

**TECHNICAL RECOMMENDATIONS
FOR HIGHWAYS**

Draft TRH 15 : 1994

**SUBSURFACE
DRAINAGE
FOR
ROADS**

1994

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PREFACE

TECHNICAL RECOMMENDATIONS FOR HIGHWAYS (TRH) are written for the practising engineer and describe current, recommended practice in selected aspects of highway engineering. They are based on South African experience and the results of research and have the full support and approval of the Committee of State Road Authorities (CSRA).

This document was produced by a subcommittee of the Highway Materials Committee and, to confirm its validity in practice, is circulated in draft form for a trial period before being submitted to CSRA for final approval.

SYNOPSIS

These recommendations provide a broad appreciation of a variety of subsurface drainage problems, and draw on experience to suggest solutions which are appropriate to particular circumstances.

The text emphasises the importance of adequate drainage in, under and around the road pavement, and outlines the scope of the site investigations required to reveal the possible extent of drainage difficulties.

Given a clear understanding of subsurface drainage concepts, such as permeability, the designer is then in a position either deliberately to avoid the expected difficulties, or to resolve them by the installation of effective drains.

The features of typical installations for the control of ground water, and of surface infiltration, are reviewed and illustrated in some detail, and the functions of each of the contributory drain elements are described and defined. The use of synthetic materials such as geotextiles is also described.

The performance of subsurface drainage systems is dependant on the use of correct design, construction and maintenance methods and each of these aspects are dealt with.

KEYWORDS :

Subsurface, drainage, ground water, infiltration, geotextiles, maintenance.

SINOPSIS

Hierdie aanbevelings bied 'n omvattende beoordeling van 'n verskeidenheid van probleme wat met ondergrondse dreineringsprobleme ondervind word en maak gebruik van ondervinding ten einde geskikte oplossings vir besondere omstandighede voor te stel.

In die teks word die belangrikheid van voldoende dreineringsprobleme in, onder en om die padplaveisel beklemtoon en die bestek van die terreinondersoeke wat nodig is om die moontlike omvang van dreineringsprobleme aan te dui, in breë trekke geskets.

Wanneer die ontwerper 'n duidelike begrip het van ondergrondse dreineringsbegrippe, soos byvoorbeeld deurlatendheid, is hy in 'n posisie om of opsetlik die verwagte probleme te vermy of hulle op te los deur doeltreffende dreineringsprobleme te installeer.

Die eienskappe van tipiese installerings vir die beheer van grondwater, en van oppervlak-infiltrasie, word bespreek en tot 'n mate in detail geïllustreer, en die funksies van elk van die bydraende dreineringsprobleme word beskryf en gedefinieer. Die gebruik van sintetiese materiale soos geotekstiele word ook uiteengesit.

Die doeltreffendheid van ondergrondse dreineringsprobleme word bepaal deur die gebruik van korrekte ontwerp-, konstruksie-, en onderhoudsmetodes en daarom word al drie hierdie aspekte aangespreek.

FOREWORD

With the ever increasing cost of road construction and maintenance, it is now more than ever essential to protect the road pavement from premature failure. One of the chief causes of such failure is water.

It is probably true to say that surface and storm water are generally adequately catered for in both the literary and practical fields. However, subsurface drainage, an important element in the control of water in the road and so essential in ensuring protection of the pavement against failure, has been neglected.

This publication is an attempt at least in part to remedy these shortcomings and to provide engineers with much-needed information on this subject, including aspects such as preliminary investigation, flow characteristics and design elements. Sections on subsurface drainage construction and maintenance, given very little attention in the past, are also included.

It is of course necessary for these recommendations to be revised from time to time, and to this end comments from practising engineers will be most welcome.

CHAIRMAN
HIGHWAY MATERIALS COMMITTEE

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1. INTRODUCTION

1.1 SCOPE

This publication considers the occurrence and effect of water in road earthworks, foundations and pavements, and makes recommendations for the design, construction and maintenance of the subsurface drainage installations which are necessary to ensure the adequate performance of the road during its design life.

The publication is not concerned with surface or storm water drainage, which is fully covered by other technical literature, except where the subsurface drainage and surface drainage aspects are interrelated.

1.2 IMPORTANCE

Centuries of practical experience and extensive quantitative research, particularly in South Africa using accelerated testing¹, have illustrated the detrimental effect of water on pavement life (See *Figure 1*). The decrease of material strengths and the development of pore water pressures invariably result in the premature failure of wet pavements under traffic.

Pavement design methods currently recommended for use in South Africa² are based on the assumption that the pavement layers and subgrade are well-drained to at least material depth, which is defined as follows :

Road Category*	Material depth (mm)
A	1 000 - 1 200
B	800 - 1 000
C	800
D	300 - 600

* Road categories defined in TRH 42.

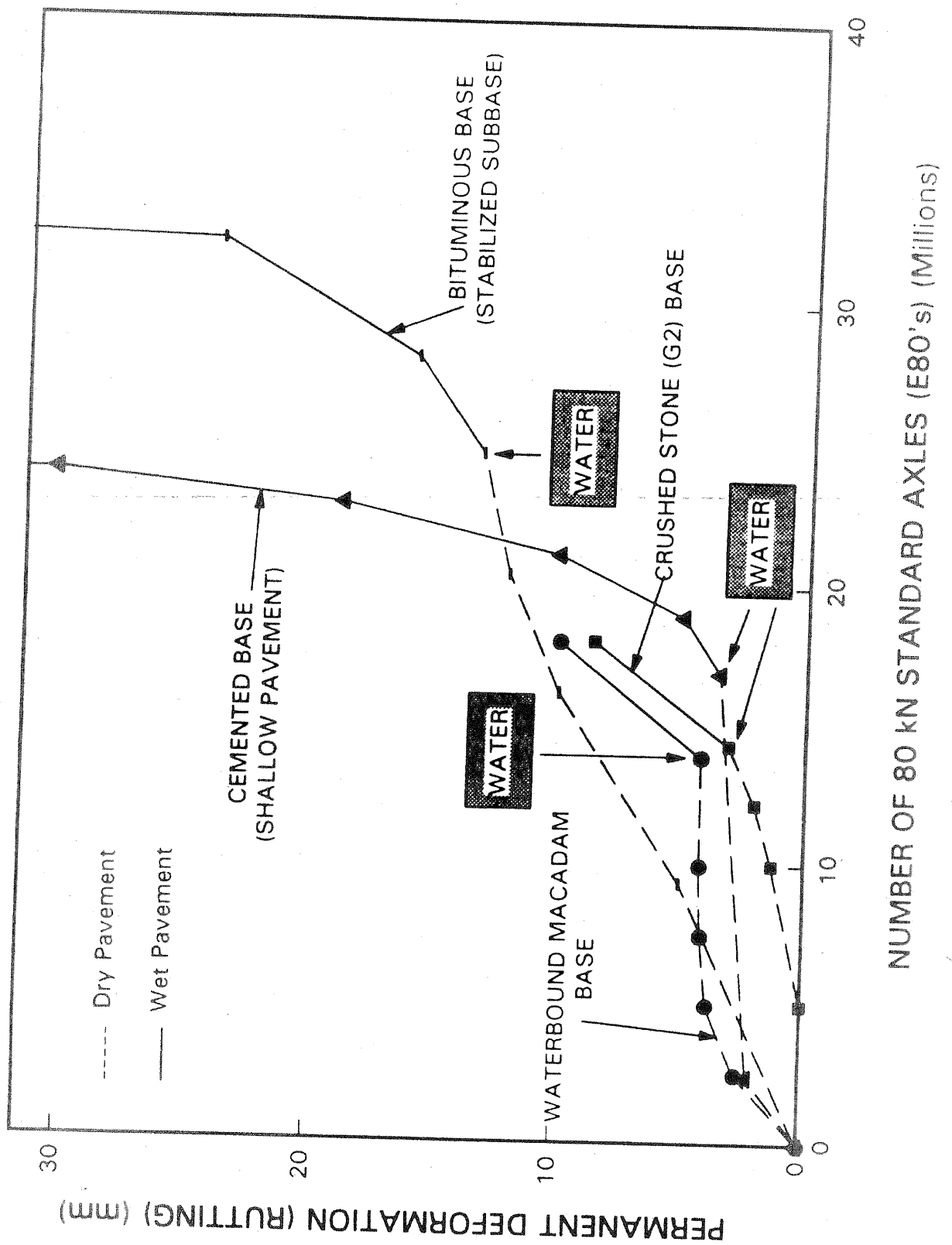


FIGURE 1

Typical HVS results showing the effect of water on pavements

1.3 PROCEDURES

It is often difficult to obtain the inputs required for a proper design of a subsurface drainage system. This, and the fact that subsurface drainage usually forms a relatively small portion of the total road construction cost, are the two main reasons for the lack of formal designs which is often evident in road construction and rehabilitation projects. However, the financial implication of drainage related failures makes the provision of adequate subsurface drainage so important that **special attention** must be given to its investigation and design and the following aspects need to be attended to during the various stages :

Stage	Subsurface drainage requirements
(a) Investigation	The collection of all the information required to appreciate and to resolve any subsurface drainage problems. This includes the collection and testing of representative soil samples and candidate subsurface drainage materials. It also includes field tests and surveys.
(b) Design : Geometric Drainage	The designer should locate and design to avoid potential drainage difficulties or deficiencies, and should be fully aware of the importance of good drainage practice. Before the construction stage, a drainage engineer should review the design as a whole, and detail the subsurface drainage required to ensure that any excess water which may accumulate in the road structure will be effectively removed. Attention should be given to every component of the system as well as to the subsurface drainage system as a whole.

Stage	Subsurface drainage requirements
(c) Construction	More comprehensive information on the site conditions must be obtained during construction, and must be used to confirm or alter the designs. Accurate detailed records must be kept of the as-built information, including positions of outlets³.
(d) Maintenance	The performance of the subsurface drainage installations should be monitored and maintained during the life of the pavement. The designs should include features that will facilitate this.

2. INVESTIGATION

2.1 GENERAL

An effective subsurface drainage system cannot be designed without a well planned, detailed investigation which should consist of a desk study as well as a site investigation. Every phase of the investigation should be carried out by competent persons and specialists should be consulted where applicable. It is important that, throughout the investigation, due allowance be made for seasonal variations in the moisture regime.

2.2 DESK STUDY

2.2.1 Climate

Rainfall is dispersed either as surface run-off, or as infiltration.

Since the well-established principles of surface drainage only cater for between 10 and 70 percent of the rainfall as run-off in unpaved areas, depending on the surface topography and vegetation, the balance of between 30 and 90 percent must enter the ground as infiltration.

A large proportion of this infiltration is subsequently removed from the ground by the natural processes of evaporation and plant transpiration, and, in broad terms, surplus ground water can only accumulate where the rate of infiltration exceeds the rate of evapotranspiration. However, ground water moves through the pavement structure in various ways en-route to the permanent water table, as illustrated in *Figure 2* which also shows other sources of water such as suction and vapour movements. The water table can also move upwards and local impermeable strata can result in perched water tables. The first requirement of a successful design for subsurface drainage is the accurate identification of the source of water that needs to be drained.

Although ground water problems can and do occur in any climatic region, they are more likely to occur where infiltration exceeds

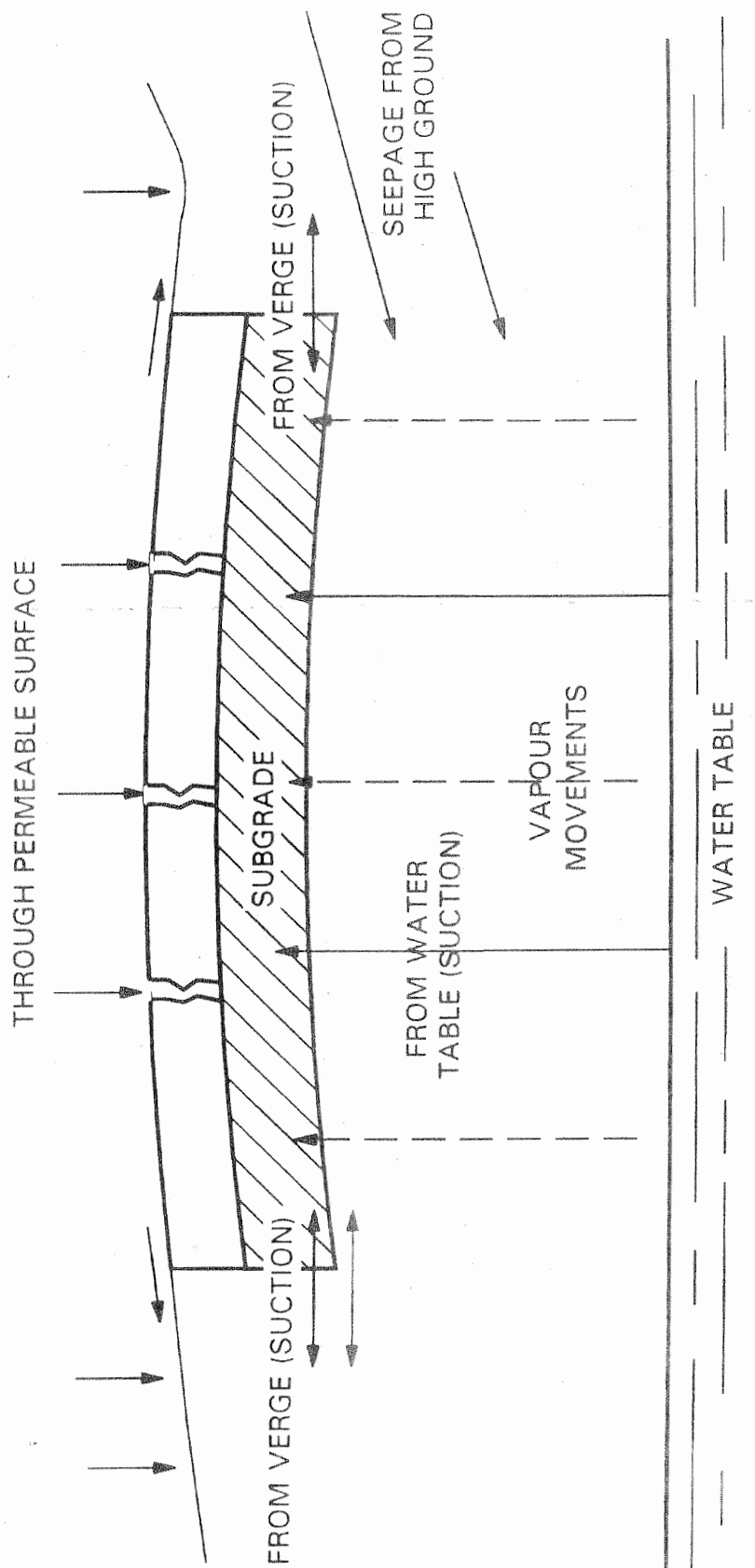


FIGURE 2
Sources of water in the pavement structure

evapotranspiration, as indicated on the map in *Figure 3*, which defines the wet areas of southern Africa according to Thornthwaite's method.

Obviously the map is a generalised analysis of long-term meteorological records, and the relevant weather patterns are subject to fairly wide fluctuations from year to year, and even from place to place inside a moisture region. Experience has shown that it only requires one wet season in a low rainfall area to cause serious subsurface drainage problems.

2.2.2 Geology

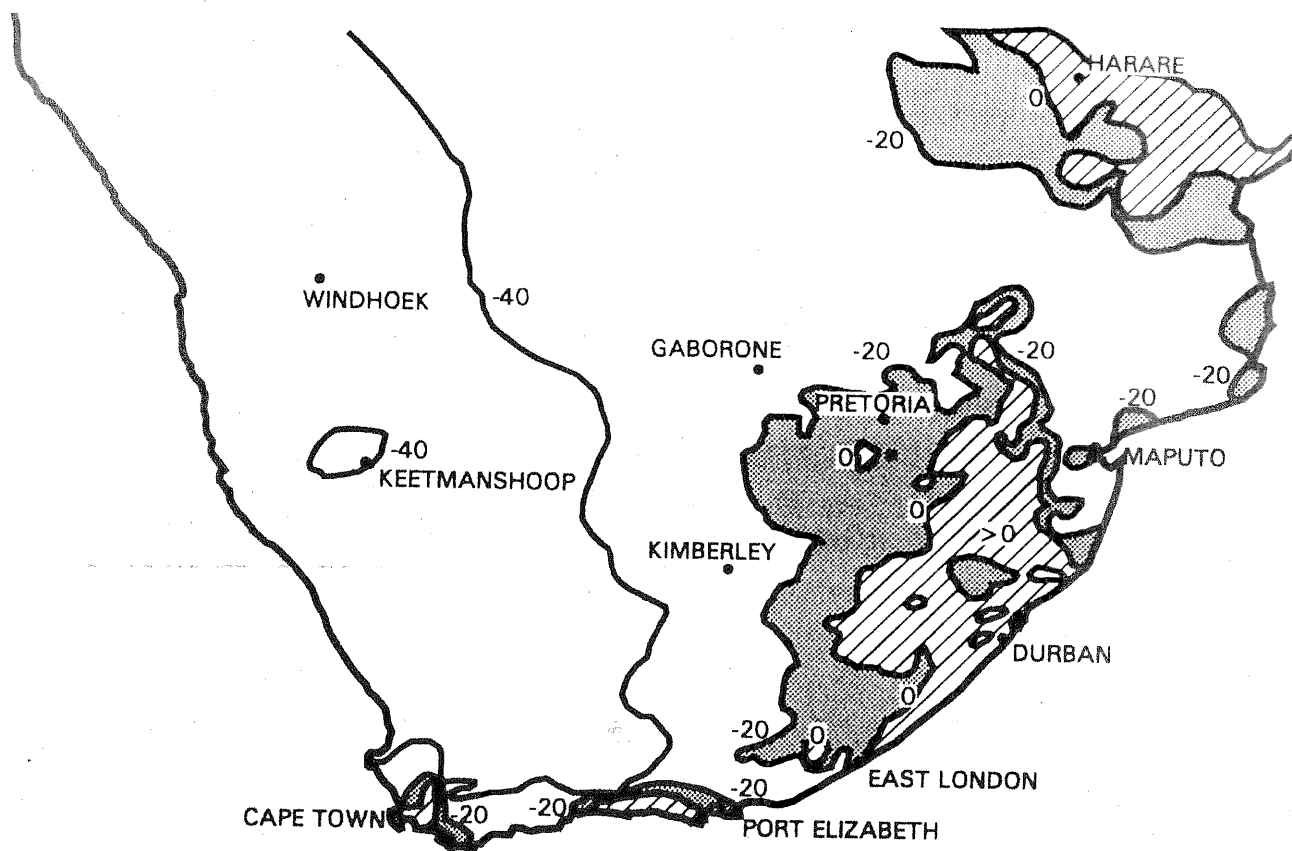
Water which enters the ground will move under the influence of gravitational, capillary and molecular forces until it is distributed, at any given time, either below the water table as ground water, or at a higher level, as pore water, adsorbed water or water vapour.

The position of the water table and its associated moisture phases depends not only on the topography of the ground surface and of the impermeable layers below the surface, but also on the infinite variations in the permeability of the soil mantle and the bedrock. Since a particular pattern of variations is characteristic of certain geological deposits and structures, the geology gives an important clue to the whereabouts of the water table and to the possibility of a perched water table, which is even more important.

The detail analysis of the relationship between the geology and the ground water should be made by an engineering geologist, who will recognise :

(a) **Structural features** that will affect the flow of ground water, such as :

- fractured, fissured or jointed rock on the one hand, and solid masses on the other;
- impermeable dykes cutting across permeable layers;
- alternating layers of sedimentary rocks, such as mudstones, shales and sandstones, with different permeabilities; and



<i>Legend</i>		
<i>Code</i>	<i>Im</i>	<i>Climatic Region</i>
	> 0	Wet (Water surplus)
	> -20	Moderate
	< -20	Dry

FIGURE 3

Moisture index (Im) according to Thornthwaite's classification

- the general dip of the stratification which defines the direction of ground water movement.
- (b) **Soil profiles** which indicate a perched water table near the ground surface, such as :
- a permeable transported sandy soil overlying an impermeable residual clay; or
 - pedogenic processes resulting in the formation of ferricretes or calcretes; and
 - the mottled discoloration arising from the solution of soil minerals in horizons which are more or less permanently saturated.

It must be emphasised that the road earthworks will disturb a ground water regime which, given seasonal fluctuations, has been in a state of equilibrium through geologic time, and this disturbance can have disastrous effects on the stability of the roadbed, and of the cut and fill slopes, unless the situation is clearly understood and effectively controlled.

2.2.3 Vegetation

While the general vegetation of the area is largely determined by climatic conditions, certain types or variations of the vegetation growing in the area may give a fairly reliable indication of ambient moisture.

For example :

- (a) hydrophilic vegetation such as reeds, rushes and sedges will define the extent of swampy conditions;
- (b) variations in the vigour and colour of the growth of surface vegetation and crops will indicate changes in soil moisture; and
- (c) in natural conditions, a line of trees will often indicate the trace of a fault line or intrusive dykes, and a clump may distinguish a pan or a prominence.

2.2.4 Topography

A study of the topography can provide valuable information on the possible movement of ground water. Topographic maps, aerial photographs and orthophotos on various scales are freely available for most of the subcontinent. Considering the fact that seepage of ground water always occurs from high to lower ground, the position of dams, rivers and marshes are important pieces of information.

2.2.5 Local knowledge and past experience

There is almost always a reservoir of knowledge about the drainage conditions and experience on previous road projects in the area, and this reservoir must be tapped by deliberate enquiries.

These enquiries should be addressed to both public and private sources of information, preferably in the form of a questionnaire itemising the specific points of concern and also inviting more general comments.

The fruitful sources are usually :

- (a) the district road authority, and its superintendents;
- (b) the consulting engineers associated with nearby projects;
- (c) the resident staff on recently completed projects;
- (d) the technical service departments of commercial suppliers of drainage materials;
- (e) local residents, e.g., farmers; and
- (f) existing facilities.

2.2.6 Remote sensing

Most of the site conditions can be deduced from a study of stereo photography⁴, and photo-interpretation should always be used to verify and to extend the information gathered by site investigation, notably with regard to fault lines and dykes in the near surface geology.

General drainage patterns and soil moisture variations can be interpreted from black and white photography, at scales preferably between 1 : 20 000 and 1 : 30 000, with detail sufficient for most road projects. Where the drainage problem is acute, the additional detail which can be interpreted from colour photography, or by infra-red line scanning, may warrant the additional expense.

The photo-interpretation must be completely integrated with the field investigation in a constructive sequence involving :

- (a) preliminary photo-interpretation;
- (b) field investigation for identification of photographic features;
- (c) detail photo-interpretation; and
- (d) detailed field investigation.

2.3 SITE INVESTIGATION

The whole purpose of field investigations for roadworks is to evaluate the nature and extent of the in-situ materials, and the evaluation of moisture contents and drainage conditions is an indispensable part of this.

2.3.1 Reconnaissance

The reconnaissance phase is of the utmost importance for obtaining an understanding of the drainage problem. Basic problems and features can be identified, previously gathered information can be evaluated and decisions can be made on the extent and detail of field testing that will be required. This phase may also indicate the need for adjustments to the road alignment to avoid obvious problems. General observations should be made on the characteristics of the site with respect to drainage, such as :

- (a) the presence and location of surface streams, springs, seepages and marshes;
- (b) any vegetation indicators, as mentioned in 2.2.3 above;
- (c) any geological exposures which will assist in the classification of the conditions in the country rock;

- (d) the evidence of patch patterns on existing roads, which should be correlated with the local topography and geology, and checked against the hypothesis that centre line patches can be attributed to a high water table, and edge patches to side infiltration or seepage;
- (e) the agricultural environment, insofar as it affects the natural drainage :
 - of surface water, by diversion or ponding on contour banks, or by storage in dams or reservoirs;
 - of subsurface water, by crop irrigation, or by desiccation resulting from long-term crops and afforestation with a high water demand; and
- (f) various soil types.

Drainage failures can frequently be traced to simple background causes which were not reported by the site investigator.

2.3.2 Field testing

2.3.2.1 Test pits

The soil profiles exposed in test pits should be examined to determine whether they exhibit any of the geological conditions mentioned in 2.2.2 (b) above; and

- (a) to determine whether they contain any standing water from a permanent or perched water table, or show any signs of seepage water;
- (b) to determine whether the soil structure is susceptible to seasonal seepage flows, for instance along fissures marked by staining, or through holes or in joints; and
- (c) to classify the excavations.

Once the need and locations of subsurface drainage systems have been identified, materials from relevant test pits must be subjected to tests to evaluate candidate materials for the drainage systems.

The number of test pits may have to be increased once the extent of the ground water problem has been determined. In deep cuts, material must be obtained from the relevant depths for testing.

2.3.2.2 Boreholes

An examination of the rock cores will reveal the details of the rock formation that are required for an appreciation of the rock structure, as discussed in 2.2.2 (a) above, and the water level should be measured and monitored over a full year to get an idea of the range of fluctuation.

In situations where it may be necessary to lower the water table permanently, boreholes will be required to test the permeability of the surrounding materials by measuring the draw-down characteristics during pumping.

2.3.2.3 Stand pipes

Stand pipes should be installed to monitor the water level before, during and after construction. They can be installed in boreholes used for the investigation or in specially drilled holes. *Figure 4* shows typical details of stand pipes.

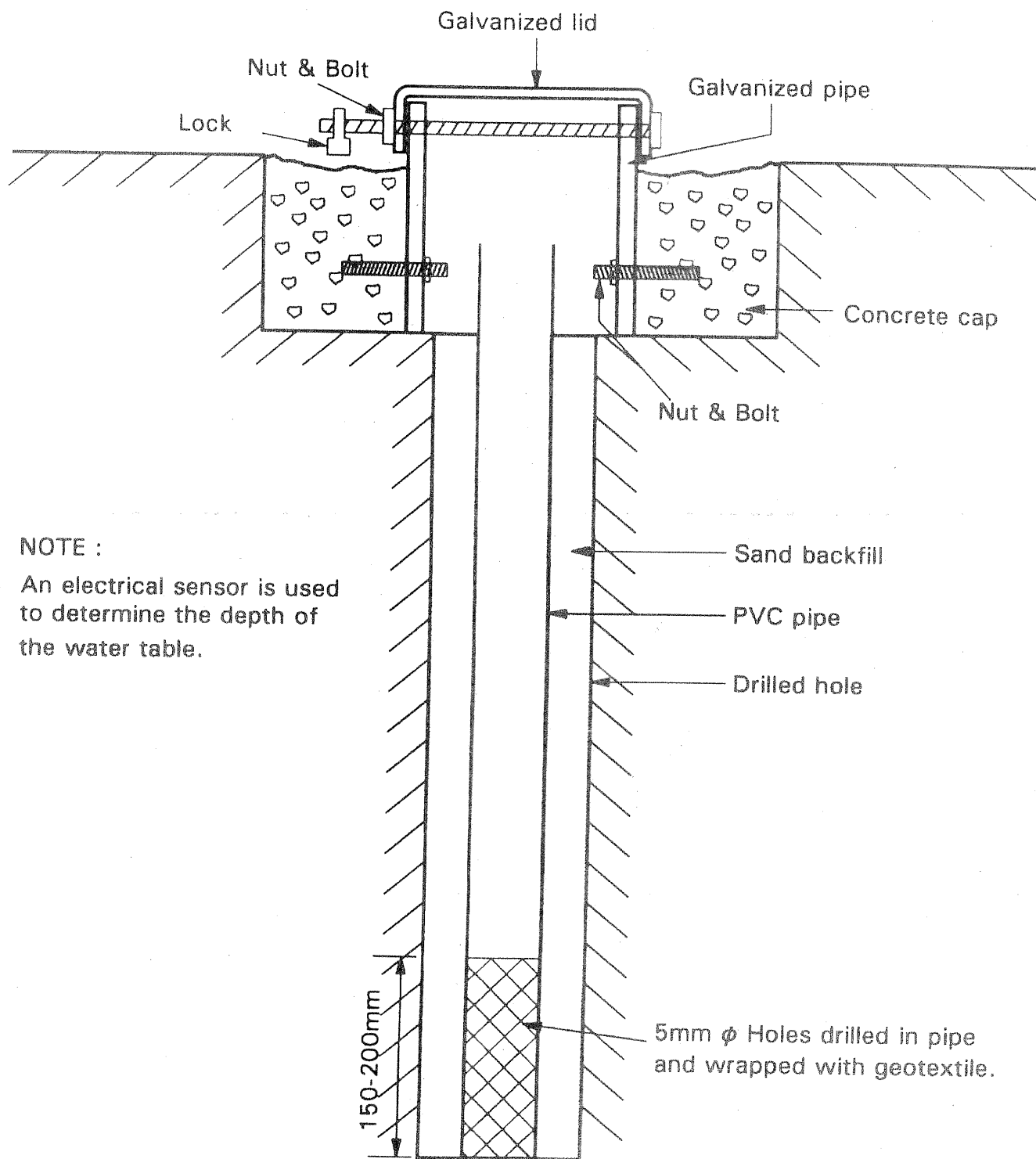


FIGURE 4
Typical standpipe detail

3. SUBSURFACE DRAINAGE CONCEPTS

3.1 PERMEABILITY

3.1.1 General

The permeability of a material is a measure of its ability to allow a fluid to flow through it. The movement of water into, through and out of the road structure is determined by the permeabilities of the various pavement layers, the surfacing, the underlying material (subgrade), the material adjacent to the road structure as well as the permeability of the filter materials. The calculated examples in Appendix A (Examples A1 to A3) show how permeabilities can be applied to calculate flow volumes.

