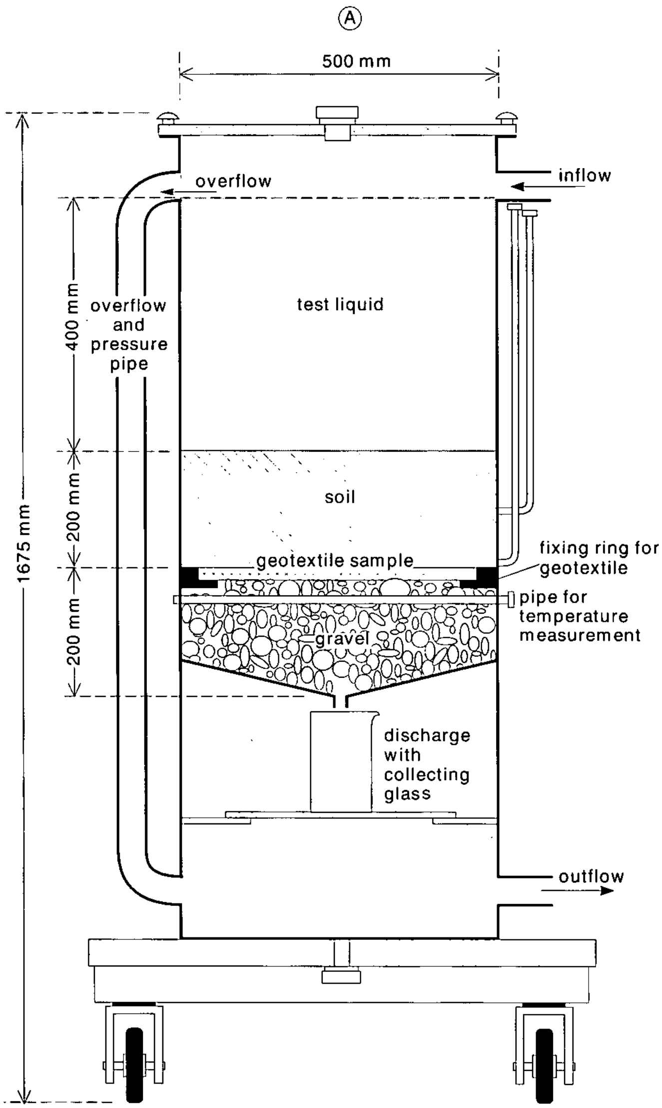


Figure 67 Examples of permeameters.
A Large-scale permeameter (Gartung *et al.* 1994).

The main argument in favour of using the upward flow permeameter is that the condition most likely to cause the most unstable situation is where upward forces (flow drag) balance the gravity effects (critical gradient of approximately 1). This situation occurs below the drainpipes and generally only when the water table is not above the pipe or the pipe is flowing full with a water table above the pipe (Figure 38a and b). When the water table is above the pipe and the pipe is not flowing full, gradients considerably higher than

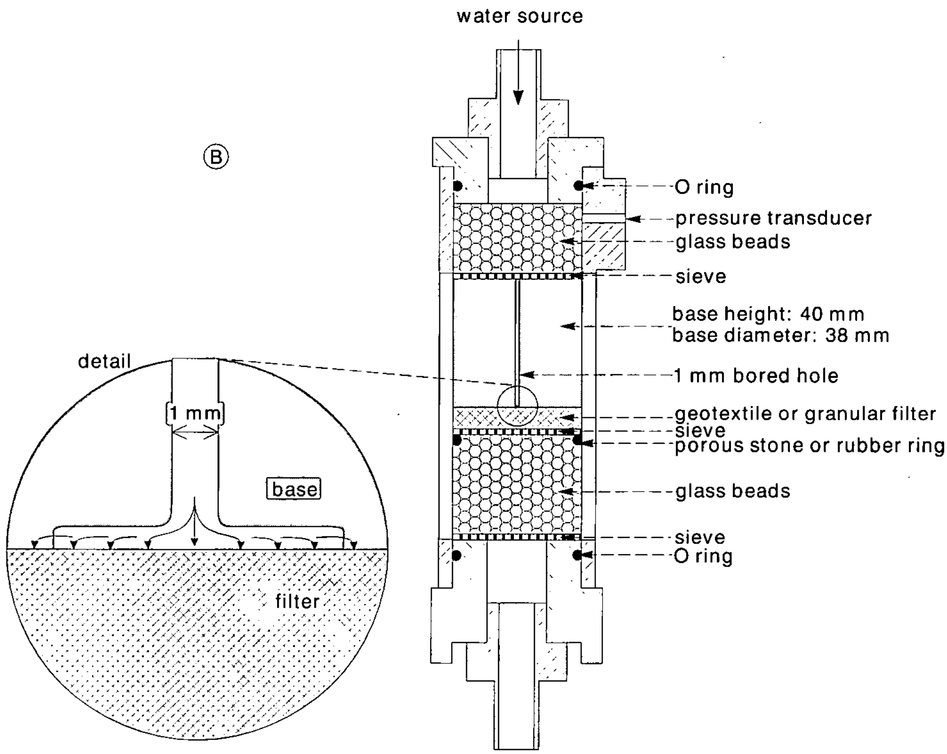


Figure 67 Cont. Examples of permeameters.
B No erosion filter test (Lafleur *et al.* 1994) for geotextiles and slot and hole test (Sherard and Dunnigan 1998) principle for granular filters (see detail).

1 can occur and the forces caused by friction and bridging against the drag forces will be the dominant factor in preventing soil particles from flowing into the drainpipe. This situation will occur all around the pipe; the maximum forces, however, will be at the top of the pipe where gravity and drag forces are complementary. Here, a downward flow permeameter would have a slight advantage in that it can test slightly higher gradients than the upward flow permeameter. In view of the fact that the unstable situation will occur before the situation with high forces, the permeameter with upward flow would have the advantage as it is capable of testing both failure situations. It is also important to keep a close eye on what is happening in the laboratory at the interface at gradients close to 1.

In the following sections the most common laboratory tests for hydraulic performance of the soil-envelope combinations, such as the Long-term Flow Test (LTF), the Gradient Ratio Test (GR) and the Hydraulic Conductivity Ratio Test (HCR) will be described briefly. This is followed by a section about a comprehensive permeameter testing methodology, which achieves most if not all of the objectives of the tests mentioned above (LTF, GR and HCR). Although this last section is research oriented, abbreviated applications of the test procedure will also satisfy the indicator function mentioned earlier.

5.7.1 Long-term flow test: clogging test

One of the main questions that primarily relates to synthetic envelopes is the expected behaviour over a long time (10 years); the possibility of gradual clogging due to fines becoming lodged in the pores of the synthetic fabric. This is primarily a potential problem with non-woven synthetics. Koerner (1994) describes a Long-Term Flow test (LTF test) of at least 500 hrs, preferably up to 1000 hrs (40 days) using a downward flow permeameter (Figure 68). Chin *et al.* (1994) performed tests lasting up to 2000 hrs. When discharge versus

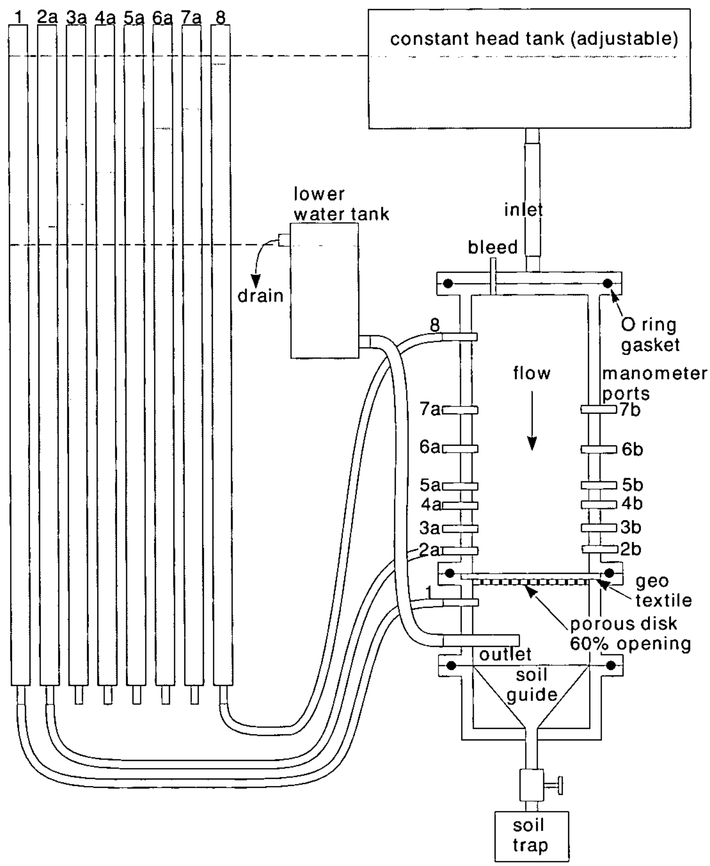


Figure 68 Example of Long-Term Flow test apparatus.
Example shown has manometer ports protruding in the soil sample. (after Li *et al.* 1994) and can be used for the GR test as well.

time was plotted a decrease in flow during the first 10 - 200 hrs was observed as a function of soil compaction, after which the soil-envelope combination started to be dominant and the discharge became steady for acceptable soil-envelope combinations (Figure 69). When the discharge continued to decrease over time, indicating progressive clogging, the synthetic envelope was not deemed appropriate for the particular soil. The major drawbacks of this test are its duration, the potential of algae growth in the cylinders, and the need for de-aerated and/or de-ionised water for the duration of the test. A test that takes considerably less time is the Gradient Ratio test pioneered by the US Army Corps of Engineers (Test CW-02215).

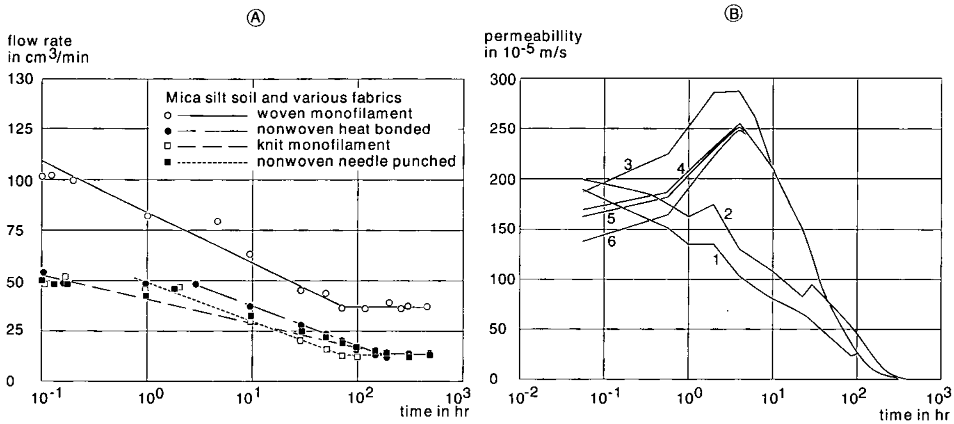


Figure 69 Example of Long-Term Flow test results.
A After Koerner (1994).
B After Li *et al.* (1994). The numbers indicate the permeameters used. Permeameters 1,4 and 6 had extended manometer ports (protuding into the soil sample, Figure 68), while the others had manometer ports that were flush with the inside of the permeameter.

5.7.2 Gradient ratio test: clogging test

ASTM has adopted the test developed by the Army COE with minor modification as the D 5101-90 Test Method (Figure 70). ASTM D 5101-90 gives clear descriptions of the procedures to follow but does not give the indicator values for judging the results (i.e. GR < 3 is based on the Army COE findings). The GR test uses a downward flow permeameter. Instead of measuring flow rates the hydraulic head at various locations in the soil-envelope columns are measured and the following gradient ratio calculated:

$$GR_{25} = \frac{\Delta h_{es+1S} / l_{es} + 1.0}{\Delta h_{2S} / l_{2.0}} \qquad \text{Eq. 91}$$

where,

GR_{25} represents the Gradient Ratio with downward flow permeameter including 25 mm of base soil with the 'envelope' gradient: i_{es};

Δh_{es+1S} the head change from the bottom of the geotextile to 25 mm (1 inch) of soil above the geotextile;

$l_{es+1.0}$ geotextile thickness (T_g) plus 25 mm (1 inch) of soil;

Δh_{2S} head change between over 50 mm (2 inches) of soil above the geotextile; and

$l_{2.0}$ 50 mm (2 inches) of soil.

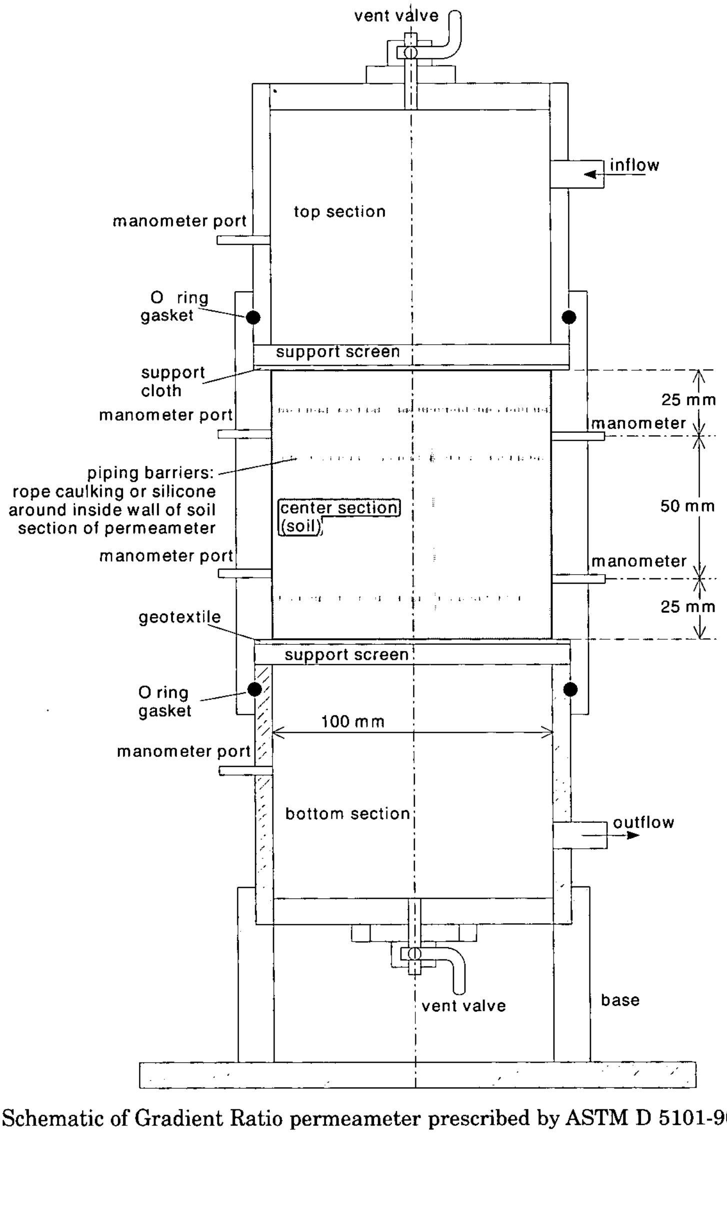


Figure 70 Schematic of Gradient Ratio permeameter prescribed by ASTM D 5101-90

When $GR_{25} > 3.0$ the geotextile is not acceptable for the type of soil (US Army Corps of Engineers 1977). Tests performed by Haliburton and Wood (1982, in Koerner 1994) indicated that woven mono-filament fabrics had GR_{25} ratios that remained well below 3 for constructed soils with silt up to 40%. Other fabrics (including also one woven mono-filament, as well as a non-woven needle punched material) had GRs greater than 3 for silt > 20% (Figure 71).

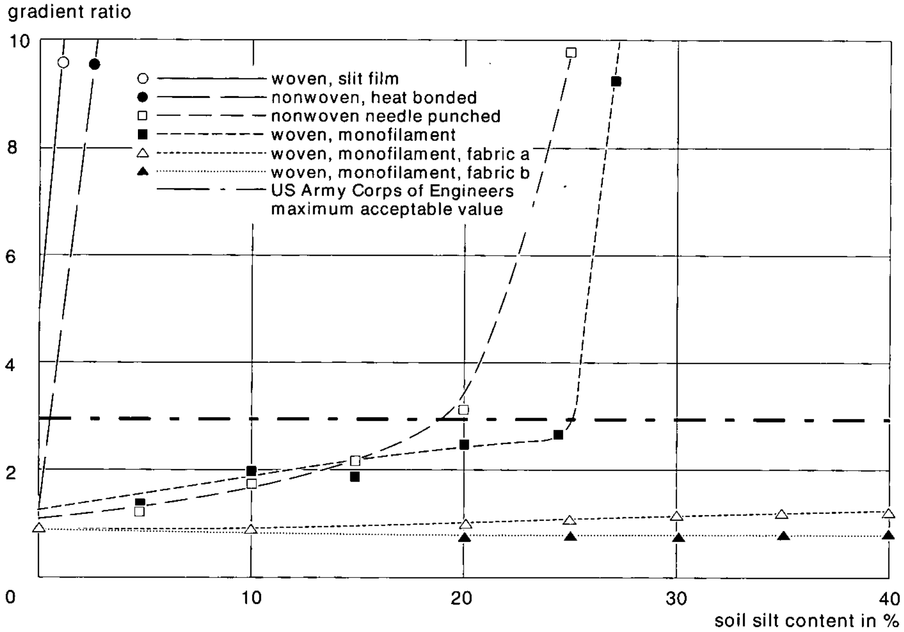


Figure 71 Example of results of Gradient Ratio tests.
(Haliburton and Wood 1982 in Koerner 1994).

As with the LTF test problems with stability of the GR over time, air pockets in the soil and piping along the test cylinder walls were reported. Williams and Luettich (1990) list the following disadvantages:

- no effective confining stress is placed on the soil;
- the soil is rained or tremmied in place which induces segregation;
- sample placement condition may cause difference in hydraulic conductivity by order of 1 - 2 magnitude for the same soil, implying that replicability is questionable;
- structure, density and void ratio of the soil are neither controlled nor measured during the test;
- compressibility, porosity and pore size distribution are not monitored during the test;
- the GR test is practicable only for soils with hydraulic conductivities greater than 10^{-4} cm/s (0.086 m/d);

- side wall piping often occurs, particularly when Ks < 10^{-4} cm/s; and
- absence of back-pressure capabilities often causes incomplete saturation in the sample resulting in poor reproducibility.

Shi *et al.* (1994) improved on the concept of the GR test by improving the procedures for reproducibility of test results. They modified the standard permeameter by adding an energy dissipater below the inlet to prevent disturbance of the top surface of the soil during high flows, and added additional ports above the geotextile specimen to better monitor the variation of head in the sample (at 89.5, 50.8 and 8.0 mm above the geotextile). In addition, sample preparation techniques were modified to enhance replicability of the soil composition in the various permeameters (details of the sample preparation are reported by Fannin *et al.* 1994, while recommended procedures are also given in Section 5.7.4). Rather than using GR_{25} they used GR_8 and modified the indicator value for this to $GR_8 < 7.4$.

The GR_{25} includes 25 mm of soil with the envelope, the GR_8 8 mm. Hence it seems strange to prescribe a $GR_{25} < 3$, which means that the 25 mm of soil in contact with the envelope together with the envelope is allowed to have a K-value as low as 33% of the base soil. With a GR_8 these values are $GR_8 < 7.4$ and a K value as low as 14% of the base soil. Shi *et al.* (1994) conclude, however, that this reduction in K_{es} does not have serious implications for performance. They further remark that if GR_{25} or GR_8 is less than 1, the permeability of the soil-geotextile composite is higher than that of the soil. When the GR is between 1 and the indicator value (3 or 7.4) the permeability of the soil-geotextile composite is less than the soil but is not inhibiting. When the GR is higher than the indicator value, the soil-geotextile composite is prone to potential clogging, but there is little test data to confirm this.

It is further interesting to note that none of the typical permeability requirements of the US states (Koerner 1994) allows K_e/K_s to be less than 1, instead several of the states prescribe $K_e/K_s > 10$. However, Giroud (1982) makes a case for allowing $K_e/K_s > 0.1$, which if Darcy holds, could be interpreted as $i_e/i_s < 10$. Later Giroud (1996) modified the $K_e/K_s > 0.1$ ratio to $K_e > 10\ K_s.i_s$, where the gradient is varied from 1 to more than 10 (more on this in Section 6.2.4, p 285). Although both are mentioned here, the last-mentioned ratio is more correct. The Giroud (1982) reference has since be widely used (e.g. in Fisher *et al.* 1990 and Wilson-Fahmy *et al.* 1996).

Li *et al.* (1994) investigated different set-ups for testing geotextile permittivity in isolation using permeameters with inside diameters of 44, 93 and 193 mm (nominal pipes of 50, 100 and 200 mm, respectively). Also investigated were the effect of inside diameter of the permeameter, soil column height (150 and 250 mm), system hydraulic gradient (varied between 1 and 3.6) and the manometer port type (flush with the inside of the permeameter or protruding into the permeameter, Figure 68, Figure 69B) on system flow rate, permeability and the Gradient Ratio (GR_{25}). The tests were conducted following the Australian Standard for determination of permittivity (AS 3706.9 - 1990). The permeameter was of the downward flow type. They found that there was a significant difference in permittivity of the geotextile between the 44 and 93 mm diameter permeameter (T-test at 1% level) but not between the 93 and 193 mm diameter permeameters. The reason for this was the edge effect in the smaller diameter permeameter (effect of clamping force on the fabric, which reduced thickness of the geotextile near the clamp).

AS 3706.9 prescribes a minimum area of 2000 mm^2 and approach velocities between 0 and 0.035 m/s. To obtain Darcian flow conditions at 20° C the Reynolds number should be less than 2100[33] and for the 44, 93, and 193 mm diameter permeameters velocities should stay below 0.048, 0.023, and 0.11 m/s, respectively. This means that in compliance with AS 3706.9 the maximum diameter allowed is 60.2 mm (2846 mm^2) at v = 0.035 m/s in order for the flow to remain laminar. The larger diameter permeameter gave smaller GR values. Non-protruding, pressure-reading points (ports) gave lower GR values because of reduced edge effects. In the case of Li *et al.* (1994) the GR test could only be used to predict failure of the geotextile and not the soil/geotextile combination. This conclusion seems to be linked to the reported trouble with contamination of the upper soil layer by impurities in the water, which caused the control of the flow to be at the top layer of the soil rather than at the soil/geotextile interface. Chin *et al.* (1994) also reported a similar problem. The test run by Li *et al.* (1994) lasted approximately 500 hrs from which they concluded that only when the test apparatus and conditions are the same as used in establishing the indicator GR ratios, can one use the results for recommending certain materials (e.g. GR >3 can be used as indicator value when ASTM D5101-90 is followed to the letter!). Note, the GR_{25} test does not include a drainpipe section and water (and soil) can flow freely through the geotextiles in contrast to the permeameter test described in Sections 5.7.3 and 4.

5.7.3 Hydraulic conductivity ratio test: clogging test

Another test, using slightly different procedures and a hydraulic conductivity ratio rather than a gradient ratio is the HCR test. This test was suggested by Williams and Abouzakhm (1989) and further evaluated by Williams and Luettich (1990). The test was reported to simulate field conditions (more so

[33] Valid only for smooth circular pipes, see Section 5.1.3, hence the logic of Li *et al.* (1994) is questionable; R_e for soils should be < 1-6 for flow to remain laminar!

than the GR or the LTF test). A flexible wall permeameter customary for the triaxial test (ASTM D 5084) is used. With the geotextile on top of the soil the first downward flow is applied from which K_s is determined. Then flow is applied upward and the combined soil-envelope hydraulic conductivity K_{es} is determined. A hydraulic conductivity ratio of the equilibrium values of K_s and K_{es} is calculated from:

$$HCR = \frac{K_{es}}{K_s} \qquad \text{Eq. 92}$$

where,

K_{es} is the hydraulic conductivity of the soil and geotextile (thickness of soil included is not reported; and

K_s the hydraulic conductivity of the soil.

The hydraulic conductivity of the geotextile can be calculated from:

$$K = \frac{T_g\, K_s\, (HCR)}{T_g + T_s\,(1 - HCR)} \qquad \text{Eq. 93}$$

where,

K_e is the hydraulic conductivity of the geotextile;

T_s the thickness of the unaltered soil in the HCR test; and

T_g thickness of the geotextile used in the HCR test.

For vertical wall drainage applications of geotextiles the HCR should be greater than 0.2 or the hydraulic head against the wall will become too high (Luettich and Williams 1989). A HCR value of 0.2 corresponds roughly with a K_e/K_s ratio of 100, which is a considerably higher ratio than evaluations by Giroud (1982, see also Eqs. 99-103). Tests performed by Williams and Luettich (1990) with three different geotextiles using silty sand (K_s = 0.346 m/d [4 x 10^{-4} cm/s]), poorly graded sand with silt (K_s = 8.64 m/d [0.01 cm/s]), and clean well-graded concrete sand (K_s = 25.9 m/d [0.03 cm/s]) resulted in HCR values of 0.02 - 0.90. Using HCR = 0.2 as the boundary between failing or passing the HCR test correctly predicted failure of the soil for which failure in the field was also documented. Note, no reference is made to agricultural field drainage and hence caution should be used with the HCR = 0.2 boundary value.

High values of HCR suggest soil loss through the geotextile, low values suggest excessive clogging, and intermediate values suggest soil-to-geotextile equilibrium. The limitations are (Williams and Luettich 1990):

- HCR equipment is currently limited to soils with K_s > 86.4 m/d;

- changes in geotextile properties due to ageing, chemical or biological effects cannot be modelled; and,
- initial sample placement must model field placement conditions.

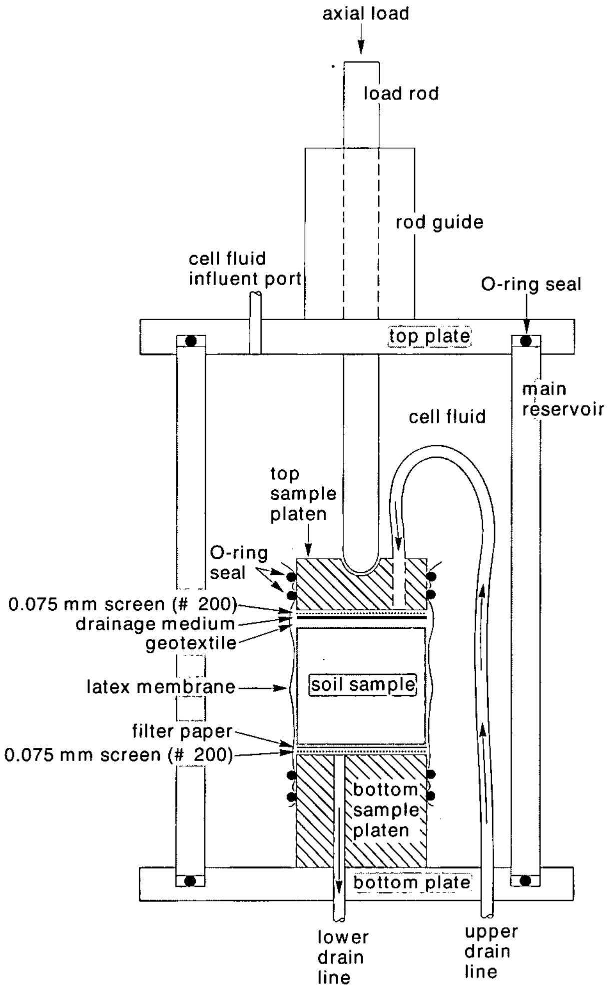


Figure 72 Schematic of Hydraulic Conductivity Ratio test with flexible wall permeameter. (after Williams and Luettich 1990, and ASTM D 5084-90).

The $K_s > 86.4$ m/d above sets this test outside the range for typical agricultural soils with typical K values of 0.002 - 50 m/d (Table 14). This statement seems to be in conflict with ASTM D5084 which suggests that the HCR test be used for soils with $K_s < 0.864$ m/d (0.001 cm/s) and the constant head method for $K_s > 0.864$ m/d (ASTM D2434-68/94). It is not clear whether the recommendations of Williams and Luettich are different because HCR is used with geotextile-soil combinations rather than soil only as described in ASTM

D5084. It is also strange that two of the three soils tested by them are not in the range of the recommendation. It could be a typographical error: $K_s < 86.4$ m/d, in which case the HCR test would be applicable for most agricultural soils.

As the flexible wall permeameter provides better control over effective stress, especially with fine-grained soils, it is expected that the reproducibility of the HCR test will be better than that of rigid wall testing (Williams and Luettich 1990, discussion section).

5.7.4 Recommended permeameter tests with granular and synthetic envelopes

Most of the aspects in the LTF test, the GR test and the HCR test are considered in the permeameter test described in this section. Please note that manometer (tap or tube) and riser tube are used inter changeably. The procedures and methods of analyses are intended for research purposes from which eventually the filter design criteria and/or the laboratory simulation design methods can be finalised. At present, a state-of-the-art range of criteria are available, some of them have been mentioned in foregoing sections, others are elaborated and summarised in Chapter 6. Simple sets of design criteria for the granular and synthetic envelopes selected from among the methods are given in Chapter 3.

A significant difference between this permeameter test and the foregoing three (LTF, GR and HCR tests) is that the flow is upward and that the envelope is pushed against a flattened drainpipe or a drain plate with actual or maximum perforations allowed (Figure 73). Hence it is believed that the test actually simulates field conditions closer than the three downward flow tests described earlier. To test the available envelope materials for research purposes, instead of trying to simulate actual field conditions, a soil type considered most unfavourable (unstable and with a good percentage of fines) is selected. If it satisfies this extreme condition the material is likely to satisfy conditions in the field. Nevertheless, permeameter results cannot directly be related to the field because so many other factors during construction could affect the performance of the envelope.

Materials

The permeameter, made of good quality, clear, Plexiglas, has an internal diameter of 100 mm and is 250 mm high. Visual inspection is important during tests. Ten riser tube taps are placed at selected intervals. Tap no.1 is always below the soil sample and tap no.10 is always above the top plate which simulates the drainpipe (the top plate may actually be a flattened piece of drainpipe, but this is not necessary). Upward flow is applied, the pressure

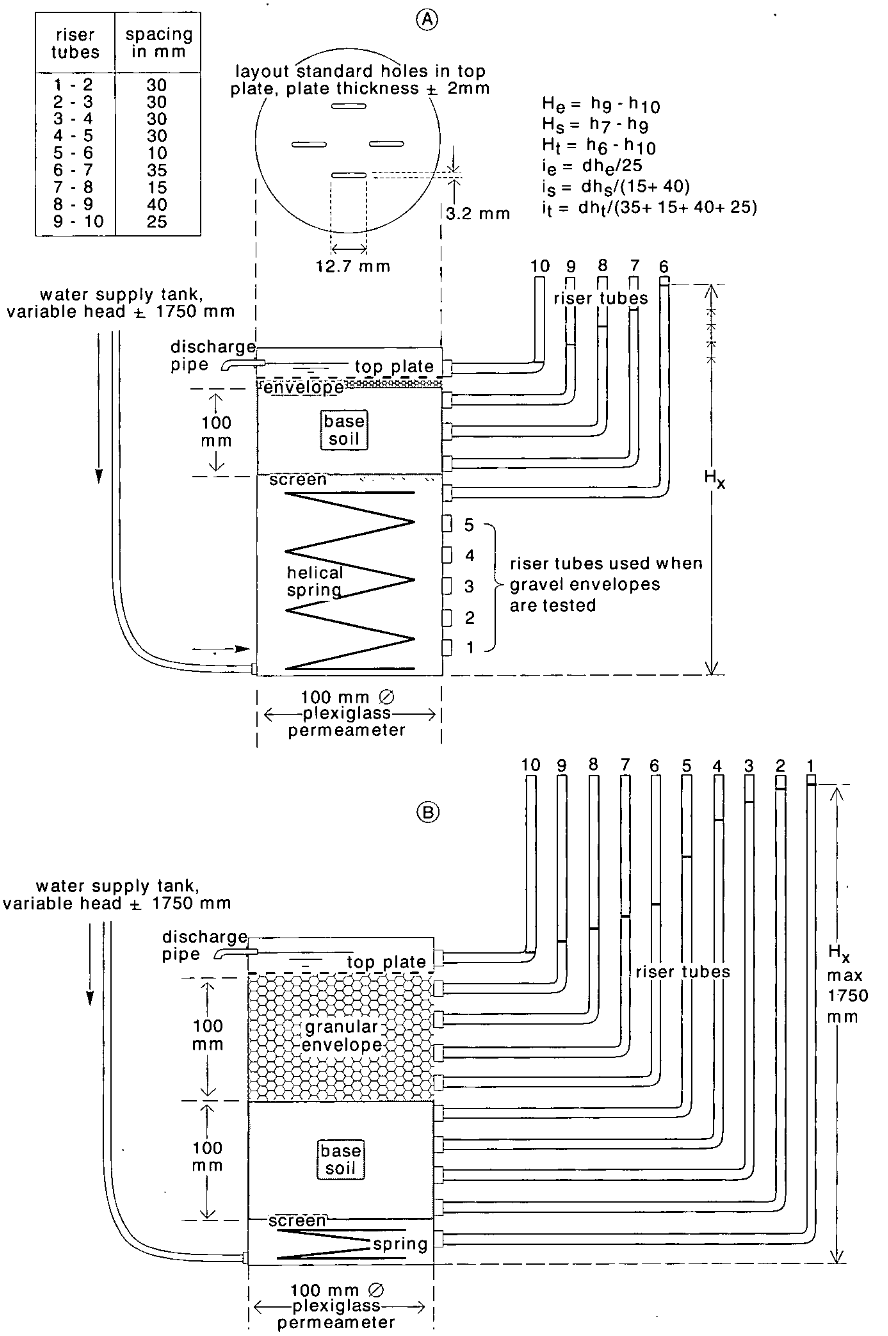


Figure 73 Schema for testing of granular and synthetic envelope with base soil in circular permeameter.
A Setup for synthetic envelope.
B Setup for granular envelope.

of which is controlled by a variable head supply tank. A coarse metal screen with a geotextile material of high permeability supported by a spring pushes the soil and envelope against the drainpipe, and assures that even when soil migrates through the envelope and the top plate, the soil and envelope will remain firmly against the top plate. The fabric on the screen prevents soil from falling through. The centre to centre spacing of the riser tube taps is shown in Figure 73.

The bottom of the top plate is 10 - 12 mm above the centre line of tap no. 9 depending on the thickness of the top plate or flattened drainpipe. As with the GR test it is not possible to measure the properties of a synthetic envelope in the permeameter exclusively: a certain amount of soil will always be included in the measurements between taps 9 and 10. For granular envelopes, on the other hand, the envelope properties can be measured exclusively between tap 9 and those below it. If comparison with the GR test is necessary a special tap 25 mm below the top plate could be installed. Like the HCR test the synthetic envelope properties can be derived using Eq 93 (K_e).

The top plate has four perforations with a combined area of 161 mm^2. The dimensions were based on the maximum allowable perforation size for the Fourth Drainage Project in Pakistan (Vlotman *et al.* 1993c). They can be modified as desired, however, it should be kept in mind that control of water flow should not be determined by the perforations but by the hydraulic conductivity of the soil. For this permeameter an opening area of at least 120 mm^2 was necessary to exclude restricting effects of the top-plate perforations on the hydraulic flow properties of the permeameter (test).

Both standard soil height and gravel height are 100 mm. However, to create higher gradients, the soil height can be reduced to 50 mm but not less to avoid effects caused by uneven compaction during the filling process and reduce piping along the Plexiglas. Piping along the Plexiglas has not been a problem when the filling procedure as outlined below was followed. An envelope layer similar to the intended thickness in the field is recommended. A minimum of three permeameter cylinders connected to the same variable supply tank is recommended.

Base soil preparation and permeameter filling

As mentioned earlier for research purposes the most unstable soil type may be selected for the tests. However, it was found that those who need to accept the laboratory research results (the client), often distrust results based on soils composed in the laboratory with no linkage to the soils encountered in the field. It is therefore recommended to select the most unstable soil encountered at drain depth in the field, and to prepare the disturbed soil sample as outlined below. Make sure that the quantity of soil selected is sufficient for all intended tests and replications, and that the soil used for filling the permeameter is fully analysed following procedures described in Section 5.5.1. The procedure for non-cohesive soil is similar and explained below as well.

Soil sample preparation for slightly cohesive soils

The sample soil should be brought to a moisture content between 8 and 15% in order to be sieved under most desirable conditions.

Sieve manually small amounts of soil, for not more than 30 seconds through sieves with opening sizes of 5 mm, 4.76 mm, 3.36 mm and 2mm (Table 15). No sieving apparatus should be used as this will promote the formation of spherical clods.

All fractions, except the one on the 5 mm sieve, are collected and kept separately from each other in plastic bags containing the specified amount of weight for the fraction. The plastic bags are carefully sealed to obtain homogeneity of the enclosed soil aggregates and to prevent them from drying out during preparation of the sample for each permeameter (minimum of three replications). Using a 40-40-20 composition, a soil sample consists of:

- 40% of the fraction less than 2 mm
- 40% of the fraction between 2 mm and 3.36 mm
- 20% of the fraction between 3.36 and 4.76 mm

The ratio of the fractions can be changed according to requirements. Make up an amount of about 2 kg for one permeameter. Put this quantity in a plastic bag, seal it and turn slowly until a homogeneous mixture is obtained.

The volume of the cylinder to be filled has to be determined. If this volume is 785 cm^3 (cylinder diameter 10 cm, height of the soil column 10 cm), then the amount of dry soil at a density of 1.5 g/cm^3 required to fill the cylinder is 1177.5 g. If the soil has a moisture content of 12% on dry weight base, an amount of 141.3 g/cm^3 of water is in the soil sample and hence a total weight of 1318.8 g has to be put in the cylinder to obtain a density of 1.5 g/cm^3 on dry weight basis.

Filling procedure

The filling of the cylinder is accomplished by experience and the best way is found by trial and error. The art is to fill the cylinder in such a way that the soil surface coincides with the top of the cylinder flange in the case of synthetic envelopes. Therefore the weighed amount of soil is poured onto an open dish and the filling of the cylinder is started in layers of about 1.5 cm each. Each layer of the soil surface is smoothed out and the soil slightly and carefully compacted as equally as possible by means of a stamp. The top is roughened slightly before the next 1.5 cm is placed. The filling is perfect when the mark on the cylinder in the case of gravel envelope or the top of the flange is reached, when the contents of the cylinder appear to be homogeneous, and when all the weighed soil has been used. If there is a shortage of soil or if soil is left, then the filling has to be done over again. Scraping the top by means of a lath can do further corrections of soil surface.

Soil sample preparation for non-cohesive soils

- It is assumed that the PSD of the non-cohesive soil is known; if not, the PSD has to be determined according to the procedure in Section 5.5
- Sieve small amounts (approximately 300 g) of the dry soil sample for 10 min using a sieve apparatus on sieve no. 10 (2.00 mm) to separate small stones that may be present in the non-cohesive sandy base soil.
- Sieving is continued till enough base soil material is available (about 2 kg for one permeameter) to fill the permeameters.
- The passed base soil is put into a plastic bag and thoroughly mixed to achieve homogeneity.
- To know the amount of soil used to fill the permeameter, the total amount of soil as well as the remaining amount after filling the permeameter is weighed.
- A small layer of about 1.5 cm soil is brought in the permeameter and the water head slowly raised to the soil level. This procedure is repeated till the permeameter is completely filled: for the granular envelope to the top of the mark on the cylinder, for the synthetic envelope to the top of the cylinder flange.
- At the top of the cylinder flange, the soil or gravel layer is scraped flat by means of a ruler and the rim cleaned thoroughly.
- Calculate the bulk density of the soil in the permeameter cylinder from the weight and the volume.
- This filling procedure has to be followed with each replicate to obtain, within certain limits, the same bulk density to prevent it being the cause of differences between replicates. If the bulk density deviates too much, permeameters with deviating bulk density should be refilled.
- Subsequently, the synthetic or granular envelope material is put in place and covered with the perforated top plate (perforated transparent plate or flattened portion of corrugated drainpipe), then carefully closed with the upper flange.
- Connect manometer tubes if not yet done, and remove all air inclusions after which the permeameter experiment can start.

Gravel preparation and permeameter filling

- Determine the weight percentages of each fraction of a gradation curve.
- Calculate the required amount of each fraction for a pre-set total weight of 2 kg/cylinder (permeameter filling and backup).
- When all fractions are finished put all these weights together in a strong

plastic bag and mix gently.

- Fill the permeameter cylinder evenly with gravel above the base soil.
- Once the top of the cylinder flange is reached, the gravel layer is scraped flat by means of a ruler. The rim is cleaned thoroughly.
- Be sure to follow the same procedure with each replicate; compaction is not necessary. The remaining gravel has to be weighed.
- Calculate the density. If not the same as in other replications take out carefully, except for a small layer at the interface with the base soil. Then refill and keep track of quantities used.
- Place the perforated transparent top plate and carefully close the cylinder.
- Connect manometer tubes and water supply reservoir; remove all air inclusions.
- Start the experiment. A checklist of steps necessary with the permeameter testing is given in the next section.

Running of tests and measurements

Outflow rates, hydraulic heads, temperature and thickness of the soil column are measured during the permeameter test, and passed particles collected. Deduce from these measurements, the performance of envelope materials, either gravel or synthetic envelopes. A typical scenario would be:

- Assume at this stage that all preparations such as analysis of the soil and envelope materials and filling of the permeameters have been completed as described in Section 5.5, 5.6 and the foregoing Sections of 5.7.
- Connect the manometers in the right order making sure that the water supply to the permeameters (replications) is still closed.
- When all connections have been made, lower the variable supply tank until the water is at the same level as the lowest manometer tap in the soil. Before connecting the water supply tube to the permeameter, open the valve to remove all air bubbles. Connect the supply tube to the permeameter, open the valve and tilt the permeameter forwards.
- Rock the top of the permeameter gently from left to right, looking underneath the supporting screen. All air bubbles on the screen have to be removed before raising the water level into the soil.
- Move the variable supply tank to the next manometer tap level and loosen the tap below to remove any air in the soil and manometer tap.
- Repeat this process until the entire soil column, and gravel envelope column if applicable, is saturated. Occasional tapping of the permeameter and manometer riser tubes may be necessary to remove the air in the soil and in the connections. If necessary water can be added to the riser tubes from the top of the riser tube.
- Once the soil envelope is saturated the variable supply tank can be raised to the initial test level. Preferably the first couple of test levels are at gradients smaller than one (< 1), to then gradually move past the critical gradient during subsequent steps. Observe closely what happens at the

soil-envelope interface. Usually 24 hours is needed to let the permeameter stabilise at the new level.
- Raise levels until the maximum has been reached in daily steps (it can be tested whether shorter intervals are acceptable from stabilisation point of view).
- Lower levels in daily steps again to below a gradient of one.
- Now, check a plot of total gradient against soil and soil-envelope hydraulic conductivity, assuming that the test has not yet failed due to soil particle movement or other reasons, to see whether or not the hydraulic conductivities have stabilised.
- If not, start raising and lowering the water supply tank levels as necessary until the soil-envelope stabilises or failure occurs.

The test is finished when:
- soil particles flow out of the top of the (drain) plate (failure of the envelope);
- soil and envelope hydraulic conductivities have stabilised without failure;
- a sudden increase or decrease of soil hydraulic conductivity or envelope hydraulic conductivity is observed (Willardson and Ahmed 1988); and
- other disturbances have occurred which render the results questionable (e.g. soil cracks are visible, during the night water supply failed and air entered the soil).

The above is the most elaborate testing procedure that can be executed and may last anywhere between 1000 and 2000 hrs (6 - 12 weeks), hence, it might not be suitable as indicator test procedure. It was found that the level could be raised several times during the day, allowing 2 - 3 hrs of stabilisation (including one long overnight period), while the increment of water supply level may be increased from the usual 5 - 10 cm to 20 - 30 cm each time the range of gradient 1 has been passed. If no failure is apparent at a gradient of 1, the exact point of failure with the larger increments might be missed. However, for indicator tests what is usually of interest is whether or not failure occurred. If the GR_{25} or the HCR are to be used, testing at the critical gradient range(s) should be executed more intensively (smaller incremental changes).

At each increment of the head the following need to be measured:
- the heads of all riser tubes, before and after the change (after the change when equilibrium has been obtained, otherwise the measurements are useless!);
- the discharge before and the discharge after the change (ibid.);
- the temperature of the discharged water;
- inspect the soil and envelope columns for cracks etc.;
- if sediment passes determine the quantity;
- note any other observations such as power cuts, or water leakage from taps, or anything that could affect the pressure distribution in the permeameter

of any of the replications; and
- measure soil and gravel column heights after each change of head.

When the test is finished perform sieve analysis on base soil and granular envelope material taken carefully from the permeameter as directed in Section 5.5. Compare with PSD of the same material before the test. Exclude the soil of the soil-envelope interface, which is the soil between the taps where the interface was located. This soil is expected to have an altered PSD due to fines that may have been deposited in front of the envelope. The intent of the PSD determination at this stage is to confirm the loss of particles through the envelope, if any.

Analyses
The hydraulic conductivity of the soil and of the envelope in contact with the soil is calculated from Darcy's law, applicable to laminar flow conditions in the soil (Eq. 18, p 133). Applying the temperature correction (Eq. 22) to calculate K at 20 °C we get:

$$K_{20} = \frac{V}{At}\,\frac{L}{H}\,\frac{\eta_T}{\eta_{20}} = \frac{V}{Ati}\,\frac{\eta_T}{\eta_{20}} \qquad \text{Eq. 94}$$

with η_T/η_{20} the temperature correction factor based on the ratio between the dynamic viscosity at test temperature and the standard temperature of 20 °C (either Eq 21 on p 135, or Eq 76, p 212).

Hydraulic conductivity values can be calculated for the soil if head losses within the soil are known, and over the envelope in contact with the soil. Also the overall hydraulic conductivity can be calculated if desired. The hydraulic gradient is derived from the riser tube readings and the spacing of the riser tube taps, as indicated in Figure 73.

The head loss, H_{es} over the drainage envelope or geotextile including the drainpipe, is determined from the water levels in riser tubes 10 and 9 and the gradient i_{es} is given by:

$$i_{es} = \frac{H_{9\text{-}10}}{L_{9\text{-}10}} \qquad \text{Eq. 95}$$

where,

$L_{9\text{-}10}$ is the distance measured from the centre of tap no. 9 to the top of the top plate. Not the centre of tap no. 10 because the hydraulic head as it appears in riser tube 10 will be the same anywhere above the top plate.

Similarly the gradient in the soil can be derived from the taps in contact with the soil. Therefore, head losses between taps 9 and 8, or between 8 and 7, or between 9 and 7, can be used. Generally the head loss H_s between taps 9 and 7 is considered, hence the hydraulic gradient, i_s in the soil is given by:

$$i_s = \frac{H_{7\text{-}9}}{L_{7\text{-}9}} \qquad \text{Eq. 96}$$

To check for laminar flow conditions Reynolds number (R_e) is to be calculated from Eq 20 (p 135).

To evaluate clogging or blocking of the envelope either the permeability ratio or the gradient ratio can be compared:

$$\frac{K_{es}}{K_s} \geq 1 \qquad \text{Eq. 97}$$

or

$$\frac{i_{es}}{i_s} \leq 1 \qquad \text{Eq. 97}$$

Up to 1994, references using Dierickx as source generally referred to this as the envelope-soil ratios (i_e/i_s or K_e/K_s) but as was shown earlier neither the envelope gradient nor the hydraulic conductivity of the envelope are true envelope values. They include soil properties as well as the properties of the top plate holding the envelope material in place with upward flow permeameters. Consequently, both the gradient ratio as in Eq 98 as well as GR_{25} do not classify true envelope characteristis but rather apparent envelope characteristics. The different gradient ratios can be used to compare results in a qualitative manner as was done by El-Sadany *et al.* (1995), who performed tests on Egyptian soils and envelopes in two independent laboratories.

The value of $i_{es}/i_s = 1$ is a reflection of the fact that the envelope in contact with the soil should have at least the same permeability as the surrounding base soil. The gradient ratio, GR = 3, recommended by the Army COE as cut-off point, is based on tests that include the 25 mm of soil adjacent to the envelope (GR_{25}).

Data Interpretation

To determine the performance of the soil-envelope permeameter test the following interpretations are suggested:

1. Plot the various heads from which the gradients and hydraulic conductivities are calculated against time. Vlotman *et al.* (1993c) observed that the head loss between riser tubes should be more than 5 mm, otherwise the reading errors would be too large and affect the calculation of the hydraulic conductivity, as well as make the gradient ratio assessment meaningless.
2. Plot the gradients against time (i_{es}, i_s and i_t, where i_t is the gradient over the soil and envelope sample) to see their time dependent behaviour. This allows judgement of whether continuous higher gradients were applied, whether gradients went up and down, and/or whether surging was performed. While this can be also assessed from the previous plot, here it could help with judgement of the GR plot.
3. Plot the hydraulic conductivities (K_{es}, K_s and K_t, where K_t is the overall hydraulic conductivity of the soil and envelope sample together) against time. On comparison with the first two plots, it can be judged whether behaviour was normal, unexpected, or caused by too small head loss (first plot) differences between head changes. Note, during the first 10 - 200 hrs of the permeameter test, any decrease in permeability observed could have been caused by soil compaction in the permeameter. Furthermore, it might be questionable to use K_{es}, and Eq. 93 could be used instead to try to calculate the actual K_e.
4. Plot discharge versus total gradient. Discharge will continue to increase with increasing gradient as long as there no blocking or clogging and laminar flow conditions exist (Darcy's law holds). This may be checked by calculating Reynolds number (R_e), which should be in the order of 1000 – 2200, if average velocity (envelope flow velocity) without considering porosity is used (straight tubes concept referred to in Section 5.1.3). It should be less than one ($R_e < 1$) when the porosity and/or actual tortuous flow paths (Figure 42) are used. If the discharge deviates from a straight line relationship with the gradient, non-laminar flow conditions have occurred
5. Plot the various gradients against total gradient. On a regular scale they should be straight rising lines; on logarithmic scale they will be parabolic in shape. Disturbances will show up and need to be explained. The main purpose, however, is to assess which gradient caused the gradient ratio to change (next plot).
6. Plot the gradient ratio (GR) i_{es}/i_s versus the total gradient i_t. According to Eq. 98 this ratio should remain below 1; the envelope-soil performs at least as good as the soil. A trend upward could indicate clogging; i_{es} becomes higher due to clogging and i_s becomes smaller due to hypothesised higher hydraulic conductivity in the soil due to loss of particles. A downward trend that stabilises could signify some initial particle movement in the soil, followed by a stabilised soil-envelope interface. However, it is important to check points 1 and 5 above, to make sure the trend of

the ratio indeed indicates the expected. If GR is calculated from Eq. 98 the values cannot be related to GR_{25} (Eq. 91). A special tap at 25 mm below the (synthetic) envelope would have been necessary, in which case the ratio would need to remain below 3 (Note, however, that one test is with upward flow, and the other with downward flow).

7. Plot the hydraulic conductivity of the soil-envelope interface, that of the soil, and that of the granular envelope (if applicable) against the total gradient. Particle movement from the soil to the envelope should result in a lower K_{es} and higher K_s. When the soil-envelope in the permeameter has stabilised, K_s and K_{es} should remain constant. Any sudden changes observed indicate internal particle movement, but not (yet) failure of the envelopes functioning.
8. Plot the hydraulic conductivity ratio (HCR) against total gradient. This plot essentially tells the same as the GR against total gradient, except that hydraulic conductivities should remain constant with changing gradients if nothing untoward happens.
9. Determine density and/or porosity of both base soil and the gravel envelope (if applicable) based on measurement of the height of the soil and gravel columns before and after the experiment, to assist with possible explanation of differences in K-values amongst the replications.
10. Passage of sediment through the top plate would indicate possible failure of the envelope depending on the amount of sediment passed. Van der Sluys and Dierickx (1990) indicate passage > 1% of the total soil in the permeameter as unacceptable.
11. Visual observations of piping along the Plexiglas, or cracks appearing in the soil after head changes could mean termination of the particular replication, but does not necessarily indicate failure of the envelope.
12. For tests of long duration check (visually) for algae growth in the permeameter and appurtenant equipment. If K-values are not stabilising (items 3 and 7 above) then a progressive reduction in hydraulic conductivities might indicate biological activities.
13. Sudden reductions in hydraulic conductivity, or reduction of discharge to practically zero soon after the start of the test, could indicate high concentrations of sodium carbonates in the soil samples used, which will react swiftly when fresh water is applied (Vlotman 1990). A drop of phenolphethalene that will colour the water red if sodicity is a problem can test this.

The above checklist of items is not exhaustive, and it is up to the individual researchers to add and/or modify the above assessments, as well as give their own interpretation to the results. However, the tests do cover the LTF, GR and HCR test described earlier and it is strongly recommended to try and standardise the laboratory testing as much as possible, or to cover at least the various options reported by researchers worldwide.

Limitations

Naturally, the tests have various limitations as well:

- The synthetic envelope material is not tested under a soil load and hence care should be taken to adjust results to include effects of pressure on the properties of the synthetic material (Figure 58 and Figure 64).
- Biological clogging could occur with long-term tests (1000 - 2000 hrs), therefore, tests with the Plexiglas permeameters described are preferably executed in the dark, or in darkened areas, to prevent growth of algae, etc. Note, ASTM D1987 (ASTM 1996c) actually gives a standard test method for biological clogging of geotextile or soil/geotextile filters, which mentions that the test may take 1000 h or more to initiate biological growth). Prevention of algae growth can also be achieved by applying commercially available liquid algaecide without affecting the tests (Shi *et al.* 1994), but not without some risk to the environment. Bonnell *et al.* (1986) used 50 parts per million mercury chloride to inhibit microbial growth.
- Air bubbles can affect the results significantly, and great caution is needed to prevent air-bubble formation by proper filling of the permeameter initially, and by using de-aerated water.
- Highly permeable envelopes might not allow proper measurement of head differences between adjacent riser tube (manometer) taps, and consequently it could be difficult to assess these type of envelopes using some of the plots indicated before. However, the permeameter is not meant to measure the hydraulic conductivity of the envelope material. So, although some of the analyses cannot be performed as indicated, as long as the soil is retained it can be concluded that the envelope material functions properly.
- The GR_{25} ratio that includes 25 mm of soil with the synthetic envelope might be adequate to calculate a meaningful gradient (i_{es}) and permeability K_{es} value. However, it might not express changes in the synthetic envelope but in the soil adjacent to the envelope instead. Note, K_{es} could also be controlled by the number of perforations in the top plate of the permeameter, if insufficient, Vlotman *et al.* (1993c). This may be all that is needed but caution should be exercised with interpreting GR results, whether based on ASTM procedures or those described herein.

5.7.5 Conclusions

Indicator values for the judgement of potential envelope materials can be derived from laboratory permeameter testing of soil-envelope combinations, provided that the limitations with the methods described in Section 5.7 are taken account of. There are two distinctly different methods of testing the soil-envelope combinations, one with downward flow and the other with upward flow through the permeameter. The advantage of upward flow methodology is that it will also test the unstable situation that may occur in non-cohesive soils at or near the critical gradient ($i_c \cong 1$), when the upward forces balance the downward force of gravity and create a highly unstable particle arrangement. This might cause soil particles to flow into the envelope, and if an improper envelope is selected, into the drainpipe as well. Gradients considerably higher than 1 may occur and these can be tested with both methodologies. The downward flow permeameter will be able to test slightly higher gradients than the upward flow permeameter under the same conditions, but this is hardly an advantage over the upward flow test. Therefore, it is recommended that the upward flow permeameter be used as it will also test the unstable condition, which could occur below the pipe in the field.

The LTF test (Section 5.7.1) and the GR test according to ASTM guidelines (Section 5.7.2) both use the downward flow principle. The HCR test (Section 5.7.3) uses downward and upward flow alternately with a flexible wall permeameter customary for the triaxial tests prescribed by ASTM D5084. ASTM indicates that the HCR test is for soils with $K_s < 0.864$ m/d, which are fine-textured soils, while Williams and Luettich (1990) suggest that the HCR test is mostly used for soils with $K_s > 86.4$ m/d (10^{-1} m/s). If the latter is correct this essentially makes the HCR test unsuitable for agricultural soils with a typical K_s value of between 0.002 and 50 m/d (Table 14). It is recommended that further confirmation should be sought of the true range of applicability of the HCR test when soil-geotextile combinations are tested (the ASTM HCR test only tests for soil permeability).

In Section 5.7.4 a methodology with an upward flow permeameter is described already in use in Belgium, Egypt and Pakistan, which combines the objectives of the LTF, GR, and HCR tests. The permeameter was initially used for research purposes, but can be used with some adaptations in the followed testing procedures, as well as after some adjustment of the apparatus (extra riser tube taps at specified distance from the envelope), be used just as well for the indicator tests (LTF, GR and HCR).

The intent of prescribing riser tubes at fixed locations is to ease comparison between tests performed at different locations. However, this might not be important when local soil-envelope combinations are tested and there is no need for comparison with tests done elsewhere with the same or similar soil-envelope combinations. Assuming certain degrees of uncertainty, when testing similar envelope materials one could argue that GR_{25}, GR_{10} or GR_5 (the number implies the height of base soil included in the observation of the synthetic envelope material) should all produce qualitatively equivalent results. Hence, comparing results qualitatively might be feasible and acceptable.

6. Design of drain envelopes: existing criteria and field experience

6.1 Drain envelope need

The need for a drain envelope is primarily dependent on the soil characteristics of the region. Whether the experiences of one region can be transferred directly to another has not yet been confirmed. To assess the need for an envelope in an existing field installation, the amount of sediment in a drainpipe without envelope can be estimated from trial lines in the field. Lagacé and Skaggs (1982, in Rajad Project Staff 1995) proposed that an envelope is not needed if sediment in the pipe is less than 10 mm deep. An envelope is recommended when 10 - 30 mm sediment is found in the pipe, and it is strongly recommended if the sediment layer is > 30 mm. No time limit for the sedimentation has been proposed but one year would seem to be reasonable.

Abdel-Dayem (1985) used the percentage of flow area blocked as indicator; hydraulic capacity calculations include a safety factor of 25% for collectors, which implies that if 25% of a collector is blocked it would still function in keeping with the design. Similarly, the safety factor for lateral drains is 40%. The transport equation (uniform flow) was used for hydraulic design of collectors, while for laterals the drainage equation (non-uniform flow) was used. The non-uniform flow equation implies a safety factor of 0.58 (Cavelaars *et al.* 1994). Cavelaars *et al.* (1994) give a complete overview of the hydraulics of pipes, which goes beyond the scope of this book.

In the Netherlands, a layer of 15 mm of sediment in 60 mm diameter pipes is regarded as unacceptable (Dekker and Ven 1982) because of the adverse effect on flow capacity. Generally, some sedimentation will occur at the first working of the drainpipe. If there is no sedimentation at the beginning, there is little risk of sedimentation at a later stage unless some disturbance takes place. An overview follows of selection criteria used in the past described per country from which they originate, the order being largely determined by the logical flow of the text.

6.1.1 USA

One of the earliest guidelines to determine the need for a drain envelope was published by the Soil Conservation Service (1971, 1973, also in Dieleman and Trafford, 1976). They used the Unified Soil Classification system (USBR 1985, Figure 74) as indicator for the need for a filter around a drainpipe. To determine the soil classification the Particle Size Distribution (PSD), the Liquid Limit, the Plasticity Index (PI), the Coefficient of Uniformity (C_u), the Coefficient of Curvature (C_c), and other parameters are needed. After the soil classification has been done Table 28 is used to determine the need for an envelope. For a number of soils (GP, SC, GM, and SM-coarse, see Figure 74) local experience needs to be collected before arriving at a definite decision on the need for the envelope. One of the first guidelines on using synthetic drain envelopes (spun bonded nylon fabric) derived from the archives of the SCS (Wenberg 1996) is reproduced here as a potential indication of synthetic envelope need (Figure 75).

UNIFIED SOIL CLASSIFICATION, INCLUDING IDENTIFICATION AND DESCRIPTIO

FIELD IDENTIFICATION PROCEDURES excluding particles larger than 3 inches and basing fractions on estimated weights					GROUP symbols **	TYPICAL NAMES
COARSE GRAINED SOILS more than half of material is **larger** than no 200 sieve size*	GRAVELS more than half of coarse fraction is larger than no 4 sieve size ***	CLEAN GRAVELS (little or no fines)	wide range in grain size and substantial amounts of all intermediate particle sizes		GW	well graded gravels, gravel-sand mixtures, little or no fines
			predominantly one size or a range of sizes with some intermediate size missing		GP	poorly graded gravels, gravel-sand mixtures, little or no fines
		GRAVELS WITH FINES (appreciable amount of fines)	non-plastic fines (for identification procedures see ML below)		GM	silty gravels, poorly graded gravel-sand-silt mixtures
			plastic fines (for identification procedures see CL below)		GC	clayey gravels, poorly graded gravel-sand-clay mixtures
	SANDS more than half of coarse fraction is smaller than no 4 sieve size ***	CLEAN SANDS (little or no fines)	wide range in grain sizes and substantial amounts of all intermediate particle sizes		SW	well graded sands, gravelly sands, little or no fines
			predominantly one size or a range of sizes with some intermediate sizes missing		SP	poorly graded sands, gravelly sands, little or no lines
		SANDS WITH FINES (appreciable amount of fines)	non-plastic fines (for identification procedures see ML below)		SM	silty sands, poorly graded sand-silt mixtures
			plastic fines (for identification procedures see CL below)		SC	clayey sands, poorly graded sand-clay mixtures
COARSE GRAINED SOILS more than half of material is **smaller** than no 200 sieve size*	IDENTIFICATION PROCEDURES ON FRACTION SMALLER THAN No 40 SIEVE SIZE					
		DRY STRENGTH (crushing characteristicts)	DILATANCY (reaction to shaking)	TOUGHNESS (consistency near plastic limit)		
	SILTS AND CLAYS liquid limit less than 50	non to slight	quick to slow	none	ML	inorganic silts and very fine sands, rock flo silty or clayey fine sands with slight plastic
		medium to high	none to very slow	medium	CL	inorganic clays of low to medium plasticity, gravelly clay, sandy clay, silty clay, lean cla
		slight to medium	slow	slight	OL	organic silts and organic silt-clays of low plasticity
	SILTS AND CLAYS liquid limit greater than 50	slight to medium	slow to none	slight to medium	MH	inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		high to very high	none	high	CH	inorganic clays of high plasticity, fat clays
		medium to high	none to very slow	slight to medium	OH	organic clays of medium to high plasticity
HIGHLY ORGANIC SOILS		readily identified by color, odor, spongy feel and frequently by fibrous texture			Pt	peat and other highly organic soils

* the No 200 sieve size is about the smallest particle visible to the naked eye
** boundary classifications: soils posessing characteristics of two groups are desinated by combinations, for example GW-GC, well graded gravel-sand mixture with clay bin
*** for visual classifications, the ... size may be used as equivalent to the No 4 sieve size

Figure 74 Unified Soil Classification Chart.
(USBR 1985, ASTM D 2487-93)

Samani and Willardson (1981) first published the theory of Hydraulic Failure Gradient (HFG) in 1981. The HFG is defined as the gradient measured in a permeameter (such as shown in Figure 45 and Figure 73) at which the soil particles bridge over the characteristic opening size (COS) of the envelope fails (whether a granular or synthetic material). The HFG of a particular soil can be related to basic soil properties such as the PI and the saturated hydraulic conductivity of the soil (K_s). Figure 43 shows a number of soils for which the HFG have been determined. It was found that a positive correlation exists between PI and HFG but that this relationship was not transferable between arid and humid regions. Further details are given in Section 5.3.