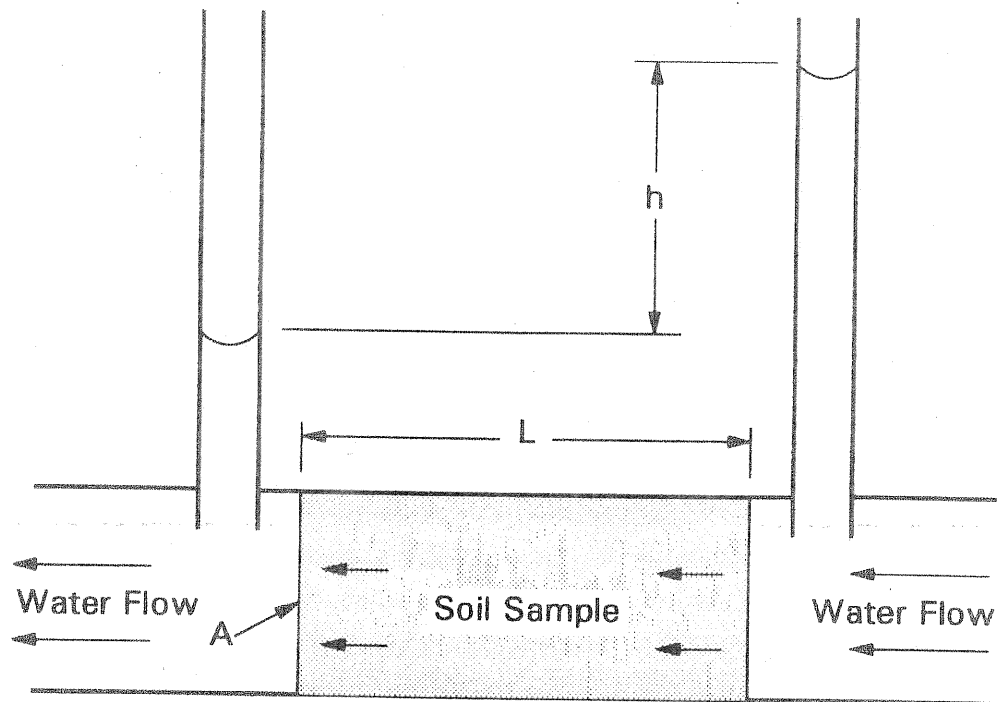
Permeability under saturated conditions is defined by Darcy's law, as illustrated in *Figure 5*, by the coefficient of permeability, k . Saturated conditions result in the highest flow rates. Under unsaturated conditions lower flow velocities and hence lower flow rates occur. Darcy's law does not apply under these conditions and complex finite element analyses must be used to calculate flows.

Permeability is very sensitive to small changes in materials. Therefore the natural variability of materials and particularly the occurrence of discontinuities such as cracks, result in large variations in permeabilities. The difference between the permeability of a small sample of soil that is determined in the laboratory and that of the large soil mass from which it was obtained, can be orders of magnitude if any discontinuities are present in the field situation.

3.1.2 Typical values of permeability

The permeability of any soil is determined by a number of factors, the most important being⁵ :

- particle size distribution - permeability decreases with decreasing grain size and is extremely sensitive to the quantity of fine soil;
- particle shape and texture - irregular shape and rough surface texture tends to decrease the permeability;



$$\text{Flow} = Q = kiA$$

where $i = h/L = \text{hydraulic gradient}$

$A = \text{Area}$

$k = \text{permeability coefficient}$

FIGURE 5

Darcy's Law: Saturated coefficient of permeability

- mineralogical composition;
- voids ratio, expressed in terms of either the voids ratio e or the porosity n ;
- degree of saturation - in soils with low degree of saturation, the air occupies voids making it difficult for the water to flow. In those cases Darcy's Law is no longer valid. Permeability increases when the degree of saturation increases;
- particle arrangement (structure or soil fabric) - permeability is affected by sorting or stratification and by the orientation of particles;
- dispersion of fines - some materials, such as montmorillonite, occurring through the soil mass in small amounts, can result in variations in permeability by orders of magnitude;
- discontinuities - the presence of cracks or any preferential flow path will increase the permeability of the soil considerably;
- density - the denser the soil the smaller the voids, therefore the lower the permeability; and
- nature of fluid - the coefficient k differs for fluids other than water and for water at different temperatures and different treatments. As the temperature of water has a significant influence on its viscosity and therefore on the permeability of the soil, the laboratory tests are standardised by performing the tests at 20 °C.

Typical values of permeability for different types of material can be estimated using results of earlier investigations. However, it is extremely important to remember that small changes in some factors, e.g., density and grading, can cause a considerable change to permeability and that "typical values" must only be used as a first indication or as a check.

Figure 6 shows typical ranges of permeabilities of various soil types and road building materials^{6,7}.

Typical permeability values of single sized stone and sand that are used as drainage materials in subsurface drains, are as follows :

COEFFICIENT OF PERMEABILITY (m/sec)														
10⁰ 10⁻¹ 10⁻² 10⁻³ 10⁻⁴ 10⁻⁵ 10⁻⁶ 10⁻⁷ 10⁻⁸ 10⁻⁹ 10⁻¹⁰ 10⁻¹¹ 10⁻¹²														
DRAINAGE RATING	VERY RAPID				RAPID	MODE-RATE	SLOW	VERY SLOW						
SOIL TYPES	Clean gravel				Clean sand, gravel & clean mixtures	Very fine sands, silts, sand, silt & clay mixtures. Stratified clay deposits.	Homogenous clays							
					Impervious soils modified by effects of weather, vegetation, fissures and cracks.									
ROADBUILDING MATERIALS					Graded filter materials	Bituminous seals and asphalt courses								
									Gravel and shoulder materials		Densely graded granular pavement materials			
DETERMINATION OF COEFFICIENT OF PERMEABILITY	DIRECT	Field testing reliable, if properly conducted. Considerable experience required.												
		Constant head permeameter Little experience required												
	INDIRECT	Turbulent flow for hydraulic gradients larger than 10				Reliable, little experience required				Unreliable, considerable experience required				Fairly reliable, considerable experience required
		Computations from grain sizes. Applicable only to clean, cohesionless sands and gravels								Horizontal capillarity test. Little experience needed. Useful for rapid field testing.				Computations from consolidation tests. Considerable experience and expensive laboratory equipment required.

FIGURE 6

Typical values of permeability and methods of determination^{6,7}

Material	Approximate Permeability m/sec
Single sized stone :	
19,0 mm	1,0
13,2 mm	0,2
9,5 mm	0,05
6,7 mm	0,01
Coarse Sand :	0,005 - 0,001

A number of formulae have been established to calculate permeabilities from grain size distributions.

The best known of these methods is the Moulton formula^{8,9} :

$$k = \frac{C(D_{10})^{1,478}(n)^{6,654}}{(P_{0,075})^{0,597}}$$

where k = permeability coefficient in m/sec

C = constant = 1,94 for units given here

D_{10} = diameter for the 10% passing size, i.e. the size of the sieve that allows 10% of the material to pass through, in mm

n = porosity

$P_{0,075}$ = Percentage passing the 0,075 mm sieve

Porosity can be calculated as follows :

$$n = 1 - \frac{\tau_d}{G}$$

where τ_d = dry density

G = Specific gravity

The use of this method is illustrated in example A4 in Appendix A.

3.1.3 Determination of permeability

Test methods that are available for the determination of permeabilities are well described in various soil mechanics text books^{10,11}. *Figure 6* is a useful guide to the test procedures which are appropriate to typical values of permeability.

3.1.3.1 Direct

Direct methods are those where actual time, flow and hydraulic gradient measurements are made. These include laboratory tests (constant head permeameter and falling head permeameter) as well as field tests. In general, constant head permeameter tests are more reliable for the more permeable materials and falling head permeameter for the less permeable materials. It will be noted that the falling head permeameter test on materials on the borderline between slow and rapid drainage (10^{-5} m/s) requires considerable experience in execution and interpretation, the test results being very sensitive to entrained air, and to any filter skin of fines on the sample.

A simplified borehole test method¹² that can be used to determine the permeabilities of a uniform soil mass in the field is given in Appendix B (Method B1).

It should be noted that anisotropic permeability is likely in sedimentary deposits with high permeability in the horizontal direction and low permeability in the vertical direction.

3.1.3.2 Indirect

The most useful of the indirect methods is the calculation based on grain size distribution as described in Section 3.1.2 above.

3.1.4 Drainability

Drainability is the ability of a soil to give up its pore water under gravity drainage. Where gravity drainage can remove a perceptible amount of pore water from the soil in a relatively short time, the soil is described as "drainable".

In coarse-grained soils with large particle sizes, pore spaces and permeability, gravity drainage is effective, but in fine-grained soils with minute pore spaces :

- the movement of water under gravitational forces is very, very slow; and
- a large proportion of the water will be retained in the soil by capillary forces, and as adsorbed films, in spite of free draining conditions¹³.

As shown in *Figure 7*, the drainability of silty or clayey soils is really quite limited, and in practice subsurface drainage installations in soils of this type, with more than 35 percent passing 0,075 mm, can only be justified on the basis of the permeability of the soil structure as a whole, and not of the material *per se*.

3.1.5 Effective permeability

Effective permeability refers to the field permeability, i.e., the permeability of the larger soil masses, taking into account the various discontinuities.

Some of the more common features which affect the effective permeability of an arrangement of materials are as follows :

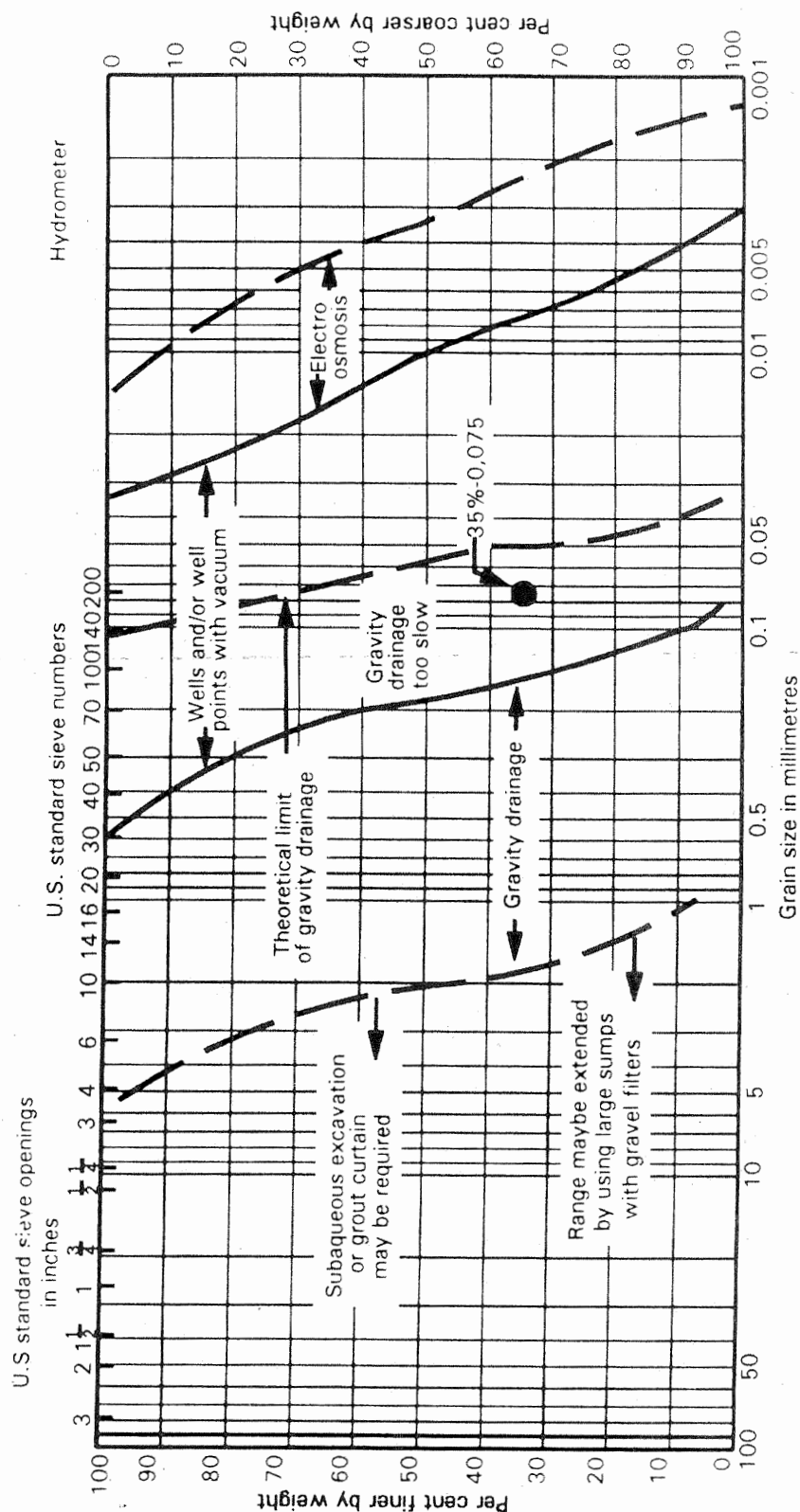


FIGURE 7

Drainability of soils¹³

	GRAVEL			SAND			SILT or CLAY	
	Coarse		Fine	Coarse		Fine	FINES	
	GRAVEL			SAND			percent by mass < 0,075mm	
	0	2,5	5	10	0	2,5	5	10
Percentage of total water in voids drainable by gravity	80 (1)	70	70	40	65 (2)	57	50	25
		60	60	30		50	35	18
		40	20	10		35	15	8

1) 75% > 4,75mm
2) well graded
3) on Unified Classification System

Residual soils

- crack patterns due to shrinkage or movement;
- original rock joints or fractures which remain in the weathered soils; and
- holes due to piping erosion, root decay or burrowing animals such as moles, crabs and worms.

Transported soils

- layers or partings of fines in an otherwise coarse sediment; and
- accumulation of coarse particles in the pebble marker.

Rock

- joints and fractures in otherwise impermeable rock masses;
- fault line breccia;
- differences in permeability between successive rock strata usually marked by seepage on the bedding planes; and
- flow lines blocked by intrusive dykes or sills, often impervious in the weathered state.

Pavements

- cracks due to deformation, shrinkage or fatigue;
- joints, where the joint seal deteriorates with time;
- segregation of coarse and fine material into separate streaks or layers during construction, e.g. layer surface virtually sealed by slushing during construction;
- interlayer surfaces with segregation at the bond; and
- granular materials choked by the migration of fines under gravity flow, or by pumping.

3.2 CAPILLARITY

Permeability is usually associated with the flow of water under a positive head, but in fine-grained materials the negative (suction) head caused by capillary forces in the soil matrix will frequently exceed the gravitational head, and the water will rise faster than it will fall.

This is the reason for the well-known fringe of capillarity water above the water table, but the height of capillary rise is not generally appreciated. In clays this rise can be as much as 4 to 5 m above the water table.

In areas with a naturally high water table, or a potentially high water table due to ponding, the empirical rules of road building (e.g., form at least 1 m above the water table) are not an adequate basis for design, and the actual capillarity of the subgrade material should be taken into account for the assessment of probable moisture contents.

3.3 CONTINUITY

Unless there is a steady increase in the capacity of each of the elements along a drainage flow line, water will accumulate at the restrictive boundary.

To avoid this situation it is necessary to make a detailed analysis of the boundary conditions, considering not only the relative permeability of the adjacent elements, but also the available flow areas and gradients.

For example, where water infiltrates vertically into the surface area of a pavement layer, and drains laterally through the same layer on a small cross fall towards the edge area, accumulation in the layer is inevitable.

4. SUBSURFACE DRAINAGE DESIGN

4.1 GENERAL

It must be stressed that the design of an effective subsurface drainage system must be done at the design stage of a project and not be left until construction. Various investigations need to be carried out and tests need to be performed, all of which take up a considerable amount of time and careful planning. It is inevitable, however, that due to the seasonal and variable nature of ground water, drainage designs must be verified and possibly altered during construction.

While the design of storm water systems is aimed at removing surface water as rapidly as possible to minimise infiltration, subsurface drainage design covers both the prevention of water infiltration as well as the collection and removal of ground water to prevent damage to the road.

One of the most important stages in the design of any subsurface drainage system is the identification of the source of ground water that is to be drained away. If seepage is encountered or expected to occur at the top of the subgrade, for example, it would serve no purpose to perform a sophisticated design for an interceptor drain unless the water originates from adjacent higher ground. The main sources from which water can enter the pavement structure are shown in *Figure 2* and those for embankments and cut slopes are dealt with in this section.

The main elements that are required in a subsurface drainage system to effectively collect and remove the ground water without disturbing the surrounding soil are a suitable filter, a permeable collector and an open conveyer. Depending on the soil type and local conditions it is possible for one material (e.g., a graded crushed stone) to perform all three of these functions. However, two or three different materials are usually employed, e.g., a geotextile (filter), crushed stone (collector) and a pipe (conveyer).

It is more often than not an extremely difficult task to determine the inputs required for an accurate design. The designer should nevertheless obtain information from whatever sources that are

available, including local knowledge and experience, and make allowance for uncertainties by using adequate factors of safety. A certain amount of "under-provision" of subsurface drainage can also be tolerated on lightly trafficked roads, since it is the combination of water and traffic that usually results in premature pavement failures. The following can be used as a guideline for the quality of drainage that should be provided for different road categories¹⁴.

<i>Drainage quality</i>	<i>Water removed within</i>	<i>Recommended for Road Category</i>
Excellent	2 hours	A
Good	1 day	B
Reasonable	1 week	C
Poor	1 month	none
Very Poor	Water will not drain	none

The decision on whether or not to install subsurface drainage systems should be based on a risk and cost analysis. Experience in a specific area may show, for example, that only one out of every twenty cuttings fail if no subsurface drainage is installed. The risk of failure must then be weighed against the cost of installing subsurface drainage systems in all cuttings, even if no ground water is encountered.

The cost of providing subsurface drainage systems must also be weighed against the cost of using pavement materials that are less water sensitive, e.g., stabilised layers and bituminous bases.

4.2 OPTIMISATION OF ROAD GEOMETRY

4.2.1 Location

While roads are generally located to avoid the obvious drainage difficulties such as surface water in pans and swamps, the less obvious hazards are often overlooked.

Designers should therefore be aware of the subsurface drainage implications :

- (a) of areas where there is a high or perched water table close to the surface of the ground;
- (b) of seepage zones on sidelong slopes, especially where they are generated by faulting;
- (c) of man-made waterworks such as canals, reservoirs and outfall works, which often leak, and of irrigated farming lands; and
- (d) of road locations on topography which is so flat that it is impossible to achieve the desirable fall for drainage.

Such situations cannot always be avoided, because of other constraints, but at the least they should be recognised as major factors in the selection of the road alignment.

4.2.2 Grading

The fundamental objective of good grade line design is to raise the grade line sufficiently above the ground so as to isolate the formation from the influence of ground water.

This objective is not always compatible with geometric standards, but at least it should govern until overruled by desirable minimum standards.

Beyond that, since all drainage problems rely on gradients for their solution, the grade line should aim to provide the maximum falls consistent with good profile design, and at least to avoid the common pitfalls shown in *Figure 8*, which are :

- (a) grades of less than 0,75 per cent over more than a 300 m length of cutting;
- (b) grades of less than 0,50 per cent of any fill where localised settlement might flatten the surface to a negligible grade of less than 0,25 per cent;
- (c) flat spots on superelevation developments, or on grade changes prolonged by the use of lavish vertical curves in the sag, or on the crest;

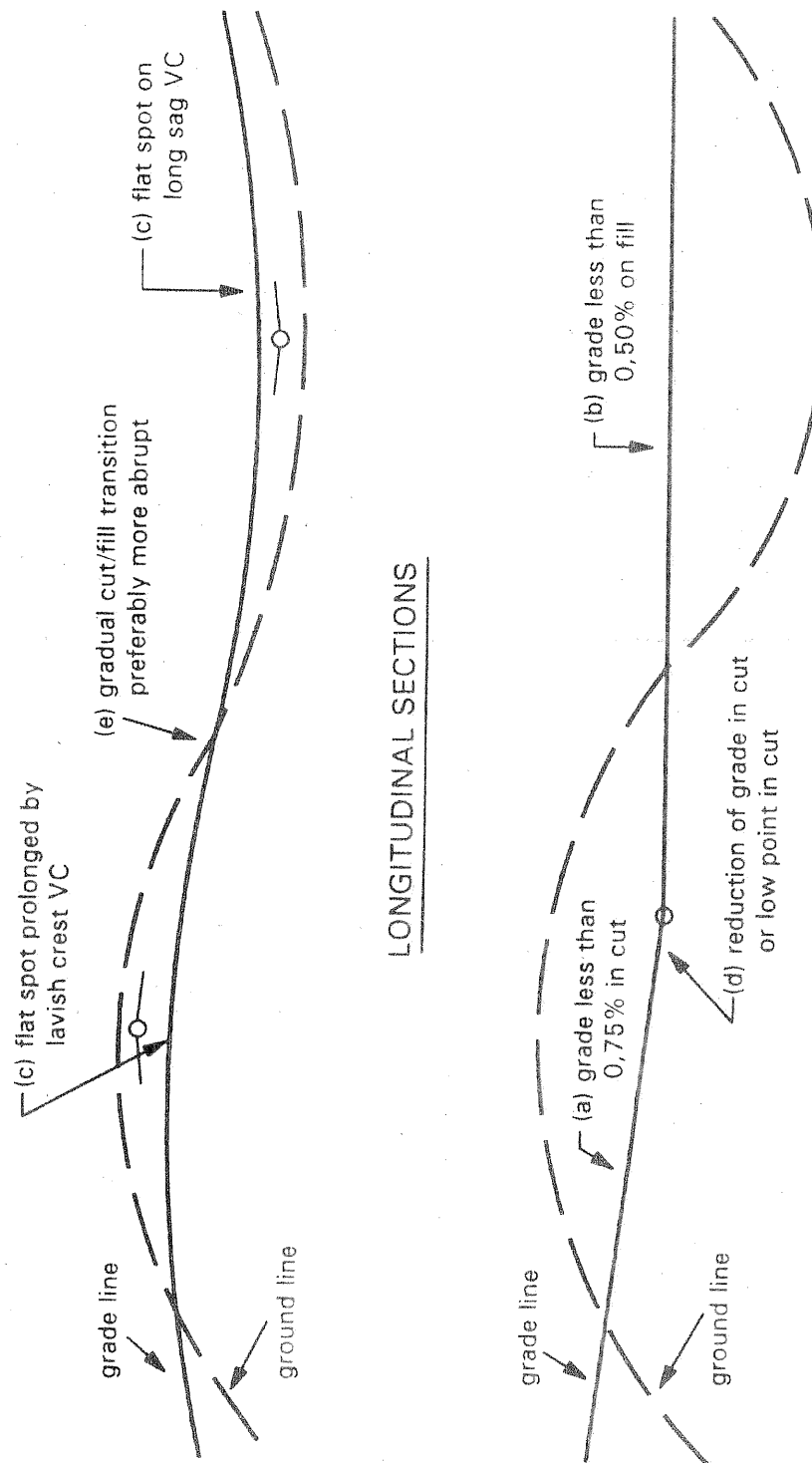


FIGURE 8
Grading drainage pitfalls

- (d) changes from steeper to flatter grades looking downhill in cuts, where the reduction of the hydraulic gradient results in the storage of seepage water at the grade change;
- (e) gradual transitions from cut to fill, where a more abrupt change would shorten the vulnerable length at the prick of cut; and
- (f) the cost of providing subsurface drainage for a shallow cut may warrant eliminating the cut by lifting the grade line.

4.2.3 Cross-section

4.2.3.1 Cross falls

Although the parabolic camber has been abandoned in the interests of mechanisation, the objective which it was designed to achieve - namely to shed the maximum amount of surface water in the shortest possible time - remains the ideal in cross-section design.

The linear cross fall for paved roads is generally 2,0 per cent but can be increased to a maximum of 3,0 per cent for multi lane roads or to cater for surface irregularities.

On gravel roads cross falls should vary between 3,0 and 4,0 per cent.

The exaggeration of the cross fall towards the edges of the pavement, and over the shoulders, will improve the surface drainage efficiency.

On multilane roads, particularly where they are not cambered, the length of the drainage path on the cross fall tends to become excessive, and many authorities consider that the interception of the flow by slot type drains is warranted by considerations of drainage and safety.

4.2.3.2 Interfaces between layers

Water which enters the pavement will tend to flow on the boundaries between layers of different permeability, and the outflow will be encouraged if the slope on these interlayer surfaces is greater than the slope on the finished surface.

Below the pavement, for instance in the top of subgrade or treatment in place, it is not difficult to exaggerate slopes, but in the pavement itself it is necessary to accept the rationale of a tapered design in order to realise the resulting drainage advantages.

Any tendency for this seepage water to accumulate under the outer lane or wheel path should, of course, be counteracted by providing for its prompt removal in collector drains.

4.2.4 Roadside features

4.2.4.1 Shoulders

Permeable shoulders are a common point of entry for subsurface water, and, from a drainage point of view, a shoulder with a bituminous or concrete surfacing is much to be preferred.

Research¹⁵ has shown that the zone of significant seasonal moisture variation extends about 0,6 to 1,0 m into the pavement from the sealed edge, and, with unsurfaced shoulders, the outer wheel path lies on this zone.

If a surfaced shoulder cannot be implemented due to budget limitations, provision should at least be made :

- for all permeable shoulders on the high side of super elevations or cross falls to fall away from the pavement (subject to geometric constraints), or to be sealed; or
- for the shoulder material to be carefully selected and placed in two layers, the lower being permeable enough to allow for some drainage of the base, and the upper sufficiently impermeable to shed the surface water without undue infiltration; or
- if a single-layer shoulder is to be used, for the permeability of the shoulder to be greater than that of the base, failing which adequate precautions will have to be taken to ensure the drainage of the basecourse "bathtub"; or
- to increase the cross fall of unsurfaced shoulders on steep grades to avoid longitudinal erosion.

4.2.4.2 *Side and median drains*

Deep, wide side drains play a key role in subsurface drainage. Although the drain slopes are usually sufficient to encourage run-off, the grade on the drain invert is often inadequate to prevent ponding, particularly where maintenance is poor, and these ponds will cause trouble in the subgrade.

The surface drainage design should therefore allow:

- for all side and median drains to be graded so that the invert is sufficiently below the adjacent layers to avoid any soaking of the materials within the design depth;
- for all drains where there is a likelihood of ponding to be lined with a smooth concrete or asphalt surface;
- for the drain flow to be limited to its proper purpose by the diversion of surface water above cuts along berms, terraces or other channels wherever possible; and
- for the drain section to be protected against blockages caused by material eroding from the cut faces, or from unsurfaced shoulders.

4.2.4.3 *Raised medians*

Raised medians should only be used where they add more to road safety than they detract from good drainage, and then only provided they are designed to prevent infiltration, or provided there is effective subsurface drainage.

The rain-water which soaks into unpaved raised medians inevitably enters the adjacent pavement layers, quite apart from the subgrade, and the drainage designer is well advised to refuse to allow the median to be used as a flower bed, tree nursery or any other ornamental feature.

4.2.4.4 Kerbing and channelling

The kerbed channel is a very effective device for the removal of surface water, but precautions should be taken to ensure that subsurface water in the pavement layers does not accumulate against the sett or the kerb foundations.

This situation is aggravated by the crack which frequently occurs at the channel edge, but can be prevented by the use of no-fines concrete in the kerb founds, either throughout, or at intervals, with piped discharge into the storm water or subsurface drainage.

4.2.4.5 Pavement surround

Although a new bituminous surface might appear to be impermeable, the road structure must be designed on the probability that water will enter through the surface at some stage of the pavement's life.

If the pavement is contained between shoulders or sidewalks, and upon a subgrade such that it is surrounded by less permeable materials, the accumulation of water in the pavement layers is inevitable. This is the classic "bathtub", "trench", or "box" situation.

Although the "bathtub" can and should generally be avoided on rural roads, local design requirements and restraints make it a typical form of urban and township construction, and premature distress is likely to result unless effective drainage is provided, or the "non-susceptible" base materials are used.

Effective countermeasures are suggested in *Figure 9*.

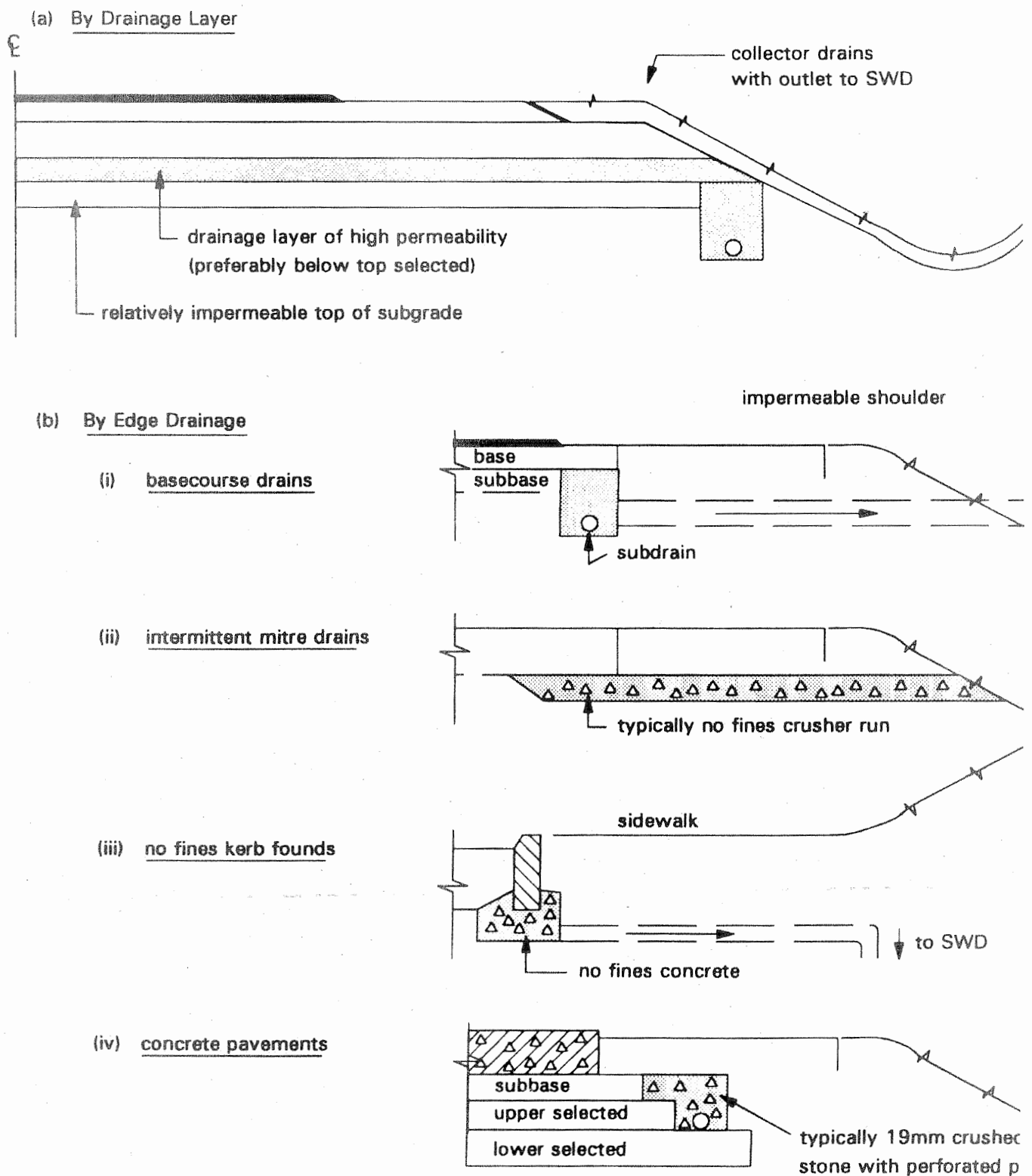


FIGURE 9
Pavement drainage

4.3 GROUND WATER CONTROL

4.3.1 Embankments

The general problems of embankment stability and settlement are considered in TRH9¹⁶ and TRH10¹⁷, and the following notes are confined to drainage considerations.

4.3.1.1 Settlement

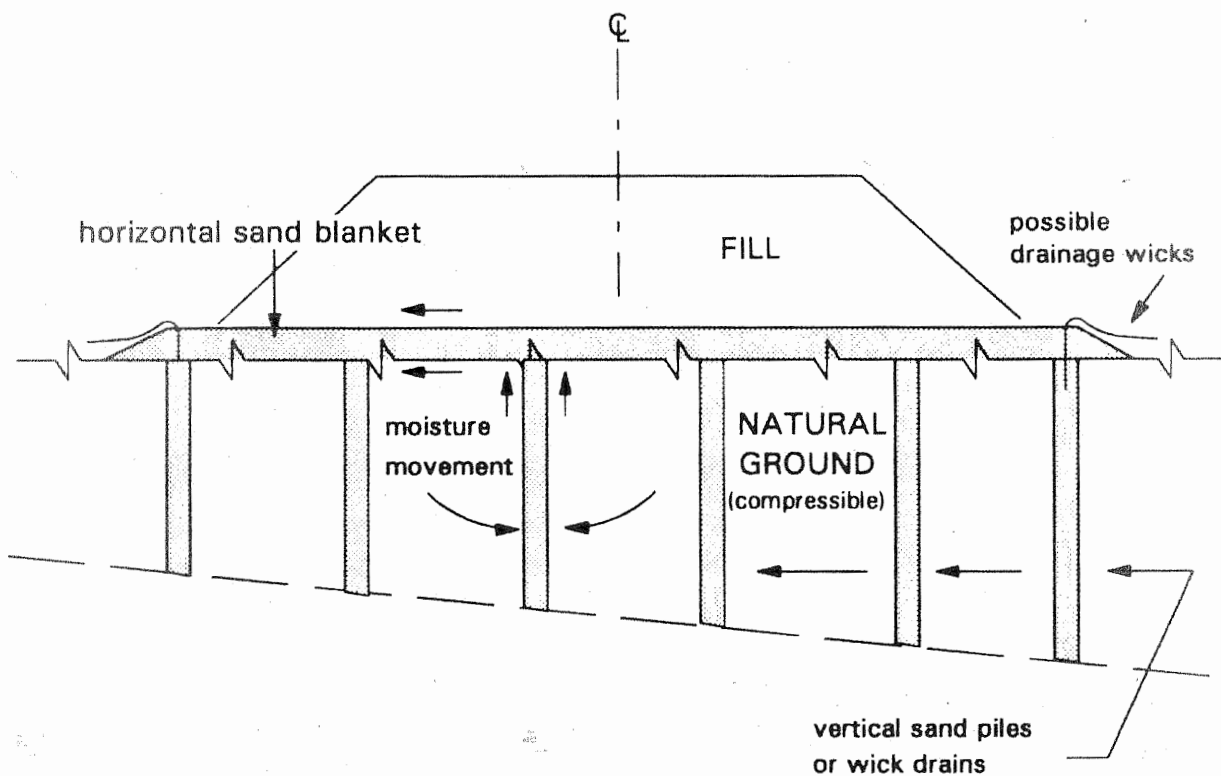
Since the rate of consolidation in saturated soils depends upon the rate at which the excess water is removed from the soil, settlement may be accelerated by the provision of subsurface drainage under fills (*Figure 10*).

The design objective is to provide artificial drainage paths through the soil mass, by means of vertical sand or gravel piles, or by the installation of a proprietary type of strip drain or wick drain.

Usually the permeability of the in-situ material is extremely low, unless it is stratified to include layers of coarser materials. As a result the drains have to be closely spaced if they are to provide effective drainage within a reasonable time, and the installation becomes expensive.

Where it is not possible to extend the construction period to allow for all of the primary consolidation to take place before surfacing the road, investigations into the subsurface drainage alternative should be assigned to specialist geotechnical engineers.

The cost of preventive measures should be carefully weighed against the damage, distress or inconvenience likely to arise from the settlement of the embankment, which may well be imperceptible, or at least acceptable, to the road user.



- Notes**
- (i) Spacing of vertical sand piles is diagrammatic - generally much closer than shown.
 - (ii) Costs of accelerated settlement should be weighed against costs of deferred surfacing.
 - (iii) Simple sand piles are largely outdated by proprietary strip drains of paper or geotextiles.

FIGURE 10

Drainage under fills to accelerate settlement

4.3.1.2 Stability

(i) Construction

Where the ground water lies close to the surface, the weight of construction equipment will often induce pore pressures sufficient to make the ground unstable. This situation can be avoided either by reducing or distributing the equipment loads, or by draining to a depth sufficient to provide support, along the lines shown in *Figure 11*.

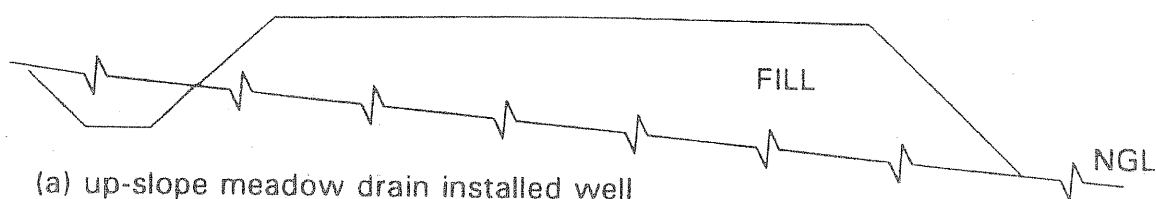
Wherever possible the construction should be programmed to take advantage of the seasonal lowering of the water table, and the fill should be pioneered across swampy ground using selected materials such as rock fill, and suitable equipment and techniques.

Temporary subsurface drainage will only be effective if the ground is sufficiently permeable to drain into rudimentary facilities within the time available, and this is not often the case. Before any reliance can be placed on this technique, trial pits should be excavated to assess, and if possible establish, the in-situ permeability as a basis for the design of the spacing and depth of drains.

Experience seems to indicate that if the ground is amenable to rapid drainage then fill settlement is not likely to be a problem, and temporary subsurface drainage makes little contribution to the performance of the permanent embankment.

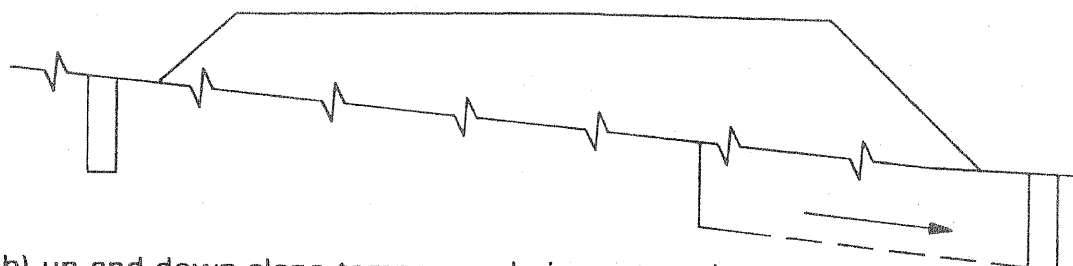
(ii) Seepage of ground water

On sidelong ground the stability of an embankment may be endangered by seepage water entering the fill from the natural ground, and intercepting subsurface drainage is then mandatory. It must be appreciated that the weight of the embankment will often be sufficient to consolidate the underlying ground, thereby reducing its permeability and causing the accumulation of ground water under considerable pore pressures. Water entering the fill will eventually seep to the toe, and dangerous slides may result from the combination of saturation and seepage forces.

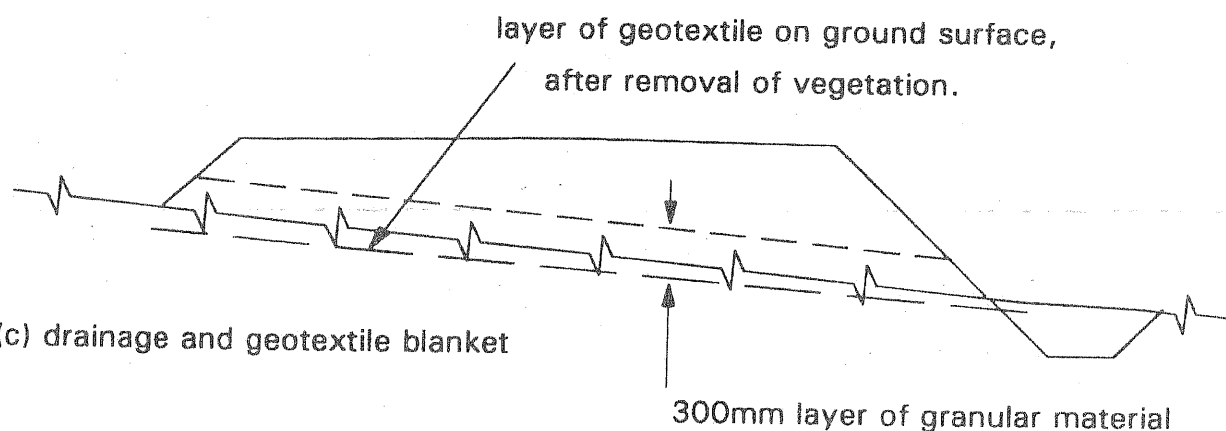


(a) up-slope meadow drain installed well ahead of construction

roll to seal fill area if possible



(b) up and down slope temporary drainage trenches, with extra laterals and herring bones where required



(c) drainage and geotextile blanket

300mm layer of granular material

(d) use pioneer techniques with rock fill

FIGURE 11

Construction drainage under fills

The consequences of failure are so serious that the designer should provide not only for the interception of the ground water by subsurface drainage, but also for the relief of any errant seepage pressures through a stable rock toe.

Three types of subsurface drainage which have been used in this situation are shown on *Figure 12* and the choice depends entirely on the designer's appreciation of the potential area and extent of ground water seepage.

The drains themselves must be designed to combine the basic elements of drainage layers, filter materials and pipes to provide an increase in capacity at each successive drainage stage.

(iii) *Seepage of surface water*

Water can also enter the embankment through surface cracks in the pavement and shoulders, and from leaks in any pipework placed in the fill. Although these possibilities should obviously be excluded by design, many large fills have the propensity to shrink after construction, due to natural drying out, and consequently longitudinal cracks along the shoulders and the fill slopes will occur. These cracked zones must be suitably sealed.

4.3.2 In situ subgrade

The most common application of subsurface drainage is to control the ground water in the road subgrade, either by intercepting the inflows or by encouraging the outflows.

Notes

(a) Benching should :-

- (i) be wide enough for construction (approximately 4,0 m)
- (ii) slope towards collector drains
- (iii) have longitudinal slope $\geq 1,0\%$

(b) Drainage must be completely reliable

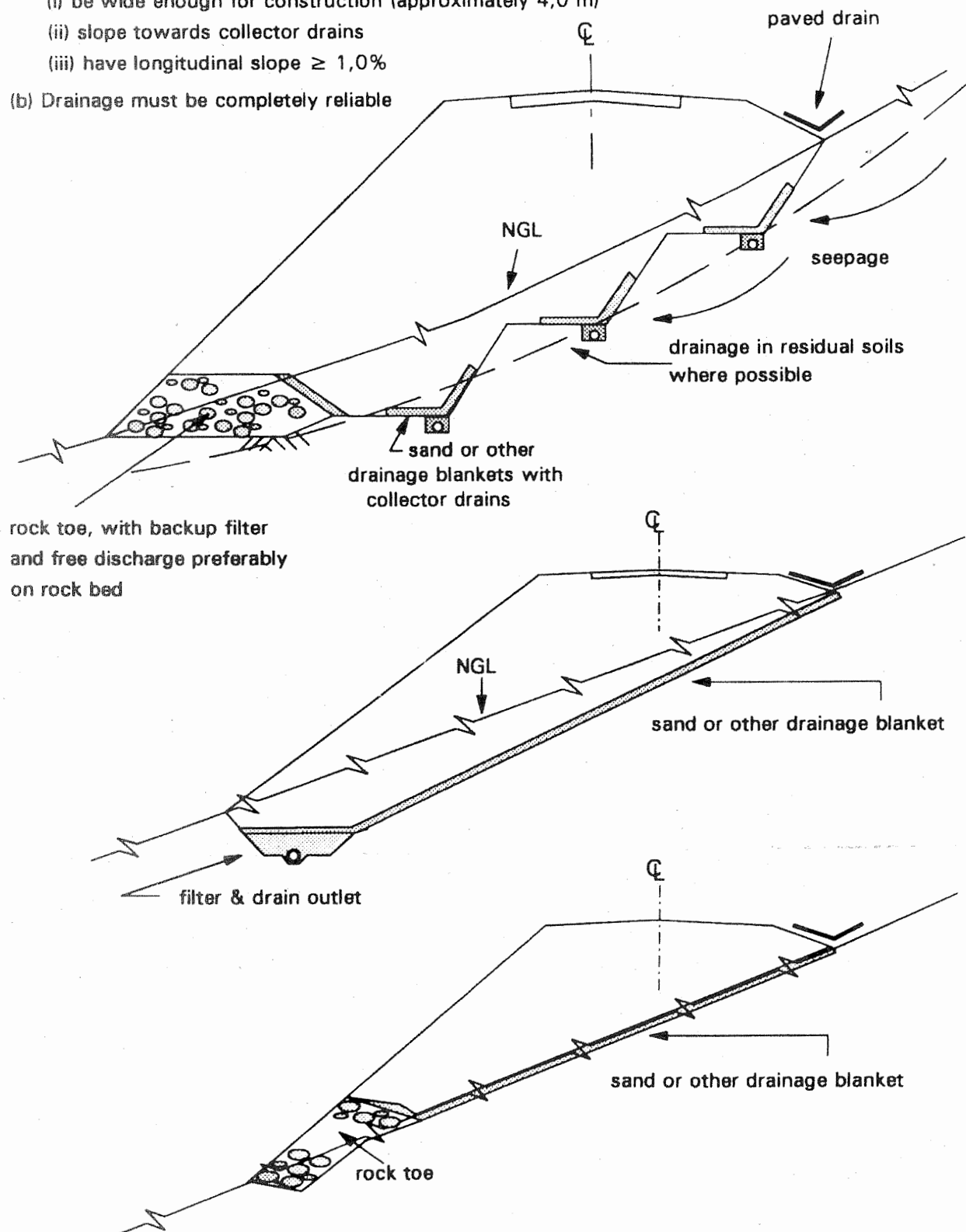


FIGURE 12
Drainage under fills to improve stability

4.3.2.1 *Interceptor drains*

Part of the subgrade in a side cut may lie in a seepage zone supplied from the uphill slope, in which case a subsurface drainage trench, such as the one shown in *Figure 13*, will serve to cut off the supply.

It should be emphasised that the idealised case shown on the drawing will very rarely occur in practice, because it assumes a fairly homogeneous drainable material below the water table. In practice the water usually arises from seepage above the water table in localised seepage zones; and in heterogeneous soil or rock formations.

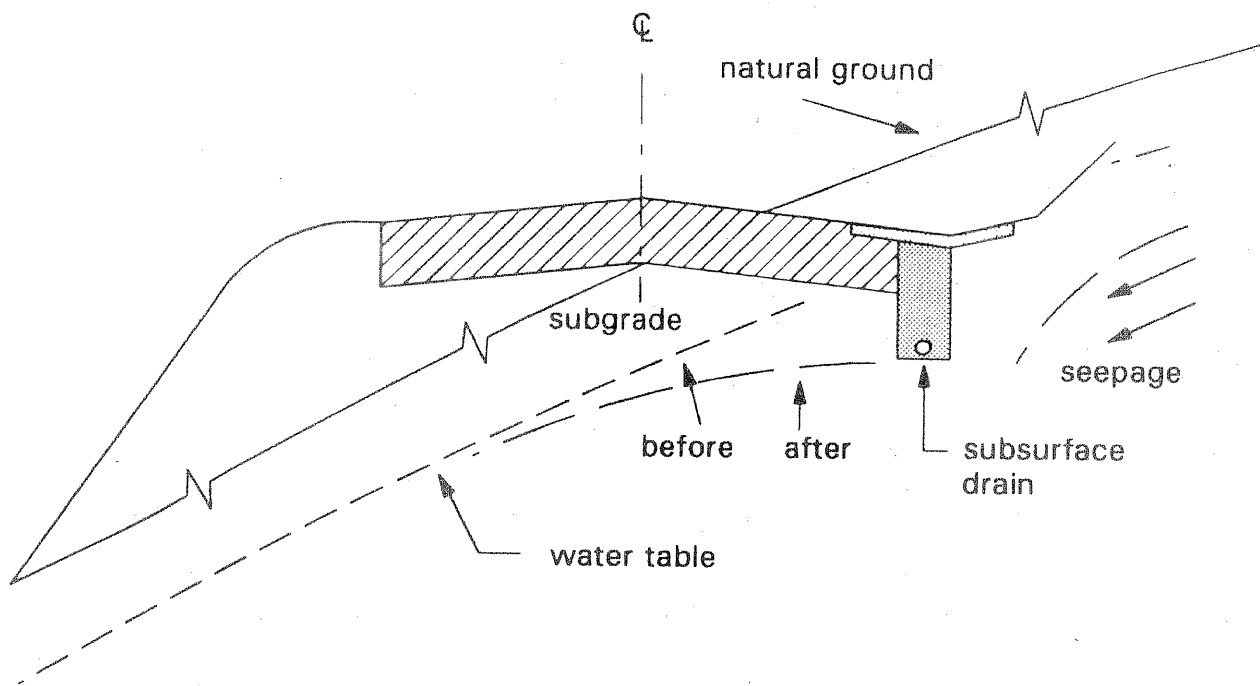
The interception of the flow, and the design of the filter material, is therefore best verified in the field on the basis of an inspection of the sides of an open trench.

Example A1 in Appendix A shows the method of calculating flows in the idealised case of a homogenous, saturated inclined aquifer. Some comments are given at the end of the example on methods of handling non-standard situations.

4.3.2.2 *Ground-water lowering*

Where the level of the subgrade has perforce to be located below the water table, subsurface drainage must be installed to drop the water table to below material depth.

In extreme circumstances gravity subsurface drainage may not be effective, and some major roads have to rely on continuous pumping from well points for their stability¹⁸. In the more usual situation the roadbed can be dewatered by a subsurface drainage arrangement of drainage pipes. The effectiveness of the arrangement depends entirely on the shape of the draw down curve originating at the drain pipes, and the characteristics of these curves are related to the permeability and geometry of the system. Empirical methods have been developed (e.g. McLelland¹⁹, Hooghoudt²⁰, van Schilfgaarde²¹) which can be used for a preliminary assessment of the problem, by making reasonable assumptions as to the permeability and porosity of



- Notes :-
- (a) locate interceptor as close to pavement as possible, without affecting structural layers.
 - (b) determine nature of seepage flow, whether through pore spaces, cracks, fissures or joints, as input data for filter material requirements.

FIGURE 13
Subgrade drainage by interceptor drain

the soil. However, final designs, even in homogeneous soils, require extensive testing to establish the actual soil properties, and complicated design procedures using electrical analogy flow nets⁶. Examples A2 and A3 in Appendix A illustrate the use of the McLelland and Hooghoudt methods.

4.3.2.3 *Top of subgrade*

Springs and seepages frequently occur on the floor of the excavation in cuttings, and these have to be drained in order to prevent local or general saturation of the top of subgrade.

Attempts should first be made to track down the source of the flow, because it is not unusual to find that the seepage comes from artificial sources, such as ponding above the cut slope or in adjacent lands which are easy to deal with.

Where the seepage is from an unstoppable natural source, the recommended treatment depends on the seepage pattern.

(i) *Localised springs*

If the water flows from identifiable areas in an otherwise impermeable subgrade, each flow should be separately captured in a sump of filter material, and led away for disposal in the storm water drainage system.

The possibility of new springs emerging after construction should be carefully considered in the light of the seasonal conditions and the long range weather cycle, and of the acceptability of installing additional drainage through the completed pavement.

(ii) *General seepage*

If the seepage is widespread, more extensive subsurface drainage will be required, but it should first be established that the wet surface areas are in fact evidence of widespread seepage through the subgrade, and not just the extended surface appearance of localised springs.

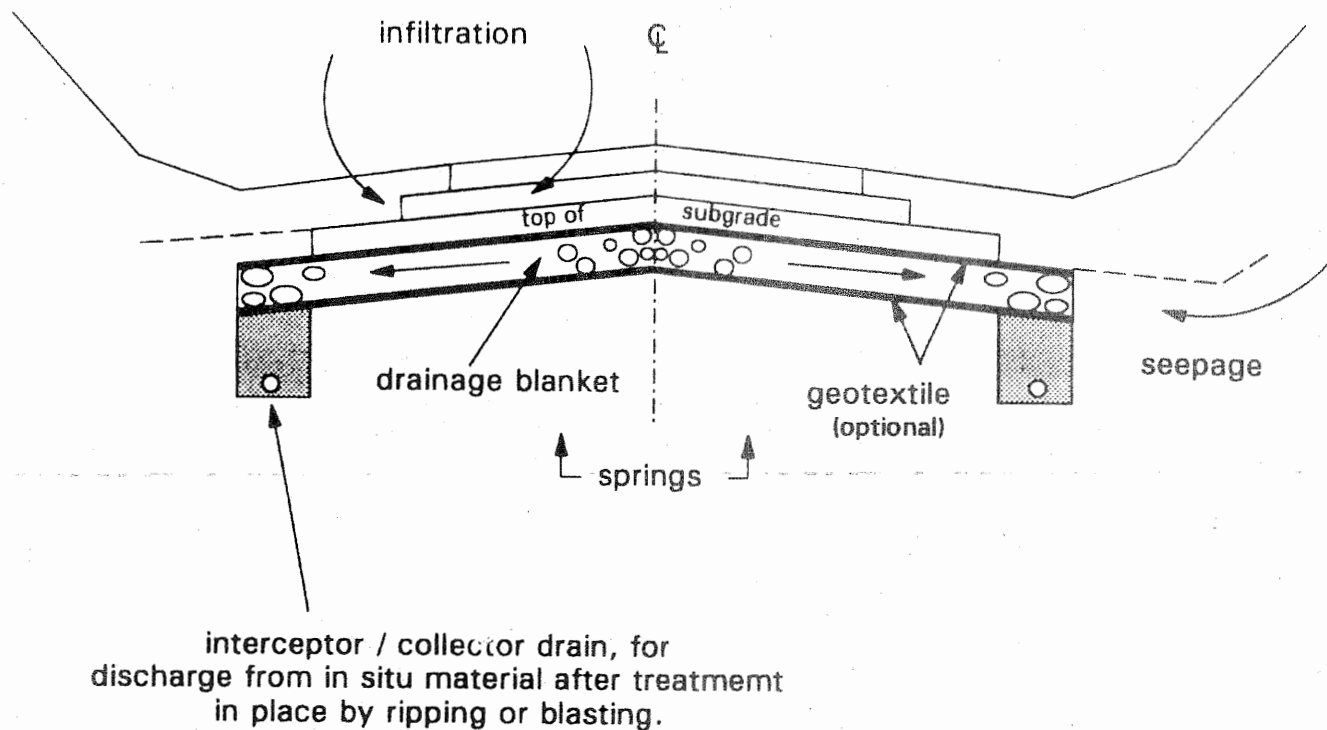
"Herringbone" drains are a popular remedy for general seepage, but, unless the laterals happen to tap the bulk of the localised seepage zones, they do not really solve the problem, although they certainly confine the distress in compartments between drains. They are more effective, provided that they are correctly spaced in a permeable material, in controlling the level of a general water table.

Drainage blankets are recommended for the interception of general seepage under the top of subgrade, and these blankets can be provided either by the in-situ materials, or with imported materials.

In excavation through hard materials or rock, erratic seepage into the imported layers can be prevented by "treatment in place". The purpose of this treatment is to fragment the in-situ subgrade to a depth sufficient to ensure that the seepage flows will be dissipated and drained well below the road formation. In practice it is impossible to provide completely free drainage at the bottom of the treated depth, and, since ponding will inevitably occur, this treatment should only be used in materials which are not susceptible to deterioration under continuous saturation. In addition, the depth of ponding should be limited as far as possible by careful control of the depth of ripping or blasting, and by exaggerating the cross fall on the underside of the treated layer.

In soils and weathered materials the seepage is generally intercepted at lower selected level and discharged by a blanket layer of imported permeable granular material. This layer must be designed to cope with the estimated seepage flows, during the rainy season, and to conform with the usual filter criteria, although some latitude may be allowed in view of the practical boundary conditions. Geotextiles can be incorporated in the design as filters and/or separators to be used below and/or on top of the drainage layer. The indiscriminate use of sand blankets for this purpose should not be encouraged, since many "sands" have a very low permeability.

Typical details of subgrade drainage blankets are shown on *Figure 14*.



Note: exaggerated crossfall is desirable.

FIGURE 14
Subgrade drainage blankets

4.3.2.4 Cut-fill transition

The area of the subgrade in the transition zone from cut to fill is inherently troublesome because it rests on the transported soil horizons, which are typically more susceptible to ground water. Ground water may be seasonally recharged by the lateral movement of water from adjacent ground slopes.

The soil profiles which create this problem can sometimes be identified by an inspection of the problem areas on existing roads in the vicinity, but, wherever the problem can reasonably be expected to arise, the subgrade should be drained and reinforced over the full length of the suspect area, as shown in *Figure 15*.

The key to the solution lies in creating a "fill in cut" condition which not only improves the side drainage, but also gives a smoother aspect to the whole transition²².

4.3.3 Cut slopes²²

4.3.3.1 Stability

While subgrade failures due to excessive ground water are relatively slow to develop, cut slope failures are often sudden and unexpected, and expensive to repair.

The ground water affects the stability of the slope mainly :

- by reducing the shear strength on the potential failure surface;
- by increasing the mass of the potential slide volume; and
- by generating seepage pressures along the flow line.

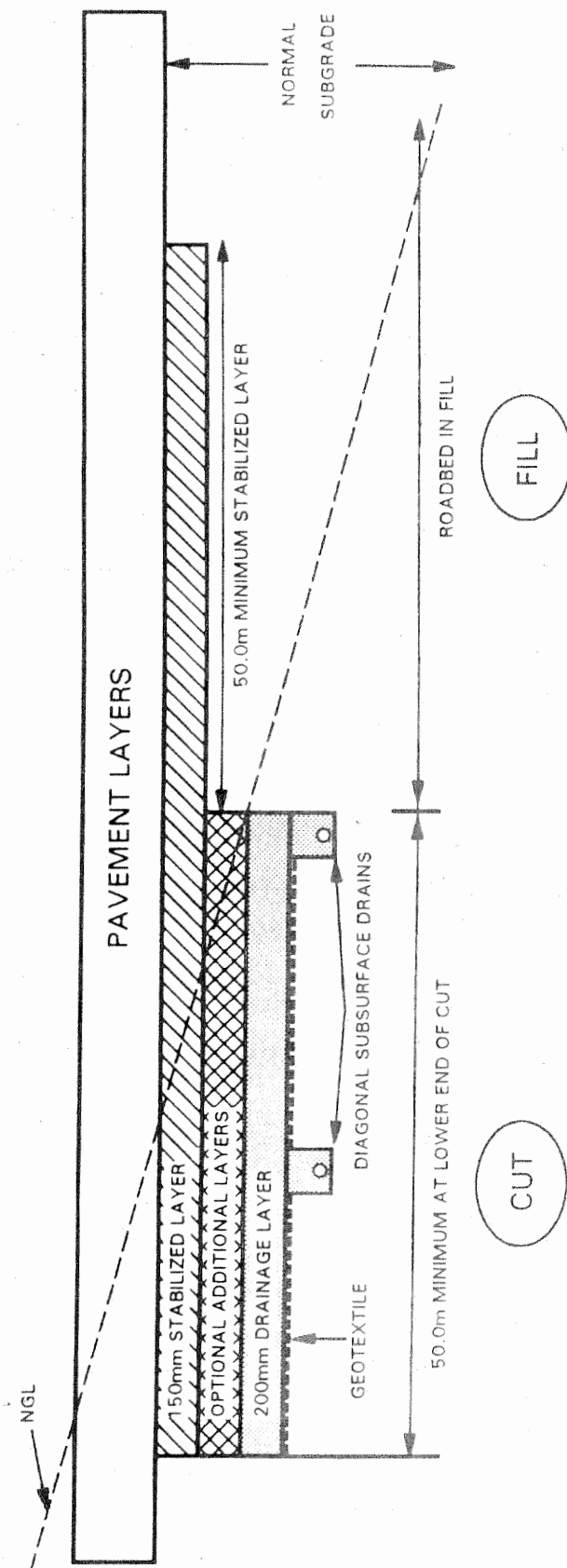


FIGURE 15
Subgrade drainage at cut-fill transition

4.3.3.2 Control

There are numerous techniques for controlling ground water in cut slopes, many of which are costly and complicated. Before any method is considered, a ground-water study should be conducted to investigate the quantity, location and sources of water. Random installation of drains must be avoided because only the necessary volume of hill slope should be drained. The natural draw-down of the ground water caused by excavation of the cut must be considered in the drainage design.