UNIFIED SOIL CLASSIFICATION

INFORMATION REQUIRED FOR DESCRIBING SOILS		LABORATORY CLASSIFICATION CRITERIA		
give typical name, indicate approximate percentage of sand and gravel max. size, angularity, surface condition, and hardness of the coarse grains, local or geological name and other pertinent descriptive information and symbol in parentheses for undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics EXAMPLE silty sand: gravelly; about 20% hard; angular gravel particles ½ in. maximum size; rounded and subangular sand grains coarse to fine; about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM)	use grain size curve in identifying the fraction as given under field identification	determine percentages of gravel and sand from grain size curve, depending on percentage of fines (fraction smaller than no 200 sieve size) coarse grained soils are classified as follows: less than 5 % GW, GP, SW, SP; more than 12 % GM, GC, SM, SC; 5 % to 12 % borderline cases requiring use of dual symbols	$C_U = \frac{D_{60}}{D_{10}}$ greater than 4, $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between one and 3	
			not meeting all gradation requirements for GW	
			Atterberg limits below "A" line or PI less than 4	above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols
			Atterberg limits above "A" line or PI greater than 7	
			$C_u = \frac{D_{60}}{D_{10}}$ greater than 6, $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			not meeting all gradation requirements for SW	
			Atterberg limits below "A" line or PI less than 4	above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols
			Atterberg limits above "A" line or PI greater than 7	
give typical name, indicate degree and character of plasticity, amount and maximum size of coarse grains, color in wet condition, odor, local or geologic name, and other pertinent descriptive information, and symbol in parentheses for undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions EXAMPLE clayey silt: brown; slightly plastic; small percentage of sand; numerous vertical root holes; firm and dry in place; loess; (ML)		PLASTICITY CHART for laboratory classification of fine grained soils		

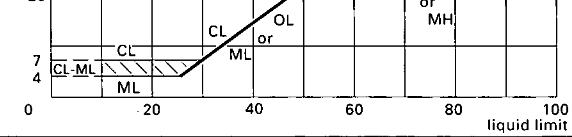


Table 28 SCS classification to determine need for drain filters and minimum velocities in drains. (after: SCS 1973, also in Dieleman and Trafford 1976, and ASCE 1994).

Unified soil classification	Soil description	Filter recommendation	Envelope recommendation	Recommendation for minimum drain velocity
SP (fine)	Poorly graded sands, gravely sands.	Filter needed.	Not needed where sand and gravel filter is used but may be needed with flexible drain tubing and other type filters.	None
SM (fine)	Silty sands, poorly graded sand-silt mixture.			
ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity.			
MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.			
GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.	Subject to local on-site determination.	Not needed where sand and gravel filter is used but may be needed with flexible drain tubing and other type filters.	With filter - none. Without filter - 1.40 feet/second (0.43 m/s).
SC	Clayey sands, poorly graded sand-clay mixtures.			
GM	Silty gravels, poorly graded gravel-sand silt mixtures.			
SM (coarse)	Silty sands, poorly graded sand-silt mixtures.			
GC	Clayey gravels, poorly graded gravel-sand-clay mixtures.	None	Optional. May be needed with flexible drain tubing.	None - for soils with little or no fines. 1.40 feet/second for soils with appreciable fines (0.43 m/s).
CL	Inorganic clays of low to medium plasticity, gravely clays, sandy clays, silty clays, lean clays.			
SP, GP (coarse)	Same as SP and GP.			
GW	Well-graded gravels, gravel-sand mixture, little or no fines.			
SW	Well-graded sands, gravely sands, little or no fines.			
CH	Inorganic, fat clays.			
OL	Organic silts and organic silt-clays or low plasticity.			
OH	Organic clays of medium to high plasticity.			
Pt	Peat.			

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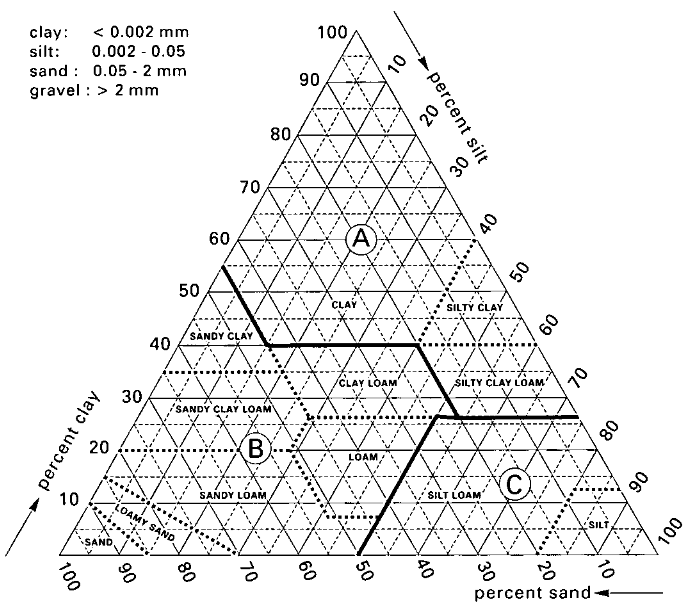


A filters are not needed
B sand and gravel filter or spun handed nylon fabric to restrict movement of particles that would deposit in drain
C sand-gravel filter as material movement of particles that would deposit in drain

Figure 75 Guide for use of spun-bonded nylon fabric to protect corrugated drain tubing. (after: Wenberg 1996).

6.1.2 Germany

Dieleman and Trafford (1976) also give the criteria used in the former German Democratic Republic; their standard TGL 20 286 of 1971, reproduced in Table 29. In the same section the C_u and the PI are also used as indicators for silting tendencies (Table 30). Unfortunately, Dieleman and Trafford (1976) do not give the original sources of these indicators.

Table 29 Tendency of mineral soils to cause siltation in drainpipes. (Dieleman and Trafford 1976).

Soil types	Silting tendency	Particle size		
		Clay < 2 µm	Silt 2 -20 µm	Sand 20 - 600 µm
Sand, sandy loam, loamy sand	Considerable	< 8%	< 25%	> 70%
Sand, sandy loam, loamy sand	Slight	8 - 10%	< 20%	> 70%
Loamy sand, sandy loam, loamy silt	Considerable	< 8%	25 - 55%	40 - 70 %
Sandy loam, loamy silt, silt loam	Slight	8 - 12%	20 - 55%	35 - 75%

Data from TGL 20 286, 1971.

From later sources (Lawson 1990) it would appear that the West German code of practice for geotextiles (FSV, 1987) gives less detail and uses primarily the PSD curves to make judgements of the need for filtering geotextiles for drainage applications. Figure 76 is based on FGSV 535 (1994). Region I is described as having adequate soil cohesion resulting in essentially stable soil structures. For soils which fall in this area of the PSD curves it is suggested that more open geotextile filters can be selected. Region II covers silts and fine sands (designated 'problematic' soils) requiring close attention to filter design. Region II has been defined as an area which has the following properties (FGSV-535, 1994):

1. soils with particles < 0.06 mm and $C_u < 15$;
2. 0.02 mm < d_{50} < 0.1 mm; and
3. soils with d_{40} > 0.063 mm, when PI < 0.15 or clay/silt content ratio[34] < 0.5.

Table 30 Coefficient of Uniformity or Plasticity Index as indicators of soil silting tendency. (Dieleman and Trafford 1976 and Sherard 1953).

Silting tendency	Cu	or	PI	PI (Sherard 1953)
No tendency	≥ 15		>12	>15
Limited tendency	5 - 15		6 - 12	6 - 15
High tendency	£ 5		< 6	< 6

Region III comprises granular soils for which design criteria for filter design are less severe than for Region II. Lawson reported that there was no indication of whether these boundaries were established on the basis of monitored performance tests or by general consideration. The curves closely resemble boundaries of Canadian problem soils (Figure 80) reported by Rollin *et al.* (1994). For comparison, the problem soil ranges of Pakistan and those indicated by Irwin and Hore (1979, also in Cavelaars *et al.* 1994) of Canada are also shown in Figure 76.

[34] Clay < 0.002 mm, silt: 0.002 - 0.060 mm in Germany.

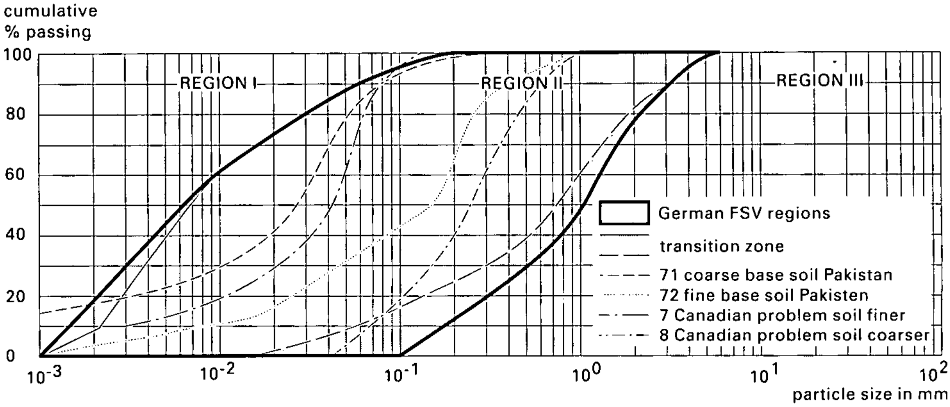


Figure 76 German classification of soils using PSD curves and Pakistan and Canadian soils. (Lawson 1990, FSV 1987, FGSV-535 1994, and Irwin and Hore 1979 [soil nos. 7 and 8]).

6.1.3 The Netherlands

Van Zeijts (1992) and also Cavelaars *et al.* (1994) reported relationships between clay and silt contents of humid soils found in the Netherlands and the need for drain envelopes (Table 31). In addition, the appropriateness of certain envelope types (gravel, organic, synthetic thin and voluminous) for certain soil types are indicated. Drains can be installed without an envelope in heavy soils containing more than 25% clay. Based on numerous field observations it was concluded that soils with clay > 17.5% had general minimal sedimentation problems (Stuyt and Oosten 1987). A layer of 15 mm of sediment in the drainpipe was taken as the boundary for acceptance or rejection. PVC pipes with a diameter of 60 mm are predominantly used. Typical soils in the Netherlands in which drainage systems have been constructed are shown in Figure 77.

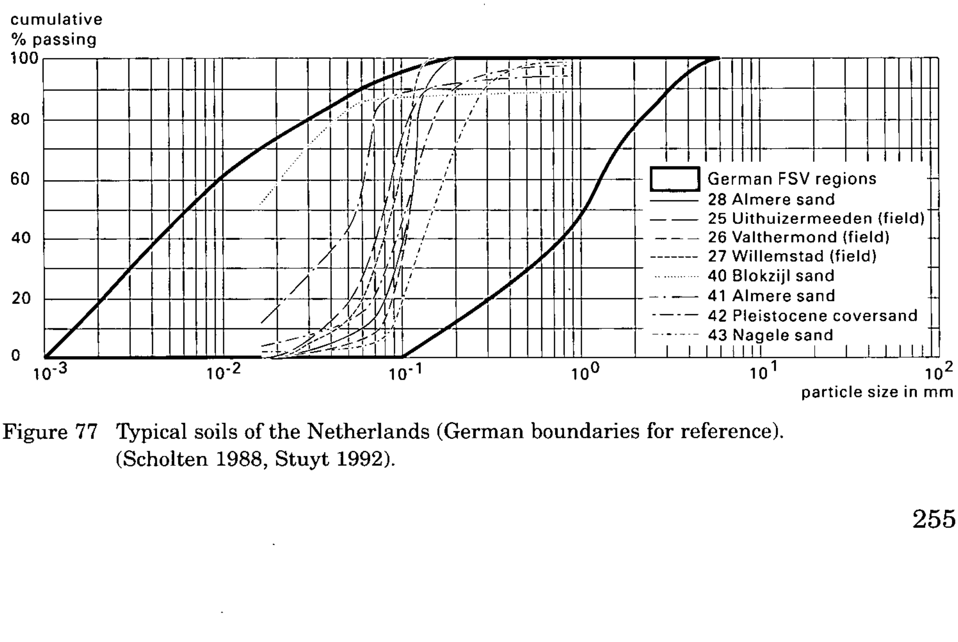


Figure 77 Typical soils of the Netherlands (German boundaries for reference). (Scholten 1988, Stuyt 1992).

Table 31 Recommendations on the use of the drain envelopes in the Netherlands based on soil type. (after Van Zeijts 1992, Cavelaars *et al.* 1994 and KIWA 1990).

Soil				Envelopes[1]				
Type based on percentage clay and silt particles[2]	Geological formation	Remarks	Characteristics related to envelopes[3]	Function	Material: Gravel	Material: Voluminous[4]: Organic	Material: Voluminous[4]: Synthetic	Material: Thin[5]
				thickness:	100 mm	4 - 7 mm	3 - 10 mm	< 1 mm
				O_{90} [7]:	NA	650 µm	650 - > 1750 µm	200 - 400 µm
> 25% clay	Alluvial; marine/fluvial	Ripe	Stable; high K	–	No envelope necessary			
		Unripe	Stable; low K	Hydraulic (temporary)	+	+	+	–
> 25% clay[6]		Ripe	Unstable; high K	Filter	+	–	+	+
		Unripe	Unstable; low K	Filter and hydraulic	+	–	+	–
< 25% clay < 10% silt	Marine	d_{50} < 120µm	Unstable; high K	Filter	+	–	+	+
< 25% clay < 10% silt	Aeolian	d_{50} > 120µm	Initially unstable; high K	Filter (temporary)	+	+	+	+
< 25% clay > 10% silt	Aeolian, fluvial or (fluvio) glacial		Initially unstable; low K	Filter (temp.) and hydraulic	+	+	+	–

1 + = suitable; - = not suitable
2 texture in soil profile above drain level, clay particles are < 2 mm and silt particles are 2-50 mm
3 high hydraulic conductivity: K ≥ 0.25 m/day, low K ≥ 0.05 m/day
4 voluminous envelopes in the Netherlands are primarily pre-wrapped loose coconut or synthetic fibres
5 only suitable if there is no risk for biochemical clogging (ochre/iron primarily)
6 at drain level > 25% but lighter layers with < 25% clay in soil profile above drain level
7 KIWA (1990).
NA: not applicable.

6.1.4 France

In France two types of clogging have been distinguished (Lennoz-Gratin 1987): primary clogging that occurs during installation or immediately after and is dependent on installation conditions; and secondary clogging that occurs gradually during successive drainage functioning periods. Mainly fine sandy soils ($d_{50} \approx 100$ µm) and loamy or sandy loam with clay < 15% are likely to cause clogging of drainpipes. According to the definition, fine sands in France constitute particles of between 70 - 150 µm, or 50 - 200 µm (Figure 50), therefore French criteria for soils susceptible to drain-clogging problems are:

1. > 50% fine sand (50 - 200 µm);
2. < 15% clay (< 2 µm); and
3. $C_u < 5$ (these soils are considered to have a uniform particle size distribution).

In Figure 78 are the Hjulström curves presented by Lennoz-Gratin (1987), which shows that particles of 200 µm are the easiest to erode. The finest particles are difficult to deposit, very easy to transport, but very hard to erode

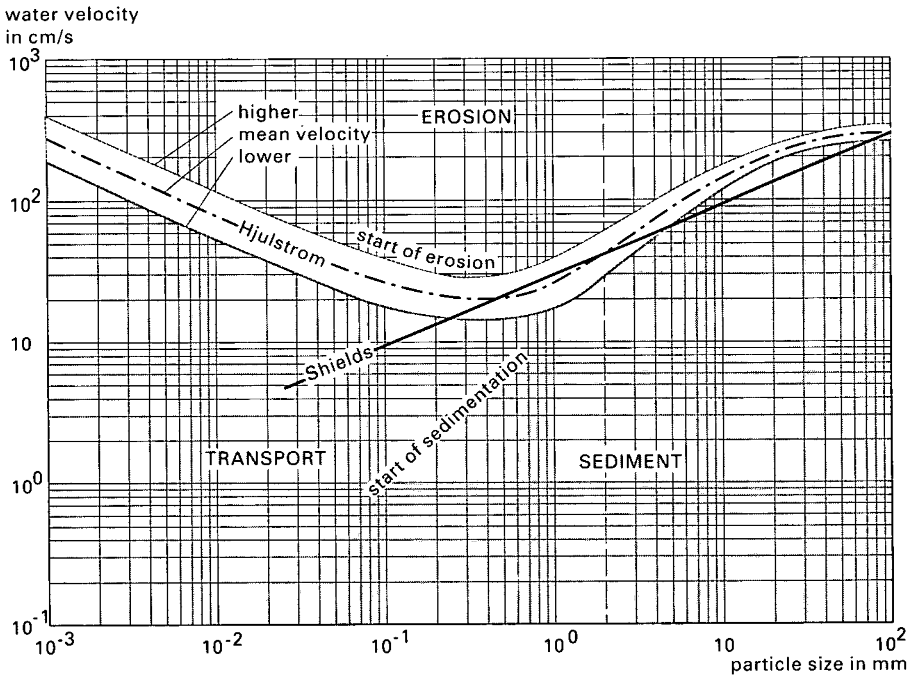


Figure 78 Erosion, transport and sedimentation-particle flow velocities.
(after ASCE 1967, Cestre 1985, Escher 1905, Lennoz-Gratin 1987, originally published by Hjulström in 1935, original reference is available in CUR/RWS 1995 [Shields curve]).

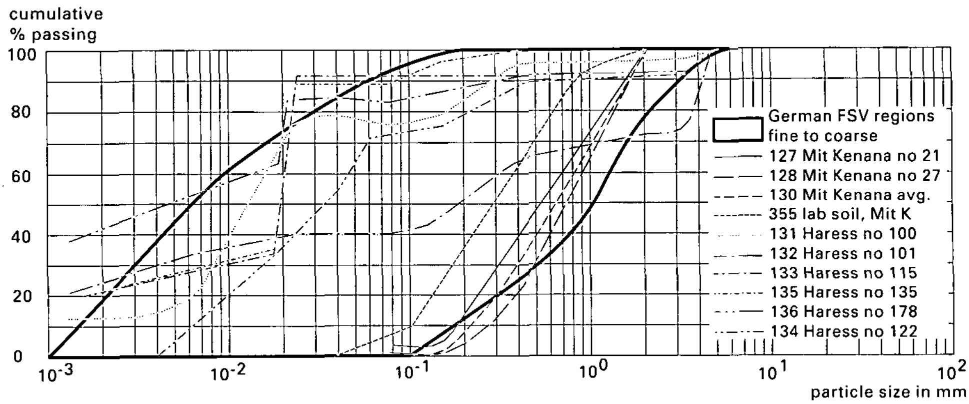
after deposition. It was concluded that particles between 30 and 200 µm are likely to account for the highest clogging risks. Other sources (CUR/RWS 1995) limit the application of the Hjulström curves to water depth of > 1 m but this may be only for the coarser materials (gravel and rock); critical flow velocities for silts and clays are well represented in the Hjulström curves. CUR/RWS (1995) suggest applying a correction factor to the critical flow velocities (those at which particles start to move) in Figure 78 as follows: for depth 0.3 m use 0.8; for depth 0.6 m use 0.9. The Shields bottom velocity shown in Figure 78 is the critical velocity primarily for sand and gravel. The flow velocity is the depth-averaged velocity. Other disturbance may cause deviations from the flow velocities indicated here.

6.1.5 Egypt

Observations during an intensive study of drainpipe siltation carried out (Abdel-Dayem 1985) in the Nile Delta revealed that, contrary to expectations, heavier soils in the Eastern Nile Delta showed serious siltation in both the concrete and plastic lateral drains without envelopes. The main reason for this was thought to be poor construction of the 30 cm long concrete pipes (joints of concrete collectors were covered with tar-impregnated burlap only). No adequate explanation for the same observation with the PVC laterals could be given. It does point out, however, that soil stability is not necessarily dependent on clay content only. Nevertheless, the same report concludes that the previously used criteria for not needing an envelope at a clay percentage of > 40% was somewhat conservative and could be lowered to > 30%. No data to support this conclusion were presented.

Subsurface drains in the Nile Delta are generally laid at a depth of 1.0 - 1.5 m below the land surface. Therefore, since there is a relationship between soil type, clay content and hydraulic conductivity, the drainage implementing agency generally used the design drain spacing as an indicator for needing a drain envelope: spacing of 40 m (clay) no envelope needed; spacing of 50 m (silt) and 60 m (sand) envelope needed. A recent study where laterals with and without envelope material were constructed in the Abu Matamir Area of the Nile Delta (Abdel-Hadi *et al.* 1998) confirmed the decision to put the clay boundary for the need of drain envelopes at 30%.

Soils in the Nile Delta tend to be rather clayey and like the soils used to determine the HFG formula are in the finer half of Region II of the German classification (Figure 79).



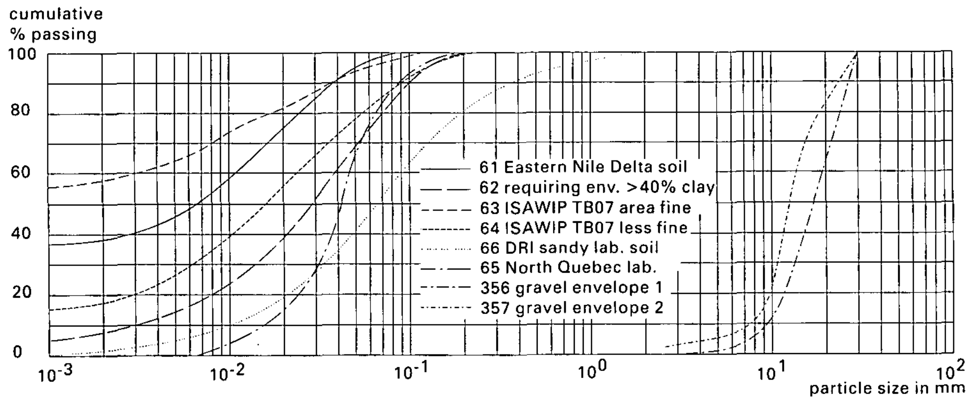


Figure 79 German problems soils and typical soils of the Nile Delta. (Abdel Dayem 1985 and Metzger *et al.* 1992)

6.1.6 India

Singh *et al.* (1992) presented the results of the Dabhou subsurface drainage experimental site (Gujarat). Sediment (excavated three years after construction) was observed in drains with coir-mat, gravel, plastic netting, and geotextile envelopes (listed in order of decreasing amount of sediment). The clay percentage in the soil around the envelope was between 25 and 34%. The drains without envelope (clay 23%) had 4 to 20 times the amount of sediment. The Rajad Project team (Rajad staff 1995) concluded that at the Dabhou site soils with a clay percentage of < 40% and a SAR > 13, and soils with a clay percentage < 30% (regardless of the SAR) were both found in need of drain envelopes. This was based on Singh *et al.* (1992) and on personal communication with the authors. Soil salinity at Dabhou varied between 0.48 and 2.94 dS/m, but water qualities were not given. At the Rajad Project itself (Rajad staff 1995) it was found that synthetic envelopes were required with clay < 40% and SAR > 8, using the sediment depth criteria mentioned before (< 10 mm, 10 - 30 mm, > 30 mm for pipes with a diameter of 80 mm, p ••) for experiments constructed with the trenchless V-plough method. Synthetic envelopes were recommended regardless of the SAR clay content < 30%. It was found that envelope protection was needed for drains constructed with a backhoe in soils with a clay percentage < 40% regardless of the SAR. Table 32 gives chemical soil and water qualities during the first year of operation of the various test sites.

Table 32 Chemical properties of soils and drainage water before 1993 at various Rajad Project test sites.
(Rajad staff 1995).

Test site	EC_d dS/m	SAR	pH	EC_e dS/m	Drain depth m.
Digod	3 - 4	4 - 8	7.8	1.5 - 2.5	0.8 - 1.3
Rangpuria	1 - 2	3 - 12	7.6	0.7	0.85 - 1.05
Prempura	0.75	2.5 - 10	8	0.7 – 2	0.95 - 1.35
Gaglawada	1 - 1.4	8 - 15	8 - 9.2	1.7 - 2.8	1 and 1.3
Hatnapur	2.3 - 5	15 - 35	8	2.5 – 7	0.8 -1.15

EC_e is average soil profile salinity of the saturated extract.
EC_d is salinity of drainage effluent.
Data are pre-1993 during first season of system operation. Drain spacing was 15, 30, 40, 60, 75 m. Values in tables are range of values encountered with different spacing. No clear trends in development of chemical levels as function of spacing are apparent for 1994 and 1995 data.

Data sets of matching sediment and chemical data collection points were not given, rather all data presented were processed in averages. Drains were excavated approximately one year after construction. In both the Dabhou and Rajad Project perforated corrugated plastic tubing was used for the subsurface drains.

Table 33 gives the range of values reported in the literature for using SAR and clay percentages as indicators for need of drain envelopes. However, experiences with these phenomena are only preliminary and caution should be used in using the values given in Table 33. The relationship between SAR and sediment in the pipe was also affected by the method of construction (vertical plough or hydraulic excavator/backhoe (Rajad Project Staff 1995). It is not reported whether sediment accumulations in the manholes were observed. The Rajad Project also include drain slopes as an indicator for potential blocking by sediment deposits in the pipe; they assume that certain flow velocities in the pipe will flush out the sediment particles. Excavations revealed that drains with a slope > 0.004 had no sediment deposits in the pipe.

Table 33 Indicators to determine need for a drain envelope.

Source	No envelope	Envelope needed
Canada and India	Clay > 30%	
Rajad Staff (1995)	–	clay < 30% irrespective of construction method clay< 40% and SAR > 8 for V-plough construction clay < 40% for backhoe construction
Netherlands	Clay >25%	
Egypt	Clay > 30 - 40%	

The slope indicator has three disadvantages: (1) velocities may not always be the same and deposits will take place at lower discharges; (2) it is undesirable to have sustained particle movement as the soil structure surrounding the drainpipe could destabilise; and, (3) the land might not slope enough, or the depth of outlets might be insufficient to allow a 0.004 slope. With regards to (2) the soil needs to be stabilised so that sediment inflow ceases almost immediately. With sustained particle movement the likelihood that the pipe will eventually become blocked is great. Making the slope steep enough to carry the sediment away does not solve the problem. Soil will settle during low flow (Figure 78) and fill up any depressions in the pipe. It is probably better not to suggest high slopes as a means of prevention of clogging.

6.1.7 Canada

Some of the earliest problem soil bandwidths given in the literature originate from Canada. Irwin and Hore (1979) reported Canadian problems soils with a narrower bandwidth (Figure 80) than those reported by Rollin *et al.* (1994). The soil bandwidth based on Irwin and Hore (1979) has been reproduced in later publications as problematic soils (Cavelaars *et al.* 1994).

There is a move from Canada to include not only the percentage of clay but also the SAR of the soil moisture extract as an indicator for needing a drain envelope (Broughton 1993, Rajad Project Staff 1995). Soils with SAR > 13 will disperse when fresh (irrigation) water is applied (EC_w < 0.3 dS/m) and bring fine particles in suspension. Usually this effect is connected with reduction in infiltration capacity of soils under irrigation. At this stage it is not clear whether the sedimentation of soils observed in drainpipes in soils with high SAR is only an initial problem, say in the first couple of years, or that it continues longer (Rajad Project staff 1995 reports results of the first two years). One would expect that the reduction in hydraulic conductivity of the soil due to dispersion of the aggregates would lead to reduced inflow in subsurface drains, and therefore result in a less serious problem of particle inflow.

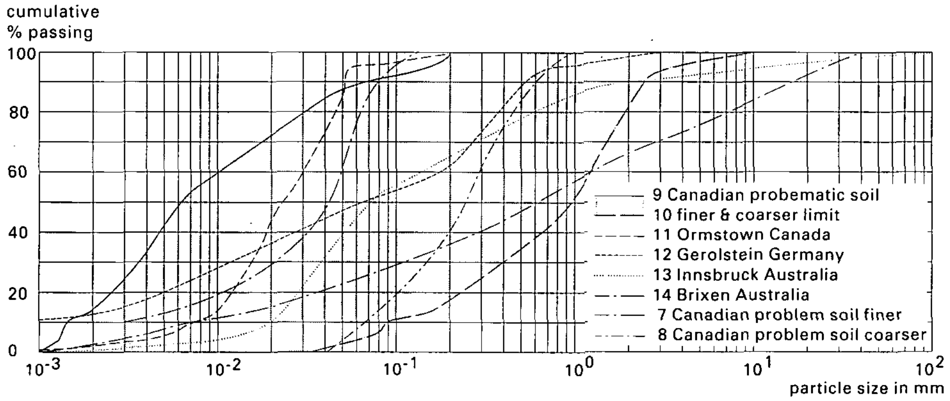


Figure 80 Canadian problem soils with Australian and German soils in need of drain envelopes.
(Irwin and Hore 1979 [soil nos. 7 and 8] and Rollin *et al.* 1994 [soil nos. 9 – 14])

Alternatively, this reduction in hydraulic conductivity could lead to higher gradients near the drain, which is in favour of enhanced sedimentation. However, sodicity effects, such as a major reduction of hydraulic conductivity near the subsurface drains have not been reported in the literature to date. In the laboratory, sodic soil from drain depth became impermeable almost immediately when fresh water was used in drain envelope permeameter experiments in Pakistan (Vlotman *et al.* 1990). Since this was not desirable in the permeameter test, soils were merely tested for sodicity and the value of SAR was not recorded. Also, no attempt was made to apply non-fresh water to the original soils in the permeameters, as this was not the objective of the tests.

Johnston *et al.* (1963) reported a decrease in drain discharge with increasing SAR values. However, further research is needed.

Irwin (1985) reviewing the studies on sediment in drainpipes performed in Canada concluded that soils with particles in the range 0.05 - 0.15 mm could enter the pipe when perforation size does not allow bridging to occur (median particle size > 3 times perforation size). Soils with a 15% clay content are usually sufficiently cohesive to prevent sediment movement. Gallichand and Lagacé (1986, 1987a) tried to predict the depth of sediment in subsurface drains as function of hydraulic radius of the opening and the d60 of the material using a mathematical model based on the physics of sedimentation and tests with glass beads, but no practical guidelines are given. In a follow-up paper (Gallichand and Lagacé, 1987b) recommended that further development of the model is needed to account for seeping water, radial flow towards the drain, and the size of stable soil aggregates.