A brief summary of some of the most important and useful techniques are as follows : (See *Figure 16*)

(i) *Sub-horizontal drains*

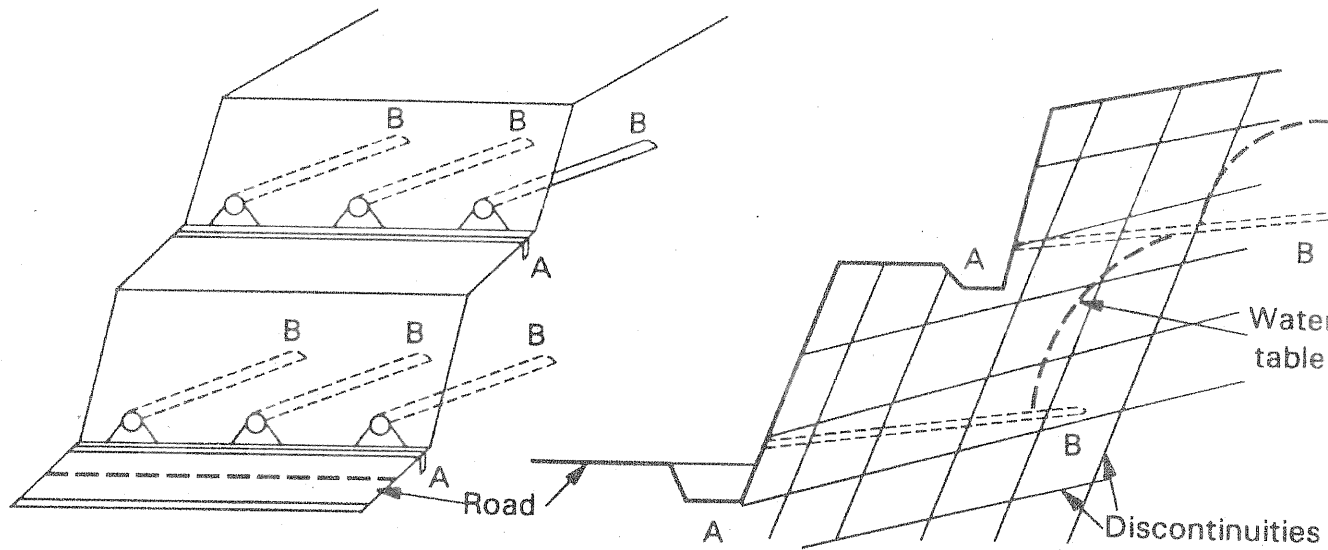
Sub-horizontal drains (*Figure 16 (a)*) consist of small-diameter holes (usually 25 to 50 mm in diameter) drilled on a grade of 5° to 8° and fitted with perforated pipes for soils and pipes perforated at depth for rock slopes. To drain the slope it is imperative that the drains intersect the water-bearing strata or structures. These are probably the least expensive drains as they may be constructed with percussion drills (although core drilling gives a better idea of whether the water paths have been intercepted) and are drained by gravity. Sub-horizontal drains should be installed during construction of the cut.

(ii) *Trench drains*

Trench drains (*Figure 16 (b)*) can be excavated down the face of the slope to a depth of 5 m or 6 m and be backfilled with free-draining granular fill. If they are deeper and extend into stronger material beneath possible slides, a reinforcing action occurs. Interceptor drains cut parallel to and behind the slope, are a specific type of trench drain.

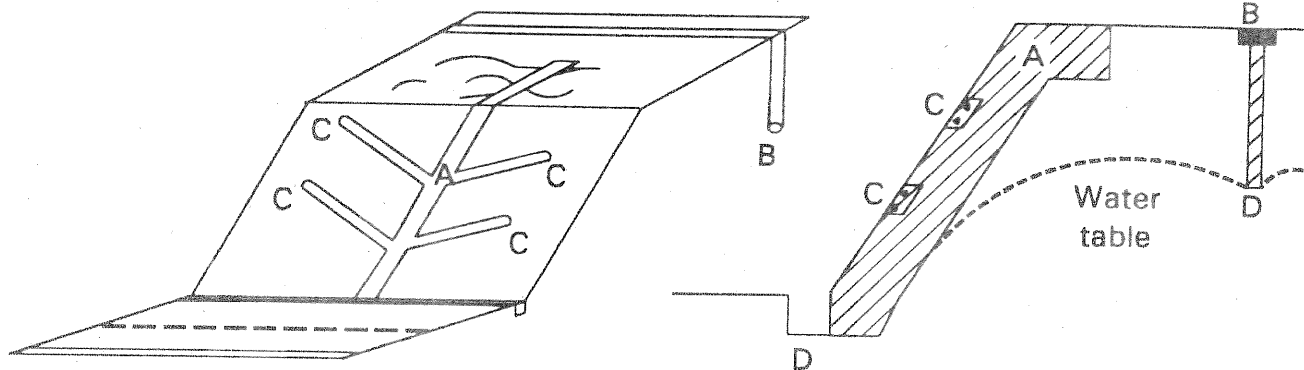
(iii) *Vegetation*

The cheapest, and one of the most effective forms of drainage is revegetation above and behind the slope. The effect of trees



(a) Horizontal drains:

A - Collector drain
B - Subhorizontal drain



(b) Trench drain

A - Trench drain
B - Cut-off drain
C - Herringbone drain
D - Collector drain

FIGURE 16
Cut slope drainage

(especially non-deciduous trees such as the eucalyptus species as well as some indigenous species such as the Karee, or *rhus lancea*) in lowering the ground-water table by evapo-transpiration cannot be over-emphasised. In addition, the root growth reinforces the material in the slope. As trees take some time to establish, other de-watering measures are often required during the early stages of the project. It is important to remove as few trees from behind the slope as possible during construction.

It is important to drain the pavement within a cut as pore water pressures due to the available head in the adjacent cut may lead to pavement distress. Provision for drainage must be made in the road foundation.

4.4 CONTROL OF SURFACE INFILTRATION

The sources from which water will frequently enter the pavement structure are illustrated in *Figure 2*.

4.4.1 Drainage of pavement structures

4.4.1.1 Considerations

According to Draft TRH4² the basic philosophy is to provide effective drainage at least to the material depth (as defined in Section 1.2).

Drainage for the control of ground water has been described in the previous section, but the drainage of water which might infiltrate through the pavement surface must also be considered.

Most designs assume that the surface will be sufficiently watertight to shed the bulk of any rainfall, and that the balance which enters into the pavement will have a negligible effect, but maintenance experience proves that this assumption is frequently wrong.

Since the addition of drainage, as a maintenance operation, requires almost complete reconstruction, many designers are now inclined to adopt the "fail-safe" philosophy used in the design of reservoirs, and

to provide drainage to cope with leaks in a structure intended to be watertight.

4.4.1.2 Susceptibility

The warrant for drainage provisions in the pavement structure depends on the appreciation of each of the following factors :

(i) Climate

Surface drainage problems are caused by the rainfall, and aggravated where the rainfall is characteristically of a low intensity over long periods. The problem is compounded by the fact that development, and hence traffic, tends to be associated with the higher rainfall areas.

In areas with an average annual rainfall of less than 250 mm, surface infiltration is minimal, but where the figure exceeds 1 000 mm, pavement distress due to excess water is to be expected.

(ii) Pavement condition

The "new" condition of the pavement surface immediately after construction is completely irrelevant to the drainage analysis. The designer must forecast the condition to which the pavement surface will be reduced before any rehabilitation measures are taken, because it is in the pre-rehabilitation stage that deterioration occurs most rapidly, particularly where cracking is the typical mode of pavement distress.

(iii) Materials

Some materials are less susceptible to water damage than others - for example concrete, sound aggregates and clean sands will generally perform better than cohesive gravels. But the use of the so-called "non-susceptible" materials in part of the pavement structure will not necessarily prevent the failure of the whole, as

evidenced by :

- pumping at concrete pavement joints;
- stripping of bitumen-bound materials;
- pore pressure failures in granular bases; and
- deposits of fines along the cracks in stabilised bases.

(iv) *Traffic*

The amount of pavement deterioration will depend upon the traffic loading applied during the period when the moisture content in the pavement structure is at or near the saturation level. Since traffic is largely independent of rainfall, the degree of susceptibility is directly related to anticipated daily traffic loads.

4.4.1.3 *Recommendations*

The guidelines²³ prepared for the Federal Highways Administration of the USA suggest that the pavement structure should have designed drainage capacity to remove the expected infiltration, except where the average annual rainfall is less than 250 mm, or, in the case of rigid pavements, where the daily traffic is less than 250 E80s, where :

- the expected infiltration is $\frac{1}{3}$ - $\frac{2}{3}$ of the maximum hourly rainfall returning on an annual frequency, the fraction depending on the pavement type and condition; and
- the drainage capacity takes into account both the vertical capability of the pavement structure and the lateral transmissibility of base and drainage layers.

South African experience seems to show that these stringent requirements are unduly conservative, but it is at least recommended that adequate drainage capacity should be provided on road facilities with a design traffic exceeding Class E2 ($0,8$ to $3,0 \times 10^6$ E80s), in areas with an average annual rainfall exceeding 500 mm, to cope with an infiltration factor of $\frac{1}{3}$ and a two-hour drainage period (See also the recommendations under 4.1).

4.4.2 Shoulders and verges

The edge of the pavement is particularly prone to distress because :

- unless adequate pavement drainage has been provided, surface infiltration will accumulate under the outer edges;
- where the subgrade is relatively dry, suction forces will draw the water which enters through the unpaved margins of the road;
- unless surfaced shoulders have been provided, the heaviest traffic loads which occur in the outer wheel path will be near the pavement edge; and
- an uncompacted over-built road prism (uncompacted overfill) will inevitably lead to surface infiltration.

Recommended techniques for the drainage of the pavement edges are shown in *Figure 9*.

It should be emphasised that whatever system is adopted must be designed to cater for the estimated inflow from the pavement base and drainage layers, because any shortcomings in the continuity of the system will result in the accumulation of water in the most vulnerable areas of the pavement.

Intermittent mitre drains need to be spaced with special attention to their permeability, slope and section in order to cope with the cumulative flow from the pavement edges.

4.4.3 Side drains

The provision of proper side drains is the cheapest insurance against excess moisture under the pavement edges, and the drain should be designed not only with the capacity to handle surface flows, but also :

- (a) with an invert below the bottom of the top of the subgrade;
- (b) at a grade sufficient to ensure rapid run-off without ponding, or with a lining when grades flatter than 1,0% are unavoidable; and
- (c) with adequate protection against scour, silting or the growth of vegetation so that only negligible maintenance will be required.

It is not acceptable to base the design on the assumption of routine maintenance. The deterioration likely to arise under service conditions must be identified, for example as :

- clogging by material eroded or weathered from cut slopes;
 - silting as a result of erosion on the shoulders and verges;
 - growth of rank vegetation; or
 - public litter,
- and dealt with by appropriate preventive or protective measures.

4.4.4 Cut slopes

Since the drainage for the removal of ground water behind cut slopes is relatively expensive, more attention should be given to tackling the problem at the source of surface infiltration.

4.4.4.1 Where the infiltration occurs naturally over an extensive area there is little chance of prevention, by way of afforestation or surface drainage, without the co-operation of the landowner, but often the infiltration will come from a localised source such as :

- ponding in natural watercourses, or in natural ground depressions;
- artificial ponds created by agricultural contour banks, catchwater banks, and berms thrown up during the course of construction;
- leaking water mains or reservoirs;
- soak pits, such as abandoned borrow pits;
- domestic wastes; and
- irrigation.

4.4.4.2 Once a source of this nature has been identified, the infiltration can usually be prevented by relatively simple surface drainage improvements, or can at least be intercepted at a safe distance behind the cut slope.

Interceptor trenches should always be carefully analysed to ensure :

- that they will, in fact, capture the seepage flow;
- that they have the capacity to discharge the flow harmlessly, without storage in the trench; and
- that they are not located so as to encourage or exaggerate the formation of tension cracks along the line of any potential slip circle.

4.5 PIPES

The function of the pipework is to collect and discharge the water entering the subsurface drainage system, and the selection, installation and maintenance of the pipes is therefore of fundamental importance.

4.5.1 Types

The type of pipe should be selected on the basis of :

- durability in relation to the surrounding soil and water conditions;
- strength under the design cover and the superimposed loads (compaction and traffic); and
- cost and availability.

Pipes that are currently available are manufactured from pitch fibre, unplasticized polyvinyl chloride (uPVC), and high density polyethylene (HDPE). Asbestos cement and porous concrete pipes have also been used, but are currently not widely used in subsurface drainage systems. The method of manufacture of the pipes include smooth pipes with holes or slots (pitch fibre, HDPE and uPVC), ribbed or corrugated flexible pipes with slots (uPVC) and extruded geopipes (HDPE).

A number of SABS specifications control the quality of the various types of pipes^{24,25,26}. However, with the exception of the specification

for pitch fibre pipes, these specifications are aimed at smooth, unperforated and unslotted pipes and are therefore not directly applicable to subsurface drainage pipes. The strengths of pipes are severely affected by holes, slots and ribs. It is also difficult to compare the strengths of various types of pipes due to the different test methods that are applied to different types of pipes. The SABS specifications can therefore do little more than to control the quality of the material from which the pipes are manufactured. Material quality is nevertheless an important aspect and the use of the specifications or other relevant documents²⁷ is therefore recommended. The designer must also assure himself that the specified pipes are sufficiently UV resistant and that the time of exposure to direct sunlight is limited to what the particular material can withstand.

For pipe strength design, the designer must rely on his own estimation of the required strength and compare these with parameters that are quoted by the suppliers. In most cases the pipes used in subsurface drainage applications are flexible or semi-flexible. For these pipes the bedding and backfill conditions are much more critical than for rigid pipes. Where the pipe is surrounded by granular material, such as a granular filter, there is little danger of crushing or deformation, provided the construction is carried out with the necessary care. Where the pipe is surrounded by soil, such as at the outlets, the bedding conditions become critical. Cohesive soil ($PI > 6$) must be avoided and the material must be compacted around and above the pipe as required by the design method used or as recommended by the manufacturers.

4.5.2 Size

The pipe size can be determined by hydraulic design using either formulae (e.g., Colebrook - White or Manning) or nomographs and charts that are often provided by the suppliers. Whatever method is used, reasonable allowance must be made for turbulence created by slots or corrugations.

The Manning formula is as follows :

$$V = \frac{R^{2/3} S^{1/2}}{n}$$

where : V = Velocity (m/s)

R = Hydraulic radius (m) = $\frac{\text{Area}}{\text{Wetted perimeter}}$
 = D/4 for full flow, with D = diameter (m)

S = Slope (m/m)

n = Manning n value

Flow can then be calculated by :

$$Q = VA$$

where : Q = flow (m³/s)

A = Area (m²).

Typical Manning n-values are as follows²⁸, but values are often available from manufacturers which can be used if they are known to be more accurate and experimentally verified :

<i>Type of pipe</i>	<i>Manning n</i>
Plastic, unperforated	0,008
Asbestos - Cement, unperforated	0,009
Perforated PVC	0,010
Perforated pitch fibre	0,010
Concrete	0,011
Corrugated plastic pipe	0,025

In practice the flow is very often indeterminable, and the designer must rely on past experience, to the effect that :

- given a reasonable outlet spacing of 200 to 300 m, pipe sizes of 100 to 200 mm diameter, typically 100 mm, will have adequate capacity;
- for laterals and minor collectors, such as base course edge drains, pipe sizes of 50 to 100 mm diameter, typically 75 mm, will be sufficient; and
- it is unnecessary to design to cope with short-term peak flows, because a reasonable time and amount of "ponding" in the drained layers is acceptable (See Section 4.1).

If outlets cannot be provided at suitable spacings, pipe sizes can be increased to cope with higher flows further downstream of the system.

The actual flow volumes must be verified, where possible, during the installation of the system. The open sides of the trench can reveal the origin and the volume of water. Adjustments can be made to the design at this stage, making allowance for the particular season.

4.5.3 Slope

The "half full" flow velocity should not be less than 0,75 m/s desirable, 0,60 m/s absolute minimum, to ensure that the invert is self cleaning.

The slope should preferably increase along the flow line, or, if not, continuity should be assured by an increase in capacity.

Where lower velocities are unavoidable it is necessary to prove by field tests that the filter material effectively bridges the pipe perforations, and to provide facilities for routine pipe maintenance.

4.5.4 Perforations

The spacing of the perforations should be such that there are sufficient openings to accept the inflow, without seriously affecting the strength of the pipe.

The diameter of the hole, or the width of the slot, should be 3 to 8 mm for usual filter materials, but the actual hole or slot should preferably be dimensioned to suit the proposed filter material, in that :

$$\text{hole diameter} \leq D_{85}F$$

$$\text{slot width} \leq D_{85}F \times 0,83$$

where $D_{85}F$ is the size of the sieve that allows 85 percent of the filter material to pass through⁹.

Recommended practices concerning the alternative of "holes up" or "holes down" are as follows :

- for general purposes in collector zones and where pipes are to remain permanently below the water table over their full length, the holes or slots should be located towards 4 and 8 o'clock around the pipe bore, i.e., "holes down"; and
- in carrier zones where subsurface drainage is not required unperforated pipes should be used or consideration can be given to using the "holes up" installation.

4.5.5 Cleaning eyes

The use of cleaning eyes is recommended for cleaning as well as for locating the positions of blockages that may occur in the form of silting, plant growth, etc. Typical details of cleaning eyes are shown on *Figure 17*.

4.5.6 Outlets

The efficacy of a subsurface drainage system is totally dependent on the provision of a well-constructed, permanent outlet structure that will allow the continuous, undisturbed flow of water. Typical detail of such an outlet is shown on *Figure 17*. Outlets may also be accommodated in the storm water system, but dedicated subsurface drainage outlets are easier to inspect and monitor. All outlets should be clearly and permanently marked. The pipe should be suitably covered at the outlet to prevent rodents and other small animals from entering and making nests in the pipes.

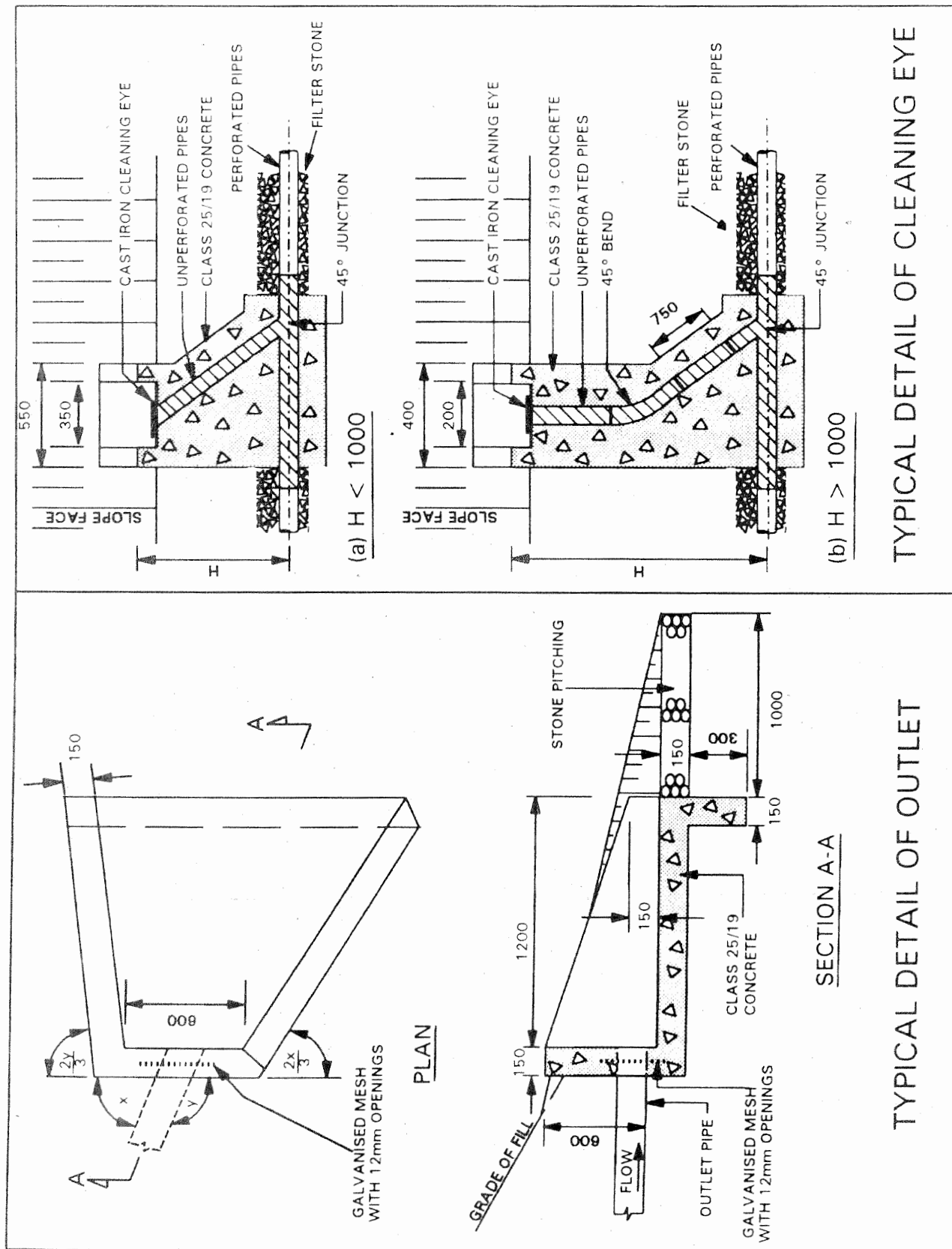


FIGURE 17

Typical outlet and cleaning eye details

4.6 FILTERS : GENERAL

The two main categories of filters between which the designer must choose are granular filters and synthetic filters (geotextiles). Granular filters have been used in road subsurface drainage and other applications for many years and successful long term performance has resulted in general acceptance of their design criteria. Synthetic materials, on the other hand, have had a relatively short history and the rate of development of new materials and products has been such that design methods and selection criteria have not always been available for these materials. However, geotextiles have now been used in subsurface drainage systems for approximately 20 years in South Africa and generally satisfactory performance has been reported. Test methods and design criteria have also been developed on which the designer can base his selection.

The respective advantages of granular filters and geotextiles may influence the designers final choice. These can be summarised as follows :

The main advantages of **granular filters** over geotextiles are :

- granular filters have had a long history of successful applications and there is more knowledge of their long term performance than that of geotextiles;
- design methods and selection criteria are better established and there is more general consensus on their applicability than for geotextiles;
- the durability of granular filters are generally considered to be adequate while the durability of geotextiles needs to be specified and controlled since some polymers from which geotextiles are manufactured may be subject to degradation under certain chemical and biological conditions;
- exposure to sunlight of granular materials is generally not a problem but for geotextiles it must be limited since ultra-violet (UV) light has a detrimental effect on all geotextiles to some extent, depending on the type of polymer used (See Section 4.8.1.1); and
- field quality control for installation is less critical than with geotextiles. Geotextiles are less "forgiving" and if they become

damaged, torn, severely soiled or are not adequately overlapped it could affect the effectiveness of the whole system.

The main advantages of **geotextiles** over granular filters are :

- the properties of geotextiles are factory controlled with the accompanying ease of achieving uniformity and quality control;
- the logistics of obtaining and deploying geotextiles on site are considerably easier since they are relatively lightweight and freely available as opposed to the geographic location, availability and transportation of granular filters;
- where the geotextile is used in conjunction with granular materials such as in a geotextile wrapped drain, the granular material need only be designed for hydraulic conductivity;
- geotextiles have tensile strength and are available in various weights, strengths and types and are therefore more versatile;
- the installation of geotextiles is quicker, easier and uses less labour than granular filters; and
- due to all the above, geotextiles are usually cheaper than granular filters.

In selecting a filter the two basic requirements must be considered, namely :

- the openings in the filter must be small enough to prevent the soil particles from passing through the filter (piping criterion); and
- the openings in the filters must be large enough to allow the flow of water at a sufficiently high rate (permeability criterion).

These are clearly opposing requirements and therefore upper and lower limits are usually applied. This also means that there is no single ideal filter, and that design limits must be based on acceptable performance.

Research^{29,30} that was carried out in the laboratory to evaluate geotextiles as filters showed that when any filter, granular or geotextile, is operated over a long period in a permeameter, a reduction in permeability is inevitable. This is partly due to the forming of a filter zone in the soil (See Section 4.8.3). It is therefore advisable to evaluate the long term performance of any filter. A long term flow test using a permeameter (Method B3, Appendix B) that was

developed for geotextiles can also be used to evaluate the long term permeability of granular filters and to do comparative tests with geotextiles.

4.7 GRANULAR FILTERS

The piping and permeability criteria for granular filters are based on grain sizes.

Extensive experimental work has shown that these filters must satisfy the criteria given below in order to ensure satisfactory performance, where :

D is the grain size diameter
F represents the filter material
S represents the soil to be drained

(For example, $D_{15}F$ means the size of the sieve that allows 15 percent of the filter material to pass through)

and U is the uniformity coefficient $\frac{D_{60}S}{D_{10}S}$

(a) *Cohesionless soils*

Piping criteria	Particle size relationship
generally	$D_{15}F \leq 5 \quad x \quad D_{85}S$
but, for different	
uniformity coefficients,	
$U < 1,5$	$D_{15}F \leq 6 \quad x \quad D_{85}S$
	$D_{15}F \leq 20 \quad x \quad D_{15}S$
$U > 1,5$ but $< 4,0$	$D_{15}F \leq 5 \quad x \quad D_{85}S$
	$D_{15}F \leq 20 \quad x \quad D_{15}S$
$U > 4,0$	$D_{15}F \leq 5 \quad x \quad D_{85}S$
	$D_{15}F \leq 40 \quad x \quad D_{15}S$
Permeability criteria	
for all soils	$D_{15}F \geq 5 \quad x \quad D_{15}S$
Compatibility criteria	
for all soils	$D_{50}F \leq 25 \quad x \quad D_{50}S$

It is also recommended^{31,32}

- that the filter material should not be gap-graded, nor have more than 5 percent passing 0,075 mm;
- that the filter design should be based on the finer fraction of the drained soil where such fraction is separately distinguished in the soil layers, or below a gap in the grading; and
- that, if $U > 4$, the filter design be based only on the fraction of soil that passes the 4,75 mm sieve³³. The South African Department of Water Affairs recommend that filter design **always** be based on the fraction of soil smaller than 4,75 mm, irrespective of uniformity³⁴.

The application of these criteria is illustrated diagrammatically in Figure 18.

(b) *Cohesive soils*

Since the small seepage velocities in these soils are generally insufficient to overcome the cohesion between the particles, the criteria given above that were developed for cohesionless materials, are generally conservative. The most widely used criteria for these materials are^{35,36} :

$$D_{15}F \leq 0,4 \text{ mm}$$

or

$$D_{15}F \leq 0,7 \text{ mm}$$

Dispersive soils are soils of which the cohesion breaks down under the influence of water. Although there has been some concern about applying the criteria for cohesive materials to these soils, recent experience has shown that they work well with most dispersive soils in Southern Africa³⁴.

The designer will be fortunate if filter materials that satisfy the required grading criteria are readily available from local resources of natural or processed materials. If they are not, the design should be reviewed, and alternatives should be considered, to avoid the disproportionate expense of custom-built products.

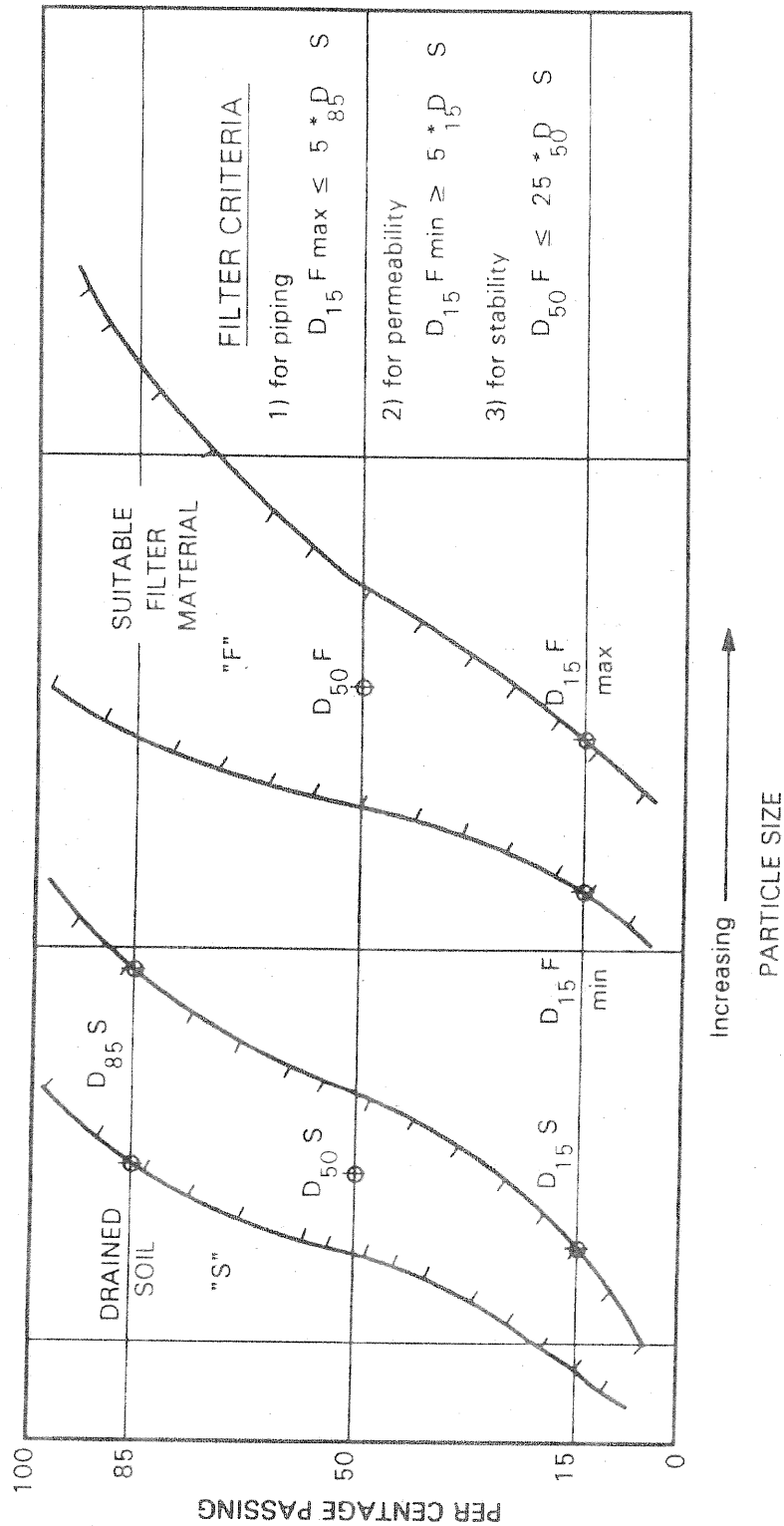


FIGURE 18
 Illustration of filter criteria

4.8 GEOTEXTILES^{37, 38, 39, 40, 41, 42, 43, 44, 45, 46, 47}

4.8.1 Composition and manufacture

The properties of geotextiles are determined by their composition (i.e., the raw materials from which they are manufactured) as well as the method of manufacture of which there are three main categories namely wovens, nonwovens and knitted. *Figure 19* shows a classification of geotextiles and on *Figure 20* some examples of geotextiles are shown.