6.1.8 Pakistan

To date there are no established citeria to determine the need for drain envelopes. Most auger holes during pre-drainage investigations tended to collapse and required screens for the auger hole method for hydraulic conductivity determination. The collapse was a clear indication of the need for a drain envelope. Most of the soils in Pakistan (Figure 76 and Figure 86) requiring envelopes fall within the ranges of reported problematic soils.

6.1.9 Conclusions on drain envelope need

At present there are no global guidelines to determine the need for a drain envelope. The most popular index has been the clay percentage of the soil, but as has been shown in Egypt and India, other factors such as the quality of construction (perforations/openings more than a few millimetres) and the SAR of the soil moisture extract (from drain depth) can play a role as well. Nevertheless, the percentage of clay from pre-drainage investigations and the stability of the auger holes are likely to be the first indicators available. Subsequent testing of the soil in the laboratory will yield more parameters. Common to several guidelines is the use of the Plasticity Index (PI) as an indicator of sedimentation potential, and, in combination with hydraulic conductivity, as an indicator of the Hydraulic Failure Gradient (HFG) of the soil. HFG is not only dependent on the soil, but on the type of soil-envelope-pipe interface as well. Perforation that are too large do not support bridging.

The method of HFG determination never gained popularity in and outside the USA, probably because its application during the design phase was not clear. Examples of its application were given in the original article (Samani and Willardson 1981). In Egypt the method was certainly noted, but HFG calculations based on laboratory PI and auger hole method hydraulic conductivity values resulted in such, seemingly, high HFG values (pages 39 - 73, Abdel-Dayem 1985) that comparison with expected gradients at drain depths was not considered. Nevertheless, it seems the only empirical calculation method available that might succeed in providing a global standard. Moreover, the reported modification in the HFG formula and recent checks on various earlier works on HFG, would indicate that a closer look at the methodology is warranted.

To date, the best method to determine the need for drain envelopes is the construction of field test lines in a range of soils expected in the area under consideration for drainage. This is expensive and time-consuming (including planning ahead a minimum of 3 - 5 years). Construction of pilot areas generally lags behind the deadline for decision making (ISAWIP project in Egypt

(ISAWIP 1994), Rajad Project in India (Rajad staff 1995) and Fordwah Eastern Sadiqia project in Pakistan (personal communication with project staff, 1995). It is therefore recommended that more attention be paid to applying the HFG method for determination of the need for an envelope and, at the same time, stimulating research in the field. The effect of a high SAR also requires detailed laboratory research to come up with definite recommendations. Seeking relations between SAR, hydraulic conductivity and possibly other factors might enhance the HFG method results.

Not specifically mentioned in the foregoing text (and the literature) is the role of laboratory permeameter testing to determine the need for an envelope. HFG determination of typical project soils is one aspect, but combinations of envelopes and soil taken from drain depth can be tested as well. Testing soil-envelope combinations, however, implies anticipation of the need for a drain envelope.

6.2 Overview of criteria for design of drain envelopes

Recommended procedures for envelope design given in Chapter 3 are based on previous design criteria and recent research (1985 - 1994) performed in the Netherlands (Scholten 1988, Huinink, 1992, Stuyt 1992, Santvoort 1994), Pakistan (Vlotman *et al.* 1992 and 1994a), and Belgium (Dierickx and Van der Sluys 1990, Dierickx 1993a and 1994). Investigations are ongoing in Canada, USA, Pakistan, Egypt and India to primarily test some of the procedures and findings reported herein for local conditions. Field experiments with synthetic materials are ongoing in Pakistan, Egypt, India and the Netherlands. Findings by researchers in other countries not mentioned before have also been considered and can be found, among others, in Corbet and King (1993), Koerner (1994), and Karunaratne *et al.* (1994). A listing of conferences and the like, where material related to drain envelope design has been found, is given in information Box 23 and 24.

Box 23 Workshops and conferences related to drain envelopes and with published proceedings.

DRAINAGE (either editors or one of the articles are referenced).	
1978	*International Drainage Workshop, Wageningen, The Netherlands considered the First International Workshop on Land Drainage (ICID/CIID). (Wesseling, J. (Ed.)).*
1982	*Second International Workshop on Land Drainage (ICID/CIID). Washington DC, Dec 5 - 11, 1982, USA (Ochs and Willardson, (Eds.)).*
1982	*ASAE Fourth National Drainage Symposium, Chicago, USA, Dec. 13 - 14, 1982 (Kriz. G.J. (Ed.)).*
1986	*International Seminar on Land Drainage, Helsinki University of Technology, Finland (Saavalainen J., and Vakkilainen, P. (Eds.)).*
1986	*Second International Conf. On Hydraulic Design in Water Resources Engineering: Land Drainage, Southampton, UK April 1986 (Smith and Rycroft (Eds.), 1986).*
1987	*Third International Workshop on Land Drainage (ICID/CIID). The Ohio State University, Department of Agricultural Engineering, Columbus, Ohio, USA Dec. 7-11,1987 (Nolte(Ed.), 1987).*
1987	*Drainage Design and Management, Proc. of the 5th National Drainage Symposium. American Society of Agricultural Engineers Publ. 07-87, St. Joseph, Michigan, USA.(Carman and Gilmore 1987).*
1990	*4th International Drainage Workshop of the ICID/CIID, Cairo, Egypt, Feb. 23 - 24, 1990 (Lesaffre, B. (Ed.), 1990b).*
1990	*Symposium on Land Drainage for Salinity Control in Arid and Semi-Arid Regions, Feb 25 - March 2, 1990, Cairo, Egypt*
1990	*Workshop on Drain Envelope Testing, Design and Research, Lahore, Pakistan, Aug 23, 1990 (Vlotman (Ed.)).*
1990	*International Drainage Workshop, Rabat, Morocco, Nov. 27-30, 1990 (ICID, Daniane (Ed.),1990).*
1992	*5th International Drainage Workshop of the ICID/CIID, Lahore, Pakistan, Feb. 8 - 15, 1992 (Vlotman (Ed.)).*
1992	*Sixth International Drainage Symposium of the ASAE; Drainage and Water Table Control, Dec. 13 - 15, 1992, Nashville, Tennessee (Bonnell et al. 1992).*
1996	*6th International Drainage Workshop of the ICID/CIID, Lublijana, Slovenia, Apr 21 - 29, 1996 (Maticic 1996).*
1997	*7th ICID International Drainage Workshop. Penang, Malaysia. Nov 17 – 21, 97, (Vlotman et al. 1997).*
1998	*7th International Drainage Symposium. A Technology Update in Drainage and Water Table Control. Orlando, Florida, USA, Mar 8 - 11, 1998 (Karaman et al. 1998).*
2000	*8th International Drainage Workshop of ICID/CIID, Jan 31 - Feb 4, 2000, New Delhi, India. (Omara and Vlotman 2000).*

Box 24 Workshops and conferences related to geotextiles with published proceedings.

GEOTEXTILES (either editors or one of the articles are referenced).	
1977	*International Conference on the Use of Fabrics in Geosynthetics, Paris, France (no Eds.).*
1982	*Second Int. Conf. on Geotextiles, Las Vegas, Nevada, USA (Giroud, J.P. 1982).*
1986	*Third International Conference on Geotextiles, Vienna, Austria (Faure et al. 1986).*
1987	*Geosynthetics'87, New Orleans, IFAI, St. Paul, 1987 (Lombard and Rollin, 1987)*
1990	*4th Int. Conf. on Geotextiles, Geomembranes and Related Products, May 28 - June 1, 1990, The Hague, The Netherlands (Hoedt, G. den. (Ed.), 1992).*
1993	*UK Chapter of the International Geotextile Society. Sep 23, 92. Churchill College, Cambridge, UK (Corbet, S. and King, J. (Eds.)).*
1994	*5th Int. Conf. on Geotextiles, Geomembranes and Related Products, Vol. 1, 2, 3 & 4, Sep. 5- 9, 1994, Singapore(Karunaratne et al.(Eds.))*
1996	*Proc. Symposium on Recent Developments in Geotextile Filters and Prefabricated Drainage Geocomposites, ASTM (Bhatia, S.K., and Suits, L.D. (Eds.)).*
1998	*6th International Geotextile Conference, March 1998, Atlanta, USA (Rowe 1998)*
1998	*Filtration and Drainage Conf., ASCE, Oct 18 – 21, 1998, Boston, MA, USA (Vlotman 1998).*

6.2.1 Granular envelopes

In 1922, Terzaghi presented a mechanics-based theory for the piping and seepage forces that develop beneath hydraulic structures (Terzaghi 1922, Terzaghi and Peck 1948/1967, Prieto 1967). Terzaghi patented what he termed a 'reverse filter' which he then used to control seepage under an Austrian dam built on a pervious foundation. By relating the particle size of D_{15} of the filter to four times the particle size d_{15} of the base material he achieved that the filter material becomes roughly 10 times as pervious as the base material. Similarly, he established that by relating the D_{15} of the filter to 4^*d_{85} of the base material (Table 34; part A) he was able to prevent particles from washing through the filter material. The D_{15}/d_{85} ratio is the relative coarseness of the envelope material: the higher the ratio, the coarser the envelope material. If the relative coarseness of the envelope material is too high, the soil particles will be washed through it and into the drain.

Luthin and Reeve (1957) presented one of the earliest overviews of drain envelope design criteria. With reference to the work by Terzaghi, they reported on Bertram's work (1940) in the laboratory with permeameters in which layered filters were tested, on the work done by the US Waterways Experiment Station (1955, see also Vicksburg 1941), and on the work done by the United States Bureau of Reclamation (USBR, Table 34, part B).

The next overview of envelope materials was done by Willardson (1974) in which he distinguished organic, inorganic (naturally graded coarse sands and fine gravels), and man-made materials (fibreglass mats and sheets primarily).

Geotextiles are mentioned but no guidelines were available. The additional guidelines presented by Willardson are given in Table 34 parts C, D, E, and I. Some USBR guidelines were given (Karpoff 1955), but these were replaced by the guidelines published by USBR in their Drainage Manual of 1978 and presented in the revised reprint of 1993. The 1993 USBR guidelines contain one addition concerning the lower and upper boundaries of the hydraulic conductivity of the granular filter material (Table 34, part F). The 1973 publication of USBR on small dams gives the US Waterways findings but does not mention the D_{50}/d_{50} criteria (Table 34, part G).

At the same time Thanikachalam and Sakthivadivel (1974) of India presented an overview that included sources from Japan, Germany, US, France and a former East Block country, but the majority of the criteria are D_{50}/d_{50} ratios similar to those in Table 34, parts A, B, and E. Having access to some of the original data and combining them with their own work they developed some empirical formulas relating C_u and D_{10}/d_{10}, which are not reported here because essential background information on the logic of the formulas were not given.

Karpoff (1955) indicated that the gradation curves of envelope material and the base soil material should be approximately parallel on the semi-logarithmic graph for soils with fine sizes (0.1 - 0.002 mm, fine sands to clay). The idea was that the stability of the soil and the effectiveness of the filter depends on the skewness of the gradation curve of the envelope towards the gradation curve of the fines in the base soil material. Sherard *et al.* (1984b) found nothing to support for the need for parallel curves in their investigations of granular filters (see below for more detail).

The design of gravel packs for wells is thought to be similar to the design of envelopes for subsurface drains, except that the gradients are deemed higher, and conditions at the envelope-well interface, more severe than with subsurface drains. However, soil-envelope-drain interfaces in pumped subsurface drainage systems are also temporarily exposed to high gradients, when water remains standing above the pipe just after pumping has started and the pipe is not flowing full any more (Figure 38). As pumped subsurface drainage systems will also be subject to surging, it is appropriate to review the pack-aquifer ratios (D_{50}/d_{50}) determined by Kruse (1962), Table 34 part D. The uniformity coefficient (C_u, Section 5.5.1) was also considered as a design factor. According to Kruse (1962), material with a uniformity coefficient of up to 1.78 is uniform and therefore less desirable as a filter material. Sand movement into the well was reduced by increasing the uniformity coefficient of the gravel pack at all pack-aquifer ratios.

Pillsbury (1967) reports on experiences in the Imperial and Coachella Valleys of California, as well as the work by Qazi (1961) and Des Bouvrie (1962). They used a relationship between the standard deviation of the envelope material (Section 5.5.1, p 161) and the D_{50}/d_{50} ratio as a criteria for the effectiveness of a drain envelope (Figure 81) showing two zones: one indicating a satisfactory filter; the other an unsatisfactorily filter. Des Bouvrie further found that for Filter/Aquifer (F/A, or D_{50}/d_{50}) ratios of about 12, a filtering envelope need only be 0.5 - 1.0 inch thick. Gravel filters with successful F/A ratios and standard deviation combinations permit envelope thickness of: 3 inches (75 mm) for F/A = 12 - 24, 6 inches (150 mm) for F/A = 24 - 28, and 9 inches (230 mm) for F/A = 28-40. This led Pillsbury to conclude that on economic grounds it is advisable to recommend F/A = $D_{50}/d_{50} \leq 24$ (Willardson 1974). All of these relationships are empirical and are based on observations of envelopes that failed in the Imperial Valley of California. It was further observed that standard concrete sand (ASTM C33-64 in Pillsbury 1967, now ASTM C33-93, see Figure 10, p 51) which is produced in large quantities, and is therefore relatively inexpensive, can perform satisfactorily as filter material provided the F/A ratio and the standard deviation (s) combinations comply with Figure 81. Assurance of satisfaction can be attained if $D_{50} \leq 1$ mm and s ≥ 1 mm. The F/A ratio is also known as the pack-aquifer ratio used for the design of filter packs (gravel) placed around the casings of wells to increase the effective diameter of the well and to help prevent sediment entry (Kruse 1962).

The Soil Conservation Service (1971) seems to have combined the results of the research reported up to the late sixties, as described up to this point, into

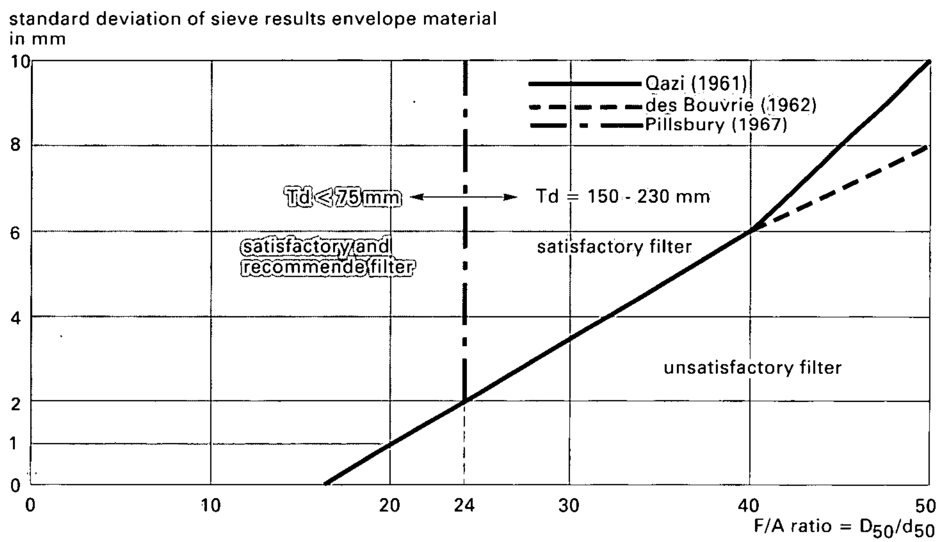


Figure 81 Filter aquifer ratio for granular drain envelope design. (after Des Bouvrie 1962, Qazi 1961 and Pillsbury 1967).

a specification for evaluating pit run and artificially-graded granular materials for use as drain envelope materials. The recommendation is for naturally-graded pit run materials or a mixture of medium and coarse sand with fine and medium gravel (Table 34, part I). Strangely enough, the SCS 1973 publication entitled Drainage of Agricultural Land does not recommend the SCS 1971 criteria for envelope design. Instead, it specifies the USBR criteria by Karpoff (Table 34, part E). For more global usage, the FAO in Rome published guidelines in FAO Irrigation and Drainage Paper No 28 (Dieleman and Trafford 1976). Design guidelines presented in FAO 28 are those in Table 34 parts B, F, I, and L.

In the USBR Drainage Manual (1978) guidelines for improving the flow towards drains by graded envelopes are given (Table 34, part F). They are based on work by Winger and Ryan (1970b) who concluded that filter criteria of the US Waterways Experimental Station (Table 34, part B) were too restrictive for agricultural use. Borings should be taken every 180 m (600 ft) along the centreline of the drain to determine the most permeable base material for significant lengths and based the envelope design on this. The envelope should be well-graded (C_u and C_c criteria), free of vegetative matter, clay and other deleterious materials, which could in time change the hydraulic conductivity. In Pakistan the following conditions were added to those of USBR (1978): carbonate content should not be more than 5%, the hydraulic conductivity should be between 15 and 305 m/d (50 - 1000 ft/d), and particles which are consistently disproportionate in length along one axis are not acceptable (Vlotman *et al.* 1990, USBR 1989, FDP 1987). When the majority of the envelope material passes sieve no. 30 (0.6 mm, pit run material), material passing US standard sieve no. 200 (0.074 mm) should be removed and hydraulic conductivity tests run on the remaining sample. This is to avoid uneconomical envelope particle ranges appearing among the criteria. Although the text does not mention this, it seems to imply that if in such a sample the hydraulic conductivity with respect to the base material is acceptable, the sample could be used for an envelope as long as it is to improve hydraulic conductivity only, with no need for a filter. Samples that have all material retained on US standard sieve no. 30 (0.6 mm) can be expected to have an adequate hydraulic conductivity. The particle size of the base material for which 60% is smaller is used to determine the lower and upper limits of the envelope gradation band (Table 34, part F). The 1993 publication (USBR 1993) added that in order to prevent segregation of the gravel envelope, the hydraulic conductivity of the envelope material should be limited to $K_{env} < 150$ m/d. Most likely, this is based on the experiences in Pakistan at the Fourth Drainage Project (see below) where gravel envelopes of crushed rock had very high K_{env} values and caused failure of the envelope. At the same time K_{env} should also remain 10 times higher than that of the base soil.

Table 34 Overview of existing design criteria for the use of sand and gravel around drainpipes.

A. FILTERS FOR HYDRAULIC STRUCTURES		
Terzaghi (1922)		
Permeability criterion	$D_{15} > 4d_{15}$	filter approx. 10 times as permeable as base material.
Retention criterion, all soils	$D_{15} < 4d_{85}$	
Terzaghi and Bertram (1922 and 1940)		
Uniform non-cohesive soils	$4 \leq D_{15}/d_{15} \leq 9$	
B. GRAVEL ENVELOPES		
Bertram 1940 (In: Luthin and Reeve, 1957)		
Loss of stability at	$D_{15} = 9\ d_{15}$ and $D_{15} = 6\ d_{85}$ for uniform sands.	
US Waterways Exp. Station 1941 (In: Luthin and Reeve, 1957)		
Loss of stability at	$D_{15} = 5d_{85}$	
US Waterways Exp. Station (In: Dieleman and Trafford 1976)		
For filtration	$D_{15} \leq 5d_{85}$	
	$D_{15} \leq 20d_{15}$	
	$D_{50} \leq 25d_{50}$	
For permeability	$D_{15} \geq 5d_{15}$	
USBR		
Uniform material	$D_{50}/d_{50} = 5 - 10$	
Graded material	$D_{50}/d_{50} = 12 - 58$	
C. GRAVEL ENVELOPES IN FINLAND (in Willardson 1974)		
Juusela (1958)		
Permeability criterion	$D_{15} > 4\ d_{15}$	
Filter criterion	$D_{15} < 3\ d_{85}$	

D. GRAVEL PACKS FOR WELLS (in Willardson 1974)

Kruse (1962) to prevent excessive movement of particles the largest permissible pack-aquifer ratios are:

Aquifer	*Gravel Pack*	*Pack-Aquifer Ratio* (D_{50}/d_{50})
Uniform	Uniform	$D_{50}/d_{50} < 9.5$
Uniform	Non-uniform	$D_{50}/d_{50} < 13.5$
Non-uniform	Uniform	$D_{50}/d_{50} < 13.5$
Non-uniform	Non-uniform	$D_{50}/d_{50} < 17.5$

E. USBR-PROTECTIVE FILTER CRITERIA FOR HYDRAULIC STRUCTURES (After Willardson 1974)

Karpoff (1955) for inverted filter with hydraulic structures and additional criteria for drains.

Uniform filter (natural)	$D_{50}/d_{50} = 5\text{-}10$	sub-rounded particles, non-cohesive.
Graded filter (natural)	$D_{50}/d_{50} = 12\text{-}58$	sub-rounded particles, non-cohesive.
	$D_{15}/d_{15} = 12\text{-}40$	sub-rounded particles, non-cohesive.
Graded filter (crushed rock)	$D_{50}/d_{50} = 9\text{-}30$	angular particles, non-cohesive.
	$D_{15}/d_{15} = 6\text{-}18$	angular particles, non-cohesive.
Additional (for drains)	$D_{100} \leq 80$ mm	to minimize segregation and bridging during placement.
	$D_5 \geq 0.074$ mm	to prevent excessive movement of fines into filter and drain.
	$D_{opening} \leq 0.5\ D_{85}$	opening of drain perforation to be adjusted to filter material used.

Curves of filter and base material parallel in the range of finer sizes.
If base material contains particles > 4.76 mm (US standard sieve no. 4) then base design on PSD curve of d < 4.76 mm material only.

F. USBR GRAVEL DRAIN ENVELOPE DESIGN (USBR 1978, 1993)

Non-filter envelope	$D_{100} < 38$ mm	(1.5" US standard sieve series, Table 15).
	$D_5 > 0.3$ mm	(sieve no. 50 US standard sieve series).
For well graded material	$C_u > 4$	for gravel.
	$C_u > 6$	for sands.
	$1 < C_c < 3$	for both gravel and sand.
new in 1993:	$K_{env} \geq 10\ K_s$	when $K_{env} > 150$ m/d material difficult to place without segregation.

gradation relationship between base material and diameters of graded envelope material:

Base soil limits for d_{60} (mm)	Lower limits (mm) percentage passing						Upper limits (mm) percentage passing					
	100	60	30	10	5	0	100	60	30	10	5	0
0.02-0.05	9.52	2.00	0.81	0.33	0.30	0.074	38.10	10.00	8.70	2.50	–	0.59
0.05-0.1	9.52	3.00	1.07	0.38	0.30	0.074	38.10	12.00	10.0	3.00	–	0.59
0.1-0.25	9.52	4.00	1.30	0.40	0.30	0.074	38.10	15.00	13.10	3.80	–	0.59
0.25-1.0	9.52	5.00	1.45	0.42	0.30	0.074	38.10	20.00	17.30	5.00	–	0.59

G. USBR FILTERS WITH SMALL DAMS (USBR 1973) based on Vicksburg (1941) and USBR (1955)

Permeability	(1) $D_{15}/d_{15} = 5 - 40$	provided that (2) $D_5 > 0.074$ mm (sieve no. 200).
Stability	(3) $D_{15}/d_{85} \leq 5$	PSD curve of filter roughly parallel to that of base soil.
	(4) $D_{85}/D_{opening} \geq 2$	all 4 criteria for natural gravel and sand, or crushed rock.
Segregation	(5) $D_{100} < 75$ mm	also to prevent bridging of large particles during placement.
for PSD curve of base material	(6) $d_{100} < 4.74$ mm	Sieve no. 4. For base soils that contain gravel.

H. US ARMY, OFFICE OF THE CHIEF OF ENGINEERS, ENGINEER MANUAL (US Army Corps of Engineers 1978)

valid for all soils except CL and CH soils (Figure 74), then D_{15} may be as great 0.4 mm and discard second criterion.

Stability (retention)	1. $D_{15}/d_{85} \leq 5$	criteria do not apply for gap graded soils.
Stability (retention)	2. $D_{50}/d_{50} \leq 25$	PSD curve of filter roughly parallel to that of base soil.
Permeability	3. $D_{15}/d_{15} \geq 5$	assumes $K_s = f(D_{15}^2)$, implying $K_{env}/K_s \approx 25$.

Criteria for perforated pipes/screens: $D_{50}/D_{opening} \geq 1$ (circular openings), D_{50}/D_{slot} width ≥ 1.2 (slotted openings).

I. SCS DRAIN ENVELOPE CRITERIA (SCS 1971)

SCS criteria for envelope (SCS 1971, revised in 1988, see below)

Graded envelope	$D_{50}/d_{50} = 12 - 58$	minimal thickness 3" (75 mm).
	$D_{10} \geq 0.25$ mm	0.25 mm = US standard sieve no. 60 (Table 15).
	$D_{15}/d_{15} = 12 - 40$	
Uniform envelope	$D_{15}/d_{85} < 5$	
	$D_{85} \geq 0.5$ * Diameter of the perforations.	

Table 34 cont.

J. SCS REVISED DRAIN ENVELOPE CRITERIA (SCS 1988 and also in SCS 1991)		
SCS criteria for filter gradation (SCS 1988)		
	$D_{15} < 7\ d_{85}$	but D_{15} need not be smaller than 0.6 mm[35].
	$D_{15} > 4\ d_{15}$	
	$D_5 > 0.074$ mm	% passing US standard sieve no. 200 (Table 15) less than 5%.
SCS criteria for envelope (surround) (SCS 1988)		
	$D_{100} < 38.1$ mm	the whole sample should pass the sieve of 1.5" (38.1 mm).
	$D_{30} > 0.25$ mm	% passing US standard sieve no. 60 less than 30%.
	$D_5 > 0.074$ mm	% passing US standard sieve no. 200 less than 5%.

K. SCS GRADATION DESIGN OF SAND AND GRAVEL FILTERS FOR HYDRAULIC STRUCTURES (SCS 1968, 1986, 1994)

Only consider materials less than 4.75 mm (no. 4 sieve) for PSD curve: adjust percentages if material on Sieve no. 4

Base category soil	% finer then 0.074 mm[36] after regrading, where applicable.	Base soil description	Filtering criterion, Maximum D_{15}	Permeability criterion, use D_{15} before re-grading[37] Minimum D_{15}
1	> 85	Fine silt and clays	$D_{15} \le 9d_{85}$ but not less than 0. 2 mm[38]	$D_{15} \ge 4d_{15}$ but not less than 0.1 mm
2	40 – 85	Sands, silts, clays, and silty and clayey sands	$D_{15} \le 0.7$ mm	
3	15 – 39	Silty and clayey sands and gravel	$D_{15} \le [[(40\text{-}A)/(40\text{-}15)]\ [4d_{85} - 0.7\text{ mm}] + 0.7\text{ mm}]$ A = % passing #200 sieve if $4d_{85} < 0.7$ mm, use 0.7 mm	
4	< 15	Sands and gravel	$D_{15} \le 4d_{85}$	

Regrading:	using d < 4.75 mm results only to prepare PSD curve	
prevent gap-graded filter:	ratio max. particle diam. to min particle diam. ≤ 5 for any given percent passing < 60% on the PSD curve.	
coarse and fine filter bands:	$C_u \le 6$ for economic or practical reasons C_u may be adjusted but for diameters above 15% passing $C_u \ge 2$.	
	$D_{100} \le 75$ mm (3")	the minus no. 40 sieve (0.425 mm) material must be non plastic as determined in accordance with ASTM D4318.
	$D_5 > 0.074$ mm	

35 Original text read: "but not smaller than 0.6 mm" (change explained in main text).

36 SCS 1994 uses 0.075 mm, both are meant to be Sieve no. 200 of the standard sieve set.

37 Sherard *et al.* 1984b recommended K_{env} be determined from material less then Sieve No 4 only; so the regarded PSD curve (see also Section. 5.4).

38 As with the $D_{15} > 0.6$ mm of SCS 1988, Sherard *et al.* 1984b remark that for soils of category 1 that "a filter with an average D_{15} of 0.2 mm or smaller probably is conservative for the finest silt or clay". Also Sherard *et al.* 1984b found $D_{15}/d_{85} < 5$ conservative and $D_{15}/d_{85} > 9$ always failed: SCS 1988 compromised: $D_{15}/d_{85} < 7$.

K cont. SCS GRADATION DESIGN OF SAND AND GRAVEL FILTERS (SCS 1994)

Segregation criteria:

Base soil category	If D_{10} is:	Then maximum D_{90} is:
all categories	< 0.5 mm	20 mm
	0.5 - 1.0 mm	25 mm
	1.0 - 2.0 mm	30 mm
	2.0 - 5.0 mm	40 mm
	5.0 - 10 mm	50 mm
	> 10 mm	60 mm

Filters adjacent to perforated drain pipe:

steady flow conditions: $D_{85} \geq D_{opening}$

surging or reversal flow: $D_{15} \geq D_{opening}$

L. UNITED KINGDOM ROAD RESEARCH LABORATORY CRITERIA (Spalding 1970 in Boers and van Someren 1979)

For filtration	1a	$D_{15} \leq 5d_{85}$	from US Waterways Exp. St. see B.
	2a	$D_{15} \leq 20d_{15}$	from US Waterways Exp. St. see B.
	3	$D_{50} \leq 25d_{50}$	from US Waterways Exp. St. see B.
For permeability	4	$D_{15} \geq 5d_{15}$	from US Waterways Exp. St. see B.
Only for uniform soils ($C_u \leq 1.5$)			
criterion 2a changes into:	2b	$D_{15} \leq 40d_{15}$	
and for well-graded soil ($C_u \geq 4$)			
criterion 1a changes into:	1b	$D_{15} \leq 6d_{85}$	
		$D_{85} \geq$ perforation width/0.83	

M. DESIGN CRITERIA FOR DOWNSTREAM PROTECTION OF HYDRAULIC STRUCTURES (Bos 1978/1989, 1994b, Bos *et al.* 1984)

Permeability to water

1. Homogeneous round grains (gravel)	$D_{15}/d_{15} = 5 - 10$	USBR (1973) and Bertram (1940).
2. Homogeneous angular grains (broken gravel, rubble)	$D_{15}/d_{15} = 6 - 20$	USBR (1973) and Bertram (1940).
3. Well-graded grains	$D_{15}/d_{15} = 12 - 40$	USBR (1973) and Bertram (1940).
4. To prevent clogging	$D_5 \geq 0.75$ mm (0.03")	

Stability (or prevention of loss of fines)

1. Uniform soil	$D_{15}/d_{85} \leq 5$	Bertram (1940).
2. Homogeneous round gains (gravel)	$D_{50}/d_{50} = 5 - 10$	US Army COE (1955).
3. Homogeneous angular grains (broken gravel, rubble)	$D_{50}/d_{50} = 10 - 30$	US Army COE (1955).
4. Well-graded grains	$D_{50}/d_{50} = 12 - 60$	US Army COE (1955).

N. DRAIN ENVELOPE CRITERIA DEVELOPED FOR USE IN PAKISTAN (Vlotman *et al.* 1994a, 1995, Shafiq-ur-Rehman 1995)

Filter criterion	$D_{15} < 7{*}d_{85}$	
Filter and Hydraulic criterion	$D_{15} > 0.3$ mm	
Filter criterion	$D_{15} > 4{*}d_{15}$	(NB. actually not filter but rather hydr. criterion).
Hydraulic criterion	$D_5 > 0.074$ mm	prevents too low hydraulic conductivity.
Construction criterion	$D_{40} < 4.8$ mm	to prevent segregation.
Construction and Hydraulic criterion		
river run material:	$D_{100} < 19$ mm	prevents segregation and macro pores.
crushed rock:	$D_{100} < 9.5$ mm	
Construction criterion	All openings should be covered by at least 76 (3") of filter material.	

Table 34 cont.

O. DRAIN ENVELOPE CRITERIA AS GIVEN IN THIS BOOK		
Control points coarse boundary envelope material bandwidth (c and f in subscript refer to course and fine base soil bandwidths):		
1. Filter, retention criterion	$D_{15c} < 7*d_{85f}$	SCS (1988)
2. Gradation curve guide	$D_{60c} = 5*D_{15c}$	
3. Segregation criterion	$D_{100} < 9.5$ mm	based on Pakistan findings (Shafiq-ur-Rehman 1995).
Control points fine boundary envelope material bandwidth:		
4a. Hydraulic criterion	$D_{15f} > 4*d_{15c}$	
4b. Gradation curve guide(bandwidth)	$D_{15f} > D_{15c} / 5$	Based on $Cu \leq 6$ and bandwidth ratio ≤ 5.
5. Hydraulic criterion	$D_5 > 0.074$ mm	
6. Gradation curve guide (bandwidth)	$D_{60f} > D_{60c} / 5$	bandwidth ratio ≤ 5 below 60% passing on PSD curve.
7. Retention criterion (bridging)	$D_{85} > D_{opening}$	
Construction criterion	All openings should be covered by at least 76 (3") of filter material. The envelope should not contain deleterious material.	
Crushed material	no particles disproportional larger in one direction by factor 2 compared to shortest dimension. 21-sieve analysis to check for missing particle ranges. $K_{env} < 300$ m/d.	

Filter design for hydraulic structures, which may have relevance to drain envelope design was also reported by Bos (1978/1989) and are given in Table 34, part M. The same set of criteria are also given by Bos (1994b), with an example of how to design a filter with successively coarser gravel underneath rip-rap downstream of energy-dissipating structures in irrigation and drainage applications. These criteria are essentially Bertram's (1940), USBR's (1973) and of the US Army Corps of Engineers (1955, 1978). Although not directly applicable to drain envelopes, some insight may be gained from the differentiation of filter properties for differently shaped gravel: when the gravel of the filter becomes finer a larger ratio of D_{15}/d_{15} is allowed. The differentiation in classes of filter materials, such as uniform, round grains, angular grains, etc., was not reported in the what seemed to be the original sources, rather the total range of ratios from 5 - 40 (D_{15}/d_{15}) and 5 - 60 (D_{50}/d_{50}) were reported (USBR 1973, US COE 1955, 1978). Flow velocities at the outflow of a structure were related to the stability of the d_{40} particle size.

The US Army Corps of Engineers (1978) gives guidelines for drains in conjunction with levees and perforation sizes of drains to prevent infiltration of sediments into the pipe drain (Table 34, part H). The criteria are not applicable to gap-graded (or skip-graded) soils. They point out that it may not be possible to meet all the criteria within one gradation band but that two successively finer filters may have to be applied, similar to the methodology described by Bos (1994b). This might be possible when a filter is used as a blanket over the drain, which of course is not possible with modern trenching equipment.

Sherard *et al.* (1984a and b) also checked filter criteria for protection of hydraulic structures. Although not intended for application in subsurface drainage, the principles were directly applied to the design of gravel envelopes and also to the selection of synthetic envelope materials. The results were subsequently used to modify the SCS criteria for drain envelopes (SCS 1988). Both papers give excellent background material on earlier work in the laboratory by Lund (1949), Bertram (1940), and USBR (1955) as well as their own work for SCS. The reader is advised to obtain copies of both papers. A summary of the key results of their work is given in the next two paragraphs.

Sherard *et al.* (1984a) noted that the critical or minimum sizes of pore channels for a sand or gravel filter are governed more by the d_{15} than by the d_{50} particle size. Therefore, there is no theoretical or experimental basis for using $D_{50}/d_{50} < 58$ (USBR 1955) or any other limits referred to in Table 34, parts B and E. Use of D_{50}/d_{50} is sometimes defended on the grounds that it will prevent segregation, but segregation is more controlled by prescribing the range of particle sizes, rather than a limit to individual particle sizes and has nothing to do with the base material properties. Sometimes $C_u < 20$ is used to prevent segregation, but this is also not considered satisfactory as the particle size greater than D_{60} such as D_{90} generally governs the magnitude of segregation. Similarly, the upper limits imposed on D_{15}/d_{15} ratios have no theoretical or experimental basis. Sherard *et al.* 1984b had many successful tests where D_{15}/d_{15} ratios exceeded 1000. However, the size ratios used for stability and filter action were found to be critical; materials with a D_{15}/d_{85} ratio greater than 9 always failed. The filter criteria $D_{15}/d_{85} < 5$ was found to be slightly conservative. Sherard *et al.* (1984a and b) also established that if a filter did not fail with the initial flow of water, it was probably safe, permanently. Well-graded materials were more successful than uniformly-sized materials.

In Sherard *et al.* (1984b), tests on filter and base soil sizes (sequence similar to SCS (1994) base soil categories in Table 34, part K) using fine textured soils resulted in the following:

1. exceptionally fine soils (d_{85} <0.02 mm): D_{15} < 0.2 mm or smaller is conservative (safe);
2. fine-grained silts of low cohesion (d_{85} of 0.03-0.1 mm, PI below the 'A' line and with liquid limit less than 30, see Figure 74): D_{15} < 0.3 mm is conservative (safe);
3. fine-grained clays (d_{85} of 0.03-0.1 mm): D_{15} < 0.5 mm is conservative (safe); and
4. sandy silts and clays (d_{85} of 0.1-0.5 mm): $D_{15}/d_{85} \leq 5$ is conservative (safe).

The SCS (1994) has made some modifications to the maximum D_{15} sizes allowed. A less conservative value of 0.7 mm seems to have been used rather than the 0.3 and 0.5 mm, although the main text of SCS (1994) does allow for the lower value in non-critical situations (situations with low gradients such as bedding below rip-rap and concrete slabs used for downstream erosion protection with hydraulic structures).

Sands and gravely sands containing fine sand sizes with a D_{15} = 0.5 mm or less would be a suitable filter for even the finest clays (Sherard *et al.* 1984b). For clays with some sand content ($d_{85} > 1.0$ mm), a filter with a D15 = 0.5 mm would satisfy the $D_{15}/d_{85} \leq 5$ criterion. For finer clays, the $D_{15}/d_{85} \leq 5$ is not satisfied, but the finer soils tend to be structurally stable and are not likely to fail. A well-graded gravely sand was an excellent filter for very uniform silt or uniform fine sand, and it was not necessary that the grading curve of the envelope be roughly the same shape as the grading curve of the soil for the finer range. A fairly uniform sand with C_u = 2 - 5, adequately prevented sediment transport of a broadly graded fine-grained clay with C_u = 50. Gravel envelopes that have a D_{15} = 0.3 mm and a $D_{15}/d_{85} \leq 5$ with less than 5 percent of the material finer than 0.074 mm will be satisfactory as envelope materials for most problem soils (Sherard et al, 1984b).

Based on the work done by Sherard *et al.* (1984a and b) the Soils Conservation Service (SCS 1988) came up with new drain envelope criteria (Table 34, part J). The 1988 SCS criteria are less prescriptive than the 1971 SCS criteria (i.e. no guidelines at D_{50} and d_{50}). The SCS suggests use of filters if: (1) local experience dictates; (2) soil materials surrounding the pipe are dispersed clays, low plasticity silts, or fine sands (ML or SM[39] with Plasticity Index < 7); (3) deep soil cracking is expected; and (4) where the method of installation may result in voids between the pipe and the backfill material. If a sand-gravel filter is specified it should be based on the properties of the surrounding base material. Use of the 1988 SCS criteria in Pakistan demonstrated that the criterion "$D_{15} < 7d_{85}$ but not smaller than 0.6 mm" caused some difficulties. The 0.6 mm limit could not be met without creating serious problems in the gradation curves of the potential/required envelope material (Vlotman *et al.* 1993a). The $D_{15} > 0.6$ mm is simply too coarse. Sherard et al (1984b) did not perform tests with material smaller than 0.6 mm and all performed well as a filter. Consequently, the conclusion was that there was no additional benefit in having D_{15} being smaller than 0.6 mm. Particle sizes smaller than 0.6 mm are allowed provided that the permeability criterion of $D_{15} > 4d_{15}$ is met. This item was discussed with James Talbot, then SCS National Drainage Engineer, and others (Vlotman *et al.* 1993a) and it was agreed that the SCS 1988 criteria actually should read "$D_{15} < 7d_{85}$ but need not be smaller than

[39] From the Unified Soil Classification System (Figure 74)
ML - Inorganic silts, very fine sands, rock flour, silty or clay fine sands.
SM - Silty sands sand - silt mixtures.

0.6 mm". Wenberg and Talbot (1987) who give an overview of the history of envelope design of the SCS also observed this.

SCS 1988 added the criteria $D_5 > 0.074$ mm, but this is not based on the work by Sherard et al (1984 a and b) which do not mention this as a criteria. They merely state that all their conclusions were based on tests and observations of sand and gravel samples that did not have any significant amount of material finer than no. 200 sieve (0.074 mm). Why this criterion was added is not mentioned in SCS 1988 or 1994. The D_5 criterion seems to be based on the fact that soil samples with more than 5% of the material passing sieve no. 200 were not tested. Furthermore, Sherard *et al.* 1984a remark that their conclusions are valid for primarily uniform filters, but do not differ much for well graded filters with C_u up to 10.

According to SCS (1988) material based on ASTM-C-33 (ASTM 1996a), fine aggregate for concrete, has been used satisfactorily as envelope material and is readily available (in the US). Standard gradations by ASTM are commonly used for filter and drain zones in embankments, retaining walls and other applications (SCS 1994), and some of the gradations applicable for drainage filters are shown in Figure 10.

The SCS did not include the observations by Wenberg and Talbot (1987) and Vlotman *et al.* (1993a) in Chapter 26 of the SCS National Engineering Handbook (SCS 1994, Table 34, part K), because as the introduction clarifies, the 1994 publication update includes more user-friendly design examples that are primarily based on the work done during 1980-1985 in the SCS Soil Mechanics Laboratory in Lincoln, Nebraska. Moreover, the criteria are meant for determining the grain size distribution (gradation) of sand and gravel filters needed to prevent internal erosion or piping of soil in embankments or foundations of hydraulic structures. Inherent in the SCS 1994 procedures is that there is first a layer that works as a filter, and then a layer of coarser material or a perforated drain with the primary function of conveying drainage water. This is somewhat different from the conditions encountered with agricultural drains.

The SCS publication also accommodates the need for some more control points in the 50 - 60% passing range of the PSD curves (something that was observed before as having been let go in the 1988 guidelines), and provides guidance for selecting the D_{90} size in relation to the D_{10} size to prevent segregation. Slightly modified control point procedures are used herein (Chapter 3). Paramount in the SCS 1994 criteria is also the regrading of the PSD curve, by excluding all material > 4.75 mm (Sieve no. 4) for various criteria (Table 34, part K). The guidelines pay attention to filters adjacent to perforated pipe and distinguish conditions that may be described as steady flow, and conditions where surging and reversal of flow may be encountered. For surging and reversal flow conditions $D_{15} > D_{opening}$ is recommended and for steady flow conditions $D_{85} > D_{opening}$. This essentially ignores potential bridging (Table 11, p 145) of finer particles for the non-steady flow conditions, which may be advisable.

Also, the design examples given in the SCS 1994 publication clearly show that granular filters are primarily expected to function as filter or as drainage medium. They are not expected to fulfil both functions at the same time. One example is given where the filter design is adjusted for use with a perforated pipe. The main adjustment is to meet the relationships of filter material with drain opening (Table 34, part K). However, their example also clearly illustrated the incompatibility of filter criteria, bridging criteria and hydraulic or permeability criteria: a three-stage filter is recommended, while in the description of additional considerations (practical and economical reason for supplying the recommended filter band as well as economics of it) the use of a fine sand and coarse gravel filter are suggested. This is neither practical nor possible with modern pipe-laying equipment unless one regresses to old practices of only a filter on top of the pipe, which of course is no longer deemed acceptable. The SCS (1994) deviates from Sherard *et al.* (1984a) on the issue of permeability: SCS recommends the application of the permeability criteria using the original PSD curve of the envelope material before regrading, whereas Sherard et al (1984a) recommend that K_{env} be determined from the D_{15} of the regraded PSD curve.

Crushed-rock drain envelopes failed in the Fourth Drainage Project (FDP) in Pakistan, even though they had been designed in accordance with specifications in the USBR Drainage Manual (Table 34, part F). This failure led to an

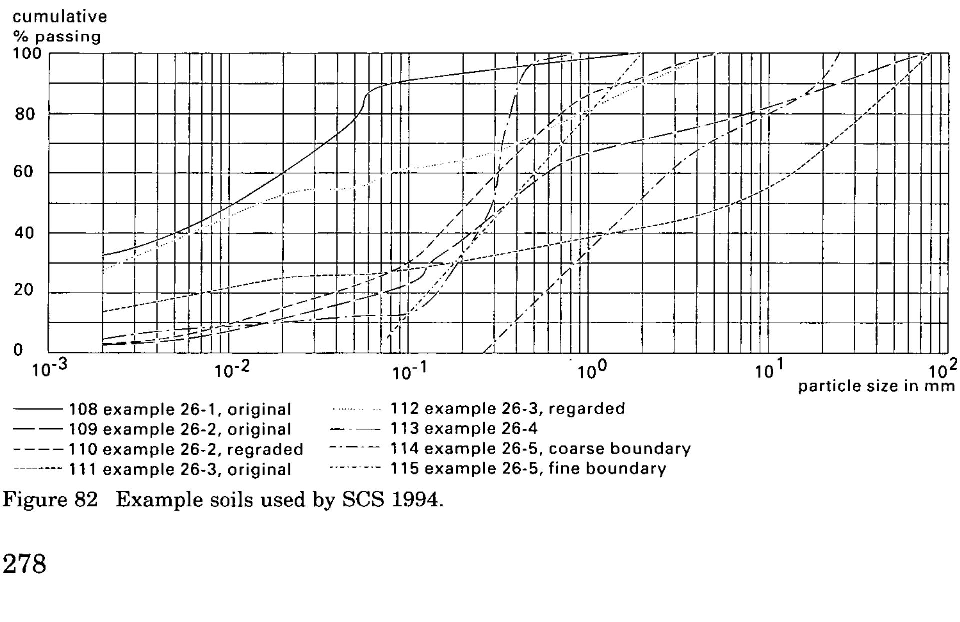


Figure 82 Example soils used by SCS 1994.

intensive review of existing design criteria for drain envelopes and to laboratory testing of potential envelope materials - both gravel and synthetics (Vlotman *et al.* 1992). The review of envelope and filter criteria - primarily those of the Soil Conservation Service (SCS 1971, and 1988) and of the United States Bureau of Reclamation (USBR 1978) - showed that drain envelopes designed with USBR criteria are coarser than those designed with SCS criteria. USBR specifications are suitable for envelopes that improve flow characteristics around the pipe (a surround), but are less suitable for envelopes that are expected to function as filters as well (Willardson and Ahmed 1988, using SCS 1971 criteria). This conclusion still applies when considering the latest publications and guidelines of USBR (1993) and SCS (1991, 1994) although the SCS 1994 allows coarser material to be used in 2 or 3 stage filters, which are not deemed applicable for agricultural drainage applications.

Analysis in the field (Vlotman *et al.* 1990) with specially-laid test lines of crushed rock (i.e. with angular-shaped particles) and river run material (i.e. with rounded particles) showed that river run material performed much better, although they are said to possess the same characteristics. Only two tests were able to distinguish between the two materials: (1) the 21-sieve analysis (Section 5.5.1) and (2) the hydraulic conductivity test (Section 5.5.4). The 21-sieve analysis showed that certain particle ranges (US standard sieves serial nos. 8 - 16, with sizes 2 – 0.21 mm) were missing (Figure 56, p 75), a fact that remained hidden in the standard semi-log particle size distribution (PSD) curves. Although the curves did not show the typical gap-graded characteristics, the missing particle sizes did represent a gap that altered the hydraulic conductivity significantly. Hydraulic conductivity of crushed rock was generally 100 times higher than the river run material with apparently the same PSD.

Findings in the Rajad Project (Rajad staff 1995) also demonstrated unsatisfactory performance of crushed rock in the field. Yet, Sherard *et al.* (1984a) found that crushed rock filters with angular particles did well in the laboratory (compacted to 70% relative density) and actually had a permeability lower than that of sub-rounded alluvial particles. One major factor leading to the trouble with crushed rock may have been attributable to the method of crushing and handling of the rock and the type of rock crushed: in Pakistan, particle sizes between 0.21 and 2 mm were lacking in the gravel envelope material that came from a commercial crushing plant. Sherard *et al.* (1984a) crushed limestone in the laboratory and therefore had better control over the gradation. Moreover, under laboratory conditions the crushing might have resulted in more uniformly-arranged particle sizes. Vlotman *et al.* (1992) reported obtaining comparable results with the rounded and angular particles in the laboratory permeameter tests, but no details on particle distribution were given.

Based on the laboratory tests and performance of envelopes in the field, the following is the list of recommendations for use in Pakistan (Vlotman *et al.* 1995):

1. To select the best potential gravel envelope material, a 21-sieve analysis and hydraulic conductivity test of the gradations proposed should be performed.
2. Drain envelopes in typical soils in Pakistan need to function as a filter. SCS 1988 criteria, with modifications as suggested below, should be used for gravel specification design based on the project base soils at drain depth.
3. The SCS 1988 criteria that the maximum particle size should be smaller than 1.5" ($D_{100} < 38$ mm) still seems to be somewhat on the coarse side. In Pakistan it is recommended that $D_{100} < 19$ mm be used (Note, Sherard *et al.* (1984b) allowed particle sizes up to 50 mm (2") when considering the danger of segregation, see also point 6 below).
4. The criteria of SCS 1988 that specifies that D_{15} should not be less than 0.6 mm ought to be relaxed; tests with D_{15} of 0.28 - 0.5 mm performed equally well in the laboratory. This essentially confirmed the interpretation of Sherard *et al.* (1984a) who found that D_{15} need not be smaller than 0.6 mm to work as a filter, but that the use of finer materials is acceptable. Hydraulic conductivity of the finer envelopes remained 10 to 20 times greater than that of the surrounding base soil. $D_{15} > 0.3$ mm is suggested (No more than 15% passing US standard sieve no. 50).
5. To partially or fully offset possible concern about increasing entrance resistance to the pipe as a result of proposed finer granular envelopes, perforations must be prescribed in every corrugation instead of every other corrugation as was common practice in Pakistan in 1993. This will halve the entrance resistance and is of particular importance when synthetic envelopes are proposed or considered.
6. Segregation during transport primarily depends on the amount of larger particles (D_{90}). For conservative practice and to avoid excessive segregation, less than 60% of well-graded envelope material should be coarser than no. 4 sieve (4.8 mm) of the US standard sieve set. Furthermore the maximum particle size should be < 50 mm (Sherard *et al.* 1984b). In other words, the material retained at sieve no. 4 should not be more than 60%. In terms of material passing no. 4 sieve this means: $D_{40} < 4.8$ mm.

The particle size gradation curve of the envelope material does not need to be parallel to the base soil curve. Rehman (1995) refined criterion number 3 to read crushed rock $D_{100} < 9.5$ mm and river run material $D_{100} < 19$ mm. Based on the above, the recommended criteria for Pakistan are given in Table 34, part N.

There is no evidence that it is necessary to prescribe the maximum content of carbonates in the envelope material (i.e. at the East Khairpur Tile Drainage Project in Pakistan, carbonate content was over 80% and at Nawabshah

Interceptor drain more than 50%. To date both envelopes continue to perform well Vlotman *et al.* 1992).

6.2.2 Conclusions on design of granular envelopes

Over the years a large array of criteria have been presented for the design of filters with granular material (mostly gravel, or sand and gravel filters). For filters with subsurface drainage pipes, filtering of fines was not the only criteria, hence criteria to assure minimum hydraulic conductivity of the envelope material to be used were given. More recently it was also found necessary to put an upper limit to the hydraulic conductivity of the envelope, because high hydraulic conductivity of the envelope material caused failure of the filtering function and seemed to enhance segregation. Whereas in the past it was found necessary to pay much attention to the form and shape of the envelope material used (i.e. oblong shapes and crushed rock), present criteria that restrict the coarse particles to less then 9.5 mm (D_{100} < 9.5 mm) take care of the need for further differentiation of the particle shape characteristics. This conclusion only holds for use with agricultural drains. For application with hydraulic structures, where filters can be built up in several layers, there is still a need for a more precise definition of the coarser filter material.

Most of the research, tests and experiences with the D_{15}/d_{85} ratio seem to confirm its usefulness as a retention criterion, while the ratio has been confined to values 4 - 9 over time from Terzaghi's first introduction in 1922 to the more recent work by Sherard *et al.* (1984a and b), SCS (1994) and work in Pakistan (Vlotman *et al.* 1994a and b, 1995). Selecting a higher value of the ratio implies less filtering capacity and a lower chance of bridging. However, it is questionable whether the D_{15}, D_{50}, D_{85} or D_{90} values of the envelope should be used for judging bridging criteria for pipe perforation dimensions (Box 15), and whether they should be one to one values or ratios such as given in Table 11 and Section 5.2, which generally recommend ratios greater than 1.

The D_{15}/d_{15} ratio seems to serve two purposes: first, as hydraulic criterion and second, as retention criterion. The hydraulic criterion is based on the relationship between the square of d_{15} or D_{15} and the hydraulic conductivity, as certain researchers reported (Section 5.4). However, neither did the data sets of gradation and hydraulic conductivity measurements nor the reported hydraulic conductivities (Rehman and Vlotman 1993, Santvoort 1994, and Muth 1994) confirm the hydraulic relationship conclusively, consequently judgement should be done with caution. A ratio of 4 for D_{15}/d_{15} implies a hydraulic conductivity of the envelope of approximately 10 times that of the base soil (16 in theory). When ranges such as 12 - 40 or 6 - 18 are given for D_{15}/d_{15} the lower boundary assures the hydraulic conductivity, while the upper boundary is intended as a filtration criterion. Hence criteria that specify a lower D_{15}/d_{15} ratio denotes permeability or hydraulic criteria, while a higher D_{15}/d_{15} denotes bridging or retention criteria. It is important to differentiate between bridging of adjacent granular materials (i.e. step filters or base soil-envelope interface) and bridging between pipe perforation (generally uniform in size) and granular envelope material. Bridging between synthetic envelope material and base soil is also important, but this is dealt with in other sections.

The D_{50}/d_{50} ratio seems to have no value as a retention or hydraulic criterion. It might have had some practical value however, because it provides a guide to approximately where the gradation curve should be - halfway on the PSD curve - and the assurance that the designed gradation range is more or less parallel to the gradation of the range of base soils. However, the most recent work by Sherard *et al.* (1984a) suggested that the curves do not need to be parallel to each other, and hence the D_{50}/d_{50} ratio was dropped from the 1988 SCS criteria. Strict D_{50}/d_{50} ratios are therefore no longer necessary. If one wishes to design envelope PSD curves more or less parallel to soil PSD curves one can use the C_u or C'_u of the base soil and determine an approximate D_{60} from $D_{60} = D_{15}/1.2 * C_u$ (where $D_{10} = D_{15}/1.2$). SCS 1994 recommends a factor of 1.2 to convert from D_{15} to D_{10}. The D_{60} is close enough to D_{50} to give some guidance for the midway point of the gradation bands recommended for envelope material (for details see examples in Chapter 3).

The USBR is particularly keen on preventing too many fine particles in the granular envelope that will reduce the hydraulic conductivity to unacceptable (low) values, while sometimes the argument that too many fines in the gravel could cause clogging of the drainpipe itself has been used as well. To prevent this they recommend the use of $D_5 > 0.3$ mm for non-filtering envelopes. As can be seen from the lower limits in Table 34, part F, all envelopes designed with USBR criteria (filtering or non-filtering) have as lower gradation limit $D_5 > 0.3$ mm. The SCS (1988, 1994) recommends $D_5 > 0.074$ mm (US Standard sieve no. 200) based on the fact that no soils were tested with significant amounts passing sieve no. 200 (0.074 mm).

Work in Pakistan showed that excavated envelope material of the natural material (river run), which worked very well, had between 10 - 15% passing at 0.3 mm and up to 10% passing at 0.074 mm, even though the material installed had less than 5% at 0.3 mm and about 2% at 0.074 mm (Figure 83A). Fines of the base material washed into the envelope before a well-functioning filter was established. Thus, it would seem that prescribing a ratio like $D_5/d_5 > 10$ would be more meaningful. This is assuming that the difference between D_5 of the envelope and d_5 of the base soil would be adequate to assure a difference in hydraulic conductivity equivalent to that achieved by prescribing $D_{15}/d_{15} > 4$. From simple calculations of amount of fine sediments in the enve-

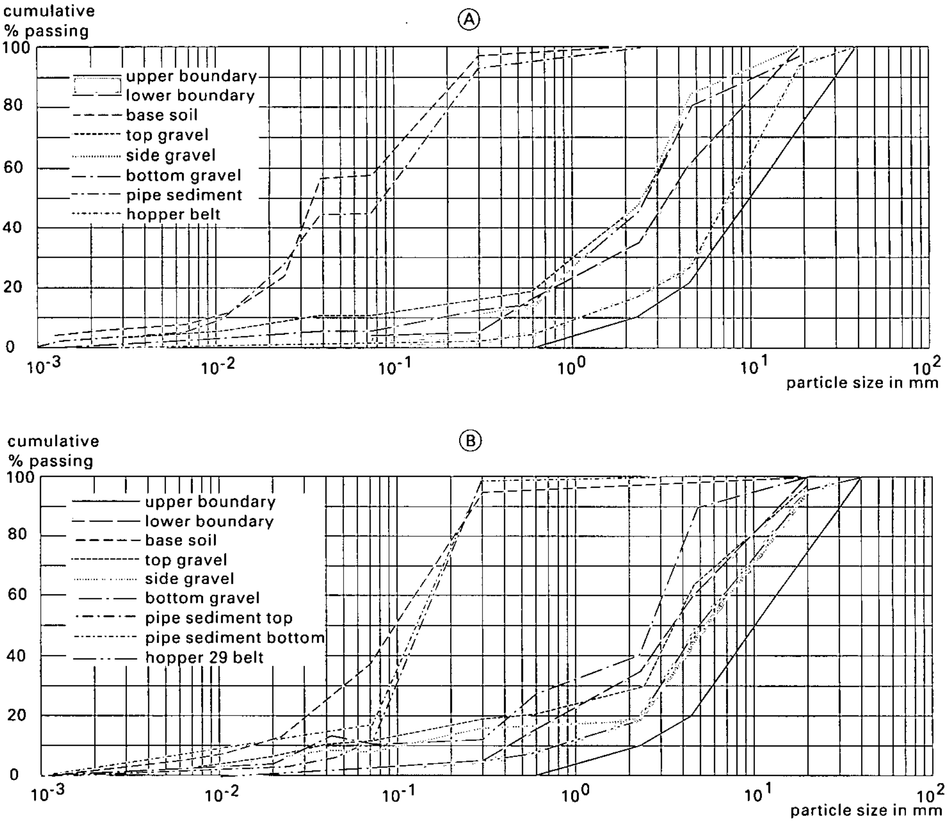


Figure 83 Gradation of envelope material before and after placement. (Vlotman *et al.* 1990)

A River run envelope material compared with upper and lower boundary of the FDP specifications before (sample taken from hopper belt just before placement) and after placement, 100 ft from manhole. Sediment in pipe also sampled. Envelope material succesfully retained fines.

B Same as A for crushed rock envelope material, 125 ft from manhole. Envelope material failed completely to retain sediment. Large amount of sediment in pipe sampled at the bottom and top of the 100 mm sediment layer.

lope, and using the safety factor of 25% common in hydraulic calculations of dimension pipe diameters, it can be shown that the amounts of fines in a 100 mm thick envelope is only critical for pipe diameters of 80 mm and less, assuming that all fines in the envelope would wash into the pipe.

Guidelines for granular envelopes, such as prescribing that the material should not contain deleterious materials, and that particles should be of certain shapes, should continue to appear in specifications. Prescribing a coefficient of uniformity or a coefficient of curvature does not seems to be necessary, however, certain limitations on the coefficient of uniformity in the design of the bandwidth for granular envelopes, such as $2 \leq C_u \leq 6$ suggested by SCS 1994, is recommended.

Naturally-graded pit run gravel is the most common and widely used drain envelope material, the best being naturally-graded coarse sand or fine gravel. Natural grading is important because it minimises the separation of particle sizes that sometimes occurs by handling during installation, when fractions of various sizes of granular materials are mixed to construct a gravel envelope. Separation can be kept to a minimum by carefully proportioning the separate sizes of granular material used to create a gravel envelope. Guidelines for this can be found in USBR 1993 (Table 34, part F) and the SCS 1994 criteria (Table 34, part K).

On several occasions reference has been made to performing a Particle Size Distribution (PSD) analysis of the finer part of the base soils only (regrading). In Willardson 1974 the guidelines of Karpoff (1955) are enhanced for drains, specifying that only material smaller than 4.76 mm (no. 4 sieve, Table 15) should be sieved and the design is based on this. Sherard *et al.* (1984b) restricts determination of the PSD to less than 4.76 mm also, to properly assess the theoretical and laboratory determined hydraulic conductivity. These recommendations were partially taken over by SCS 1994.

Most of the guidelines described in this Section found their origin in the application of filters for hydraulic structures. They should not be used without further consideration of the selection criteria for agricultural drain envelopes, which has been reflected upon in the recommended design criteria in Chapter 3.

6.2.3 Organic material

By nature, organic material is vulnerable to deterioration. Its restricted lifetime depends on micro-biological activity, which in turn is a function of temperature, soil alkalinity and the presence of oxygen. While decay can occur within a year, some organic material is more successful in surviving under certain circumstances: decay is slower in colder climates as well as under submerged conditions. Organic material can no longer be recommended for use in regions with high temperatures (Singh *et al.* 1992). Since drainage is of utmost importance in irrigated areas to prevent salinisation and to reclaim salt-affected soils, the use of organic envelopes is unjustified when climatic conditions favour decay, and certainly when other alternatives are available locally.

Very few criteria for the design of organic envelopes have been presented. The closest to some form of criteria is the selection criteria used in the

Netherlands (Table 31) and the guidelines shown in Table 5 and Table 6 prescribing minimum envelope thickness and minimum as well as maximum mass of certain pre-wrapped loose materials (PLM).

6.2.4 Synthetic material

The use of synthetic envelope materials instead of gravel envelope materials became more viable after the introduction of lightweight corrugated and perforated plastic tubing for use as drainpipes. If a large effective drain diameter is not needed for a particular soil and if the bedding requirements of a plastic pipe could be met by pre-shaping the trench bottom into a semi-circle or a ninety degree supporting groove, then a synthetic fabric wrapped around a pipe can serve as a satisfactory envelope. In addition, the relatively high costs of gravel envelopes (Section 4.1.1) also spurred the switch from granular materials to geotextiles. With a gravel envelope, the filtering function or mechanical support function is all at the interface between the gravel and the soil, so what is needed is a synthetic material that is strong and durable and presents the same opening sizes to the soil at the soil-geotextile interface. The physical characteristics and opening sizes of various geotextiles commercially available are very different, so selection and acceptance of a particular geotextile material is difficult. Geotextiles used as drain envelopes must be carefully designed to withstand the prevalent soil and hydraulic conditions so that failure, clogging or excessive sediment passing, does not occur.