4.8.1.1 Composition

The most raw materials from which geotextiles are manufactured are polymers and the most widely used are polypropylene and polyester. Polyester has better strength and creep properties as well as higher natural resistance to ultra violet deterioration than polypropylene but is also more expensive. The ultra violet resistance of polypropylene is improved by a treatment termed UV stabilising.

Other polymers that are sometimes used are polyethylene and polyamides, but very few geotextiles are available that are made from these polymers since their natural resistance to various aggressive environments is generally less than that of other polymers.

4.8.1.2 Woven geotextiles

Woven geotextiles are manufactured by weaving weft threads through warp threads. Warp threads are usually stronger than the weft threads. The end product obtained by weaving depends on the type of thread used, the method of weaving and after-treatment (stabilising) of the woven product. A classification of woven geotextiles is given in *Figure 19*.

(a) Thread type

Woven geotextiles can be manufactured from a single thread type or from a combination of thread types.

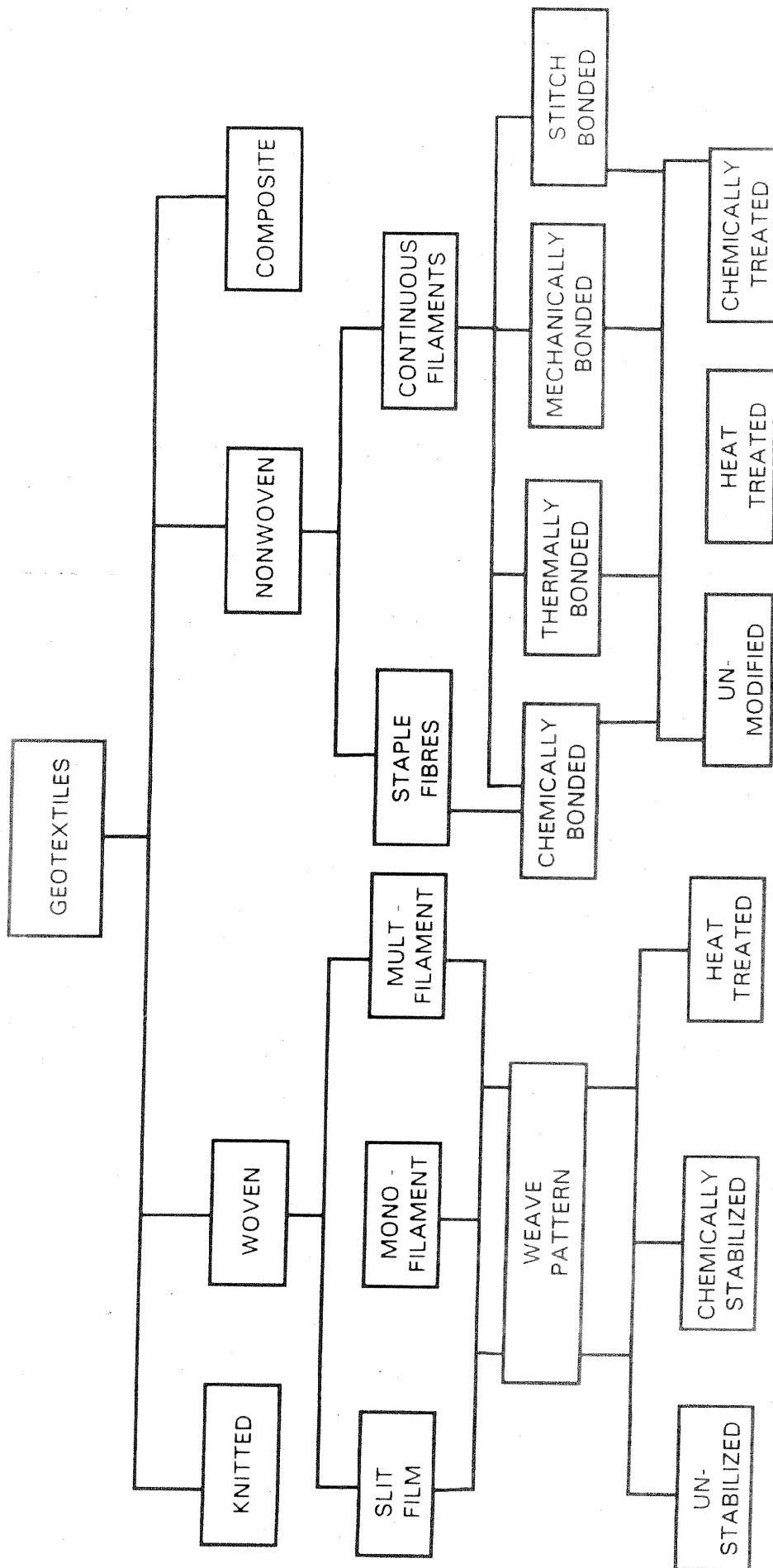
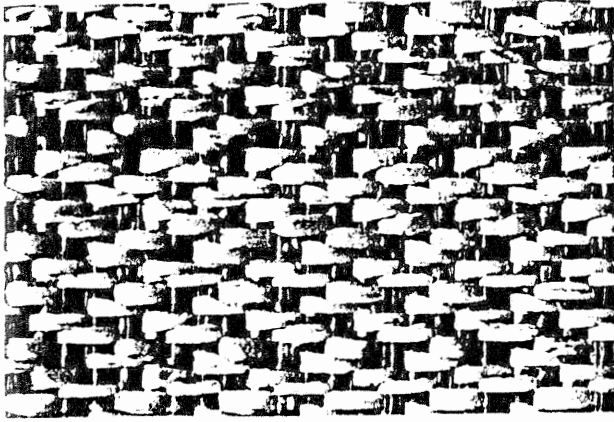
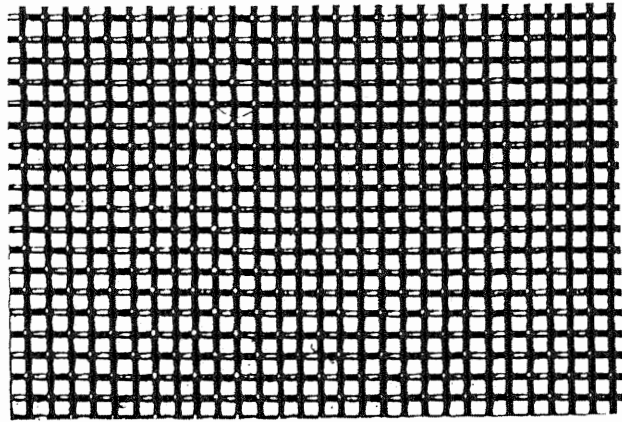


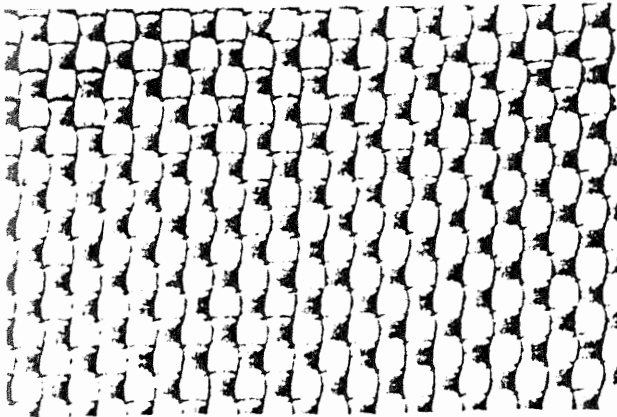
FIGURE 19
Classification of geotextiles



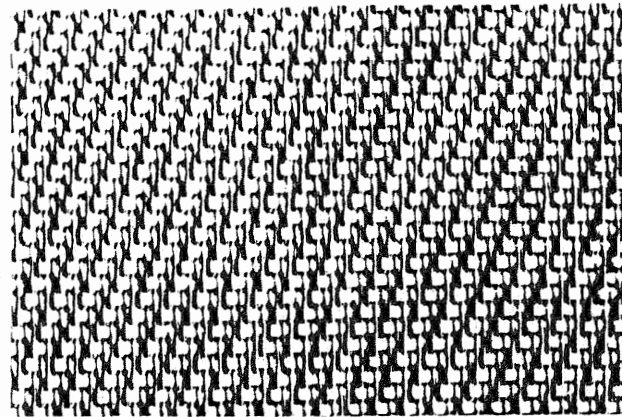
WOVEN
SLIT FILM



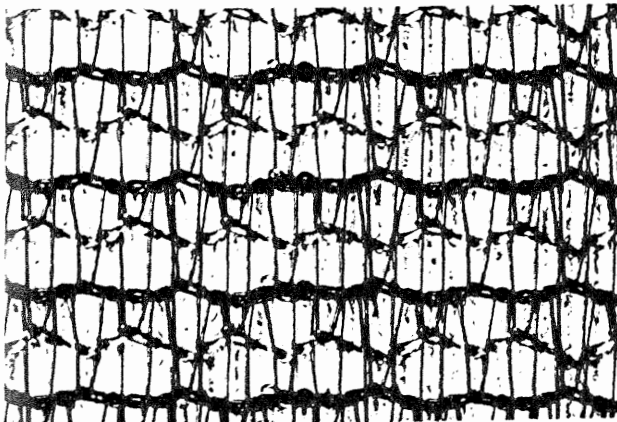
WOVEN
MONOFILAMENT



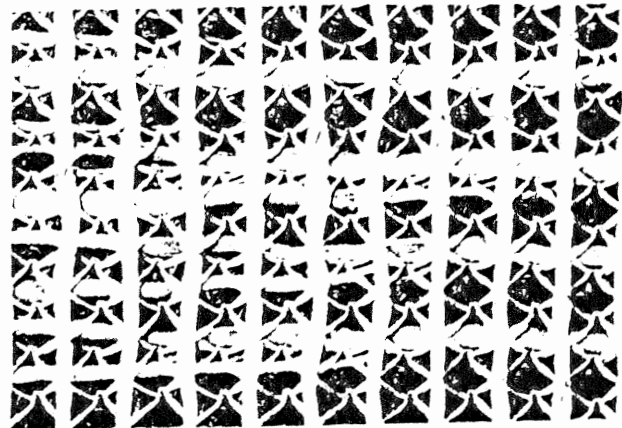
WOVEN
MULTIFILAMENT



WOVEN
WARP: MONOFILAMENT
WEFT: MULTIFILAMENT



KNITTED



COMPOSITE: NON WOVEN/KNITTED
MULTIFILAMENT

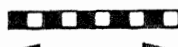
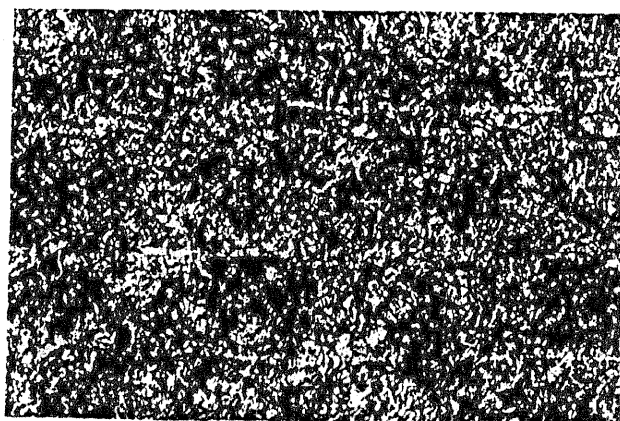
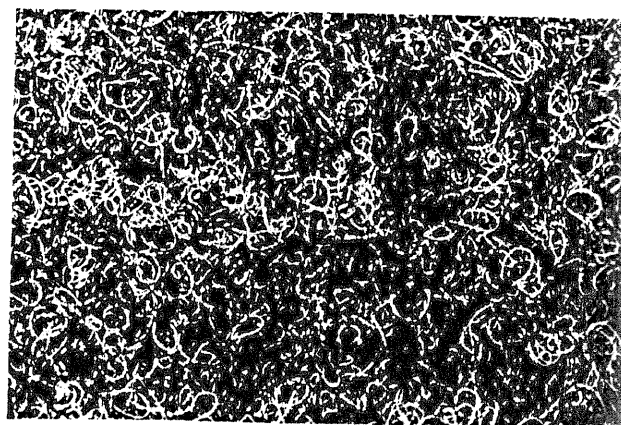
Scale: 
10 mm

FIGURE 20

Examples of geotextiles



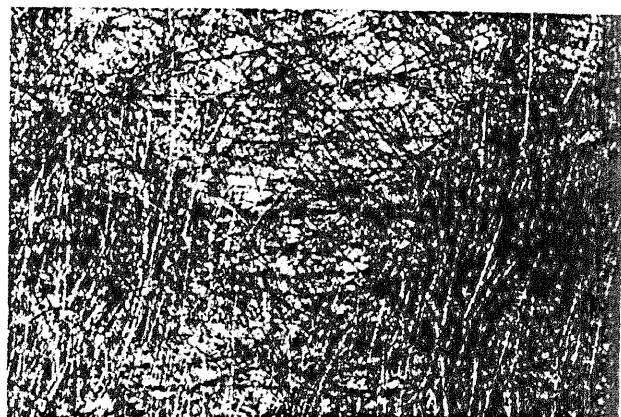
NONWOVEN
STAPLE FIBRES
CHEMICALLY BONDED



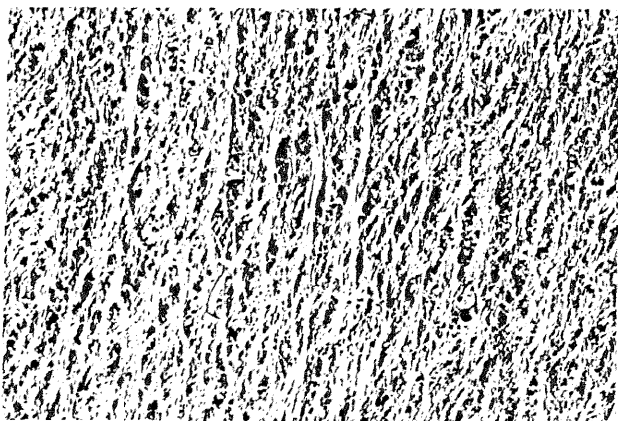
NONWOVEN
STAPLE FIBRES
NEEDLE PUNCHED



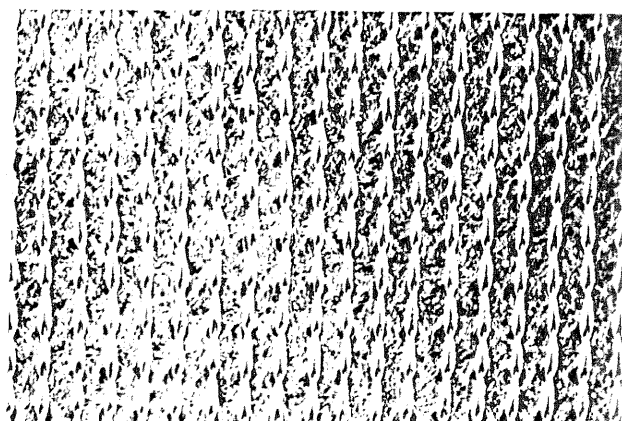
NONWOVEN
CONTINUOUS FILAMENTS
CHEMICALLY BONDED



NONWOVEN
CONTINUOUS FILAMENTS
HEAT BONDED



NONWOVEN
CONTINUOUS FILAMENTS
NEEDLE PUNCHED



NONWOVEN
STAPLE FIBRES
STITCH BONDED

Scale: 
10 mm

FIGURE 20 (Continued)
Examples of geotextiles (Continued)

Tapes and slit film : Tapes are extruded through a die which has many slots and slit-films are produced by slitting extruded polymer film. Geotextiles manufactured from these thread types are the most common wovens and are usually less permeable than those woven with monofilaments.

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

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

(b) Weave pattern

Weave pattern (or weave) is the pattern which the warp and weft threads form. There are three basic types of weave, namely Plain or Linen, Twill and Satin. In addition there are numerous derivatives such as Hopsack and Hucksback. The weave determines the physical and mechanical properties of a woven geotextile and as a result special weaves are used for particular applications.

(c) Stabilising

If a geotextile has an open weave with large openings, the threads can move relative to each other, thus distorting the openings. This can be overcome by stabilising the geotextile either by heat setting or by chemical coating, but the process is not widely applied.

4.8.1.3 Nonwoven geotextiles

Nonwoven geotextiles are produced from randomly distributed continuous filaments or staple fibres which are bonded together chemically, thermally or mechanically. A classification of nonwoven geotextiles is shown in *Figure 19*.

(a) Fibre type

- (i) Continuous filaments : Filaments are fibres which are continuously extruded from molten polymer and are referred to as continuous filaments. When used to manufacture nonwoven geotextiles the extruded filaments are spun and drawn prior to being spread and laid down on a moving belt, in the form of a fibrous web. The web is essentially an unbonded nonwoven which is then bonded in one of the ways described below. This process of spinning and bonding is referred to as the spun-bond process.
- (ii) Staple fibres : Relatively short staple fibres (20 to 150 mm), cut from continuous filaments, are used to manufacture nonwoven staple fibre geotextiles. The staple fibres are usually carded (opened, cleaned, aligned and formed), and then laid down as a fibrous web on a moving belt prior to bonding.

(b) Bonding processes

Both continuous filament and staple fibre nonwovens can be bonded in one of four ways :

- (i) Chemical bonding : The web is sprayed or impregnated with binder, usually an acrylic resin. After curing and/or calendaring (heat and pressure), strong bonds develop between the fibres. Forced air drying or a vacuum is used to remove excess bonding agent and re-establish the open pore structure.
- (ii) Thermal bonding : The web is subjected to calendaring and the fibres are melted together at the points of contact. This process can be carried out using fibres made from one type of polymer or a combination of polymers (heterofilament) which have different softening characteristics.
- (iii) Mechanically bonded (Needle punched) : The web is passed through a needle loom where barbed needles punch through the web mechanically entangling the fibres.

- (iv) Stitch bonding : Stitch bonding is usually carried out on staple fibres using conventional sewing techniques. The finished product consists of evenly spaced lines of stitching.

For specific applications nonwoven geotextiles can be treated further, either thermally or chemically.

4.8.1.4 Knitted geotextiles

Knitted geotextiles consist of a single strand systematically intertwined with itself and is manufactured with a knitting machine, instead of a weaving loom. The most popular type of knitting is warp knitting, the end product of which consists of a net-like structure of variable mesh size.

4.8.1.5 Composite Geotextiles (Multi-layered geotextiles)

These are manufactured by combining layers of different material. In some cases nonwoven geotextiles are combined with woven geotextiles. Composites also include combinations of geotextiles with other related products such as grids, meshes, membranes and drainage cores. The components can be combined by needling, stitching, chemical bonding or heat bonding. Composites usually have improved or different characteristics from their components.

4.8.2 Functions and Properties

4.8.2.1 Functions

Geotextiles can perform a number of functions in a variety of applications. Of the various functions described below, filtration, fluid transmission and separation are the most important in subsurface drainage applications.

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

- (b) Fluid transmission (drainage in the plane) : The geotextile acts as a water carrier by allowing flow in the plane. This function is important in certain drainage applications, e.g., where the geotextile is wrapped around a pipe without a natural (granular) or synthetic (geonet) water carrier.
- (c) Separation : The geotextile separates two different materials and ensures that one material does not contaminate the other. If a granular subbase is placed on a clayey subgrade a geotextile can be used to ensure that there is no movement of fine particles into the subbase material.
- (d) Protection : The geotextile protects a material when it alleviates or distributes stresses and strains transmitted to the protected material, e.g., where the geotextile is placed in conjunction with a membrane to prevent damage from concentrated stresses or where the geotextile is placed on a soil to prevent surface damage such as where it is used for erosion control or river protection.
- (e) Reinforcement : The geotextile provides tensile strength to other materials, e.g., soils or asphalt.
- (f) Moisture barrier : The geotextile performs as a moisture barrier if it is rendered relatively impermeable. This can be achieved by spraying or impregnating a properly deployed geotextile with bituminous or polymeric mixes, e.g., road surface treatments and roof waterproofing.

4.8.2.2 Properties

The type of polymer used and the method of manufacture of a geotextile will determine its basic properties, which in turn will affect its ability to perform the various functions described above.

The main properties of geotextiles can be summarised as follows :

- (a) Constructional and basic physical properties : These include material constituents, method of manufacture, mass per unit area, thickness, compressibility, stiffness, etc. These properties are

important mainly in terms of their effect on the more functional properties namely mechanical, hydraulic and durability.

- (b) Mechanical properties : These include all the strength related properties such as stress-strain behaviour, puncture behaviour, burst strength, tear propagation, creep properties, surface friction, etc. These properties play a dominant role where geotextiles are used in reinforcement functions, but are less important for filtration functions. For road subsurface drainage applications their most important role is during construction.
- (c) Hydraulic properties : These include normal permeability (permittivity), transverse permeability (transmissivity), pore size opening, porosity and percentage open area. These properties are of direct importance to all filtration functions.
- (d) Durability (also known as environmental) properties : These include resistance to ultra violet light, chemical degradation, biological degradation, temperature variations and weathering. These properties are important for all the various functions for which geotextiles are used.

4.8.3 Geotextiles as filters

Various researchers have described the mechanism whereby a geotextile functions as a filter in uni-directional flow conditions. This mechanism illustrated in *Figure 21* shows a highly permeable zone of larger soil particles forming a bridging network adjacent to the geotextile. The finer soil particles in this zone, before commencement of flow through the system, have all been washed onto, into and/or through the geotextile. Immediately behind this bridging zone the permeability of the soil decreases as the distance from the geotextile increases. This is referred to as the filter zone, behind which the soil has remained undisturbed during the formation of the bridging and soil filter zones.

Once the soil filter is established, no further soil is washed through the geotextile and the system is in equilibrium. The main function of the geotextile in such a uni-directional flow condition is therefore to act as a catalyst for the formation of a stable soil filter in the insitu soil. The

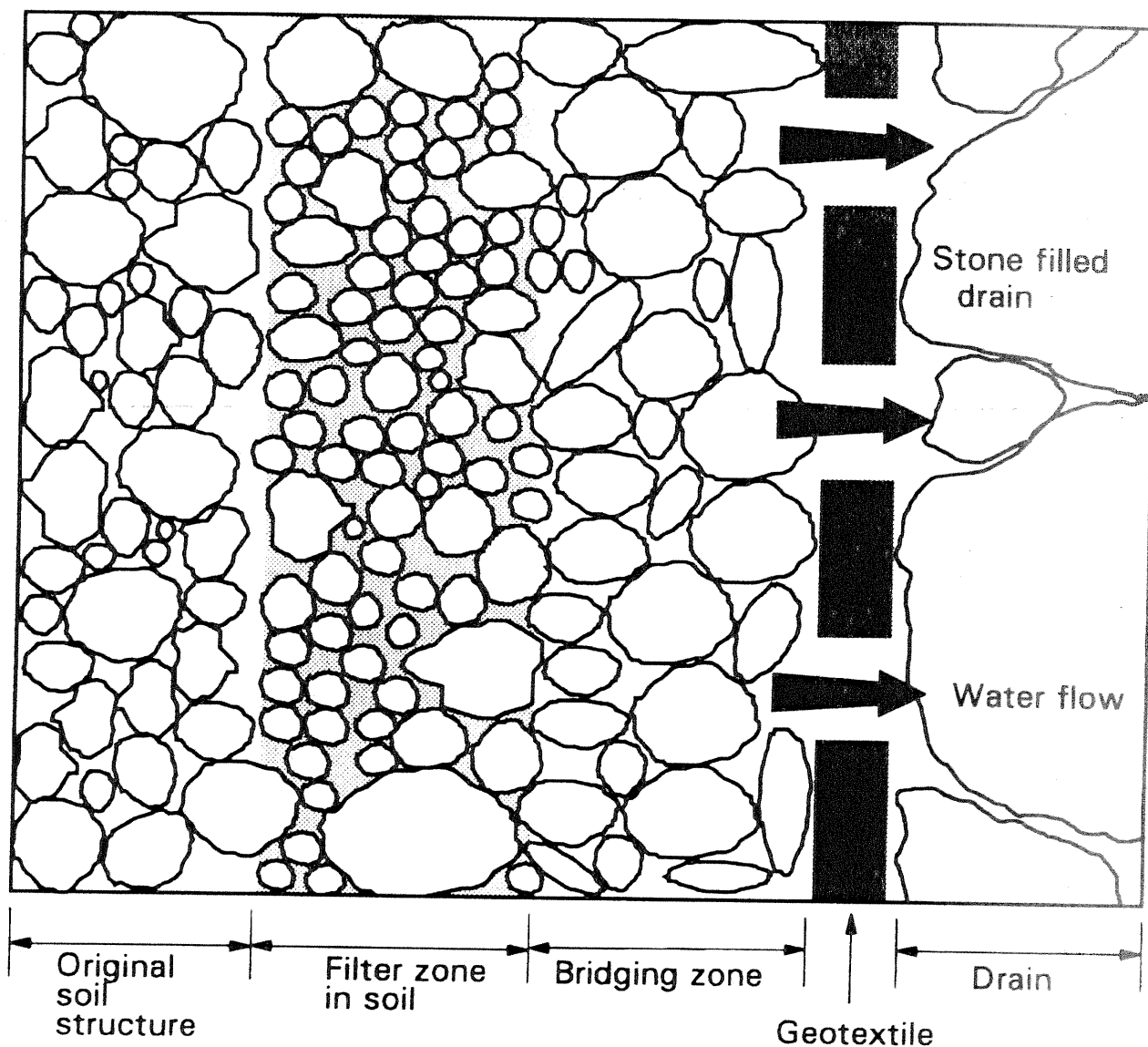


FIGURE 21

Formation of a filter zone in a soil-geotextile system

main requirements for such a system to develop and remain stable are :

- the permeability of the geotextile must be higher than that of the insitu soil,
- the effective pore sizes in the geotextile must be able to retain a sufficient proportion of the soil particles to form the bridging zone upstream of the geotextile,
- the pore sizes in the geotextile, relative to the grain sizes of the soil, must be such that soil particles do not clog or block pores in the geotextile to the extent that the permeability reduces to less than that of the insitu soil, and
- the geotextile must be placed in intimate contact with the insitu soil because if there are voids between the soil and the geotextile, large volumes of fine soil particles may move resulting in blinding of the geotextile surface.

Where reverse flow conditions occur, e.g., river bank protection, a different mechanism applies because if a filter zone does build up, it will be destroyed when the flow direction changes.

4.8.4 Tests

Table 4.8.1 gives a summary of the most widely used tests for the evaluation of geotextiles. It should be noted that other tests that are not shown in the table also exist and that for each test shown various test methods and standards are used. Some of the tests shown are also used more often than others.

A large number of tests are used in various parts of the world to determine a variety of properties. In selecting the most appropriate test methods, the following should be considered :

- importance of property for the application;
- simulation of field conditions;
- standardisation of test method; and
- availability of criteria for local conditions.

TABLE 4.8.1 SUMMARY OF GEOTEXTILE TESTS

TEST		TEST REFERENCE
SOIL/GEOTEXTILE TESTS (DESIGN)		RECOMMENDED FOR DESIGN
Long Term Flow Tests (LTFT)		Yes
Gradient ratio (GR)		Optional extra test
Interface Flow Capacity (IFC)		For specific situations
IN-ISOLATION TESTS (CLASSIFICATION)		APPLICABILITY OF TEST
Mass per unit area		High
Thickness and compressibility		Moderate
Strength tests	Penetration load	High
	Puncture resistance	High
	Tensile strength	Moderate
	Grab strength	Low
	Puncture strength	Low
	Mullen burst strength	Low
	Trapezoid tear	Low
	Creep	Low
Hydraulic tests	Normal permeability	High
	In-plane permeability	Moderate
	Pore-sizes	Moderate with caution
Durability	Ultra-violet light	Low
	Chemical degradation	Moderate
	Biological degradation	Moderate
	Temperature	Low

It is useful to separate the tests into two groups namely in-isolation and soil-geotextile tests as shown on Table 4.8.1. In-isolation tests are often used for classification and soil-geotextile tests for design. However, this is not always the case and various parameters measured with in-isolation tests are often used for design, especially strength parameters in reinforcement applications. The groupings shown are therefore not strictly correct but useful for subsurface drainage design where it is recommended that design be based on soil-geotextile tests. The terms index tests and performance tests are also used in some publications, referring to classification and design tests respectively.

4.8.4.1 Classification tests

(a) Mass per unit area

This is useful for classifying and identifying a geotextile, also in the field.

(b) Thickness and compressibility

Geotextiles are compressible and as such thicknesses must be defined in terms of the pressure under which it is measured.

Where the geotextile is used as a filter, the thickness of the geotextile is the length of the flow path in the Darcy equation to calculate permeability. However, the thickness is very small in relation to that of the soil and it varies under different pressures so that it is more convenient to measure permeability of geotextiles in terms of rate of flow, thus ignoring the thickness of the geotextile.

Where the geotextile is used as a water carrier in its longitudinal (in-plane) direction, the thickness will determine its in-plane permeability and hence the volume of water that it can carry.

(c) Penetration load

The penetration load test (sometimes referred to as the CBR test)

uses modified California Bearing Ratio (CBR) equipment to measure the load at rupture of a geotextile. The test is relatively easy to perform and resembles the mode of failure that can be expected in the field fairly closely. The test is also a standard SABS test⁴⁸.

(d) Puncture resistance

The puncture test or drop cone test is used to measure the resistance of a geotextile to puncture when a sharp object is dropped onto it. This test is probably a closer resemblance of failure in the field than the penetration load test. The test is also relatively easy to perform but is not a standard SABS test.

(e) Tensile strength

This is the most widely used test for measuring strength and elongation characteristics of geotextiles. However, specialised equipment and skilled personnel are required to perform the test accurately, especially to avoid slippage of the geotextile in the jaws.

A disadvantage of this test is the fact that strength is measured in one direction as opposed to the penetration load and puncture resistance tests that determine an average strength. The test is a standard SABS test.

(f) Grab strength, puncture strength, burst strength, trapezoidal tear

These are all ASTM strength tests that are used in varying degrees in different parts of the world. Their use in South Africa has been very limited.

(g) Creep

Creep is the deformation of a geotextile under load over time. Creep characteristics are important in reinforcement applications, but of lesser importance in subsurface drainage applications.

(h) Normal permeability

Normal permeability is of direct importance where a geotextile is used as a filter. It is usually measured in terms of flow per unit area and, due to the high flows that are usually measured, a constant head permeameter is recommended, such as is specified in the SABS test.

(i) In-plane permeability

In-plane permeability is measured under specified confining pressures which attempt to simulate field conditions. This is usually not an important parameter in road subsurface drainage applications.