It is not a problem to find geotextiles that prevent drainpipe siltation because very fine geotextiles are readily available. Laboratory research as well as practical experience revealed that thin, fine geotextiles are vulnerable to blocking and clogging. Their function as a water-carrying layer, reducing the flow resistance in the direct surrounding of the drain, is small compared to voluminous envelopes. Moreover, geotextiles that are too fine could have some wetting problems (Lennoz-Gratin, 1987, 1992), although Dierickx (1994, 1996b) reports additional head losses that seem to be of small practical value (between 5 and 30 mm in sand tank and special testing apparatus). Nevertheless, this phenomena needs to be borne in mind when pipes are constructed below the water table and rapid filling of water in the drainpipe is essential to prevent floatation problems.

The ability of a geotextile to retain soil particles is usually expressed as the ratio of a characteristic pore size of the geotextile to a characteristic particle size of the soil. This ratio is called the retention criterion, also known as bridging factor or filter criterion. It is a measure for determining the mechanical support which envelopes provide to the surrounding soil. The other major indicators to be used in the design of synthetic envelopes are the hydraulic and mechanical criteria. The latter has been extensively discussed in Section 5.6 (not to be confused with the mechanical function of the envelope, Section 2.1).

In the introduction of Chapter 5 a multitude of factors to be considered in the design of synthetic envelopes were mentioned. The conclusion arrived at was that until more rigorous theories tested in practice become available, it would be best for the time being to use a simplified approach to describe the functioning of envelopes, similar to that used for the design of granular envelopes, namely concentration on the retention and hydraulic criteria.

An overview of various retention (filter) and hydraulic (permeability) criteria is given in Table 35. Like Table 34 for gravel envelopes, this table clearly illustrates the large variability in retention criteria for geotextiles, which led Koerner (1994) to describe the situation as "an unsettled state of the art". Table 35 is the latest version of a series of tables that seems to have been started by Fischer *et al.* (1990), and found their way into articles by Dierickx (1992b and 1993a), and Mlynarek (1993, in CUR/NGO 1995b), and appears in reduced form in Cavelaars *et al.* (1994), Koerner (1994), CUR/NGO (1995b), and Stuyt and Willardson (1999). Some differences (of interpretation) and errors became apparent in the various versions of the tables found in the literature. Where possible, these have been resolved by checking the original documents, and footnotes have been added to explain the inconsistencies.

It appears that Calhoun (1972), Ogink (1975), and Zitscher (1975) published the earliest work on design criteria for synthetic filtering envelopes. The US Army Corps of Engineers (US Corps 1977) picked up some of the guidelines and developed tests such as the Gradient Ratio test (Section 5.7.2). Some new criteria seem to emerge from England where Sweetland (1977) and ICI Fibres (1978) first introduce O_{15}/d_{15} criteria. Some sources refer to the latter two as Rankilor (1981). Schober and Teindl (1979) then presented their work, which was used by Giroud (1982, 1984).

Giroud (1982) derives his criteria on a theoretical basis with reference to Terzaghi's original criteria for retention and permeability characteristics of granular filters (Table 34, part A). For retention criteria, he describes optimum interlocking of particles at $C_u = 3$ with the highest density achieved. Between $C_u = 1$ and $C_u = 3$ the maximum particle size (d'_{100}) determines the interlocking mechanism, and for values of $C_u > 3$ the largest particle is not sufficient to retain the bulk of the soil and that some characteristic particle size smaller than d'100 will be critical in prevention of the migration of fine particles (d'_x). Rather then determining C_u he recommends use of the linear coefficient of uniformity (C'_u), which, taken from the central portion of the curve, excludes the coarsest and finest particle sizes that have a negligible effect on the stability of the soil structure in the filtration process. Use of the

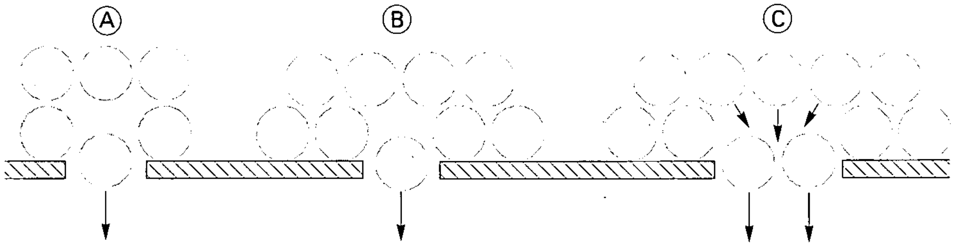


Figure 84 Influence of soil density on the retention criterion. (Giroud 1982, 1996).
A loose soil represented by a cubic arrangement.
B dense soil represented by a tetrahedral arrangement, with only one particle passing.
C dense soil with all particles passing.

linear gradation curves allowed the theoretical approximations used by Giroud (details of the linear PSD, as well as the density index are given in Section 5.5.1). He further postulates that two soil particles need to move in a densely packed soil to break the bridging, while only one particle is sufficient in a loosely packed soil (Figure 84).

For the hydraulic criterion (permeability ratio) Giroud uses Darcy's formula to determine the discharge through a soil-granular envelope (filter) and a soil-geotextile envelope (filter) and observes that to minimise flow and pore pressure disturbance caused by the envelope, the ratio of soil permittivity (K_s/T_s) to the envelope (filter) permittivity (K_e/T_f) must be small compared to 1. In geotechnical engineering a disturbance is usually considered negligible when its effect is less than 10%, while a safety factor of 10 is commonly used when soil permeability is involved. Hence (Giroud 1982):

$$\frac{T_f\,K_s}{T_s\,K_e} < \frac{10\%}{10} = 0.01 \qquad \text{Eq. 99}$$

where,

T_f, T_s is the thickness of envelope (filter) and soil respectively; and
K_e, K_s the hydraulic conductivity of the envelope (filter) and soil, respectively; the envelope (filter) can be either granular or a geotextile (T_g).

Using T_f granular = 1 m, $T_f = T_g$ = 10 mm and T_s = 10 m (Figure 85) the following hydraulic criteria result:

$$K_e > 10\ K_s \text{ for a granular filter} \qquad \text{Eq. 100}$$

$$K_e > 0.1\ K_s \text{ for a geotextile filter} \qquad \text{Eq. 101}$$

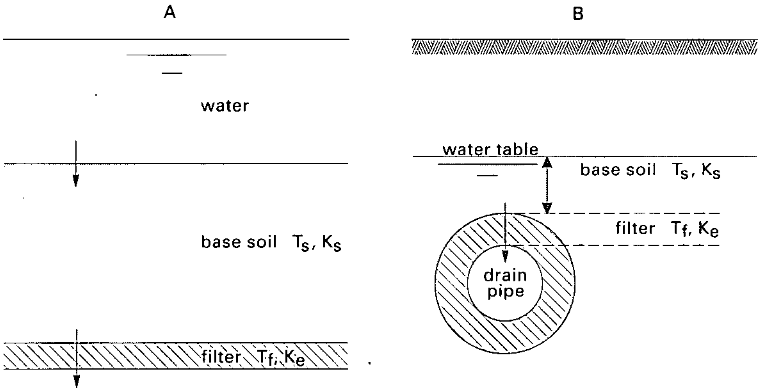


Figure 85 Typical filter situations.
A civil engineering situation.
B agricultural situations.

According to the above calculations Giroud states that the requirement for granular filters matches those generally used, but for geotextile filters they are clearly conservative. Criteria requiring that the conductivity normal to the plane of the geotextile be greater than that of the soil, actually demand more from geotextiles than the classical criterion of granular filters (Terzaghi 1922, see Table 34). Giroud retracts his conclusions in his 1988 and 1996 publications concerning the permeability required for geotextiles and recommends Eq. 103.

To compare his theoretically derived criteria (1982, retention and hydraulic) with tests performed with geotextiles, Giroud used results presented by Schober and Teindl (1979) and observes that his criteria for dense soil match those for needle-punched geotextiles and for loose soils those of other geotextiles such as wovens and heat-bonded non-wovens. A possible explanation is that contact between the soil and a geotextile with a rough surface (needle punched geotextiles) is better than with a geotextile with a smooth surface. Comparing his criteria (various O_{95}/d_{50} in Table 35 part I) with the often used retention criterion of $O_{95}/d_{85} < 1$ it became clear that his criterion will lead to excessively fine geotextile selection for $C'_u < 3$ and for $C'_u > 3$ and to O_{95} values that are too large. Giroud's retention criteria are too conservative for cohesive soils (similar with granular filters) and should not be used with gap-graded soils or when dynamic flow conditions that destroy bridging are likely to occur. Apart from these limitations presented by Giroud, a drawback of his methodology is also the need for determination of the density index. However, as adequate control of backfill with agricultural drainage is not possible in most cases, use of his criteria for loose soils would seem viable (without having to determine the density index).

In 1988 Giroud (in Williams and Luettich 1990) adjusted the hydraulic criterion to:

$$K_e > i_s K_s \qquad \text{Eq. 102}$$

and, in 1996 to (Giroud 1996):

$$K_e > 10\, i_s K_s \qquad \text{Eq. 103}$$

This resulted from the realisation that the situation generalised above using Eq. 102 was in fact a unique situation. For instance, for typical agricultural conditions a granular envelope would be 75 – 100 mm thick, a synthetic envelope 1 mm and the soil depth would be between 1 and 3 m (Figure 85). In such cases, the ratios of equations 100 and 101 range between 3 and 7.5 for granular envelopes and between 0.03 and 0.1 for synthetic filters. It may be further argued that the safety factor and the percentage disturbance which are considered negligible are not the same for agricultural applications. Giroud presented by his revised equation (Eq. 103) a series of likely gradients (Box 25).

Fisher *et al.* (1990) followed more or less a similar approach as Giroud to convert well-established granular material filter criteria to comparable ones for geotextiles. With the emergence of methodologies, which can reliably reproduce the pore size distribution of geotextiles (Section 5.6.9), they felt that values other than AOS, EOS, O_{95} and O_{90} could be used, such as O_{50} and O_{15}, for the retention and hydraulic filter design criteria. To achieve this they first established relationships between the particle size and the pore size (Section 5.1.3, Eq. 26 - 28). These were then used to convert D/d ratios to O/d ratios for different ranges of C_u. They established $C_u = 4$ as the boundary between different behaviours of the soil-filter combination. On comparing their criteria with those of Schober and Teindl (1979) and of Giroud (1982), Fischer *et al.*

Box 25 Selected exit gradients in civil engineering. Giroud (1996).

Application	*Typical hydraulic gradient*
Standard dewatering trench	*1.0*
Vertical wall drain	*1.5*
Pavement edge drain	*1*
Landfill leachate collection / detection removal system	*1.5*
Landfill leachate collection removal system	*1.5*
Landfill closure surface water collection removal system	*1.5*
Dam toe drains	*2*
Dam clay cores	*3 to > 10*
Inland channel protection	*1*
Shoreline protection	*10*
Liquid impoundment with clay liners	*>10*

(1990) found that their own criteria resulted in design AOS values in between the two (Table 35, parts G and I). However, the criteria of the latter cannot be implemented unless the O_{50} and the O_{15} of the geotextile are known.

The US Federal Highway Administration (FHwA) actively researched the application of geo-synthetics for use with highway drainage purposes (see also Section 6.3.3) and proposed several criteria over time (Christopher and Holtz 1985, 1992, parts L and Q in Table 35). Although not all criteria may be directly applicable they do present a major set of criteria used nationwide in the USA. From France similar sets of criteria are reported (Table 35, parts M and N). Dominant among these criteria is the use of O_{95}/d_{85} ratios for the retention criterion, with values varying between 1 and 3. Elsewhere in Europe the O_{90}/d_{90} ratio is more favoured as retention criterion (Table 35, parts B, K, S, T and N), and even as hydraulic or permeability criterion (Part S, Table 35) rather then traditional K_e/K_s ratios.

It is difficult to set a lower limit for detrimental soil invasion into geotextiles, but envelopes for which $O_{90}/d_{90} < 1$ are considered inappropriate (Faure, 1991) and are likely to clog. Experiments for the benefit of the Antwerp harbour (Belgium) showed that geotextiles with $O_{90}/d_{90} < 1$ rapidly clogged when placed in turbid water, whereas no obvious clogging was observed with materials with O_{90}/d_{90} of approximately 1.25 (Dierickx, 1990). The ratio $O_{90}/d_{90} = 1$ is a minimum beyond which no noticeable passage of particles occurs, i.e., the envelope material becomes a geometrically closed filter. Geotextiles with O_{90}/d_{90} ratios near the higher (i.e. safer) end of the recommended range of values are generally preferred, thus minimising the risk of mineral clogging of the envelope while providing adequate mechanical support for the soil.

Stuyt (1992) investigated the functioning of various geotextiles installed in experimental fields in the Netherlands and found that the O_{90}/d_{90} ratios as proposed by Dierickx (1987), Dierickx and Van der Sluys (1990), and Dierickx *et al.* (1992), which were also summarised in Dierickx (1992a, 1993a), were valid for the three problem soils (Figure 77) he investigated. Most of the applied envelopes had comparatively high O_{90}/d_{90} ratios (4 to 5). For coarser textured soils ($d_{90} \geq 200$ µm), the O_{90}/d_{90} ratio may be less than 1 on condition that the O_{90} of the geotextile is at least 200 µm and that the permeability is not affected. Geotextiles which are made too fine ($O_{90} < 100$ µm) are likely to clog and subsequently the natural filter that may have formed in the adjacent soil may clog as well (Stuyt 1992).

For use in Pakistan, the following criteria, based on the work by Dierickx (1992a, 1993a, Table 35, part S) and on extensive literature review were finally recommended for use with synthetic and organic drain envelopes (Vlotman *et al.* 1994a and b):

1. $1 \leq O_{90}/d_{90} \leq 2.5$ for envelopes thickness ≤ 1 mm (hydraulic and retention criterion);.
2. $1 \leq O_{90}/d_{90} \leq 5$ for envelopes thickness ≥ 5 mm (hydraulic and retention criterion).
3. $O_{90} \geq 200$ µm (200 µm = 0.2 mm, hydraulic criterion; anti clogging).

For envelopes with a thickness between 1 and 5 mm, it is recommended that the ratio be linearly interpolated. Geotextiles with O_{90} between 100 and 200 µm may be considered for use but under certain soil conditions clogging may eventually occur. To be safe the 200 µm boundary is recommended. A reason for allowing slightly finer filters could be the ready availability of the material, as long as laboratory testing of the material does not show a clogging trend. Rehman (1995) tested a good number of non-woven fabrics with $O_{90}/d_{90} < 0.5$ in the laboratory and did not find evidence of clogging, while hydraulic conductivity remained acceptable.

Comparison of O_{90}/d_{90} with other ratios proposed by Ogink (1975), Heerten (1983) and Santvoort (1994), which seem to be mostly retention criteria, with those from John and Watson (1994) and Dierickx (1993a) is not particularly helpful as both prescribe ratios > 1 and < 1 for seemingly similar conditions. In general, it can be observed that criteria prescribing $O_{90}/d_{90} > 1$ are hydraulic or permeability criteria, while those prescribing $O_{90}/d_{90} < 1$ are retention criteria. As with granular envelopes the designer will have to weigh which of the criteria may be relaxed under certain conditions.

None of the criteria described thus far referred to criteria to prevent long-term clogging, except those by Dierickx (1993a). Few exist and most guidelines prescribe laboratory tests with permeameters (Section 5.7.1 and 2).

Table 35 Existing filter criteria for geotextiles[40].

Geotextile	Soil	Flow type	Criteria	Remarks
A. Calboun (1972)[41] in Haliburton *et al.* (1982)				
woven	Non-cohesive ($d_{50} \geq 74$ µm)	dynamic and steady	$O_{95}/d_{85} \leq 1$	dry sieving, glass bead fractions reported originally EOS = O_{95} EOS = equivalent opening size Origin: USA
			4% ≤ POA ≤ 40%	POA = percent open areas per unit area POA reported only in CUR/NGO 1995b.
woven	Slightly cohesive ($d_{50} < 74$ µm)	dynamic and steady	$O_{95} \leq 200$ µm 4% ≤ POA ≤ 10%	in: Fisher *et al.* 1990 in: CUR/NGO 1995b
B. Ogink (1975)				
woven	sand	steady	$O_{90}/d_{90} \leq 1$	dry sieving, sand fractions
non-woven	sand	steady	$O_{90}/d_{90} \leq 1.8$	Origin: The Netherlands
woven	sand	dynamic	$O_{98}/d_{15} \leq 1$	without built up of natural filter and sand passage < 15%
non-woven	sand	dynamic	$O_{98}/d_{85} \leq 1$	provided built up of natural filter possible
C. Zitscher(1975) in Rankilor (1981)				
woven	sand, $C_u \leq 2$ and	steady	$O_{50}/d_{50} \leq 1.7 - 2.7$	
	100 µm ≤ d_{50} ≤ 300 µm	dynamic	$O_{50}/d_{50} \leq 0.5 - 1$	
	cohesive soils.	not specified	$O_{50}/d_{50} = 25 - 37$	
D. US Corps (1977) in Haliburton *et al.* 1982.				
woven and non-woven	$d_{50} \geq 74$ µm (sieve No 200)	not specified	$O_{95}/d_{85} \leq 1$ $O_{95} \geq 149$ µm	Based on Calhoun's work of 1972, piping criteria. Origin: USA
	$d_{50} \leq 74$ µm ≤ d85		149 µm ≤ O_{95} ≤ 211 µm (US standard sieve nos. 100 and 70)	
	$d_{85} \leq 74$ µm		do not use fabric	
E. Sweetland (1977) in Dierickx 1992, Fisher *et al.* 1990.				
non-woven	$C_u = 1.5$	not specified	$O_{15}/d_{85} \leq 1$	Origin: UK
	$C_u = 4.0$		$O_{15}/d_{15} \leq 1$	
F. ICI Fibers (1978) in Rankilor (1981)				
non-woven	20 µm ≤ d_{85} ≤ 250 µm[42]		$O_{50}/d_{85} \leq 1$	
	$d_{85} > 250$ µm		$O_{15}/d_{15} \geq 1$	

[40] Primary sources for this table were: Haliburton *et al.* 1982, Fisher *et al.* 1992, Dierickx 1992, Mlynarek 1993 in CUR/NGO 1995b, Wilson-Fahmy *et al.* 1996, and original references.

[41] The original Calhoun document was not available, and various sources used here did not report the same. In 1977 the US Army COE published guidelines (US Army Corps 1997 in Haliburton et al. 1982), which enhanced Calhoun's work to include non-wovens. "The criteria were originally developed for use in coastal engineering applications, but have also been used in design of filration/drainage systems since 1972". The CUR/NGO (1995b) publication first mentions the POA ranges.

[42] Refered to as Rankilor 1981 in Wilson-Fahmy *et al.* 1996 as 0.2 mm ≤ d_{85} ≤ 0.25 mm.

Geotextile	Soil	Flow type	Criteria	Remarks
G. Schober and Teindl (1979)				
woven and thin non-woven $T_g \leq 1$ mm	sand	steady	$O_{90}/d_{50} \leq B_1 (C_u)$	dry sieving, sand fractions $B_1(C_u) < B_2(C_u)$ and are factors depending on the coefficient of uniformity C_u
thick non-woven	sand	steady		$B_1(C_u) = 2.5 - 4.5$;
$T_g \geq 1$ mm (2 mm in CUR/NGO 1995b)		steady	$O_{90}/d_{50} \leq B_2(C_u)$	$B_2(C_u) = 4.5 - 7.5$ Origin: UK
H. Millar *et al.* (1980)				
woven and non-woven		steady	$O_{50}/d_{85} \leq 1$ $O_{50}/d_{15} \geq 1$	Origin: New Zealand
I. Giroud (1982, + 1984 reprint)				
	all soils cohesionless	steady[43]		<u>all</u> criteria theoretically; $K_f \geq 0.1\ K_s$
wovens[44] and heat-bonded, non-wovens	loose ($I_D < 35\%$)			ID is density index (see section 5.5.1)
	$1 < C'_u < 3$		$O_{95}/d_{50} < C'_u$	
	$C'_u > 3$		$O_{95}/d_{50} < 9/C'_u$	C'_u is linear Coefficient of Uniformity as defined in section 5.5.1.
	medium dense ($35\% < I_D < 65\%$)			Values for medium dense soil were arbitrarily determined between the values for loose and dense soils.
	$1 < C'_u < 3$		$O_{95}/d_{50} < 1.5\ C'_u$	
	$C'_u > 3$		$O_{95}/d_{50} < 13.5\ C'_u$	
needle-punched non-woven	dense ($I_D > 65\%$)			Wilson-Fahmy et al 1996 remark that Giroud assumes soil migrates for large C_u.
	$1 < C'_u < 3$		$O_{95}/d_{50} < 2\ C'u$	
	$C'_u > 3$		$O_{95}/d_{50} < 18/C'_u$[45]	
J. Carroll (1983) in Fisher *et al.*. (1990)				
woven and non-woven			$O_{95}/d_{85} \leq 2\text{-}3$	$K_f \geq (1 - 10)\ K_s$ Koerner 1994 favours this criteria.
K. Heerten (1983)				
woven and nonwoven	cohesionless ($d_{50} \geq 60\ \mu m$)	dynamic	$O_{90}/d_{50} < 10$	wet sieving, graded soil Origin: Germany
	$C_u > 5$	steady	$O_{90}/d_{50} < 10$ $O_{90}/d_{90} < 1$	Franzius Institute, Hannover Also given in Santvoort (1994)
	$C_u < 5$	steady	$O_{90}/d_{50} < 2.5$ $O_{90}/d_{90} < 1$	
	cohesive ($d_{50} \leq 60\ \mu m$)	steady and dynamic	$O_{90}/d_{50} < 10$ $O_{90}/d_{90} < 1$ $O_{90} \leq 100\ \mu m$	

43 Although the derivation by Giroud is theoretical, he mentions that they cannot be used when the soil structure is repeatedly destroyed by turbulent flow (dynamic, i.e. surging and wave action with bank protection).

44 Giroud did not test geotextiles but compared his results with those of Schober and Teindl (1979).

45 Dierickx 1992b and 1993a, Cavelaars et al. 1994, and Stuyt and Willardson (1999) had a value of 13.5 for this ratio; the correct value is 18. In addition they mixed needle-punched and wovens, which compared results of dense and loose soil respectively; not the other way around!

Table 35 cont. Existing filter criteria for geotextiles.

Geotextile	Soil	Flow type	Criteria	Remarks
L. Christopher and Holtz (1985) in CUR/NGO (1995b), and Fisher *et al.* (1990)				
not specified	$d_{50} > 0.074$ mm			US standard sieve No 200
	$C_u < 2$ and $C_u > 8$	steady	$O_{95}/d_{85} \le 1$	(Table 15). Origin: USA
	$2 \le C_u \le 4$	steady	$O_{95}/d_{85}5 \le 0.5$	based on experience with
	$4 < C_u \le 8$	steady	$O_{95}/d_{85} \le 8/C_u$	
		cyclic	$O_{95} \le d_{15}$ and $O_{50}/d_{85} < 0.5$	
not specified	$d_{50} < 0.074$ mm			US standard sieve No 200
woven		steady	O_{95}/d_{85}	
thin woven		steady	$O_{95}/d_{85} \le 1.8$ and $O_{95} < 0.3$ mm	
not specified		cyclic	$O_{95}/d_{85} \le 0.5$	
			$O_{95}/d_{15} \ge 3$	$K_f \ge (1 - 10)\ K_s$ (in Fisher *et al.* 1990)
M. CFGG (1985) and Rollin *et al.* (1987), and in Wilson-Fahmy *et al.* (1996)				
woven and non-woven		all steady	$O_{95}/d_{85} \le C$	hydrodynamic sieving, graded
			$C = C_1C_2C_3C_4$	soil. Origin: France
	$C_u > 4$ well graded		$C_1 = 1$	O_{95} = FOS
	$C_u < 4$ uniform		$C_1 = 0.8$	also given in Santvoort 1994
	unconfined loose, less dense		$C_2 = 0.8$	
	confined and dense		$C_2 = 1.25$	
	$i < 5$		$C_3 = 1$	
	$5 < i < 20$		$C_3 = 0.8$	
	$20 < i < 40$		$C_3 = 0.6$	
	filter function only		$C_4 = 1$	
	filter and drainage function	$C_4 = 0.3$		
	cohesive		$O_{95} \ge 50$ µm	
permittivity			$\Psi > 10^5\ K_s$	critical (Wilson-Fahmy *et al.* 1996)
			$\Psi > 10^4\ K_s$	less
			$\Psi > 10^3\ K_s$	clean sand
N. Fayaux *et al.* (1984) and Rollin *et al.* (1987)				
			$1.5 < O_{95}/d_{85} < 3$	Origin: France
	uniform silty soil		$O_{95}/d_{85} < 1.5$	based on other sources from France
O. AASHTO (1986), Task Force # 25 in Koerner (1994), and Wilson-Fahmy *et al.* (1996)				
no limitations	$d_{50} < 74$ µm	–	$O_{95} < 595$ µm	Origin: USA
	$d_{50} > 74$ µm	–	$O_{95} < 300$ µm	possibly $K_e \ge 10\ K_s$
P. Fisher *et al.* (1990)				
	function of C_u (Fig 1 in Fisher *et al.* 1990)		$O_{50}/d_{85} \le 0.8$	Tested on composite soil prepared in laboratory
			$O_{50}/d_{15} \le 1.8 - 7$	
	function of C_u (Fig 2 in Fisher *et al.* 1990)		$O_{50}/d_{50} \le 0.2$[46] $- 2$	Guidelines based/derived from traditional graded granular filter criteria such as those in Table 34, part E. Origin: USA

[46] Value given as 0.8 by Wilson-Fahmy et al. (1996).

Geotextile	Soil	Flow type	Criteria	Remarks
Q. Christopher and Holtz (1992) in Wilson-Fahmy *et al.* (1996)				
not specified	d_{50} < 74 µm (sieve No 200)	steady	$O_{95}/d_{85} \leq B$	Federal Highway Admin.
	$2 \leq C_u \geq 8$[47]		$B = 1$	(FHwA). Origin: USA
	$2 < C_u \leq 4$		$B = 0.5$[48]	
	$4 < C_u < 8$		$B = 8/C_u$	
woven	$d_{50} \geq 74$ µm (sieve No 200)	steady	$O_{95}/d_{85} \leq 1$	
non-woven	$d_{50} \geq 74$ µm (sieve No 200)	steady	$O_{95}/d_{85} \leq 1.8$	
both	$d_{50} \geq 74$ µm (sieve No 200)	steady	$O_{95} \leq 300$ µm	(US standard sieve no. 50)
all materials	all soils	dynamic	$O_{95}/d_{85} \leq 0.5$	incl. pulsating and cyclic[49]
R. John and Watson (1994)				
all materials	sand and all soils[50]	–	–	based on model results reten-
	$1 < C_u < 8.35$	–	$2 < O_{90}/d_{90} < 4$	tion criterion (see Table 10)
	$1 < C_u < 8.35$	–	$O_{90}/d_{90} > 0.5 - 1$	permeability criterion
S. Dierickx (1992a, 1993a, 1994, 1996)				
				Origin: Belgium, Egypt, Pakistan
all materials	–	steady	$O_{90}/d_{90} > 1$	permeability criterion
thin geotextiles ($T_g \leq 1$ mm)		–	$O_{90}/d_{90} < 2.5$	retention criterion
voluminous ($T_g \geq 5$ mm)		–	$O_{90}/d_{90} < 5$	retention criterion
			$O_{90} > 200$ µm	hydraulic criterion and anti clogging
T. Santvoort (1994)				
non-woven		steady	$O_{90}/d_{90} \leq 2$	Origin: Holland, Delta Works
woven		steady	$O_{90}/d_{90} \leq 1$	and generally no soil movement allowed.
natural filter will form under cyclic conditions			$O_{98}/d_{85} \leq 2$	Delft Hydraulics for extreme
natural filter cannot form under cyclic conditions			$O_{98}/d_{15} \leq 1.5$	load conditions.
U. DESIGN CRITERIA AS PRESENTED IN THIS BOOK				
thin geotextiles ($T_g \leq 1$ mm)		–	$O_{90}/d_{90} < 2.5$	retention criterion
voluminous ($T_g \geq 5$ mm)		–	$O_{90}/d_{90} < 5$	retention criterion
$1 \leq T_g \leq 5$ mm		–	interpolate between	O_{90}/d_{90} = 2.5 and 5
		–	$O_{90} > 200$ µm	hydraulic and anti-clogging criterion
		dynamic and steady	$K_e \geq a\ K_s$	hydraulic criterion a = 1 for non critical conditions[51], and a = 10 for reverse flow conditions
		–	$O_{90}/d_{90} > 1$	anti clogging criterion
		–	$O_{90} > 100 - 200$ µm	anti clogging criterion
			for mechanical strength and other criteria see Box 13.	

[47] There is some confusion in the literature whether it should be $1 < C_u < 2$ (Williams and Luettich 1990) or $2 \geq C_u \geq 8$ (Wilson-Fahmy et al. 1996).

[48] Williams and Luettich (1990) repport $B = 0.5\ C_u$.

[49] Christopher and Holtz (1989) presented slightly different criteria for dunamic soils which must have been superseded by the 1992 reference: if soil can move beneath geotextile $O_{95}/d_{15} \leq$ or $O_{50}/d_{85} \leq 0.5$.

[50] See Table 10 in section 5.2 for details, soils may be considered the same as Giroud (1982) and Schober and Teindl (1979).

[51] A noncritical condition is where flow is steady and in one direction only (no reverse flow).

6.2.5 Conclusions on synthetic drain envelope design

There are four sets of criteria for the design of synthetic envelopes: 1) retention criteria; 2) hydraulic criteria; 3) anticlogging criteria; and 4) criteria related to the mechanical strength of the envelope material. The last-mentioned have already been discussed in Section 5.6 and are usually not mentioned along with the first three. Nevertheless, it is important that the strength criteria are also taken into account whilst designing and selecting synthetic envelopes.

Retention criteria

Essentially three types of retention criteria were encountered: 1) O_{90}/d_{90} ratios; 2) O_{95}/d_{85} ratios; and 3) a range of ratios using O_{90}, O_{50}, O_{15} in combination with d_{15}, d_{50} or d_{85}. To start with the third type of ratios, these have become more popular in recent years due to advances in determination of Opening Size Distribution (OSD) curves (Section 5.6.9c). The criteria have not been tested extensively (yet) and are not recommended for use without further research. The second type of ratio is common in the US and France but have one main disadvantage and that is the determination of the O_{95}. The EN ISO standard (EN ISO 12956, 1999) recommends O_{90} as the characterising property for filtration by synthetic materials, because the O_{95} can vary considerably as it was found to be generally located on the gentle sloping part of semi-logarithmic OSD curve (Figure 62B).

The type 1 ratios (O_{90}/d_{90}) seem to be the most practical criterion at present, even though some of the values are in conflict when applied as permeability criterion (for instance $O_{90}/d_{90} < 1$ and $O_{90}/d_{90} > 1$). Therefore it is advisable not to use this ratio for the hydraulic or permeability criterion, but the hydraulic criteria mentioned below, instead. One can wonder why d_{90} is used rather than d_{85}, which is more common with traditional granular filter designs. Both the d_{90} and d_{85} are generally located in the transition (steep) zone of the semi-logarithmic PSD curve as can be seen from the many PSD curves shown throughout the book, hence the need for a preference is not apparent. Should one wish to use d_{85} rather than d_{90} a conversion factor of 1.2 can be used, i.e. $d_{90}/d_{85} = 1.2$. Conversion between other soil particle sizes is described in Section 5.5.1 (Table 18), while conversion factors to convert COS are shown in Table 27.