(j) Pore sizes

Various tests exist that attempt to measure the pore size in geotextiles. In most tests glass beads of varying sizes are sieved through the geotextile and the sizes of the beads are equated to the pore sizes. The pore size distributions are then used in conjunction with grain size distributions (grading curves) of the soil in the development of filter criteria similar to those for granular filters. This test method and the associated filter criteria have the following shortcomings :

- the true shapes of the pores are not necessarily the same as those of the spherical glass beads;
- factors such as compression of the geotextile and the opening of the pores under stress are not taken into account;
- the test method has not been standardised and variables such as duration of test, type of glass beads, frequency and amplitude of vibration, the use of a water spray, the water pressure and type of nozzle all have a marked influence on the test results;
- the repeatability has been reported to be poor;
- pore sizes smaller than about 0,075 mm cannot be determined in this way due to interparticle forces, and

- no evidence exists of relationships between long term performance in terms of permeability reduction (See Section 4.8.4 above) and opening sizes or filter criteria based on opening sizes.

Despite these shortcomings, this type of test and design methods that are based on such tests are widely used in various countries. An example of such a method is given in appendix B (Method B6). Any such method should be used with caution and only as a first approximation. The designer must ensure that the opening sizes that are used were determined by the same method on which the design method was based. It should also be noted that various criteria exist world-wide but that none have been developed or confirmed under local conditions.

(k) Durability tests

A large variety of tests exist to evaluate various aspects of geotextile durability. Durability is dependant on the type of polymer from which the geotextile is manufactured, rather than the manufacturing process. It is recommended that geotextiles be used that are made from polymers that are known to be durable from previous international testing programmes (e.g., polyester, UV stabilised polypropylene) rather than performing actual durability tests since such tests are difficult to standardise and to relate to durability in the field. UV resistance can be evaluated simply by measuring strengths before and after exposure to sunlight (See Table 4.8.2).

4.8.4.2 Design tests

(a) Long Term Flow Test (LTFT)

The long term flow test is a relatively simple test using a permeameter with a constant head water supply from which water flows through a soil-geotextile specimen. The flow, and hence permeability, of the system is monitored over time. The

advantages of this test method are :

- the test is relatively simple;
- the actual soil/geotextile combinations that are to be used can be evaluated in a way that closely resembles the field situation;
- the long term performance in terms of permeability reduction can be monitored;
- all the uncertainties regarding actual opening sizes and the influence thereof on the forming of a filter zone are eliminated, and
- experience in the use and interpretation of the test has been gained for South African conditions.

The main disadvantage of the test is the relatively long time involved. Testing time is typically 400 hours (See Appendix B, Test Method B3).

(b) Gradient Ratio (GR)

A permeameter setup similar to the long term flow test is used. Manometers are placed at specific points in the permeameter and hydraulic gradients are calculated. The gradient ratio is defined in terms of the hydraulic gradient of a section of soil plus geotextile relative to that of a section of soil upstream of the geotextile. The gradient ratio gives an indication of the geotextile's tendency to become clogged.

The main advantage of this method as opposed to long term flow tests, is that it can be done in a relatively short time. However, the system must reach a state of equilibrium before hydraulic gradients can be measured which can also result in long waiting periods. The main disadvantage is the fact that actual long term performance is not determined and any relationships that do exist between test results and long term performance are likely to be soil and geotextile dependant. The apparatus is also more complex than that of the long term flow test. The test has been accepted as a standard ASTM test, but criteria have not been developed or confirmed under local conditions. The test can be used as an optional extra test (See Appendix B, Test Method B4).

(c) Interface Flow Capacity (IFC)

In these tests fines from the candidate soil are introduced in increasing quantities in a column of water flowing through the geotextile. The resulting decrease in flow is monitored as being indicative of the system's tendency to become clogged. The main advantage of this test is the relatively short time of testing. The main disadvantage is the fact that actual long term performance is not determined and any relationships that exist between test results and long term performance are likely to be dependant on the specific soil and geotextile used. This test can be used as an optional extra test (See Appendix B, Test Method B5).

4.8.5 Geotextile selection

Geotextile selection for subsurface drainage applications should be based on the following considerations :

- permeability reduction (clogging)
- piping (retention)
- permeability
- strength (construction survivability)
- durability.

For a geotextile to perform satisfactorily during construction as well as during its design life, it is important that it meets the requirements for all the above.

The properties of the geotextile must be such that they will perform the desired functions over the full design life of the road, which is normally 20 to 30 years. Unfortunately the performance of very few, if any, geotextiles have been monitored over such long periods and designers must base their selection on available information. Properties that are considered to be important are summarised on Table 4.8.2 with tentative minimum requirements. These requirements must be seen as guidelines and must be adjusted if local conditions and experience indicate the need to do so. The designer of the subsurface drainage system should take responsibility for geotextile selection and the required tests should therefore be done at the design stage and not during construction.

TABLE 4.8.2 SUMMARY OF RECOMMENDED MINIMUM REQUIREMENTS FOR GEOTEXTILES IN ROAD SUBSURFACE DRAINAGE APPLICATIONS

	PROPERTY	MINIMUM REQUIREMENT	TEST METHOD
SOIL/ GEOTEXTILE	Permeability reduction	k400 in medium or high range	B3, Appendix B
	Piping	Water discoloration $\leq$ 8 hours	B3, Appendix B
HYDRAULIC	Normal permeability	Water percolation $\geq$ 20 l/m ² /s [§]	SABS 0221
STRENGTH		Installation Conditions*	
		Good	Adverse
	Penetration load	Minimum load (N) at rupture	1 500 2 400 (2 100)#
		Minimum elongation (%) at rupture	10 10
	Puncture resistance	Maximum puncture diameter (mm)	32 26
	Tensile Strength (in direction of weakest strength)	Minimum breaking strength (N/m)	12 000 (5 000)# 18 000 (12 000)#
DURABILITY		Minimum elongation (%)	10 10
	Resistance to chemical attack	Properties of geotextile must be retained in soil and ground water with a pH in the range of 4 to 12 and containing salts with a conductance of up to 1,0 S/m	
	Resistance to UV degradation Biological stability	80 per cent of strength must be retained after 1 500 hours of exposure to direct sunlight Geotextiles must be completely rot-proof and not support the growth of algae	

§ This value can be reduced if validation tests, that take the variability of the material into account, show that a lower value is required.

* Good installation conditions include relatively smooth trench walls, medium to small stone size (9,5 - 19,0 mm) dumped from the edge of the drain, a trench depth of not more than 2,0 m and careful handling of the geotextile.

Adverse installation conditions include rough trench walls, large stone (37,5 mm) dumped from a height onto the geotextile lined drain and rough handling of the geotextile.

These values are recommended by the geotextile industry based on international experience. South African experience with these values is limited, but any feedback from Road Authorities will be welcomed by the compilers of this document.

4.8.5.1 *Permeability reduction (clogging)*

The permeability of a soil-geotextile system will normally reduce, as described in Section 4.8.4, until the filter zone has been formed after which the permeability should remain fairly constant. If the geotextile is prone to clogging, due to soil particles being caught in the pores, the permeability will either reduce very rapidly, or it will continue to decrease after a state of stability should have been reached.

It is recommended that long term flow tests be used to evaluate the clogging potential of geotextiles. The use of criteria based on opening sizes is not recommended for reasons given earlier.

In most cases where the total clogging of geotextiles have been reported it has not been due to a general incompatibility of the soil and the geotextile, but rather due to a specific problem, e.g.,⁴⁴ :

- gap graded soils consisting of two discrete particle sizes : one a medium sand, the other a fine silt, where the fine silt migrated out of the remaining soil structure and clogged the geotextile;
- pavement edge drains where the hydrodynamic conditions associated with pumping has resulted in a build up of fines and hence clogging of the geotextile;
- high alkalinity ground water that result in the build up of a calcium and/or magnesium salt deposit on the geotextile;
- ground water which contains high levels of organic matter, micro-organisms or ferrous oxides; and
- landfill leachates which have a high content of both suspended solids and micro-organisms.

4.8.5.2 *Piping*

A limited amount of piping immediately after installation of the geotextile is inevitable, due to the forming of the bridging zone in the soil (See Section 4.8.2). Excessive piping will occur if the openings in the geotextile are too large relative to the soil particles and will lead to silting and possible blockage of the downstream drainage system.

If the selection of geotextiles to avoid piping is based on the criteria using opening sizes and grain sizes, it should be done with caution,

for reasons stated earlier. Piping potential can be evaluated with the long term flow test, as suggested in Table 4.8.2.

4.8.5.3 Permeability

The permeability of the geotextile must be adequate to allow a free flow of water from the soil into the drain. Its permeability must therefore be higher than that of the undisturbed soil, as described in Section 4.8.2. The value given in Table 4.8.2 can be used as a guideline.

4.8.5.4 Strength

In normal subsurface drainage applications the biggest stresses are exerted on the geotextile during installation and strength requirements must be based on expected installation conditions. The recommended values given in Table 4.8.2 can be used as guidelines. The elongation limit is aimed at reducing the risk of puncturing of the geotextile in the case of protruding rocks and rough trench walls.

4.8.5.5 Durability

The required resistance of the geotextile to chemical and biological attack must be determined by the conditions under which it will operate. Resistance to ultra violet light degradation is required for the period that the geotextile is exposed to direct sunlight. Such periods must be kept to an absolute minimum.

4.8.5.6 Other considerations

(a) Cohesive soils

Cohesive soils (PI more than 10 and Liquid limit more than 30 percent) are often very impermeable so that the quantity and energy of seepage water discharging from the soil into the drain is too small to overcome the cohesive forces in the soil. In such cases the selection of a geotextile by such methods as the long

term flow test may not be possible due to the low flow rates. Consideration can then be given to one of the following options :

- The geotextile selection can be based on previous experience; or
- The geotextile can be omitted if there is certainty that the soil structure will remain intact. If the flow rates are very low the coarse aggregate in a typical subsurface drain will also serve as a filter because piping is not a threat; or
- Use a geotextile in conjunction with sand or granular material that is compatible with the geotextile.

(b) Dispersive soils

Dispersive soils are apparently cohesive soils that lose their cohesion under water, thus resulting in migration of the fine material. If a dispersive soil has the potential for piping or for clogging a geotextile, this will occur much more rapidly than would be the case if it were not dispersive. Dispersive soils should therefore be identified⁴⁹ and tested in conjunction with the candidate geotextiles. These soils can usually be subjected to long term flow tests. The Interface Flow Capacity test was developed largely for dispersive soils and it is therefore recommended for the evaluation of these soils⁴⁵.

(c) Gap graded soils

Gap graded soils, such as silty or clayey gravel, where the fine particles do not fill the voids between the larger particles, tend to be unstable. This occurs when the fine particles smaller than 0,05 mm are less than about 30 percent by mass of the soil. A geotextile suitable for use with these soils may be difficult to obtain. A filter consisting of a combination of sand and a geotextile may solve the problem. In addition, manholes for cleaning of the drains should be provided for the removal of excess fines. The use of the IFC test⁴⁵ may also be considered for these soils.

(d) Soil sampling and testing

Soil-geotextile compatibility tests such as the long term flow test require that soil samples be taken from the site where the geotextile is to be installed. This must be done at the design stage to allow enough time for the long term flow tests.

Sampling must be based on sound statistical principles. Soil samples must be obtained from the total length of the proposed installation at frequencies that will be determined by the variability of the soil. Samples must be taken in a random way to include the total depth of installation.

4.9 SUBSURFACE DRAINAGE SYSTEMS

4.9.1 General

The three main elements of a subsurface drainage system, namely the filter, collector and conveyer, can be combined in a number of ways, some of which are shown in *Figure 22*. Conventional drains incorporate granular material either as a natural filter or as a permeable collector, whereas fin drains use a synthetic collector of which the dimensions are usually considerably less than that of a conventional drain.

Attention should be given to the selection of each element in a subsurface drainage system, as described in other sections of this document. Final choices will be determined by local conditions, available materials, local experience and cost.

4.9.2 Trench drains (Conventional)

The choice between geotextile and granular filters must be based on principles described earlier. If a geotextile filter is used, the granular material merely acts as a permeable collector and the selection thereof must be based on its permeability. Single sized aggregates between 9,5 and 19,0 mm have been found to be the most suitable (See Section 3.1.2 for typical permeability values).

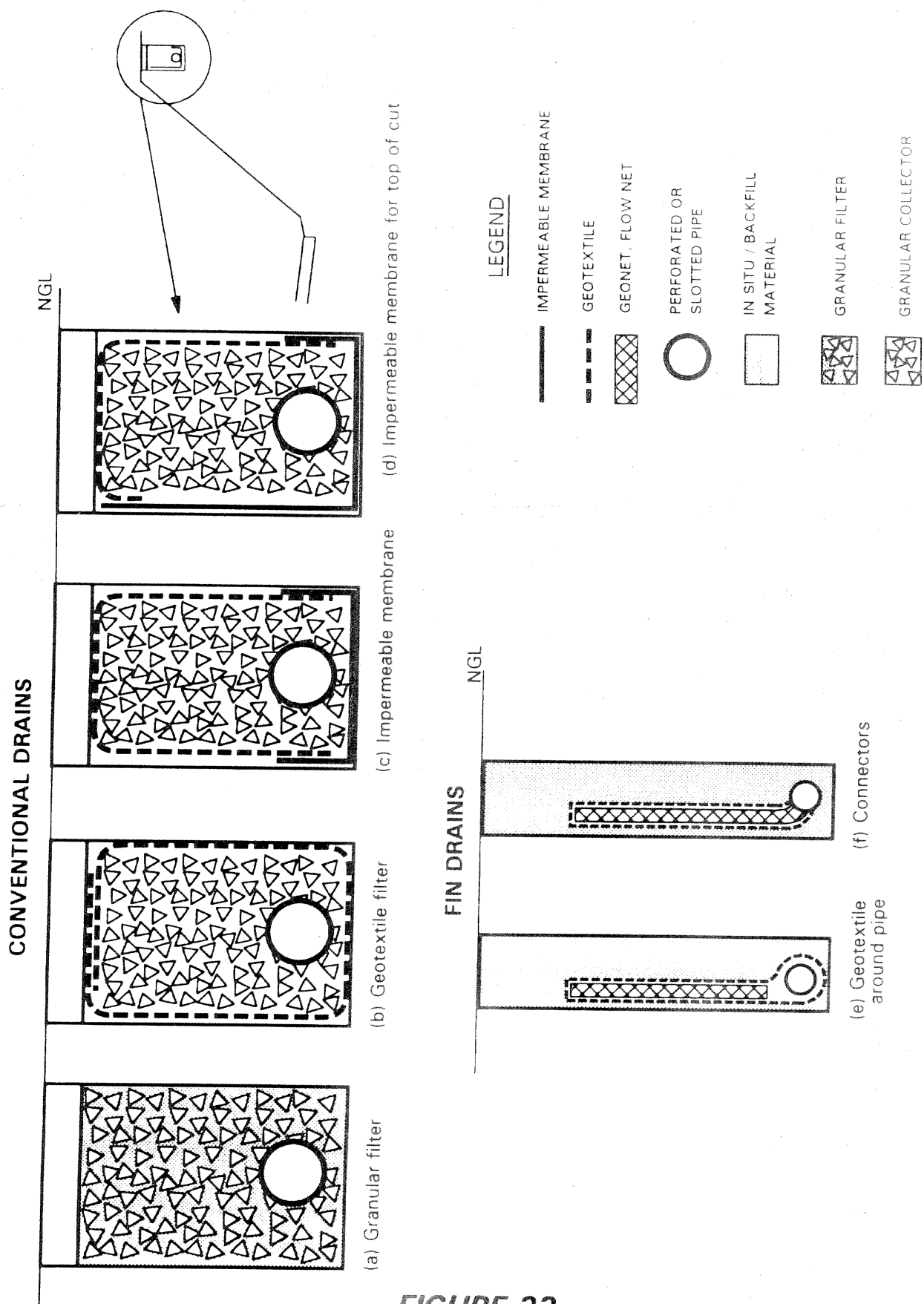


FIGURE 22

Typical subsurface drainage systems

Impermeable membranes can be used as shown on *Figure 22 (c)* to prevent water from being discharged in a potential dry zone if there is uncertainty about the extent of wet areas. Where a trench drain is used at the top of a cut, an impermeable membrane can be used as indicated in *Figure 22 (d)*.

The placing of subsurface drains over dry areas must be avoided. Where water must be conveyed from a wet zone to a discharge point some distance away, this must be done with an unperforated pipe and not with a subsurface drain.

Some typical details of trench drain dimensions and layouts are shown on *Figure 23*.

4.9.3 Fin drains

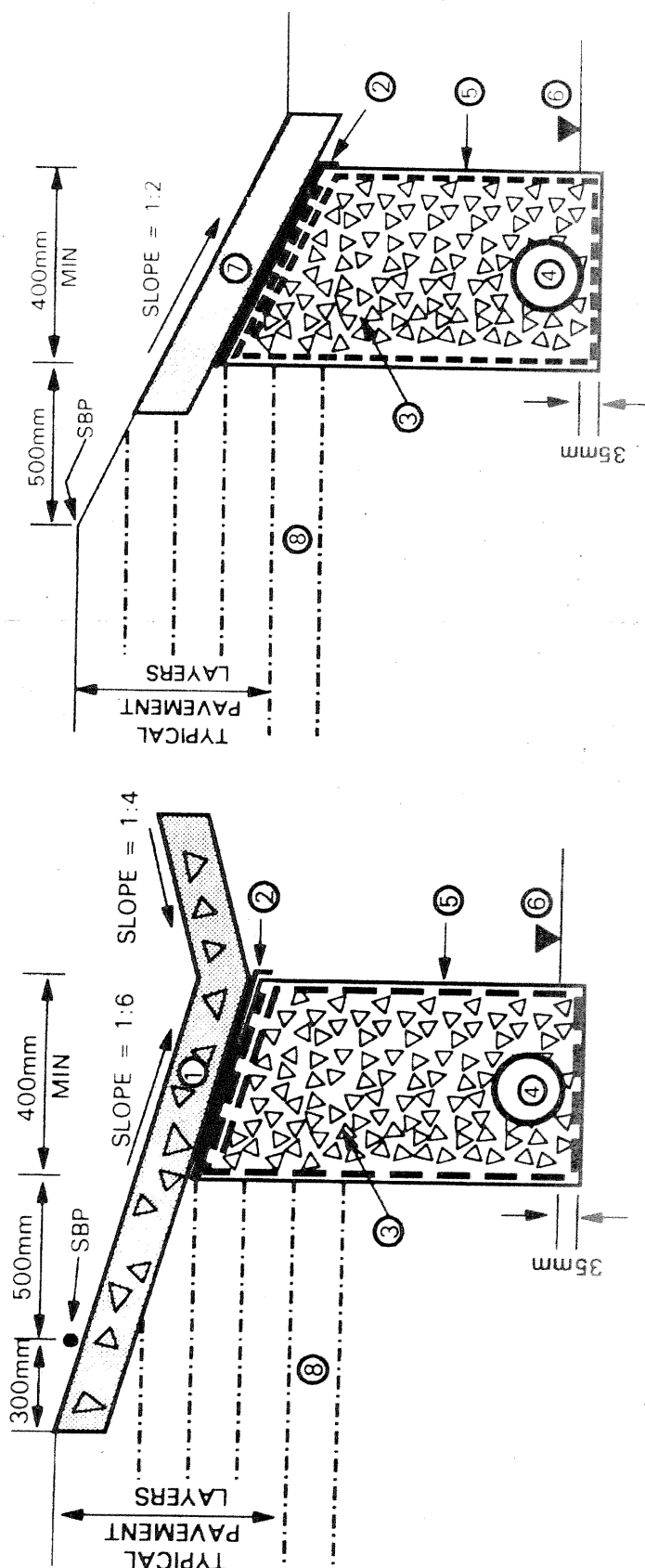
Fin drains use geotextile filters and synthetic collectors as opposed to granular materials.

The synthetic collectors are sometimes referred to as geospacers or cores. They are either net-like (flownets or geo-nets) or cusped (sometimes referred to as waffles or egg-boxes). The geotextile with the synthetic collector is sometimes referred to as a geocomposite.

The elements of a fin drain can be assembled in various ways, examples of which are shown in *Figure 22 (e), (f) and (g)*.

The main advantages of fin drains are :

- Due to the smaller dimensions, narrower trenches are required. Typical widths are 200 mm as opposed to approximately 450 mm with conventional drains. If the soil does not contain large rocks, these trenches can usually be excavated at relatively high speeds with mechanical equipment.
- Fin drains are relatively easy to construct. They are usually pre-assembled, either in the factory or on site, and the final product is installed in the trench.



(a) With a concrete side drain

(b) With an unlined earth side drain

NO.	DESCRIPTION	NO.	DESCRIPTION
①	Concrete lining	⑥	Level to which surrounding area is to be drained (typical for 12 m wide road):
②	PVC or Building paper		- 1,2 m below SBP for sandy material ($K = 10^{-1}$ cm/sec)
③	Granular filter or collector		- 1,7 m below SBP for silty sand ($K = 10^{-4}$ cm/sec)
④	Perforated pipes	⑦	Soil Cement Seal
⑤	Geotextile	⑧	Drainable selected material (if required) to be daylighted into the subsurface drain

FIGURE 23

Typical details of subsurface drains (trench drains)

- The total cost is usually lower than that of a conventional drain of the same depth.

Potential disadvantages include the following :

- Relatively little information exists on the long term performance of fin drains.
- Backfill material must be carefully selected which may result in the material having to be imported or screened. The material must be such that the fin drain is not damaged during backfill operations. It should be granular, non-cohesive ($PI < 6$) and be free from vegetation, lumps and stones of diameter exceeding 37,5 mm.
- Difficulty may be experienced with the compaction of the backfill material in the narrow trenches while the bearing capacity of flexible pipes is largely determined by the degree of compaction of the surrounding material. It may be possible to overcome this problem by using cohesionless material and hydraulic compaction methods.
- Careful supervision is required with fin drains due to the high speed of installation.

4.10 DRAINAGE LAYERS

Where the inflow of water occurs over a large area - for example by infiltration through pavement cracks, or by seepage from the natural ground - a positive means for the collection and removal of the water can be provided by a layer of permeable material.

The following criteria are generally applicable to the use of a drainage layer in any of the appropriate drainage situations :

4.10.1 Capacity

Bearing in mind that water may enter the layer over the whole surface area, but may only leave through the layer thickness, it is important to check that the discharge capacity is more than sufficient for the estimated inflow, otherwise the layer will simply act as a sponge.

The discharge can be calculated from Darcy's law :

$$q = kiA$$

where

- q = seepage quantity
- k = coefficient of permeability
- i = hydraulic gradient
- A = cross-sectional area

4.10.2 Protection

Since water will flow across the layer boundaries, the adjacent materials (geotextiles or natural materials) have to be selected to satisfy the normal criteria for filter materials, in order to prevent piping and clogging.

4.10.3 Flow

The flow line in the drainage layer must be carefully analysed, because the flow characteristics are governed by physical conditions. Unless the layer is properly aligned and confined it may only serve to irrigate areas of the subgrade which would otherwise be dry.

Stabilised layers affecting the flow path should be given special consideration, because although they seem theoretically impermeable they may be permeable due to shrinkage cracking.

4.10.4 Materials

The materials used in a drainage layer should satisfy the following requirements :

- **Permeability** sufficient to ensure the drainage capacity;
- **Workability** such that the layer can be placed without segregation, and will carry construction traffic without excessive rutting and deformation;
- **Durability** such that the materials will not deteriorate in service under repeated wet-dry cycles; and
- In the case of pavement drainage layers, **strength** sufficient, for reasons of economy, for the materials to rate as a component part of, and not just an addition to, the pavement cover design.

Materials which have been successfully used in drainage layers include^{23,51}.

- concrete sands, provided they are clean, usually in situations such as fill drainage where they have sufficient capacity in spite of their relatively low permeability;
- single size aggregates, which have high permeability, but are difficult to keep in place;
- graded aggregates, with little or no fines, bound with about 1,5 to 2 percent of bitumen to provide workability;
- under-sanded concrete mixes, gap-graded, with the minimum cement content necessary to meet compressive strength requirements; and
- crushed stone, with just sufficient fines to be workable and stable under vibro compaction, with or without a cement additive. The following grading (also shown on *Figure 24*, superimposed on recommended grading curves for effective vibro compaction), has been found to work well, particularly as a bottom selected subgrade layer, without a cement additive⁵² :

SIEVE SIZE <i>mm</i>	GRADING ENVELOPE <i>percentage passing by mass</i>
53,0	100
37,5	74 - 100
26,5	55 - 98
13,2	40 - 67
4,75	24 - 36
2,00	15 - 20
0,425	4 - 9
0,075	0 - 4

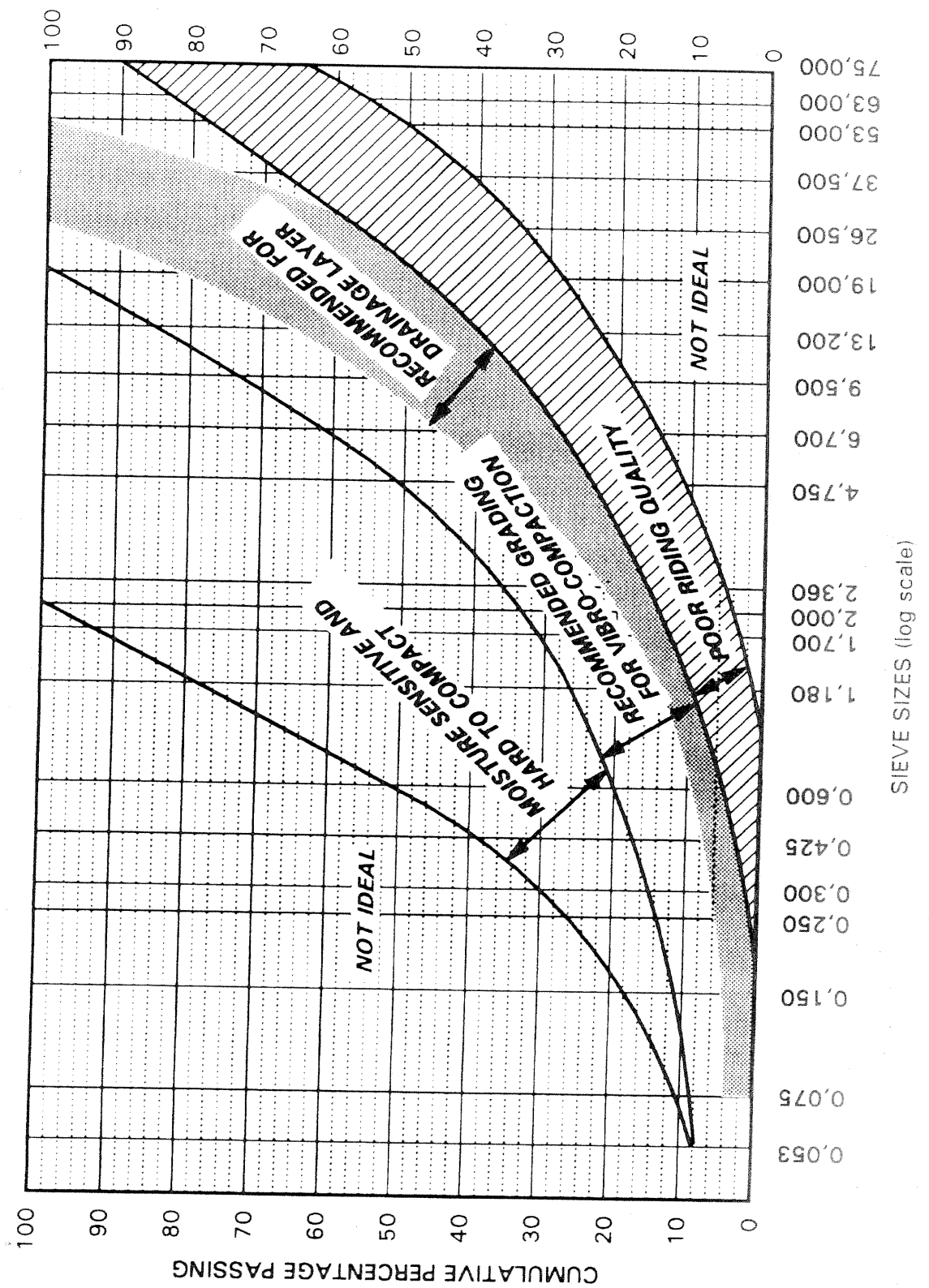


FIGURE 24
Recommended grading for drainage layer

In any particular situation the choice of materials will obviously depend on the most economical arrangement of materials to give the design capacity, but the preference should be for the naturally high capacity of relatively single sized aggregates, provided the problem of workability can be solved by choke stone, or with bitumen or cement.

5. CONSTRUCTION

5.1 GENERAL

Most failures of subsurface drainage systems are not due to inadequate or improper design, but due to construction errors that are easily avoidable. In many cases the lack of adequate supervision or quality control is the main contributing factor to a failure. Particular attention should therefore be given to construction practices and inspections should be carried out after every element of the system has been installed. **The failure of any one element will lead to the failure of the total system.**

5.2 HANDLING AND STORAGE

All the components must be handled and stored in such a way as to avoid any form of damage or contamination. Synthetic materials should not be exposed to sunlight for long periods. Any materials that do become damaged or contaminated in any way should be removed and discarded.

5.3 EXCAVATION

Excavation of the trenches must be done in such a way that damage by storm water is avoided. The sides and bottom of the trench must be smooth and without cavities or protruding items such as rocks or tree stumps. The grade and level must be carefully controlled. It may be necessary to import sand or small sized aggregate to correct the profile where over-excavation has occurred.

5.4 GEOTEXTILES

Particular care should be exercised with all aspects of geotextile installation. One of the main disadvantages of geotextiles as opposed to granular filters is the fact that they are less "forgiving" and their effectiveness is seriously affected if damaged in any way.

5.4.1 Handling, site acceptance and storage

Geotextiles must be handled in such a way as to ensure delivery to the place of installation in a sound undamaged condition. During shipment and storage geotextiles must be wrapped in undamaged protective covering.

Exposure to direct sunlight must be kept to an absolute minimum. The total time of exposure during shipment, storage, installation and delay prior to backfilling should not exceed 14 days.

The person responsible for accepting the geotextiles, must ensure that the product that was ordered is delivered on site. Each roll should have a clear identification of its type and grade.

The storage area should be flat, well drained and covered. Geotextiles should not be placed on an uncovered soil surface and must be protected from mud, soil, dust and any other potential source of contamination. Any petroleum products, e.g., fuel, oil or bituminous products should be kept well away from geotextiles.

The height to which rolls are stacked must be limited to enable ease of handling and to avoid damage.