Hydraulic criteria

Three hydraulic criteria are frequently mentioned in the literature: 1) $K_e > 0.1$ K_s, 2) $K_e > K_s$, and 3) $K_e > 10$ K_s. K_e is the clear water permeability of the material. The first criterion, $K_e > 0.1$ K_s, was later modified to $K_e > 10$ K_s i_s, while the criterion of $K_e/K_s > 0.1$ was rescinded. A fourth criterion presented by the French Committee on Geotextiles and Geomembranes (CFGG 1986,

Table 35, part M) relates permittivity (K_e/T_g) to certain values. Criteria become more stringent in ascending order, hence criterion 1 will be easy to meet (but is not correct), and criterion 4 the least easy. Wilson-Fahmy *et al.* (1996) demonstrated that although criterion 4 was not met in most cases the site drainage systems worked well (details in Section 6.3.3, Table 36). Criterion 1 has no safety built in, whereas the others do to varying degrees. For agricultural applications it would seem that criterion 3, which is similar to that used with granular envelopes, would be the most appropriate (see also Sections 5.6.10 - 12). Giroud (1996) presented a formula, $K_e > 10\ K_s\ i_s$ including criterion 3 as a function of the exit gradient. In addition to criteria relating hydraulic conductivities directly (using clear water permeability), there are a number of indirect hydraulic criteria such as $O_{90}/d_{90} > 0.5 - 1$, $O_{90} > 200$ mm, and the criteria from permeameters that use K_{es}. The latter include some soil with the filter material (see Sections 5.7.3 – 5). Those that use particle sizes should be used with the envelope design criteria with some caution. Those resulting from laboratory experiments with permeameters should be used qualitatively only (i.e. to compare laboratory results with each other but not to relate directly with field results or incorporate in design criteria).

Anti-clogging

Clogging of synthetic fabrics is a decrease of permeability in the long-term caused by particles of the base soil. This is different from blocking of synthetic envelopes, which is the immediate near total loss of permeability of the envelope by a layer of fine particles (commonly caused when smearing under wet construction conditions takes place). At present there are few established criteria and permeability tests are generally recommended (Wilson-Fahmy *et al.* 1996). Dierickx proposed $O_{90} > 200$ µm from a hydraulic and anti-clogging point of view, his theory being to prevent clogging of the fabric itself. Christopher and Holtz (1992) remark that when $C_u > 3$ then use $O_{95} > 3d_{15}$ and when $C_u < 3$ the maximum opening size allowed from the retention criteria should be used. Wilson-Fahmy *et al.* (1996) found clogging of the drain conveyance medium (pipe or inside of composites) and not much clogging of the fabric itself. Consequently, there is an obvious need for considerable further research on this particular aspect of synthetic envelope design.

Additional conclusions

Giroud (1982) mentioned that contact between soil and fabric was not satisfactory if the material has a smooth surface, which lead to mismatch between design criteria and values observed in the field laboratory.

The overriding impression on reviewing the literature on synthetic envelope design is that the design criteria for synthetic envelopes are constantly changing and that the reports encountered are somewhat sloppy in that there seem to be many differences between what are supposedly the same criteria, even within the same article! In Table 35 we have tried to resolve this mentioning where we could not reconcile the issue. In each of the articles reviewed authors seemed to model the presentation of criteria developed by others to suit their own purpose (and we are probably guilty of this too!). It is recommended that criteria only be reported as originally presented by the researchers, and then and only then (and clearly indicated) rewrite the equation or ratio to fit the particular need.

6.3 Field experiences

There are many articles and publications that report on actual field experiences, but unfortunately many do not present enough detail to compare the results with the criteria presented in various parts of this book. We therefore provided a description of essential data to be reported in Section 4.7.3, in the hope that future publications will provide the desired adequate detail. Alternatively, reports referenced in those articles, which are the basis of the results presented in the paper, can be obtained. Recent developments included in research reports and the like obtainable from databases via the World Wide Web are likely to enhance accessibility to these critical data.

6.3.1 Granular envelope

Case study California

In 1970, after some serious problems with clogging of drain envelopes were discovered in the Imperial and Coachella Valleys of California, an elaborate experiment was designed to test drain envelope materials. More than half of the drains installed in the Coachella Valley had water above the pipes, indicating that the envelope materials or the drain openings were clogged. The experiment was constructed and maintained by the Coachella Valley County Water District and was a cooperative effort with the USDA-ARS and the University of California at Davis. The soil had a fine sandy texture with little cohesion and a considerable mica flake content. Clay, concrete, and perforated plastic pipes were tested with various types of gravel envelopes. Replications included pipes without envelopes. Since the area was very arid, a solid set sprinkler system was used to apply water. Individual valves were installed in the drains so that drainage intensity (drain spacing) could be modelled by closing some of the drain outlets. The drains without envelopes failed quickly. The drains with coarse and fine single sized envelope materials allowed excessive amounts of the sand to enter the drains. Gravel envelope materials with excessive amounts of fines also failed. Envelope materials that had acceptable grading curves and did not have excessive amounts of fine materials performed successfully (Davis *et al.* 1971).

There were some problems with the envelopes in some locations, and soil contamination of the envelope material was suspected. So, a second experiment was installed in which some of the envelope material was handled according to normal practices, i.e., it was stockpiled on the ground in the field and was loaded into the drainage machine with tractor loaders as needed. The other treatment was to move the material directly from trucks into the machine so that it would not have any chance to become contaminated with surface soil from the field. Drains installed with envelope materials that had been contaminated with soil failed. Drains with clean envelope materials performed successfully. It was apparent that even small amounts of soil will seriously contaminate drain envelope materials because the fine soil particles clog the openings of the envelope material.

Case studies Pakistan

Construction of subsurface drainage systems in Pakistan has taken place at five locations since 1971 and several more are scheduled. Details of the granular envelopes at four locations (Fourth Drainage Project, Mardan, East Khairpur Tile Drainage, and Khushab) have been published. Either USBR, SCS or US Waterways Experiment Station specifications (Table 34, parts F, I and J, and B) were used for the envelope design. Although designed according to the specifications, serious problems occurred with the crushed rock envelope at the Fourth Drainage Project near Faisalabad. An in-depth review of existing criteria revealed that the selected specifications for this particular project were on the coarser side of possible gradation specifications.

For the East Khairpur Tile Drainage Project (EKTP), filter criteria as developed by the US Waterways Experimental Station (Dieleman and Trafford, 1976) and used as source by the UK Road Research Laboratory (RRL,

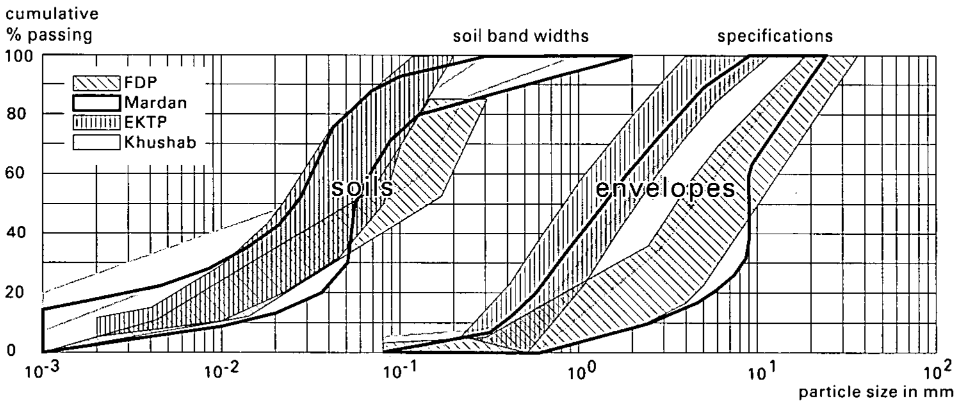


Figure 86 Pakistan base soils and granular envelope bands specified. (Vlotman *et al.* 1992)

Spalding 1970) were compared with the 1971 Soil Conservation Service (SCS and the 1978 United States Bureau of Reclamation (USBR) criteria. Composite curves resulted (Figure 86) which worked satisfactorily with the gravel materials available at the Shadi Shaheed quarry. At the Mardan SCARP and the Fourth Drainage Projects (FDP) straight forward USBR criteria were applied (USBR 1978). Khushab SCARP horizontal subsurface drainage design was more recent (1991) and for those SCS 1988 criteria were used.

Figure 86 shows the representative curves for lower and upper limits of the base soils and corresponding lower and upper envelope gradation curves for all the projects. The gravel envelope curves of the EKTP are finer than the USBR based curves. Willardson and Ahmed (1988) found that the 1971 SCS criteria result in finer envelopes than the USBR criteria and therefore are more suitable for use as filter criteria. Criteria that further describe the shape of the curves, coefficient of uniformity (C_u) and curvature (C_c), are also used at FDP and Khushab SCARP. From Khushab SCARP it was concluded that also the SCS 1988 criteria needed adjustment from practical point of view (see Section 6.2.1).

Problems with the crushed rock partly stemmed from construction problems, but when these were resolved crushed rock material still did not perform well. Analysis in the field with specially laid test lines using crushed rock (having angular shaped particles) and river run material (having rounded particles) showed that river run material with the same characteristics on paper, performed much better. Sieve analysis of samples excavated after installation showed invasion of fines in both the crushed rock and river run envelopes. The percentages passing the ASTM sieve no. 200 (0.074 mm) ranged from 10% to 17% in those samples (Figure 83).

While research was still going on, FDP introduced limitations on hydraulic conductivities of granular envelope materials (Vlotman *et al.* 1995): 15.3 m/d $< K_e <$ 153 m/d. Performing a 21-sieve analysis and following the appropriate design guidelines based on prescription for the envelope band should, however, satisfy the K_e criteria in most cases. Laboratory testing of materials straight from the quarries or other sources met with mixed success. Sand alone would not bridge over the perforations if the maximum perforation size was allowed, and crushed rock did not perform as well as some of the blended materials and the river run material. Coarse river run material also did not perform well. River run material from the quarries near Attock performed very well and would fit most gravel bandwidths in SCS 1988 criteria.

Another interesting finding was that no evidence has been found at the two projects where envelopes with high carbonate contents were used pointing to it being detrimental for the functioning of the granular envelope (Vlotman *et al.* 1995). At EKTP the carbonate content was more than 80%, while the envelope material used with the Nawabshah Interceptor drain contained more than 50% carbonates.

6.3.2 Organic envelope

Any voluminous porous organic material can theoretically be used as envelope material around a buried subsurface drain. Such material will improve the hydraulic performance of the drain and could temporarily protect the drain from sediment inflow. However, organic materials may deteriorate in the soil, due to biological degradation, and the drain could lose the protection provided by the envelope material. Many different organic materials have been successfully used as drain envelope materials, but their durability depends on the type of material and the bacteriological environment at the drain level.

Fibrous peat
Peat is one of the better organic drain envelopes with an acceptable durability experience. Suitable peat is required to contain a large amount of fibrous material that is not too dusty and does not contain large clods. Use of peat evolved from a simple loose cover over the drain to a netted pre-wrapped material. Adding coconut fibres enables pre-wrapping with synthetic threads. The intermediate form with peat being stitched onto a non-woven carrier strip has been less successful as a result of the high entrance resistance of the fine-structured carrier strip that formed the interface with the soil. Also, soil particle invasion cannot always be prevented when the material was placed under wet conditions, especially when the strip ended up beside the drain. Covering the top of a drain does not suffice to prevent soil particle invasion through the bottom perforations (Dierickx and Leyman, 1978). For efficiency, an additional underlay strip is needed. Peat was extensively used in the Netherlands, but problems with the supply of good quality peat and the fact that prewrapping was more expensive than other organic materials, resulted in its disuse. Among the organic drain envelope materials in common use, peat keeps the longest in soil.

Flax straw
Flax straw used for an envelope is the waste that remains after retting flax. After initially being stitched with synthetic threads to a strip to cover the drain, flax straw is then wrapped around the drains at the time of manufacture. The coarse fibre structure requires a thicker layer to offer an adequate protection against soil invasion, but even then flax straw was still found to be too coarse. A strip of the material used only as a cover does not prevent soil invasion from underneath and therefore requires an additional underlay

strip. The voluminous flax straw envelope has proved to be an excellent material with adequate durability. Although taking second place to peat, research on the durability of flax straw has never been done. The downturn of the flax industry and the appearance of the more favourably considered coconut fibres caused the disappearance of flax straw as drain envelope material in Europe.

Coconut fibre

Coconut fibre (coir) has been considered as a more favourable organic envelope material than flax straw because its structure is not so coarse. However, the O_{90} value for coconut fibre is more than 1.0 mm (1000 μm) which is too coarse for certain fine sandy soils and results in soil invasion in the drains. Another important disadvantage of coconut fibre is its limited lifetime, especially in alkaline soils and soils rich in organic matter (Meijer and Knops, 1977). Its mean lifetime is about 4 to 5 years although coconut fibres can decay within one year (Dierickx, 1985). Coir fibre is an organic fibre consisting of 46% lignin and 54% cellulose (Venkatappa Rao and Balan, 1994). Coir fibre is more resistant to rotting than jute, which has only 11.5% lignin.

Wood chips and saw dust

Wood chips and coarse saw dust originating from a chain saw or commercial sawmill are mainly used as drain envelope materials in the Scandinavian countries, initially as a loose cover and later wrapped around the drain inside a plastic netting material. The bulky application is somewhat labour-intensive and not so effective in preventing soil particle invasion, as the drain is not protected underneath. When used as wrapping it is an excellent material. The durability is not known, but due to the reduced micro-biological activity at lower temperatures in Scandinavia, its lifespan appears to be adequate.

Other organic materials

Obviously there are numerous other organic materials that can be used as a drain cover or as a wrapping envelope for drain tubes, depending on the inventiveness of farmers. These are materials that are mostly only locally available. They have therefore never been used on a large scale and consequently the experience with such materials is very regional and limited. Heather appeared to be an excellent material with high decay resistance. Also straw of cereals, chopped or not, has been used but cannot be recommended because of slime formation when decaying, and the restricted life. Other materials that have been mentioned are: chaff, corncobs, brushes, reeds, moss, conifer leaves and grass sod. A number of articles on the application of jute, coir and sisal as natural geotextiles are in Karunaratne *et al.* (1994, pp 849 - 902). They go beyond the scope of this publication but may be of interest when considered for use with agricultural drains. Typical topics covered in the articles include: jute application for general civil engineering treated with anti-microbial agent to extend their life in the soil (India); jute in combination with different ratios of polypropylene fibres for civil engineering (India); jute durability tests (India); jute for vertical drains (Bangladesh); jute geotextile in layered clay sand reclamation (Singapore); sisal fibres for erosion control (South Africa); and application of coir geotextiles for river bank protection and embankment stabilisation (India).

6.3.3 Synthetic envelopes

Glass fibre

Glass fibre was the first material to be been used as a strip to protect drainpipes against soil particle invasion (Johnston, *et al.*, 1963). It can be applied without any difficulty in sandy soil provided that it is not too ferrous or too organic and the amount of fine particles is limited. Glass fibre has been used in non-cohesive soil underneath the drain in combination with a peat, flax or coconut fibre strip placed on top. In this way the soil invasion from underneath was prevented while the voluminous cover strip reduced the flow pressure and the clogging risk. However, when used for clay drains sediment was often found in the drains. Furthermore, field experience demonstrated that the edge of the pipe drains often damaged the glass fibre sheet during installation, and it was concluded that glass fibre did not offer adequate protection to prevent soil particle invasion (Dierickx and Leyman, 1978).

Efficient protection was obtained when glass fibre was used in combination with plastic drains. Because of its fine structure, glass fibre was found to be vulnerable to blocking and clogging by fine soil particles, organic dust, and from iron ochre, and therefore cannot be recommended as drain envelope in many soils. These disadvantages and the fact that it irritates the skin of the labours during processing are the reasons why glass fibre is no longer used. If a fine envelope is considered to be useful, fine non-woven geotextiles are used. Only boro-silicate glass should be used as other glass fibres dissolve in the soil environment

Glass wool

Glass wool mats are 1 to 2 cm thick. For this voluminous material, clogging is less critical than for fibreglass, especially for glass wool with coarse fibres. Glass wool is ineffective when its structure is too fine. Moreover, it causes skin irritation like glass fibres. It is no longer used because of its limited use and because alternative synthetic products are available.

Rock wool

Like glass wool, rock wool is also a voluminous product. Its fine structure and the consequent problems limited its use to a very brief period only.

Polyamide string
Polyamide string wound in the valleys of the corrugations where the perforations are located can be considered as the prototype of synthetic envelopes. Although effective to prevent soil invasion, it has not been accepted because of its fine structure with the accompanying risk of blocking and clogging.

Loose polypropylene fibres: pre-wrapped loose material (PLM)
The wrapping of drainpipes with loose polypropylene fibres came into being as an alternative to organic coconut fibre wrapping, which have a limited lifetime. Polypropylene fibres virtually do not decay once installed in the soil, so they have more or less an unlimited life. The characteristic opening size of these envelopes can be adapted to that of the surrounding soil in which they lie by varying the diameter of the fibres. Envelopes of polypropylene can consist completely of finer fibres which are a waste product of the carpet industry, or new coarse fibres, or a blend of the two. The finer fibre wrappings have an O_{90} of about 200 to 350 μm, the coarse fibre wrappings have an O_{90} of 700 to 1000 μm, and the blended wrappings have an O_{90} of 350 to 700 μm. Although the finer fibre wrappings carry a potential risk of blocking and clogging, no justifiable complaints have been noted. Practical experience confirms the superiority of the coarser fibre wrappings ($O_{90} \geq 350$ μm) with a limited soil invasion under normal drainage circumstances. The materials are used extensively in the Netherlands. Guidelines for design, construction, quality control and inspection have been prepared (NEN 7090, KIWA staff 1992a and b). Looser fibres are considered voluminous and increase the effective drain diameter.

Geotextiles
Drains can be wrapped with woven and non-woven geotextiles, but as they can be made with thin and fine materials, the risk of clogging and blocking and consequently of a reduced drainage performance is high. Although thin, coarser materials can be considered, they do not have the advantage of voluminous materials and the risk of soil invasion limits the use of geotextiles as drain envelope materials. Some research and field experiments even suggest that it is likely that thin, finer geotextiles may hamper drainage performance, but there is no clear evidence of this as yet and on the whole they perform quite well. Below are a number of field studies with information relevant to the application of drains in agricultural settings.

<u>Case study 1.</u> One of the most recent studies reported, and probably a landmark for details on long- and short-term behaviour of synthetics in relation to drainage applications is the study that was executed by the Federal Highway Administration (Koerner 1994, Koerner *et al.* 1996, and Wilson-Fahmy *et al.* 1996). The study reports on the results of exhuming 91 sites in 17 states throughout the United States that were in use for between 1 and 15 years,

though age did not seem to affect the results. About 75% of the sites were in hot and cold humid areas, and 25% in dry hot climates. Typical drainage conditions investigated are shown in Figure 87 from which it is clear that few of them relate to agricultural applications of subsurface drains, however the behaviour of the geotextiles used will be indicative of what to expected under agricultural conditions. Each application consisted of three components: the filter, the drain and the system, and each was judged on its performance. The system as a whole functioned when all three components performed satisfactorily. Failure was related to construction and/or maintenance problems, to clogging of the drainage medium by sediment, and to failure of the geotextile to retain the soil (filter working).

With respect to the drainage medium, improper product use, excessive deformation of the core of geocomposites, and installation damage were reported in a few cases. A major finding was that 8 of the 10/41 geocomposite sites failed as a result of retention problems. Soil finer than the openings moved into the drainage core and completely clogged it. AOS of the fabric was 0.21 mm (no 70 sieve) while soil particles in the core were less than 0.15 mm. No further details are given, except that it was observed that backfill was improper, resulting in large voids from which suspended particles moved freely into the drain core through the fabric, probably under dynamic load. It was concluded that good contact between geotextile and base soil right from the start is important to it to be a success. Because this type of failure was deemed installation-related the 8 cases were not considered in the comparison of ground truth and design criteria (Table 36).

The types of problems encountered with the filters were excessive soil clogging; slag precipitation, inadequate strength (geotextile and seam), excessive UV degradation; and, the earlier mentioned inadequate soil retention (clogging of drainage core). Wilson-Fahmy *et al.* (1996) compare existing permeability, retention and excessive clogging criteria for geotextile filters with the results of the study (Table 36). They classified the soils as granular, mixed and fine, according to the Unified Soil Classification System. The significance of this division is not made entirely clear, except that it divides soils in certain classes. In agriculture all three types might be encountered. Regarding the permeability criteria it is noted that Giroud (1982) and FHwA criteria (Christopher and Holtz 1989) are preferable in correlating their predictive behaviour with the field sites. For retention criteria, Task Force 25, FHwA (steady flow, 1989), Ogink (1975), CFGG (1986) and Carroll (1983) all give good results. With regard to excessive clogging criteria it was observed that none were applicable, nor where they needed for granular soils. For mixed soils most were also not applicable, while for fine soils the FHwA criterion seemed most appropriate. Wilson-Fahmy *et al.* (1996) recommend for all three criteria the most recent FHwA guidelines (Christopher and Holtz 1992).

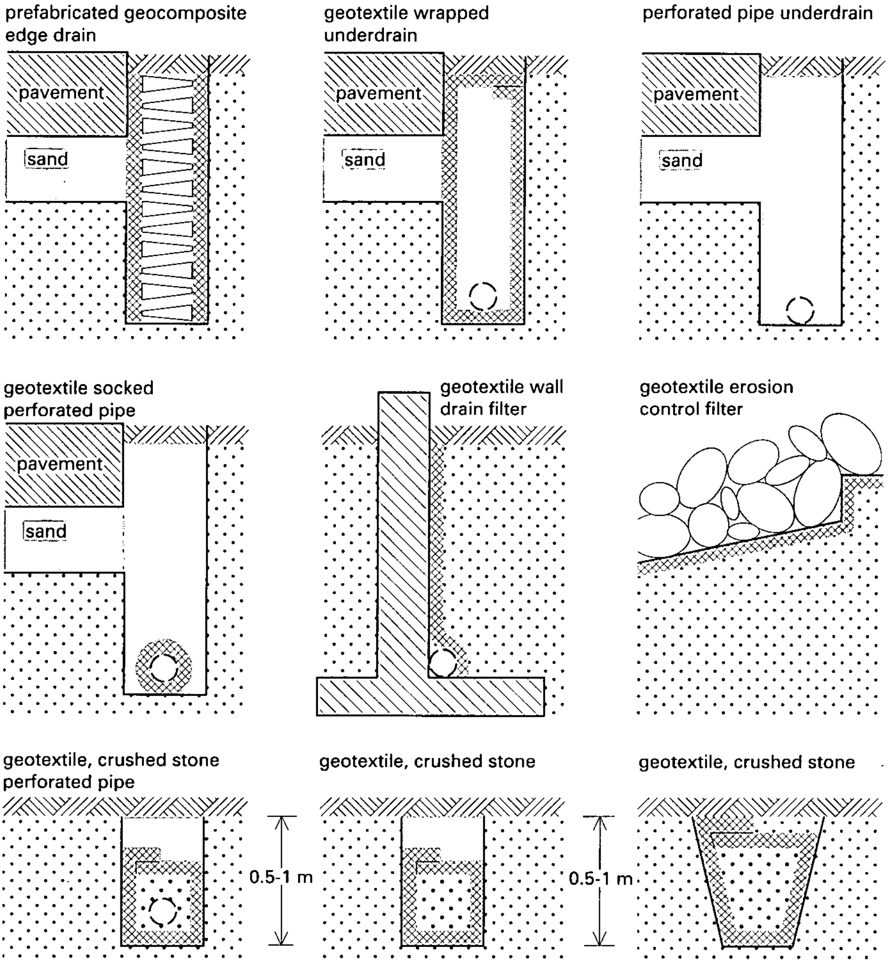


Figure 87 Examples of non-agricultural drainage systems. (After Koerner 1994, and Koerner *et al.* 1996).

Case study 2. In 1976 in the Central Valley of California, a field experiment was established using three different types of synthetic envelope materials, including a bulky fibre envelope made of carpet waste material from the Soviet Union. A gravel envelope wa s used as a control. All of the envelopes performed satisfactorily, but the gravel envelope produced much higher rates of drainage than any of the synthetic envelopes. The difference was attributed to the larger effective diameter of the gravel envelope which improved the hydraulic function of the drains (Johnston, 1978, 1981).

Miscellaneous case studies. Major sources of recent experiences with synthetic materials are the Fourth and Fifth International Conference on

Geotextiles, Geomembranes and Related Products (Hoedt 1990, Karunaratne *et al.* 1994), and the ASTM sponsored symposium in Denver Colorado (Bhatia and Suits 1996) on recent developments in geotextiles and prefabricated drainage geocomposites. Case study 1 came from the latter source. Numerous articles are available for perusal but few are directly related to the application of drain envelope for agricultural soils and conditions.

Table 36 Results of testing highway drains that use geotextiles. (after Wilson-Fahmy 1996).

Source	Part in Table 35	Granular Soil < 12% pass No 200		Mixed Soil 13-49% pass No 200		Fine Soil > 50% pass No 200	
		Agree	Disagree	Agree	Disagree	Agree	Disagree
a – Permeability criteria				n/a = not applicable			
Giroud (1982)	I	15	1	44	2	10	1
CFGG (French 1)	M	0	9	0	3	n/a	n/a
CFGG (French 2)	M	0	15	25	21	19	1
CFGG (French 3)	M	0	7	6	37	15	5
FHwA - not critical, not severe (1989)	not	14	2	44	2	19	1
FHwA - critical, severe (1989)	not	7	9	43	3	19	1
b – Soil retention criteria							
Zitscher (1974)	C	n/a	n/a	37	3	12	7
Ogink (1975)	B	16	0	38	2	19	0
Sweetland (1977)	E	16	0	12	28	0	19
Schober and Teindl (1979)	G	16	0	36	4	9	10
ICI Fibres (= Rankilor 1981)	F	16	0	12	28	2	17
Giroud (1982)	I	14	1	n/a	n/a	n/a	n/a
Carroll (1983)	J	16	0	38	2	19	0
CFGG (1986)	M	16	0	38	2	19	0
AASHTO (1986)	T	16	0	38	2	18	1
FHwA steady flow (Christopher and Holtz 1989)	not	16	0	38	2	19	0
FHwA dynamic flow (Christopher and Holtz 1989)	not	7	9	1	39	16	3
Fischer *et al.* (1990)	O	16	0	30	10	1	8
Luettich *et al.* (1992) steady flow	not	16	0	35	5	16	3
Luettich *et al.* (1992) dynamic flow	not	15	1	34	6	1	18
c – Excessive Clogging							
CFGG (1986)	M	n/a	n/a	24	22	19	1
FHwA (1989, Christopher and Holtz 1989)	not	n/a	n/a	33	13	19	1
Fisher *et al.* (1990)	O	n/a	n/a	28	18	19	1

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Abbreviations

AASHTO	American Association of State Highway and Transportation Officials, USA.
ABC	American Building Contractors, USA.
AFNOR	L'association française de normalisation, France (French Standardisation Association.
AFEID	French Committee(s) of ICID
AOS	Apparent Opening Size, USA, comparable with O95 based on dry sieving.
AR	Aramide, type of synthetic sewing yarn (plastic, similar to PET and PVC)
ARBTA	American Road Builders and Transportation Association, USA.
AS	Designation of Australian standards, followed by a number, i.e. AS 3706.9
ASAE	American Society of Agricultural Engineers, St. Joseph, MI, USA
ASCE	American Society of Civil Engineers, Reston, VA, USA
ASTM	American Society for Testing and Materials, West Conshohocken, PA, USA.
BAW	Bundesanstalt für Wasserbau, Germany
BS	Designation of British Standard Institution followed by specific number, London, UK
CEN	Comité Européen de Normalisation (French), English: European Committee for Standardisation, German: Europäisches Komitee für Normung. All standards will be published in French, English and German. CEN, Brussels, Belgium
CEN TC 189	The CEN Technical Committee 189 is in charge of developing standards and guidelines in the area of geotextiles and related products.
CFGG	Comité Français des Géotextiles et des Géomembranes, France
c.i.f.	cost, insurance, freight
COE	Corps of Engineers, US Army.
CR	crushed rock (gravel). Designation used in Pakistan to distinguish test results from those using RR (= river run) gravel material
CRBC	Chasma Right Bank Canal, irrigation and drainage project in Pakistan
CV	Coefficient of Variation: Standard deviation divided by the average (mean).
COS	Characteristic opening size without specifying at which percent passing, generic, can refer to AOS, EOS and FOS
D.I. Khan	Dera Ismalaya Khan, drainage project, Pakistan
DIN	Deutsche Industry Norm (German equivalent of ASTM, NEN, etc.)
DRI	Drainage Research Institute, Egypt
dS	Deci Siemens
EC	Electrical Conductivity (of the soil or water)
ECS	European Committee for Standardisation, see also CEN, Brussels, Belgium
EEC	European Economic Community, see EU.
EKTP	East Khairpur Tile Drainage Project, Pakistan
EN	1 - designation of standards accepted by CEN. European standard (to be reviewed every 5 years). See also prEN. 2 – Envelope number in suitability calculation
EN ISO	designation for approved standards by CEN, which are also accepted by ISO.
ENV	designation for temporary European standards i.e. ENV 12226 (1996), see also prEN, and EN ISO. European pre-standard (validity period of 2 years)
EOS	Equivalent opening size, geotextiles, O_{90} based on wet sieving.
ESP	Exchangeable Sodium Percentage
EU	European Union. Before 1995 European Economic Community (EEC)
FAO	Food and Agricultural Organisation, Rome, Italy

FDP	Fourth Drainage Project, Faisalabad, Pakistan, also known as Drainage IV
FES	Fordwah Eastern Sadiqia project, drainage project in Pakistan
FHwA	Federal Highway Administration, USA.
FIH	Franzius Institute Hanover, Germany.
FOS	Filtration opening size, geotextiles, O95 based on hydrodynamic sieving.
G4, G5, G6	4^{th}, 5^{th} and 6^{th} International Conference on Geotextiles. Geomembranes and Related Products (References: Hoedt 1990, Karunaratne *et al.* 1995, and Rowe 1998).
GIS	Geographical Information System
GR	Gradient Ratio (test).
GR_{25}	Gradient Ratio calculated according to ASTM D5101.
HCR	Hydraulic conductivity ratio (test).
HFG	Hydraulic Failure Gradient is the hydraulic gradient at which soil cannot resist the drag force of water on soil particles. The hydraulic failure gradient is defined as the gradient achieved in a permeameter at which the soil will not bridge over the characteristic opening size of the envelope (whether a granular or synthetic material). OR: ... at which soil supported by a screen with a mesh of 0.059 mm looses its internal stability and particles will flow out of the soil sample through the filter.
IGS	International Geosynthetics Society, Easley, South Carolina, USA.
ISO	International Standardisation Organisation, Geneva 20, Switzerland
ISAWIP	Integrated Soil and Water Improvement Project (approx. 1990 – 1993), Egypt
KPa	Kilo Pascal
LBOD	Left Bank Outfall Drain, Sindh, Pakistan
LTF	Long term flow (test)
msl, MSL	Mean sea level
MWD	Mean weight diameter of water stable aggregates
N	Newton
N.B.	Nota Bene
na , n/a	Not applicable
PA	Polyamide or Pennsylvania (a state of the USA)
PCN	Performance Criterion Number in suitability determination of drain envelope.
PSD	Particle size distribution
PET	Polyester
PI	Plasticity Index
PLM	Pre-wrapped Loose Material; a permeable structure consisting of loose, randomly oriented yarns, fibres, filaments, grains, granules or beads, surrounding corrugated drainpipe, assembled within a permeable surround or retained in place by appropriate netting and used in drainage applications (CEN/TC 155 WI 1261, 1994).
POA	Percent Open Area of a thin woven synthetic envelope.
PP	Polypropylene.
prEN	Draft standard by CEN, which is still under review (pre-standard).
prENV	Draft European pre-standard
PVC-C	Plasticised Polyvinyl Chloride. This is a special form of PVC with has received a post treatment with Chloride, which makes the PVC more resistant to high temperatures. Flammability behaviour depends on proportion of plasticiser present; most plasticisers burn readily with a yellow, smoky flame. Black smoke is given off. The odours are mostly esther-like, but with an unpleasant acidic smell (Koerner 1994).
PVC-U	Un-plasticised Polyvinyl Chloride. The PVC does not contain a softener (like PVC for garden hoses) and is therefore more suitable for drainage application.

	The term un-plasticised is used in European standards primarily. Burns with difficulty and is self-extinguishing. Flame is yellow, green at bottom edges, with spurts of green and yellow. White smoke is given off. The material softens on ignition and has an unpleasant acidic smell (Koerner 1994).
PVD	Prefabricated Vertical Drain. Term used in Europe for what is called wick drain in the USA.
RR	River run (gravel). Designation used in Pakistan to distinguish test results from those using CR = crushed rock.
RRL	Road Research Laboratory, UK
S	Siemens.
SAR	Sodium Absorption Ratio.
SARadj	Adjusted Sodium Absorption Ratio.
SCARP	Salinity Control and Reclamation Project, Pakistan.
SCS	Soil Conservation Service
SI	Système Internationale (d'Unités).
SRWSC	State Rivers and Water Supply Commission, Victoria, Australia.
TGL	TGL 20 286, 1971, Standard of German Democratic Republic
UK	United Kingdom
USA	United States of America
USBR	United States Bureau of Reclamation, United States Department of Agriculture (USDA).
USC	Unified Soil Classification System
USDA	United States Department of Agriculture
WAPDA	Water and Power Development Authority, Pakistan
WD	Working draft (stage in ISO corresponding to stage 11 in CEN), abbreviation used with European standards
WI	Work Item, abbreviation used with European standards
WG	Working Group, abbreviation used with European standards

Glossary

Acid sulphate soil: A soil with a pH below 4 as a result of the oxidation of pyrite to sulphuric acid.

Alkali soil: See **Sodic soil.**

Alkalinity: A property of soil or water, characterised by a pH between 7 and 14.

Allowable velocity: Flow velocity of water in an open channel, just below the velocity that would cause bed material to detach.

Apparent opening size: O_{95} for a geotextile (ASTM D 4439-95 and ASTM D 4751-95). A property, which indicates the approximate largest particle that, would effectively pass through the geotextile.

Apparent velocity: A fictitious velocity of water flowing through a porous medium (e.g. soil), better referred to as the discharge per unit area. Used in Darcy's Equation.

Arithmetic mean: See **Mean**.

Atterberg limits: See **Consistency limits**.

Augerhole method: A technique to determine the saturated hydraulic conductivity of a soil at a certain depth by augering a cylindrical hole in the soil, bailing water from it, and measuring the rate of water-level rise in the hole.

Available water: The quantity of water available to plants, defined as the quantity of water retained in the soil between field capacity and permanent wilting point.

Bandwidth: A range of minimum and maximum particle size in between which desirable properties of the soil or envelope material are located.

Bedding: A specially prepared layer (with fine gravel or sand) on which drainpipes are laid. Usually done when the trench bottom is irregular due to excavation by hand or by machine.

Blinding: The process of placing bedding material of loose stable-structured soil on the sides and over the top of the drain to a depth of 150 mm (6") with the intend to block or prevent macro pores having direct access to drain envelope material.

Bulk density: The mass of soil per unit volume in an undisturbed condition. Normally equivalent to the dry bulk density (i.e. when only the dry soil mass is considered), but sometimes to the wet bulk density (i.e. when the mass of water present is also considered).

Coefficient of curvature: C_c, Sometimes also called coefficient of gradation. A measure of the normality of distribution of the soil.

Coefficient of Uniformity: C_u, A measure of the slope of the gradation curve.

Coir: Strings of coconut fibre used as envelope material.

Collector drain: A drain that collects water from the field drainage system and carries it to the main drain for disposal. It may be either an open ditch or a pipe drain.

Consistency limits: Soil physical values indicating the ease with which the soil can be deformed (i.e. a plastic limit and a liquid limit); also called Atterberg limits.

Correlation coefficient: A measure of the linear interdependence of two variates, ranging from −1 (perfect negative correlation) to +1 (perfect positive correlation).

Denier: Denier is the weight in grams of 9000 m of a single fibre. See also: **dtex** and **Tex**

Density Index: A relative measure of the stability of the soil or gravel structure as function of actual void ratio, void ratio at its loosest and most dense packing.

Design discharge: A specific value of the flow rate which, after the frequency and the duration of exceedance have been considered, is selected for designing the dimensions of a structure or a system, or a part thereof.

Drain spacing: The horizontal distance between the centre lines of adjacent parallel drains.

Drainable pore space: The ratio of the change in soil-water content in the profile above the water table to the corresponding rise/fall of the water table, in the absence of evaporation

Drainable surplus: The amount of water that must be removed from an area within a certain period so as to avoid an unacceptable rise in the levels of groundwater or surface water.

Drainage coefficient: The discharge of a drainage system, expressed as a depth of water that must be removed within a certain time.

Drainage intensity: (1) An agricultural drainage criterion based on the ratio between the design discharge and the depth of the water table. (2) The number of drainage provisions (e.g. natural or artificial open drains, pipe drains, or tubewells) per unit area.

dtex: dtex is the weight in grams of 10,000 m, Koerner 1994 defines tex as weight per 1000 m! The d is for deci. See also: **Tex**

Dynamic viscosity: In fluid dynamics, the ratio between the shear stress acting along any plane between neighbouring fluid elements, and the rate of deformation of the velocity gradient perpendicular to this plane.

Effective porosity: See **Drainable pore space**.

Electrical conductivity (EC): The reciprocal of the electrical resistance measured between opposite faces of a centimetre cube of an aqueous solution at a specified temperature, usually 25 °C. It is a measure of the concentration of salts.

Elevation head: The vertical distance to a point above a reference level.

Entrance head: The head required to overcome the entrance resistance of a pipe drain. (See **Entrance resistance**.)

Entrance resistance: The extra resistance to water flow in the vicinity of a drainpipe, due to a decreased permeability of the material around the drain and/or to a contraction of the flow lines resulting from the small drain openings.

Envelope: Material placed around pipe drains to serve one or a combination of the following functions: (i) to prevent the movement of soil particles into the drain; (ii) to lower entrance resistance in the immediate vicinity of the drain openings by providing material that is more permeable than the surrounding soil; (iii) to provide suitable bedding for the drain; (iv) to stabilise the soil material on which the drain is being laid.

Equivalent depth: Depth to the imaginary impermeable layer, introduced by Hooghoudt to take into account the radial flow resistance near drains in deep homogeneous soils.

Exchangeable sodium percentage (ESP): The fraction of the soil's cation exchange capacity that is occupied by sodium ions. It is a yardstick of sodicity problems in soils.

Fabric envelope: May be referred to as synthetic and geotextile, and based on the manufacturing process can be further defined as loosely wrapped, non-woven, woven, needle punched.

Field capacity: The volumetric water content of a soil after rapid gravity drainage has ceased. It usually occurs about two days after the soil profile has been thoroughly wetted by precipitation or irrigation.

Filter: A layer or combination of layers of pervious materials, designed and installed so as to provide drainage, yet prevent the movement of soil particles in the flowing water.

Flowability: The way granular material flows through constricted areas under gravity, as in a trencher box.

Frequency analysis: A statistical method of analysing hydrological or other data, which uses the observed number of occurrences to predict how often a phenomenon may occur in the future and to assess the reliability of this prediction.

Frequency distribution: (1) A tabular arrangement of empirical data by classes, together with the corresponding class frequencies. (2) A mathematical expression of the relationship between a value and its theoretical frequency.

Gap-graded: soil which misses certain particles size and therefore shows horizontal section in the PSD curve.

Geotextile: A permeable, polymeric, synthetic or natural, textile material, in the form of manufactured sheet, which may be woven, nonwoven or knitted, used in geotechnical and civil engineering applications. The term geotechnical includes the land drainage application (CEN/TC 155 WI 1261, 1994).

Granular envelope: a drain envelope made up graded granular material, usually gravel mixed with sand.

Gravel mole: A mole drain filled with gravel material.

Gravel pack: An artificially-graded filter placed immediately around a well screen so as to increase the local permeability, to prevent soil particles from entering the well, and to allow a somewhat larger slot size in the well screen.

Gravimetric method: A method of measuring the water content of the soil, which involves determining the weight loss from a number of oven-dried field samples obtained by coring or augering.

Hjulström curves: Curves which show the relationship between water velocity and the mean size of the soil particle that will start moving at that velocity (scouring velocity).

Hydraulic conductivity: The constant of proportionality in Darcy's Law, defined as the volume of water that will move through a porous medium in unit time, under a unit hydraulic gradient, through a unit area, measured at right angles to the direction of flow.

Hydraulic Failure Gradient (HFG): The hydraulic gradient at which (cohesive) soil cannot resist the drag force of water on soil particles (Samani and Willardson (1981). The hydraulic failure gradient is defined as the gradient achieved in a permeameter at which the soil will not bridge over the characteristic opening size of the envelope (whether a granular or synthetic material).

Hydraulic head: The elevation of the water level in a piezometer with respect to a reference level; it equals the sum of the pressure head and the elevation head.

Hydraulic soil properties: Properties of the soil profile that affect the flow of water (e.g. hydraulic conductivity, soil-water content, specific water capacity, or diffusivity), often as a function of pressure head.

Ideal drain: A drain without entrance resistance.

Isotropic: Having the same physical properties in all directions.

Kevlar: Synthetic material

K-value: See **Hydraulic conductivity**.

Kinematic viscosity: The dynamic viscosity divided by the fluid density.

Laminar flow: Flow of water in separate thin layers, not influenced by adjacent layers perpendicular to the direction of flow.

Liquid Limit: The minimum percentage by weight of moisture at which a small sample of soil will barely flow under standard treatment. Liquid Limit is synonymous with upper Plastic Limit.

Log-normal distribution: A transformed normal distribution in which the variate is replaced by its logarithm. It is used empirically for hydrological frequency analysis.

Mean: Arithmetic mean, the average value of a list of numbers. The centre of gravity in a histogram.

Median: the number in the middle of a list of numbers, the d_{50} and D_{50} represents the median value of a gradation curve and is the size of the sediment of which 50% is finer and 50% is larger by weight.

Mechanical analysis: Determining the particle-size distribution of a soil by screening, sieving, or other means of mechanical separation.

Mode: Most frequently occurring, or repeated value in a list of numbers.

Mole drain: An unlined underground drainage channel, formed by pulling a solid object, usually a solid cylinder with a wedge-shaped point at one end, through the soil at the proper slope and depth, without a trench having to be dug.

Normal distribution: A symmetrical, bell-shaped, infinite, continuous distribution, theoretically representing the distribution of accidental errors about their mean.

Observation well: A small-diameter pipe, at least 25 mm in diameter, in which the depth of the water table can be observed. It is placed in the soil and perforated over a length equal to the distance over which the water table is expected to fluctuate.

Organic envelope: a drain envelope made of straw, jute or other stringy organic matter.

Organic soils: Soils with a high content of composed or decomposed organic carbon and a low mineral content.

Performance Assessment: the determination of the functioning of the drainage system compared with established design criteria, and to identify the cause of malfunctioning.

Permeability: (1) Qualitatively, the quality or state of a porous medium relating to the readiness with which such a medium conducts or transmits fluids. (2) Quantitatively, the specific property governing the rate or readiness with which a porous medium transmits fluids under standard conditions. Permeability is permittivity multiplied with material thickness T_g. More properly referred to as Hydraulic Conductivity. See also **Hydraulic conductivity.**

Permittivity: Permittivity is permeability divided by material thickness T_g. For a geotextile, the volumetric rate of water per unit cross-section area, per unit head, under laminar flow conditions, in the normal direction through the fabric.

pH: A measure of the hydrogen ion concentration in a solution, expressed as the common logarithm of the reciprocal of the hydrogen ion concentration in mol per litre. pH is presented on a scale of 0 – 14; 0 represents the most acid, and 14 the most alkaline. All values below 7.0 are acid and all above 7.0 alkaline. Hence pH is a measure of acidity or alkalinity of a fluid.

Piezometer: A small-diameter pipe used to observe the hydraulic head of groundwater. It is placed in, or driven into, the subsoil so that there is no leakage around the pipe. Water can only enter the pipe through a short screen at the bottom of the pipe, or through the bottom only.

Piezometric head: See **Hydraulic head.**

Piping: The continuous flow of particles into a drainpipe through macro pores which remain open; used also to indicate similar effect in permeameter tests, when flow paths along the plexiglas appear.

Plasticity Index: The numerical difference between the Liquid and the Plastic Limit, or synonymously, between the lower Plastic Limit and the upper Plastic Limit. Also called Plasticity Number.

Plastic Limit: The minimum moisture percentage by weight at which a small sample of soil material can be deformed without rupture. Synonymous with lower Plastic Limit.

Porosity: The fraction of the volume of soil pores plus solids occupied by soil pores.

Possible maximum discharge: The maximum, theoretical discharge from a lateral drain if the midway water table depth between two drains is assumed to be at the land surface (NB. For design purpose the design discharge is based on the design water table depth midway between two drains.)

Potential head: See **Hydraulic head.**

Probability: The chance that a prescribed event will occur, represented as a pure number p in the range 0 - 1. It can be estimated empirically by the relative frequency (i.e. the number of times the particular event occurs, divided by the total count of all events in the class considered).

Puddling (action): the destruction of soil structure in saturated soil to make it act like a fluid.

Quartile: the value at which 25%, 50% or 75% of the value range is achieved.

Radial flow: Groundwater flow towards the wet perimeter of a drain, whereby the flow lines resemble converging radii.

Radial resistance: A resistance against water flow, which occurs as a consequence of radially converging flow lines.

Re-grading: Re-grading is adjustment of percentage of weight retained on each sieve by excluding the combined weight of the particle sizes greater than 4.75 mm. The reason for this is that these larger particle sizes do not play an important role in hydraulic and sedimentation characteristics of soils, and better relationships concerning filtering and hydraulic criteria are possible when these larger particles sizes are excluded from the assessment.

Regression analysis: A statistical technique applied to paired data to determine the degree or intensity of mutual association of a dependent variable with one or more independent variables.

Replicates: A replicate, in research terminology, is an observation under similar conditions as others and serves to exclude the random effects that may affect observations. Commonly 3 replicates are suggested, but in envelope research at least 4 replicates, and preferably 6 are deemed appropriate.

Reynolds number: A hydraulic number representing the ratio of inertia forces to viscous forces, and allowing laminar flow and turbulent flow to be distinguished.

Ripened soil: Soil that does not have the characteristic of the un-ripened soil (see un-ripened).

Rip-rap: Broken stone or boulders placed compactly or irregularly on dams, levees, dikes, or similar embankments, and at the downstream end of structures, to protect earth surfaces from the action of waves, currents, and flowing water.

Saturated soil paste: A particular mixture of soil and water, which glistens as it reflects light, flows slightly when the container is tipped, and slides freely and cleanly from a spatula for all soils except those with a high clay content.

Saturation extract: The solution extracted from a saturated soil paste.

Saturation percentage: The water content of a soil sample that has been brought to saturation by adding water while stirring, expressed as grams of water per 100 grams of dry soil.

Selvage: Also: selvedge; the edge of a fabric woven so that it will not unravel.

Shrinkage: The change in volume of a soil, produced by capillary stresses when the soil is drying.

Smearing: The sealing with liquefied soil, mostly a problem with heavy textures soils, which creates and impermeable layer.

Skip-graded: Same as gap-graded soil. Term is used in US Army Corps of Engineers publication (1978) and also surfaces again in SCS 1994.

Sodicity: A soil feature indicating a problem of high sodium content. See **Sodic soil**.

Sodic soil: A soil that contains sufficient exchangeable sodium to interfere with soil structure and the growth of most crops, without appreciable quantities of other soluble salts being present.

Sodium adsorption ratio (SAR): A ratio for soil extracts and irrigation water that expresses the relative activity of sodium ions in exchange reactions with soil. An adjusted SAR is used to classify irrigation water according to its potential to cause infiltration problems because of its high relative sodium content.

Soil classification: The organisation of types of soil in a systematic and meaningful way, based on practical characteristics and criteria.

Soil ripening: The process that transforms a soft, water-saturated, and reduced sediment into a soil that can be used for agriculture. A distinction is made between biological, chemical, and physical ripening.

Soil salinity: The presence of salts in the soil profile that impair crop production.

Soil structure: The combination or aggregation of primary soil particles into aggregates or clusters (peds), which are separated from adjoining peds by surfaces of weakness.

Soil survey: The systematic examination of soils in the field, including the laboratory analysis of specific samples, their description, and mapping.

Soil texture: The relative proportions of the various sized groups of individual soil grains in a mass of soil. Specifically, it refers to the proportions of clay, silt, and sand below 2 mm in size (fine earth fraction).

Specific volume: The volume of a unit mass of dry soil in an undisturbed condition, equalling the reciprocal of the dry bulk density of the soil.

Spunbonded: A finished nonwoven fabric from a polymer produced in a continuous process and that is bonded by thermal, mechanical, or chemical treatment.

Standard deviation: A statistical measure of dispersion of a frequency distribution, equal to the positive square root of the mean squared deviation of a number of individual measurements of a variate from their population mean. A measure of the average spread of the frequency curve.

Staple: Short fibres in the range 1 to 8 cm long.

Task Force 25: A joint committee consisting of officials of the AASHTO, the ABC and the ARBTA that formulates a unified approach to minimum design specifications for geotextiles (Task Force 25, 1991).

Tex: Denier multiplied by 9 and is the weight in grams of 1000 m of yarn.

Textural class: The name of a soil group with a particular range of sand, silt and clay percentages, of which the sum is 100% (e.g. sandy clay is: 45-65% sand, 0-20% silt, 35-55% clay).

Textural triangle: A triangle indicating the boundary limits of the sand, silt, and clay percentages for each textural class.

Texture: See **Soil texture**.

Transmissivity: For a geotextile, the volumetric rate of water per unit thickness under laminar flow conditions, in the in-plane direction through the fabric.

Transmissivity: Transmissivity is permeability multiplied with material thickness T_g

Trencher: A drainage machine that digs a trench in which a drainpipe and envelope are laid.

Tremmied: the levelling and compacting of soil in a permeameter. Tremi: A pipe or tube used to place soil, gravel, concrete, where the material must fall for some distance. The material is poured through a pipe and can be accurately placed without segregation by moving the pipe.

Turbulent flow: Flow of water, agitated by cross-currents and eddies, as opposed to laminar flow. Any particle may move in any direction with respect to any other particle, and the head loss is approximately proportional to the second power of the velocity.

Unified: Unified Soil Classification System, USBR.

Uniform flow: Flow of water with no change in depth or any other element of flow (e.g. cross-sectional area, velocity, and hydraulic gradient) from section to section along a canal.

Unplasticised PVC: See PVC-U in abbreviations section.

Unripened soil: In the Netherlands a clayey type of soil which has not been aerated (generally has been submerged up to the point when drainage is going to be applied), and which has no signs of shrinkage and, a hydraulic conductivity of near zero (which will drastically improve during the reclamation process). Major characteristics used by soil scientists for identification are substantial clay content, blue of colour (no brown present), and will squeeze between fingers when a sample is pressed in a fist. See Soil-ripening

Void ratio: Ratio of the volume of pores to the volume of solids in a soil.

Water balance: Equating all inputs and outputs of water, for a volume of soil or for a hydrological area, to the change in storage, over a given period of time.

Weighting: A statistical method of adjusting the results of observations by taking into account the fact that not all the data may be of equal reliability or importance.

Wettability: The ease with which water flows through, primarily, new geotextiles. Or in other words, the extra resistance geotextiles show during initial wetting.

Wide width: Term used in geotextile testing for indicating certain test methodology.

Symbols and units

Symbol	Definition	Units
A	Cross sectional area of the sedimentation cylinder	cm^3
a, A	Cross-sectional area, drained area	m^2, km^2
a	Distance	m
a_c	A dimensionless contraction coefficient	–
a_e	Total entrance resistance contraction constant [the ratio of the area of synthetic envelope exposed to perforations over the total area per meter pipe length.	–
A	The area open to inflow (perforation area) per unit length	m^2/m
A_p	The area of the perforations or gaps per meter pipe length for the pipe flowing full	m^2/m
A_{pe}	The area of actual flow into the drain envelope per unit length that is a function of wetted perimeter ratio of total unit area.	m^2/m
A_{pu}	Actual area of inflow into the drainpipe	m^2/m
b	Bottom width of a canal, drain, or outlet	m
Bc	Pipe width (as seen in trench bottom)	m
B_d	Trench width	m
B/C	Benefit / Cost ratio	–
c	Compression, consolidation constant	–
c	Distance between corrugations	mm
cf_{xx}	A shape factor for clayey soils (similar to sf_{xx} but not the same)	–
c, C	Hydraulic resistance	d
C_d	Pipe load coefficient	–
C	Chézy coefficient	$m^{0.5}/s$
C_c	Coefficient of curvature	–
C_u	Coefficient of uniformity	–
C'_u	Linear coefficient of uniformity	–
d, D	Depth, thickness, height, diameter	mm, m, mm
D	Depth to impermeable layer	m
d_e	Equivalent depth as function of depth to impermeable layer (D), S and the outside pipe radius	m
d_i	Inside diameter of drain pipe	mm
d_o	Outside diameter of drain pipe	mm
$D_{opening}$	The diameter, or the width of slotted openings, of the perforation in drain pipes	mm

Symbol	Definition	Units
d_{xxy}	Particle size of the base soil at which 90% of the material has a smaller size. xx can be any percentage but common ones are 5, 10, 15, 30, 50, 60, 85, 90, 95, 100. The subscript y can be: f = filter, c - course	mm, mm
$d`_{xx}$	Particle size derived from straight line extensions on standard PSD curve at xx% passing.	mm, mm
D_{xxy}	Particle size of the base soil at which 90% of the material has a smaller size. xx can be any percentage but common ones are 5, 10, 15, 30, 50, 60, 85, 90, 95, 100. The subscript y can be: f = filter, c - course	mm, mm
d_{wtd}	Design water table depth below surface	m
e	The base of natural logarithm (2.7183)	
e_v	Void ratio, actual in-situ	–
e_{max}	Void ratio, in its densest state	–
e_{min}	Void ratio in its loosest state	–
EC	Electrical conductivity at 25°C	dS/m
EC_b	The dispersion or deflocculation boundary of the electrical conductivity of soil water	dS/m
EC_e	average soil profile salinity of the saturated extract.	dS/m
EC_d	salinity of drainage effluent.	dS/m
EC_{iw}	salinity of irrigation water.	dS/m
EC_w	Electrical Conductivity of the [irrigation] water	dS/m
ESP	Exchangeable Sodium Percentage	-
f_s	A factor accounting for the influence of the shape and structure of the soil particles on the actual length of the flow lines.	
F	Force	N
F_{up}	Upward force	N
G	Specific mass of individual soil particles	g/cm^3
GR	Gradient Ratio	–
GR_{25}	Gradient ratio including one inch (25 mm) of soil	–
G(x)	Cumulative probability function	
g	Acceleration due to gravity	m/s^2
H	Hydrometer settling depth, hydraulic head difference	m
h, H	Height, water depth, (Energy) head or head loss	m
h_c	Head loss resistance (secondary) convergence (contraction)	m
h_e	Head loss, difference between heads in observation well closest to the drain (near the envelope-soil interface, or just outside the trench boundary) and in the drain	m

Symbol	Definition	Units
h_r	Combined radial head loss in the soil	m
h_t	Head midway between drains with respect to selected reference level (usually centre line of the drain, but better is to take the invert of the drain when to be used with entrance loss calculations)	m
HCR	Hydraulic Conductivity Ratio	–
HFG	Hydraulic Failure Gradient	–
H_{max}	Maximum head midway between drains, can be taken as drain depth	m
i	Hydraulic gradient (dimensionless)	–
i_c	Critical gradient	–
i_{env}	Hydraulic gradient across the envelope	–
i_{max}	Maximum (expected) hydraulic gradient across the envelope	–
i_x	Exit gradient as calculated from the Darcy equation at the perforations of the drainpipe (without drain envelope), or at the envelope openings of the envelope-soil interface.	–
I_D	Relative density index	–
J	Julian day number	–
k	Corrugation height	m
K	Hydraulic conductivity, without specifying for which material.	m/d
K_1	Hydraulic conductivity of soil layer below the drain, usually K_s	m/d
K_2	Hydraulic conductivity of soil layer above the drain, usually K_s	m/d
K_c	Hydraulic conductivity calculated based on particle sizes	m/d
K_{cc}	The average (calculated) hydraulic conductivity in m/d or cm/s for when $D_{15}<0.2$ mm	m/d
K_e	Hydraulic conductivity or permeability of envelope material either for gravel envelopes (K_g), or synthetic envelopes (geotextiles)	m/d
K_{es}	Hydraulic conductivity as determined from permeameter results when a certain thickness of the soil is included with apparent synthetic envelope permeability. In addition the top plate used to hold fabric envelopes in place may be included as well.	m/d
K_f	Hydraulic conductivity of a filter, preferably not to be used with drain envelopes, but with other civil engineering applications.	m/d

Symbol	Definition	Units
K_g	Hydraulic conductivity for granular materials used for drain envelopes.	m/d
k'_g	Modified Gapon selectivity coefficient	$(mmol/l)^{-1/2}$
K_n	Hydraulic conductivity of geotextile material normal to the plane of the material.	m/d
K_p	Hydraulic conductivity of geotextile material in-plane.	m/d
K_r	Radial hydraulic conductivity	m/d
K_s	Hydraulic conductivity of the base soil at drain depth.	m/d
K_T	Hydraulic conductivity at temperature T	m/d
K_{20}	Reference hydraulic conductivity at temperature 20 °C. All laboratory determined values should be converted to the equivalent K at 20 °C.	m/d
KD, KH	Transmissivity	m^2/d
L	Height of the soil column, or length	m
L_1	Distance along the hydrometer from the top of the bulb to the mark for a hydrometer reading	cm
L_2	Overall length of the hydrometer bulb	cm
n	Manning's resistance coefficient	–
N_s	Suitability number (of envelope)	–
O_{xx}	The pore size diameter for which xx% of the pores are smaller. xx can be any percentage but common values are 50, 85, 90, and 95.	µm, mm
PCN	Performance criterion number, varies from 1 – 7	–
p_e	Effective pore diameter	m
P_w	Mass density of water	kg/m^3
P	Wetted perimeter	m
PI	Plasticity Index	–
P_s	The bulk density of saturated soil in kg/m3	
q, Q	Discharge, flow rate, runoff rate, flux	m^3/d, m^2/d, m/d
q	Drainage coefficient, drainable surplus	mm/d
q_d	Design drainage coefficient	m/d, mm/d
q_{dmax}	Maximum possible discharge per unit area under free flow conditions	m/d
q_l	Design discharge per unit length of lateral (q*S)	$m^3/d/m$
q_{lmax}	Maximum possible discharge per unit length of lateral	$m^3/d/m$
Q_d	Design discharge from drain	m^3/s
Q_{dmax}	Maximum possible discharge from drain	m^3/s
r	Correlation coefficient	–

Symbol	Definition	Units
r, R	Distance or radius of influence of radial flow, the distance of the nearest observation well from the centre line of the drain (but not further away than 0.2-03.m), the outer radius of geotextile specimen with permittivity determination	m
r_e	Entrance resistance per unit discharge	d/m
r_o	(ideal) outside drain radius	m
R_o	Inner radius of geotextile specimen with permittivity determination	m
R_e	Reynolds number(–)	–
RSC	Residual Sodium Carbonate	meq/l
S	Drain spacing	m
s	Distance	m
s	(Water table) draw-down	cm, m
SAR	Sodium Adsorption Ratio	meq0.5/l0.5
Sf_{xx}	A shape factor related to the characteristic particle diameter D_{xx}	
t	Time	yr, d, s
T	Temperature in °C	
T_d	Thickness of granular envelope	m
T_f	Thickness of a filter (civil engineering)	m
T_g	Thickness of geotextile under standard pressure of 2 kPa	m
u	Wetted perimeter	m
$U_{(cm)}$	Specific surface ratio. U-ratio of the soil sample specific surface of the soil particles.	–
$(U_{cm})_i$	U-ratio of the sub-fraction specific surface of the soil particle dimensionless. The U-ratio of each sub-fraction is multiplied by the weight of the fraction, and the sum of all these products is divided by the total weight of all the sub-fractions in grams in order to find the U-ratio of the sample as a whole.	–
v	Flow velocity (average)	m/s
v_e	Average linear velocity of pore flow	m/s, m/d
V	Volume of water	m^3
V_a	Volume of hydrometer bulb	cm^3
V_v	Volume of voids through which flow takes place	m^3
V_t	Total volume of voids (pores)	m^3
V	Volume	m^3
w, W	weight, Width	m
w	Soil weight	kg/m^3
W_c	Pipe load	kg/m, N/m

Symbol	Definition	Units
W_{DS}	Mass of oven-dried soil	g
w_e	Total entrance resistance	m
W_i	Weight of sub fraction "i" with U-ratio calculation	g
W_{LL}	Soil mass at liquid limit	g
W_{PL}	Soil mass at plastic limit	g
w_r	Radial entrance resistance	d
x	Distance or particle size	m, mm
x_{avg}	D_{50} or d_{50}	mm
y	Water depth	m
y	A transform determined from $y = (x-x_{avg})/s$	–
Δ, or d	Difference	–
ε	Porosity, the ratio volume over total volume	–
η	Dynamic viscosity	kg/m s
η_r	Dynamic viscosity of water adjusted for temperature	kg/m s
η_{20}	Dynamic viscosity at 20°C	kg/m s
θ	1 -Soil-water content (volume fraction)	%
	2 –Transmissivity	$m^3 min^{-1} m^{-1}$
θ_v	Water a content of the soil at the liquid limit	%
θ_{pl}	Water content of the soil at the plastic limit	%
μ	Drainable pore space, Specific yield, mass per unit surface area	g/m^2
ν	Kinematic viscosity	m^2/s
λ_{r1}	O_{90}/d_{90} ratio (permeability ratio or indicator)	–
λ_{r2}	O_{95}/d_{85} ratio	–
λ_{r3}	O_{50}/d_{50} ratio	–
ρ	Density	kg/m^3
ρ_b	Bulk density of soil or gravel material	kg/m^3
ρ_p	Particle density	kg/m^3
ρ_l	Liquid density	kg/m^3
ρ_s	Specific density of saturated soil	kg/m^3
ρ_w	Density of water	kg/m^3
σ, s	Standard deviation of a distribution	
σ_e	Standard deviation for envelope material	
ψ	Permittivity	s^{-1}

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