5.4.2 Ground preparation

The sides and bottoms of trenches, or any other areas where geotextiles are to be laid, must be evenly graded to obtain a smooth surface so that the geotextile will not bridge cavities or be damaged by protruding rocks, tree stumps, vegetation or any other irregularities.

5.4.3 Placing of geotextile

Any portion of geotextile that may have become damaged during storage or handling by tearing, creasing, compression or soiling, must be removed and discarded prior to installation. Should any portion become damaged during installation it should also be discarded.

Geotextiles must be placed manually or by various mechanical means in such a way that they are laid flat, but not stretched. There should be no creases and good contact must be obtained between the geotextile and the soil without any cavities. These precautions are important because they will determine the effective forming of the bridging and filter zones.

Where more than one sheet is required for the width or length of installation it is essential that the joints be secured in a suitable manner to maintain the integrity of the installation. The method adopted must take into consideration the type of installation, the functions which the geotextile is required to perform, and stability of the ground supporting the material. Jointing may be done by overlapping, sewing, stapling, glueing or heat welding.

Where jointing is done by overlapping the amount of overlap should be governed by the bearing capacity of the soil, the soil profile and the stress that is likely to be induced in the geotextile. The overlap should not be less than 0,3 m when laid on a level, firm surface, but should be increased to 1,0 m, or more, on soft, uneven ground, depending on the severity of the conditions. The direction of overlap should be such as to prevent lifting by the initial fill material and by prevailing winds.

The closure of the geotextile at the top of the trench should be such as to provide two thicknesses of geotextile over the top of the trench firmly secured with mechanical ties.

5.4.4 Backfilling

All geotextile installations should be inspected and approved before backfilling. Any defective or damaged fabric should be rejected and replaced.

Backfilling should be done as soon as possible after the geotextile is in place to minimise exposure to sunlight.

Backfilling must be done in such a way as to prevent damage to the geotextile. The height of dumping must be kept to a minimum,

particularly with larger sized aggregates and aggregates that have sharp edges. Material used for backfilling against the geotextile should not have stones larger than 37,5 mm, lumps or pieces of vegetation and should be non-cohesive ($PI < 6$). Where such material is not available, consideration can be given to incorporating a layer of selected material to act as a cushion. Aggregates between 9,5 mm and 19,0 mm are the most suitable as a drainage medium when used in conjunction with a geotextile filter.

5.5 PIPES

Pipes must be laid true to level and grade. Particular attention should be given to this if the more flexible pipes are used. The bedding must provide a smooth, continuous support. The resistance of flexible pipes to crushing and deformation is largely determined by the surrounding material. Where the pipe is not surrounded by granular material, such as at the outlets and where fin drains are used, the backfill material should be carefully selected and compacted (See Section 5.8).

Pipe sections must be securely joined using the couplings recommended by the manufacturers. Particular care must be taken to ensure that there is no silting in the pipes before they are joined. The upper end of the pipe system must be sealed off with a suitable end cap.

5.6 FIN DRAINS

Fin drains are usually pre-assembled, either in the factory or at the site. Care should be exercised to ensure that all the parts are positively connected as designed. Particular attention should be given to the method by which water is conveyed into the pipe. Where connectors are used they should be checked to ensure proper functioning. Overlap of the geotextile at the top and at the sides must be provided to avoid silting. The assembly should be done on a clean, flat surface to avoid any form of damage or contamination.

The fin drain must be carefully lowered into the trench and secured to

the side from which the water is expected to flow. The method used for holding the fin drain in position must be such that no damage is caused to any of the elements, particularly the geotextile. The fin drain should be carefully inspected after installation to ensure that all connections are firm and in place.

Particular care should be exercised during backfilling. The material should be selected and compacted (See Section 5.8) in such a way that no damage is caused to the fin drain.

With pavement edge drains the drain should be placed on the shoulder side of the trench rather than on the pavement side and fine aggregate should be used for backfill.⁵³

5.7 GRANULAR FILTERS AND COLLECTORS

Granular filters and collectors should be placed in layers of approximately 150 mm and lightly compacted. This must be done with care to avoid any damage to the geotextile or the pipe. Contamination of the granular material by any soil (e.g., by construction workers walking on uncovered material) should be avoided and if contamination should occur, the material should be replaced to its full depth.

5.8 BACKFILL

All material surrounding pipes, geotextiles, and fin drains, i.e., the bedding, the sidefill and the material directly above the pipes, must be carefully selected and compacted. It should be a soil of a granular non-cohesive nature ($PI < 6$) that is free from vegetation, lumps and stones of diameter exceeding 37,5 mm.

The material surrounding the pipe as well as any backfill above the pipe system should be compacted in layers to obtain a density at least equal to that of the surrounding material. A density of 90% Mod.AASHTO is usually sufficient, but where the drain is in close proximity to the pavement layers, e.g., with a pavement edge drain, a

density of 93% Mod. AASHTO should be obtained. Densities can be controlled by devices such as the dynamic cone penetrometer (DCP). The choice of compaction equipment should be such that no damage is caused to the pipe. The backfill material should also be placed and compacted in such a way as to avoid damage to any components of the drain.

Where over-excavation has occurred due to, e.g., excavation in rocky material or blasting in hard rock, some method of forming, such as steel boxing or casing, may be used to ensure that the final dimensions of the drain are as designed. This will also protect the geotextile from damage against the uneven sides of the trench.

5.9 INSPECTION

Every phase of the construction should be carefully inspected before the next phase is allowed to continue. Attention should be given to any form of damage or contamination and unsuitable materials should be replaced. Connections and overlaps should be checked to ensure that they are as designed and firmly in place. The grade and level of the pipes should be carefully checked.

After completion of the whole installation the pipes should be tested to ensure that water flows through the whole system. A special cleaning eye can be provided for this purpose or the upper end can be kept open and the end cap removed to allow the water to be pumped into the system and the outflow checked at the completed outlet.

6. MAINTENANCE

6.1 GENERAL

The effective maintenance of road subsurface drainage systems is no less important than that of other elements of a road. A maintenance programme that is systematically planned, executed and documented will not only ensure optimum performance of the systems, but also provide invaluable information for future designs.

The three main problems that can be expected to occur and result in the system to become inoperative are⁷ :

- clogged filter;
- blocked pipes; and
- blocked outlet.

6.2 DESIGNING FOR EFFECTIVE MAINTENANCE

A number of aspects need to be considered at the design stage of a subsurface drainage system to minimise the need for maintenance and to facilitate effective maintenance where it is needed⁷.

6.2.1 Filter design and construction

A clogged filter can normally only be detected when the road begins to fail. A filter that results in excessive piping due to too large openings or as a result of damage during construction, will result in blocked pipes. These problems can only be rectified by replacing the filter and must be avoided by careful filter design and by proper construction techniques as outlined in Sections 4 and 5 respectively.

6.2.2 Pipes

Blocked pipes can be avoided by proper filter design, the provision of adequate grades and the necessary care with construction. However,

some silting is usually inevitable and provision should be made for flushing the pipe system by providing cleaning eyes at regular intervals (Typical detail on *Figure 17*). Consideration should also be given to providing manholes at junctions, where direction changes occur and if the system is particularly long. Cleaning eyes and manholes should be designed and positioned in such a way that they will not allow storm water to enter the subsurface drainage system. They should also be accessible to maintenance personnel, stable and not interfere with other road maintenance activities. The position of cleaning eyes and manholes should be clearly marked, both on the as-built plans and in the field using suitable marker pegs.

6.2.3 Outlets

Outlets should be located where they are accessible and, if possible, visible from the road surface. They must also be clearly marked with marker pegs and on the drawings. The outlet should be constructed with adequate protection against scour and rodent invasion. The height of the outlet must be well above the surrounding terrain to ensure that the outflow can be observed and measured and not be obstructed by silt deposits and vegetation. Where storm water and subsurface drainage outflows are accommodated in the same outlet structure, it should be done in such a way that they can be monitored separately.

6.2.4 As-built records

The effectiveness of road subsurface drainage maintenance is largely determined by the accuracy and detail of as-built records. They should include accurate positions of all cleaning eyes, manholes, junctions and outlets and design details such as materials and dimensions. The results of any design test, e.g., long term flow tests, should also be recorded.

6.3 MAINTENANCE ACTIVITIES

Maintenance activities related to subsurface drainage consist of :

- inspections;
- data recording; and
- maintenance actions.

Decisions on maintenance needs and the resulting actions can only be made if inspections are done regularly and if the information thus obtained is recorded accurately. The records must also be maintained for the entire life of the system to determine trends that may affect decisions on maintenance. For example, if maintenance records show that silt deposits at the outlet is a continuous problem, it may indicate the need to flush the pipe system or that a tear in the geotextile needs to be repaired.

6.3.1 Outlets

The following items should be recorded when outlets are inspected :

- the outflow rate or, at least, if any outflow is evident;
- the existence of silt deposits;
- any evidence of scouring;
- vegetation growth that affects the outflow or the visibility and accessibility of the outlet;
- any other blockages that obstruct the outflow;
- any damage to the outlet structure; and
- the condition of the rodent protection.

Maintenance actions that may result from the inspection of outlets include :

- removal of vegetation;
- removal silt deposits;
- removal of blockages;
- replacing rodent protection;
- repair of any damage to the structure or that caused by scouring;

- relocation of the outlet structure;
- flushing of the pipe system;
- location and repair of damaged geotextile or pipe; and
- replacing the filter or the whole system.

6.3.2 Cleaning eyes and manholes

Items to be recorded during inspections include :

- any damage to the structure, pipes, covers, caps or locking devices;
- any missing items, e.g., locks or caps;
- the condition of any hinges or screw thread;
- any evidence of vandalism, e.g., litter forced down the pipes; and
- any evidence that surface runoff can flow into the subsurface drainage system.

Maintenance actions that may result from the above include :

- repair of any damage;
- replacement of locks, caps or covers;
- greasing or oiling of hinges and threads;
- removal of any foreign material, e.g., litter;
- securing of the cleaning eyes or manholes against vandalism; and
- protection against surface run off.

6.3.3 Marker Pegs

Inspections of marker pegs should include :

- the existence of marker pegs;
- visibility from the road;
- condition of painting and markings; and
- any damage to the peg or its foundation.

Possible maintenance actions include :

- removal of vegetation;
- repainting;
- repair of any damage; and
- replacing of marker pegs.

6.3.4 Effectiveness of the system

It is quite possible that no outflow is ever recorded during routine inspections because they are done during dry periods, or because there is no longer a ground water problem at the particular site. In either case it would be erroneous to conclude that the system is ineffective and needs to be repaired or replaced. It is, therefore, necessary to make an assessment of the road condition at each subsurface drainage site at regular intervals. It should be done by an experienced pavement evaluator and the condition of the pavement should be assessed relative to that of a "dry" section of road adjoining the "wet" section. This should be the final test for the effectiveness of a subsurface drainage system.

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APPENDIX A : DESIGN EXAMPLES

EXAMPLE A1 : INTERCEPTING SEEPAGE ALONG AN INCLINED AQUIFER (ADAPTED FROM GERKE⁷)

This example shows the basic principles of using permeability and Darcy's equation to calculate the discharge from a drainage system intercepting an inclined aquifer, as shown on *Figures A1-1 and A1-2*. The example illustrates a very special situation, namely where the aquifer thickness is known and limited and where saturated flow occurs. However, the solution for the problem that usually occurs, namely general seepage under unsaturated conditions, is complex and requires inputs that are as yet unquantified. It is therefore recommended that this method be used as a first approximation and that adjustments be made on the conservative side to accommodate the unknown factors.

A1.1 DEFINITIONS

k_a	=	permeability of aquifer
k_f	=	permeability of filter material in the trench
T	=	thickness of aquifer
W	=	width of trench
S	=	slope of aquifer
q	=	discharge (flow) per metre of pipe

A1.2 EXAMPLE

k_a	=	1×10^{-5} m/s
T	=	1,5 m
W	=	0,4 m
S	=	10% = 0,1

Determine the maximum discharge collected by a 1 m length of trench and the minimum permeability of the filter material in the trench.

(a) Maximum flow will occur when the aquifer is totally saturated.

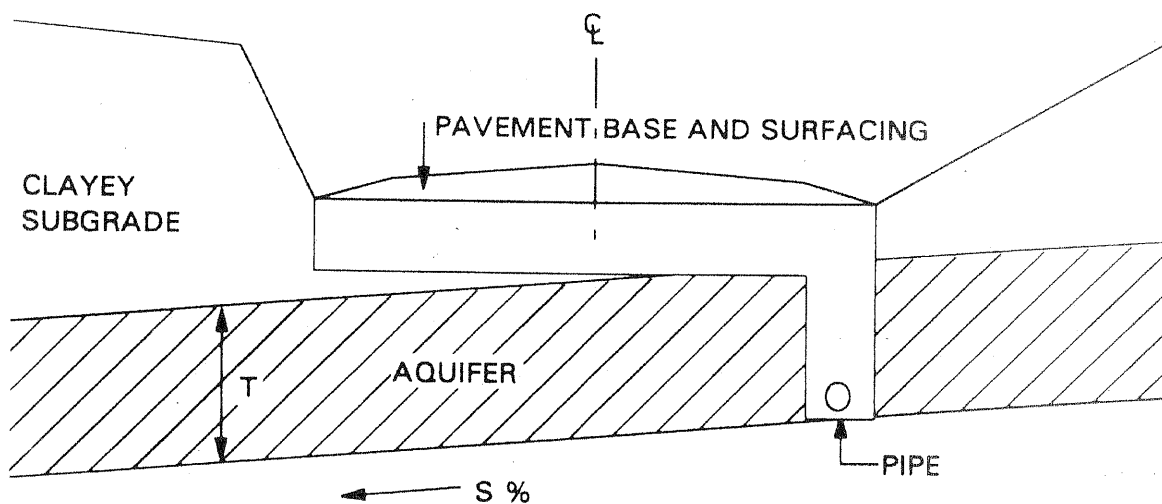


FIGURE A1-1

Subsurface drain to intercept ground water seepage

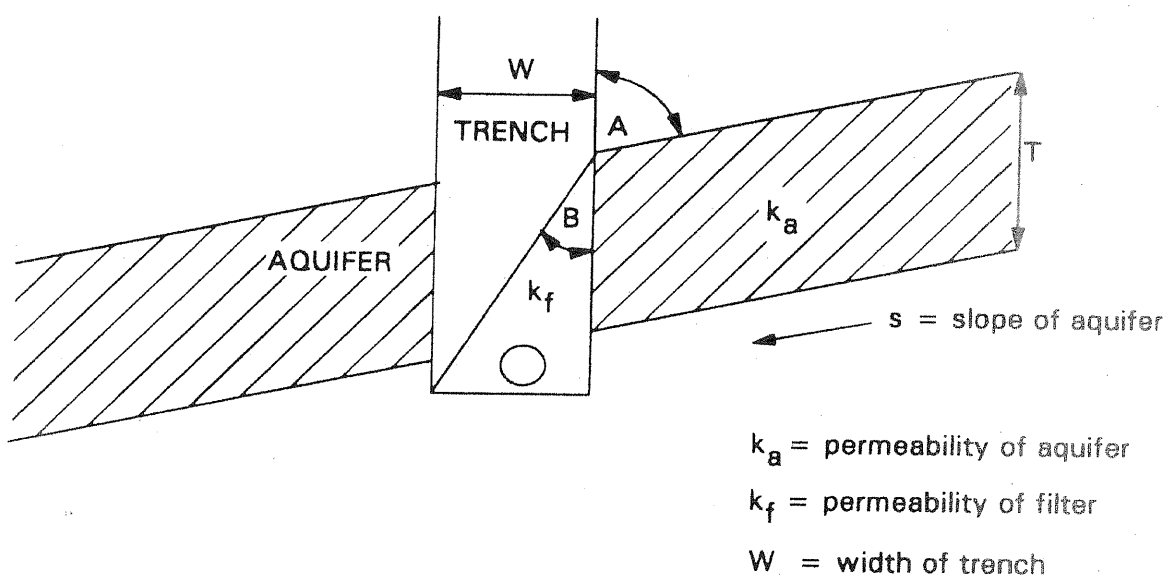


FIGURE A1-2

Trench excavated through an inclined aquifer

For saturated flow Darcy's law applies :

$$q = kiA$$

i = slope of the aquifer (valid if piezometric heads within the aquifer are equal)

$$= 0,1$$

$$A = T \times 1 = 1,5$$

$$\begin{aligned}\text{then } q &= 1 \times 10^{-5} \times 0,1 \times 1,5 \\ &= 1,5 \times 10^{-6} \text{ m}^3/\text{s}\end{aligned}$$

(b) From *Figure A1-2*

$$\frac{\tan B}{\tan A} = \frac{ka}{kf}$$

$$\tan B \cong W/T \text{ and } \tan A = 1/s$$

$$\frac{W \times s}{T} \cong \frac{ka}{kf}$$

$$\text{Then } f = \frac{ka \times T}{W \times s}$$

$$\begin{aligned}Kf &= \frac{1 \times 10^{-5} \times 1,5}{0,4 \times 0,1} \\ &= 3,7 \times 10^{-4} \text{ m/s}\end{aligned}$$

A1.3 COMMENTS

- The above example applies if the depth of the trench is approximately equal to the thickness of the aquifer. Often the thickness of the aquifer is more than the trench depth or unknown. If this method is used in such a case the flow into the system will be under-estimated because water will also enter from the bottom of the trench.
- The discharges obtained are maximum flows for saturated conditions. For unsaturated conditions, flows will be considerably less.

- If a specific aquifer does not exist and the side drain is being installed to intercept general seepage from higher ground, the slope can be assumed to be that of the ground. However, for such applications saturated flow is unlikely to occur and a reduced discharge rate must be used.

EXAMPLE A2: DESIGN OF A PIPE SYSTEM TO LOWER A STATIC WATER TABLE (McCLELLAND¹⁹)

This example illustrates McClelland's design method for lowering a static water table with a pipe system. The method is empirical and is based on model studies that were done on a system such as that shown on *Figure A2-1*. The dimensionless factors shown on *Figure A2-2* and *A2-3* are used.

A2.1 DEFINITIONS

- k = permeability coefficient of soil (m/s)
- m = non capillary porosity or the volume of drainable water per unit volume of soil (dimensionless). Typical values are 0,05 for sands and 0,02 for clays
- D = drain depth, measured from the original ground water surface to the bottom of the drain (m)
- W = drain spacing or distance between drains (m)
- T = depth of the impervious boundary below original ground water level (m)
- t = time after drainage system is installed (s)
- q = discharge (flow) per metre of pipe at time t (m^3/s)
- d = depth of the ground water surface after time t (m)

A2.2 EXAMPLE

Determine the flow in the pipes and the depth of the ground water surface after 1 day, for the following :

- k = 1×10^{-6} m/s
- m = 0,03
- W = 8,0 m and $W = 12,0$ m
- D = 1,0 m
- T = 4,0 m

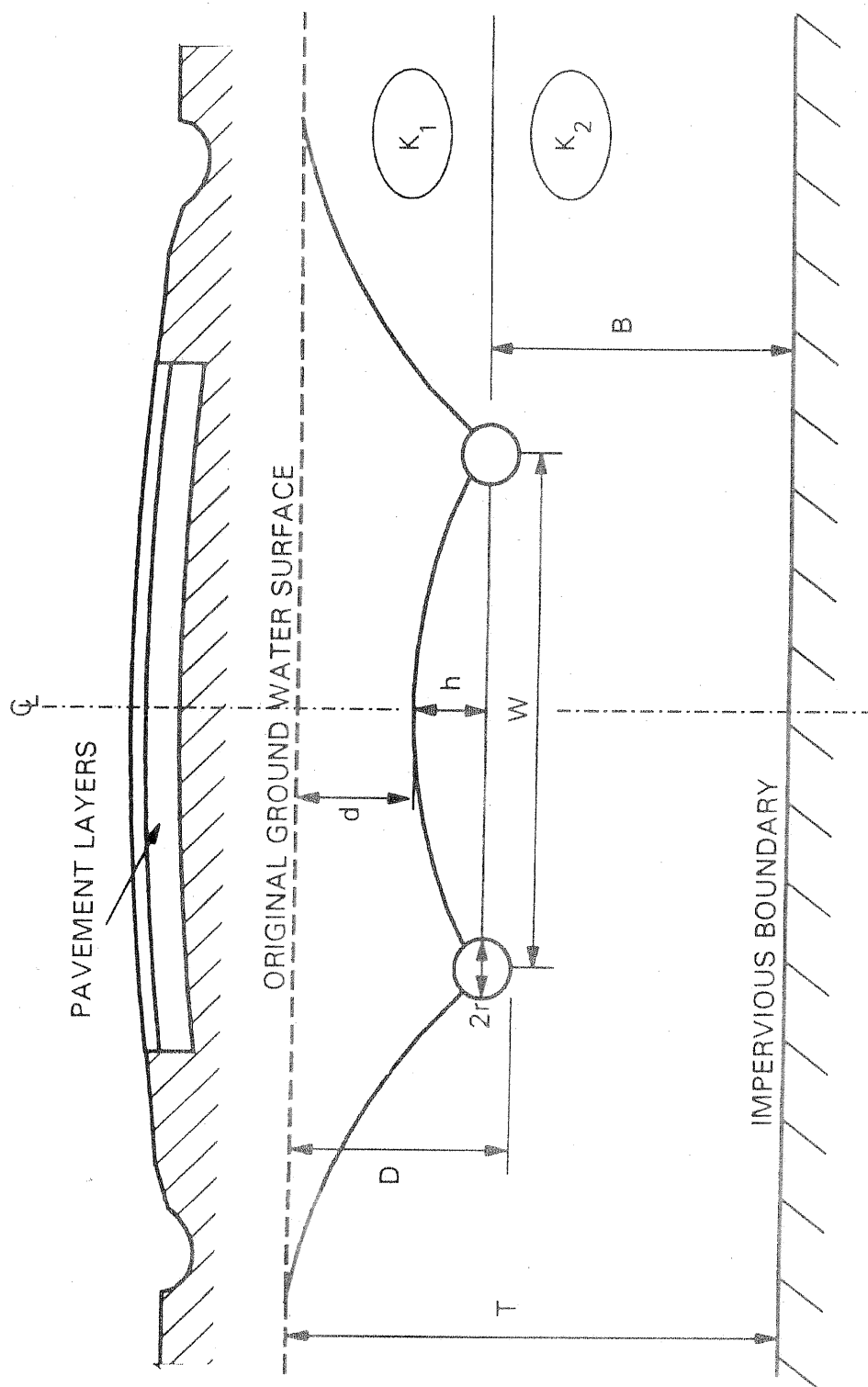


FIGURE A2-1

Design of a pipe system to lower a static water table

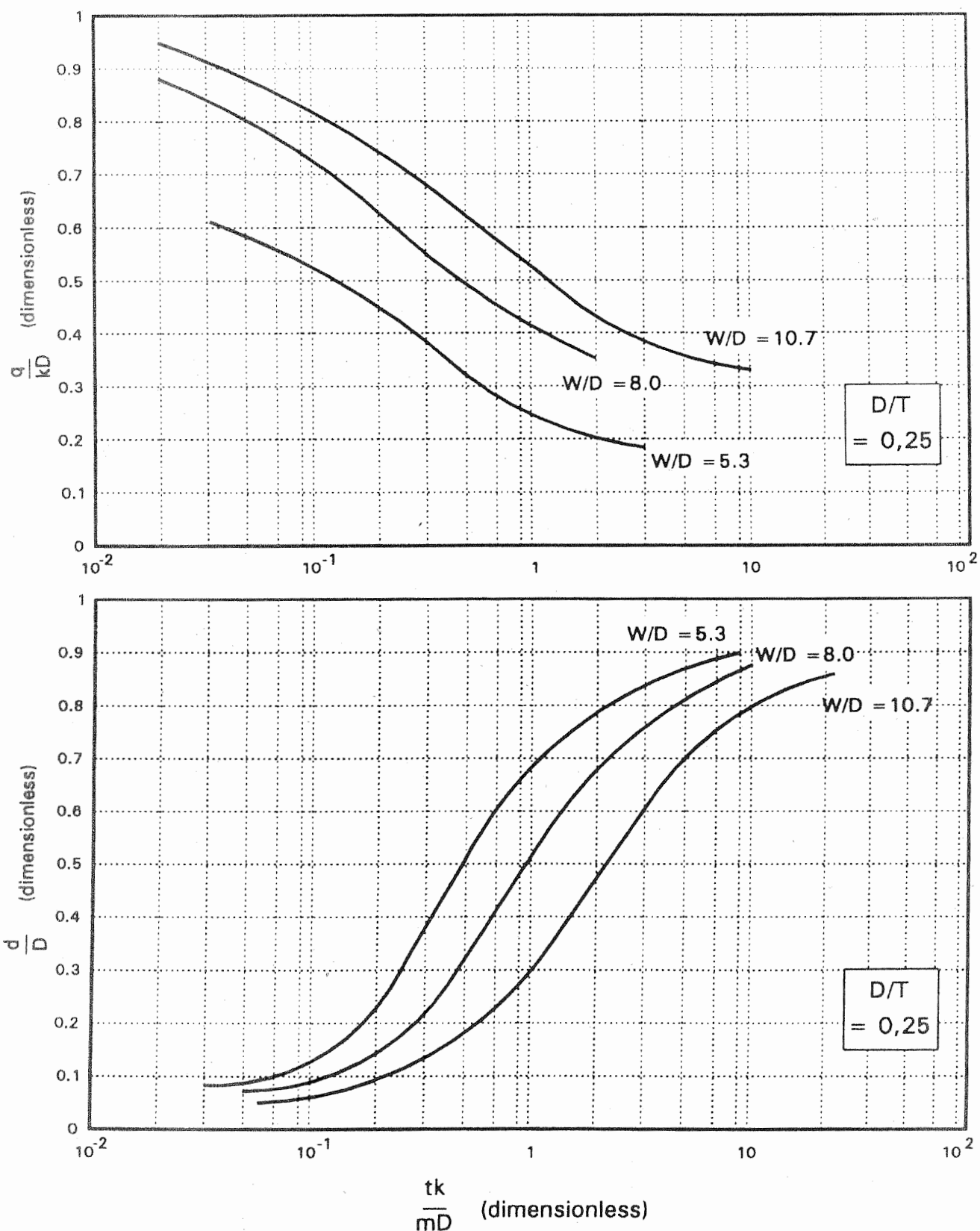


FIGURE A2-2
Effect of drain spacing^{6,7}

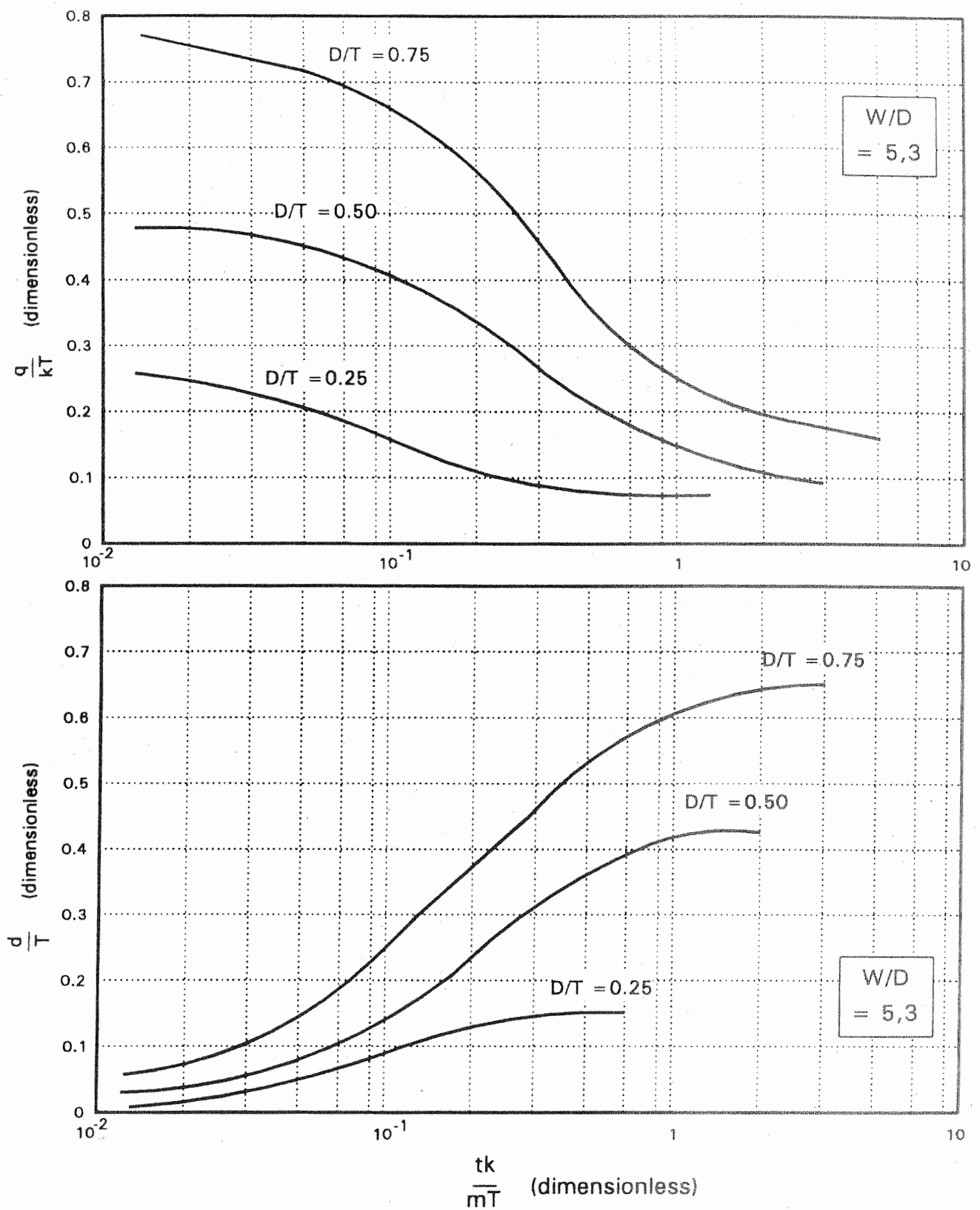


FIGURE A2-3
Effect of drain depth^{6,7}

$D/T = 0,25$, therefore *Figure A2-2* is applicable without further adjustments for depth of the impervious boundary.

(a) For $W = 8,0$ m :

$$W/D = 8,0$$

$$t = 1 \text{ day} = 86\,400 \text{ s}$$

$$\frac{tk}{mD} = \frac{(86400 \times 1 \times 10^{-6})}{(0,03 \times 1,0)} = 2,88$$

$$\text{From Figure A2-2(a) : } \frac{q}{kD} = 0,3$$

$$\begin{aligned} \text{then } q &= 0,3 \times 1 \times 10^{-6} \times 1,0 \\ &= 3,0 \times 10^{-7} \text{ m}^3/\text{s per metre of pipe} \end{aligned}$$

$$\text{From Figure A2-2(b) : } \frac{d}{D} = 0,75$$

$$\text{Then } d = 0,75 \times 1,0 = 0,75 \text{ m}$$

(b) For $W = 12,0$ m

$$W/D = 12,0$$

Figure A2-2(a) must be extrapolated by adding a line for $W/D = 12$.

$$\frac{tk}{mD} = 2,88 \text{ as before}$$

$$\text{then } \frac{q}{kD} = 0,45$$

$$\text{and } q = 4,5 \times 10^{-7} \text{ m}^3/\text{s per metre of pipe}$$

From *Figure A2-2(b)* (extrapolated)

$$\frac{d}{D} = 0,43$$

$$d = 0,43 \times 1,0 = 0,43 \text{ m}$$

A2.5 COMMENTS

- *Figure A2-2* is applicable when $D/T = 0,25$. *Figure A2-3* shows the effect of other values of D/T and can be used to adjust the values obtained from *Figure A2-2*, if required.
- In the above example fixed values were given for drain width and depth. However, this method can be used to determine optimum positions of drains.

EXAMPLE A3 : DESIGN OF A PIPE SYSTEM TO LOWER A STATIC WATER TABLE (HOOGHOUDT²⁰)

This is an alternative method that can be used to design a pipe system using formulae.

A3.1 DEFINITIONS

Referring to the layout shown on *Figure A2-1* (Example A2), the following definitions apply :

- K_1 = Permeability coefficient of soil above the drains (m/day)
- K_2 = Permeability coefficient of soil below the drains (m/day)
- W = Drain spacing or distance between drains (m)
- B = Depth of impervious boundary below drains (m)
- h = Final height of water table above centre line of drains when system has reached a state of equilibrium (m)
- r = Radius of drainage pipe (m)
- q = Drainage factor = quantity of water that must be removed from the area that is drained by the system (m/day)
- and δ = depth of an "equivalent layer", which caters for radial flow into the pipe

$$\text{Then} \quad W^2 = \frac{8K_2 h \delta}{q} + \frac{4K_1 h^2}{q}$$

$$\text{and } \delta = \frac{W}{8 \left[\frac{(W - B\sqrt{2})^2}{8BW} + \frac{1}{\pi} \ln \frac{B}{r\sqrt{2}} \right]}$$

A3.2 EXAMPLE

Using the same data that was used for Example A2 (McLelland method), calculate the final height of the water table above the pipes,

and compare with the results obtained in Example A2.

$$K_1 = K_2 = 1 \times 10^{-6} \text{ m/s} = 1,16 \times 10^{-11} \text{ m/day}$$

$$W = 8,0 \text{ m and } 12,0 \text{ m}$$

$$r = 0,150 \text{ m (not need in example A2)}$$

$$B = T - D + r = 3,15 \text{ m}$$

q = the total flow and can be chosen to represent the total infiltration from the surface. For this example the flow that was calculated in Example A2 is used

For $W = 8,0 \text{ m}$:

$$q = 3,0 \times 10^{-7} \text{ m}^3/\text{s per metre of pipe.}$$

The total area drained per metre of pipe = $8 \times 1 \text{ m}^2$

$$\begin{aligned} \text{then } q &= \frac{3,0 \times 10^{-7}}{8 \times 1} \text{ m/s} = 3,75 \times 10^{-8} \text{ m/s} \\ &= 4,34 \times 10^{-13} \text{ m/day} \end{aligned}$$

$$d = \frac{8}{8 \left[\frac{(8 - 3,15\sqrt{2})^2}{8 \times 3,15 \times 8} + \frac{1}{p} \ln \frac{3,15}{0,15\sqrt{2}} \right]}$$

$$= 1,086$$

$$\text{From } W^2 = \frac{8K_2 h \delta}{q} + \frac{4K_1 h^2}{q} \text{ and } K_1 = K_2$$

$$h = 0,2474 \text{ m}$$

By comparison, in Example A2 :

$$\begin{aligned} h &= D - d - r = 1,0 - 0,75 - 0,15 \\ &= 0,10 \text{ m} \end{aligned}$$

(b) For $W = 12,0$ m

$$q = \frac{4,5 \times 10^{-7}}{12 \times 1} = 3,75 \times 10^{-8} \text{ m/s}$$
$$= 4,34 \times 10^{-13} \text{ m/day}$$

$$\delta = 1,433$$

$$h = 0,4110 \text{ m}$$

By comparison, in Example A2 :

$$h = D - d - r$$
$$= 1,0 - 0,43 - 0,15$$
$$= 0,42 \text{ m}$$

EXAMPLE A4 : CALCULATION OF PERMEABILITY USING THE MOULTON FORMULA

It must be stressed that this method gives a first indication of the permeability. Particle shape and orientation can have a significant influence on the actual permeability.

A4.1 DEFINITIONS

As given in Section 3.1.2.

A4.2 EXAMPLE

A G1 crushed stone base is compacted to 86% of apparent density. The grading parameters are :

$$\begin{aligned} D_{10} &= 0,15 \text{ mm} \\ P_{075} &= 8\% \end{aligned}$$

Determine the permeability of the compacted layer.

Apparent density can be taken as being equal to specific gravity, since the internal voids in the particles will have no influence on permeability

$$\text{then } n = -\frac{0,86}{1,0} = 0,14$$

$$\begin{aligned} \text{and } k &= \frac{1,94 \times (0,15)^{1,478} \times (0,14)^{6,654}}{(8)^{0,597}} \\ &= 7,07 \times 10^{-8} \text{ cm/sec} \end{aligned}$$

APPENDIX B : TEST AND DESIGN METHODS

TEST METHOD B1 : BOREHOLE METHOD TO DETERMINE PERMEABILITY OF A UNIFORM SOIL MASS IN THE FIELD¹²

Method :

Drill a hole to a depth of approximately 600 mm (at least 200 mm, not more than 2 000 mm) below the water table. Empty the hole a few times and then wait until the water table has stabilised at its original level (Approximately 2 hours for sand and 12 hours for clay). Pump the water out rapidly to lower the water table by approximately 300 mm. Record the rate at which the water table returns by recording height and time.

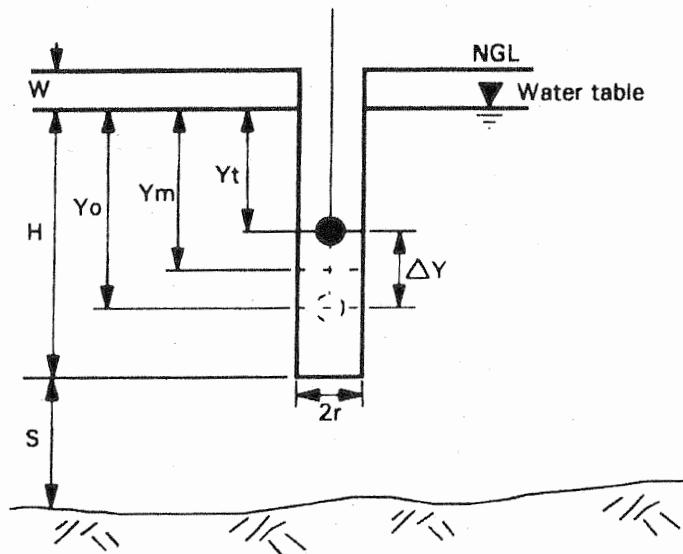
The following conditions must be adhered to :

$$30 \text{ mm} < r < 70 \text{ mm}$$

$$200 \text{ mm} < H < 2\,000 \text{ mm}$$

$$\Delta Y \leq 0,25 \text{ Yo}$$

$$0,2 H < Y_m$$



Where :

H = Depth of hole below the original water table

r = radius of hole (mm)

ΔY = Height that the water table rises (in mm) in the time interval ΔT
(in seconds)

= $Y_o - Y_t$

Y_m = Distance from the original water table to the average water level
(in mm) in the time interval ΔT (in seconds)

= $\frac{DY}{2} + Y_t$

S = Depth of an impervious layer below the bottom of the hole (mm)

Then : The permeability coefficient, K in m/day can be calculated as follows :

$$\text{If } S > H : K = \frac{400 r^2}{(H + 20 r) \left(2 - \frac{Y}{H}\right)} \times \frac{\Delta Y}{\Delta T} \text{ (m/day)}$$

$$\text{If } S = 0 : K = \frac{360 r^2}{(H + 10 r) \left(2 - \frac{Y}{H}\right)} \times \frac{\Delta Y}{\Delta T} \text{ (m/day)}$$

All lengths in mm and time in seconds.

TEST METHOD B2 : PUNCTURE RESISTANCE

B2.1 PURPOSE

To measure the resistance of geotextiles to puncture.

B2.2 APPARATUS

(i) Clamping device

A suitable means of clamping and positioning the specimen, consisting of :

- a mounting-platen/supporting-sleeve assembly; and
- an upper and a lower clamping ring provided with the necessary securing bolts, guide pins and V-grooved concentric matching rings for securing the specimen.

(ii) Mounting device

A suitable device to ensure that, when the specimen is positioned in the clamping rings, it is in a true plane, free from folds and stress, which device shall consist of :

- a metal base plate at least 4 mm thick;
- a centering device secured to the base plate; and
- a means of preventing the rotation of the upper clamping ring when the first two bolts are tightened (two short sockets welded to the base plate will retain the bolt heads).

(iii) Cone assembly (See *Figure B2-1*)

A suitable cone assembly consisting of :

- a polished brass cone with the following dimensions :
 - generating angle of cone, 45°;
 - radius of point of cone, 0,5 mm;

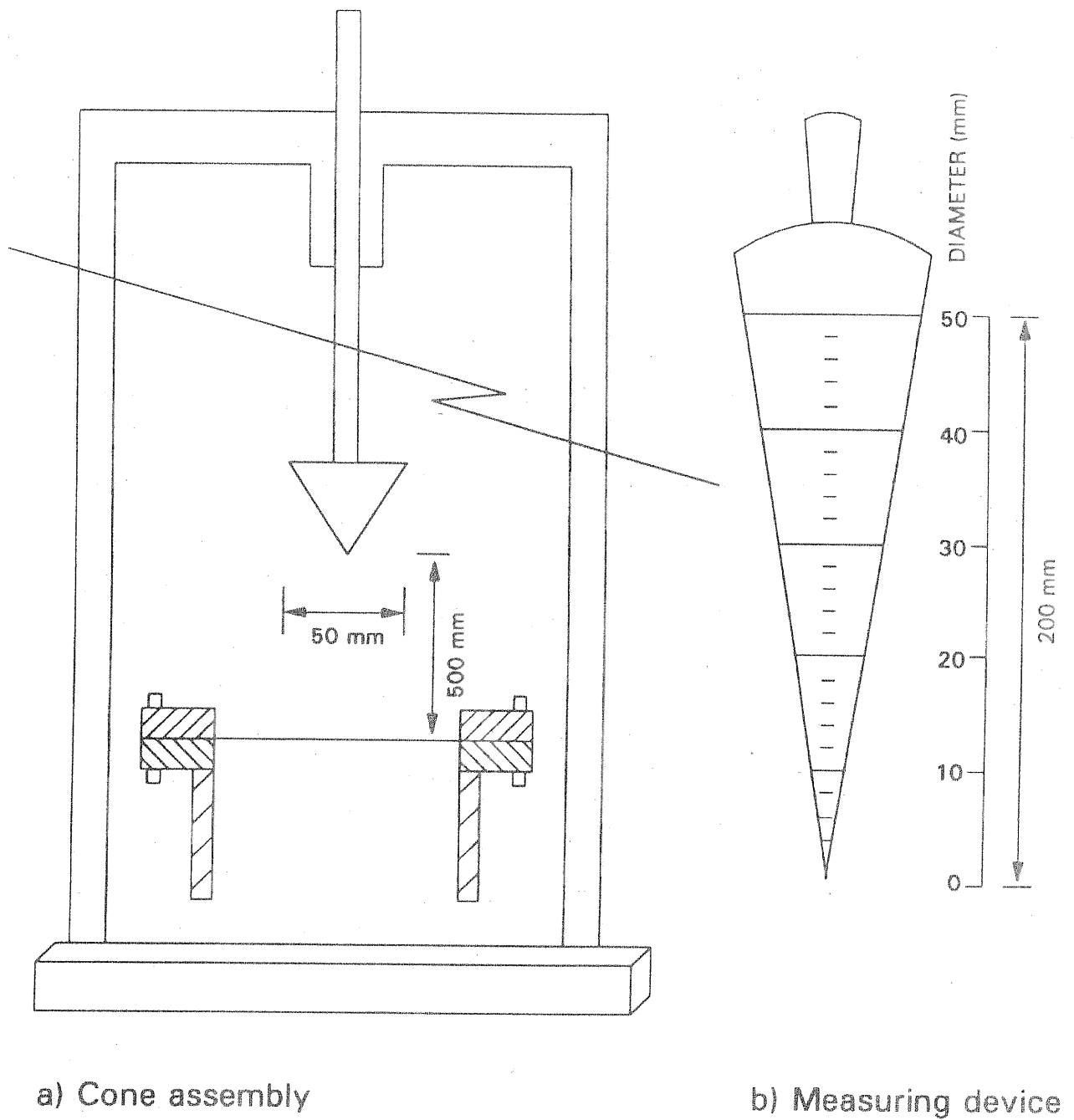


FIGURE B2-1
Apparatus for puncture resistance test

- overall diameter, 50 mm;
- mass of falling assembly, 1 kg; and
- a means of retaining the point of the cone at a height of 500 mm above the specimen together with a suitable release system.

(iv) Measuring device (See *Figure B2-1*)

A suitable conical measuring device with a mass of $700 \text{ g} \pm 10 \text{ g}$ and graduated from zero to 50 mm diameter at 2 mm intervals over a length of 200 mm to measure the diameter of the hole formed in the specimen. The surface of the device to be polished.

(v) Marking and cutting stencil

A suitable metal stencil for cutting specimens 250 mm x 250 mm square, or circular with a 282 mm diameter, with a handhold and studs to secure the stencil on the sample. The stencil shall be capable of accurately locating the position of bolt holes and locating pin holes on the specimen. The holes in the stencil shall be 13 mm diameter, i.e., 2 mm larger than the holes to be cut into the specimen.

(vi) Felt pen and punch

The position of the holes on the specimen shall be marked through the stencil with a 3 mm diameter felt-tip pen.

A punch capable of cutting 11 mm diameter holes in the specimen.

(vii) Cutting device

A suitable cutting device.

(viii) Torque wrench

B2.3 SPECIMENS

(i) Dimensions of specimens

Specimens shall be either 250 mm x 250 mm square, or circular with a diameter of 282 mm.

(ii) Number of specimens

Not fewer than 10 specimens shall be tested.

(iii) Preparation of specimens

With the aid of the stencil, mark on the conditioned sample the positions of the required specimens, together with the positions of the holes for bolts and locating pins. The specimens shall be set out in two rows equally spaced across the width of the sample, the spacing of the specimens shall be as specified, except that the distance between the two rows shall be 150 mm. Where the width of the sample does not allow all the specimens to be contained in two rows, increase the number of rows.

Carefully cut each specimen from the sample as specified. Centre the punch over the marked positions and punch out the holes.

Maintain the conditioned specimens in the specified atmosphere. Test the specimens in the testing atmosphere.

B2.4 PROCEDURE

(i) Assembly

Place the lower clamping ring with bolts in position on the mounting device and carefully position the specimen over the bolts and pins so as to ensure that the specimen is free from stress and is lying flat on the mounting device.

Position the upper clamping ring over the bolts and pins and carefully lower the ring onto the specimen. Secure the ring in position by tightening the nuts to a torque of 30 Nm.

(ii) Testing

Ensure that the mounting-platen/supporting-sleeve assembly is central with the line of the fall of the cone.

Place the clamping-ring assembly on the supporting cylinder.

Release the cone-release system.

Carefully remove the cone from the specimen and measure the diameter of the hole formed in the specimen with the measuring device. The device shall be gently lowered, vertically by hand, into the hole formed in the specimen until penetration under its own mass is stopped. The diameter of the hole is taken as the graduation nearest to the line of contact between the device and the geotextile.

Note :

It is advisable to pack the inside of the supporting cylinder with a resilient material of sufficient thickness to prevent damage to the cone in the event of complete penetration of the specimen.

B2.5 CALCULATION AND REPORTING

(i) Calculation

Record the diameter of the hole formed for each specimen and calculate the average diameter of the hole for the sample.

Round off the results correct to 2 mm.

(ii) Reporting

Report the value of the hole diameter for each specimen and the average diameter for the sample.

TEST METHOD B3 : LONG TERM FLOW TEST

B3.1 SCOPE

This is a performance test to evaluate soil-geotextile compatibility over time. The test method described below was used to develop the given criteria, using the apparatus shown on *Figure B3-1*. However, subsequent overseas research has shown that considerable information can be gained by incorporating stand pipes, such as those shown on *Figure B3-2*. If stand pipes are used, which is recommended, then the test method must be adapted accordingly. A discussion of stand pipe measurement and interpretation is given in Section B3.7 below.

B3.2 APPARATUS

- (a) Water supply. A supply of water from an overhead tank with a constant head of $1\,000 \pm 25$ mm above the geotextile sample.
- (b) Permeameter (see *Figure B3-1*) with a suitable means of mounting to ensure that the permeameter remains in a vertical position throughout the test. The permeameter consists of
 - (i) Two perspex cylinders, 90 mm inner diameter. The bottom cylinder has a recess for the support mesh and a breather hole.
 - (ii) Two perspex end caps with inlet/outlet nozzles. The end caps are machined as shown in *Figure B3-1* to fit over the cylinders and allow air bubbles to escape through the inlet nozzle. A bleeding hole is also provided.
 - (iii) Three brass rods and nuts (wing nuts).
 - (iv) Brass or stainless steel mesh with 2,67 mm openings (standard soil sieve).
- (c) Cutting device. Means of cutting out a circular specimen of diameter at least 110 mm.
- (d) Containers with capacity of at least 5 litres.

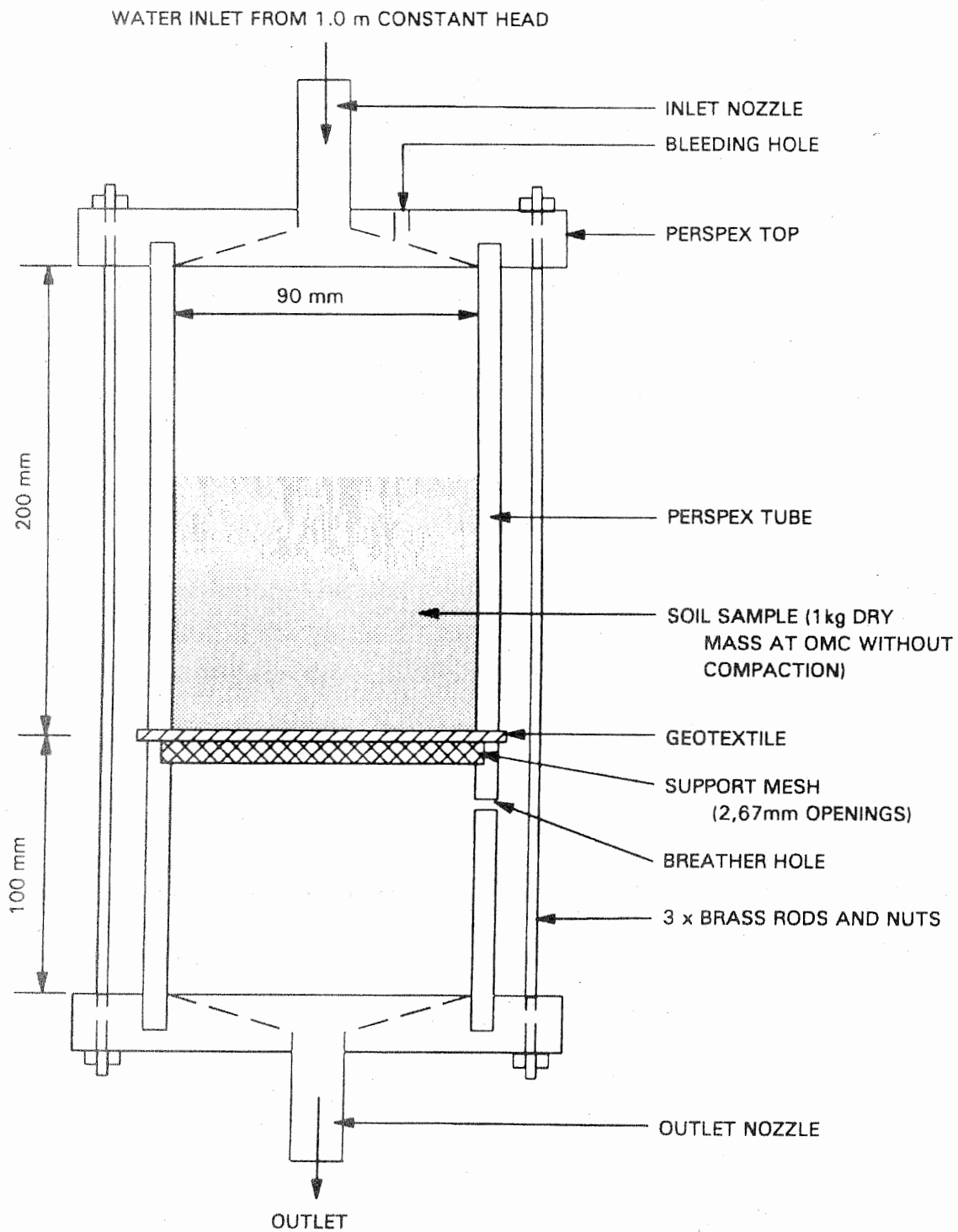


FIGURE B3-1

Permeameter for Long Term Flow Tests (Without Stand Pipes)

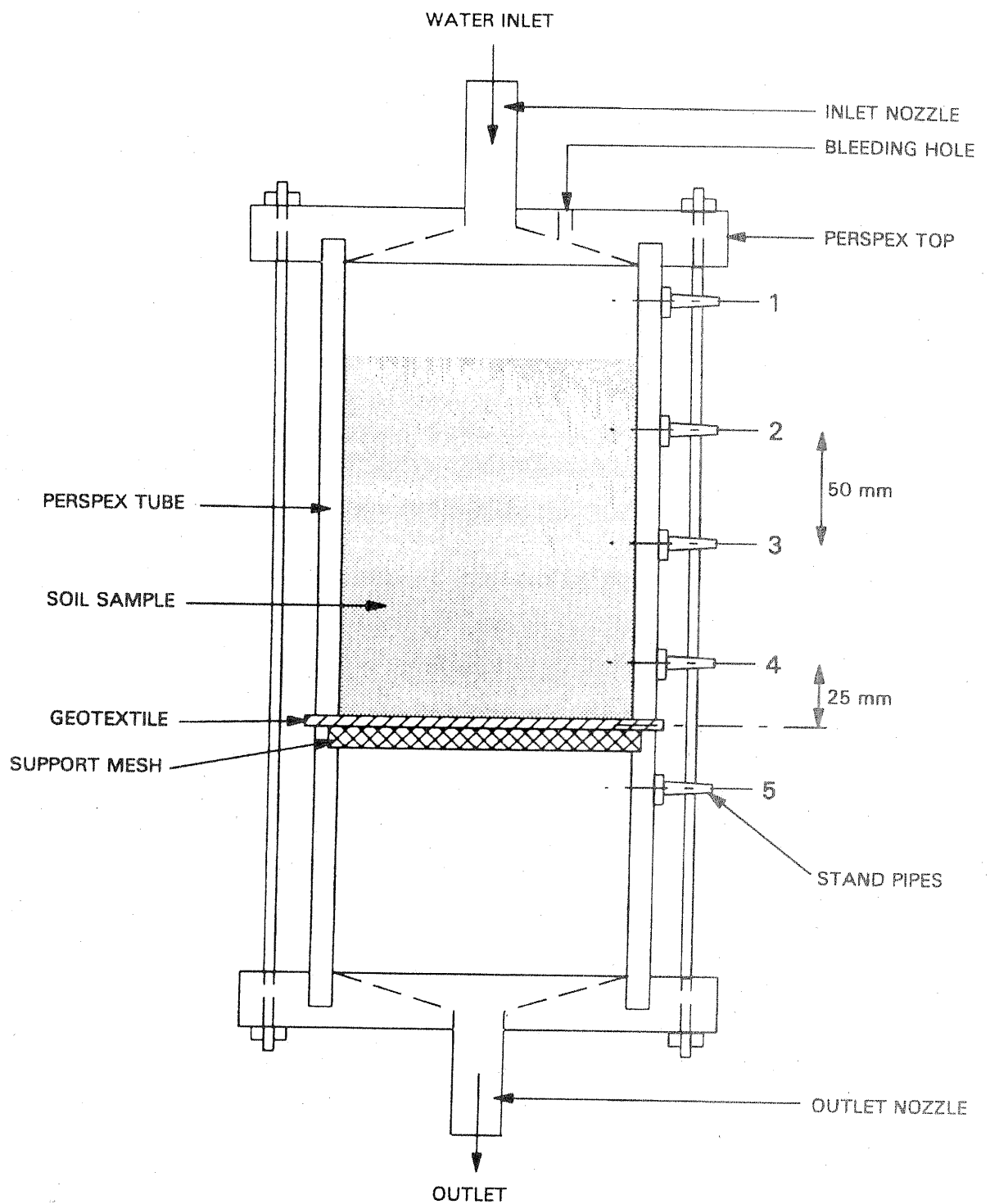


FIGURE B3-2

Permeameter for Long Term Flow Tests (With Stand Pipes)

- (e) A riffler with 25,0 mm openings.
- (f) A soil test sieve with 13,2 mm openings.
- (g) A balance to weigh up to 1 kg, accurate to 1 g or better.
- (h) Stopwatch, silicone grease, pans.

B3.3 SPECIMENS

- (a) Number of specimens : Conduct one test for each soil/geotextile combination.
- (b) Geotextile specimens : Mark and cut the required number of specimens and maintain in the standard atmosphere until tested.
- (c) Soil samples : Sieve the oven-dried soil through a 13,2 mm sieve and discard the material larger than 13,2 mm. Riffle the remaining soil to obtain specimens of $1\ 000 \pm 5$ g. Place in sample bags and label.

B3.4 PROCEDURE

- (a) Place the geotextile specimen between the two cylinders on top of the mesh. Apply silicone grease to ensure that water does not leak out between the two cylinders.
- (b) Mix the amount of water required to bring the soil sample to Mod AASHTO optimum moisture content (OMC). With dispersive soils, friable mudstones and heavy clays, remoulding often results in an impermeable soil mass. In such cases this step is omitted and the soil sample is placed dry.
- (c) Place the soil sample on top of the geotextile. Smooth the surface without compacting the soil.

- (d) Place the perspex end caps in position and fasten the rods and nuts. Place the assembled permeameter in its mounting.
- (e) Close the outlet and breather hole in the bottom cylinder and slowly fill the whole cylinder with water from the top. Care must be taken not to disturb the surface of the soil sample. This can be done by using a small diameter pipe with a spray nozzle which is inserted through the top inlet to a height just above the soil surface. Fill the permeameter to the top of the inlet nozzle, remove the small diameter pipe and connect the hose from the constant head tank. Remove any entrapped air bubbles through the inlet nozzle and the bleeding hole.
- (f) Open the outlet nozzle and breather hole in the bottom cylinder and record the time as the beginning of the test.
- (g) The first outflow measurement should be taken between 1 minute and 5 minutes after the beginning of the test. Outflow is measured using a container and stopwatch. The container is placed under the outflow and the time recorded that it takes to fill the 5 litre container, or the amount of flow that occurs in 30 minutes, whichever occurs first. It is not important that exactly 5 litres or 30 minutes be used, but the time and volume must be recorded accurately. Record the following with each outflow measurement :
- Date and time (hours and minutes)
 - Flow volume ($1\,000\text{ mm}^3 = 1\text{ ml}$)
 - Flow time (minutes and seconds)
 - Height of the sample (soil + geotextile) in mm
 - Height of the water head above bottom of geotextile in mm (ensure this height remains at $1\,000 \pm 25\text{ mm}$)
 - Whether the water at the outlet is discoloured.
- (h) Outflow measurements should be taken at the beginning of the test (1 to 5 minutes after opening the outlet) and thereafter at least once a day. The test should be continued for at least 400 hours (17 days).

B3.5 CALCULATIONS

- (a) Permeability coefficient, k . Calculate permeability coefficient for each outflow measurement as follows :

$$k = \frac{Q}{iA} \text{ (mm/sec)}$$

Where : Q = outflow (mm³/s)
 i = hydraulic gradient

$$i = \frac{\text{water head above geotextile (mm)}}{\text{sample height (mm)}}$$

A = cross-section area (mm²)
= 6 362 mm² for 90 mm diameter

- (b) Permeability reduction factor, K_{400} . K_{400} is determined at the end of the test, as follows :

$$K_{400} = \frac{K \text{ at } 400 \text{ hours}}{K \text{ at beginning of test}} \times 100$$

If an outflow measurement was not taken at exactly 400 hours, K at 400 hours may be determined by linear interpolation.

B3.6 SELECTION CRITERIA

The normal filter criteria namely piping and permeability as well as permeability reduction can be controlled with the permeability reduction test.

- (a) Piping. Discolouration of the water should not be visually discernable for longer than 8 hours after the start of the test.
- (b) Initial permeability, K . The permeability of the geotextile should be equal to or more than that of the soil. This means that the initial permeability obtained with the flow test should not be less than the permeability of the soil obtained with a porous disk or geotextile of which the permeability is known to be high.

- (c) Permeability reduction, K400. The geotextile resulting in the highest value of K400 for a particular soil should be selected.

Figure B3-3 gives ranges of K400 values for different soils (based on percentage passing 0,075 mm sieve) and can be used as a guide. Geotextiles selected should yield results in the high or medium ranges.

B3.7 Stand pipes

Stand pipes make it possible to measure head loss (therefore permeability) through the soil sample and across the geotextile. The flow test can be carried out in a closed system where there is water on both sides of the geotextile or in an open system where there is water above and air below the geotextile.

In the closed system (*Figure B3-2*) the water flows through an "air free" system. The outlet is adjusted so that the head difference between the inlet (stand pipe 1) and the outlet (stand pipe 5) is 1,0 m.

Measurements are made of the quantity of water flowing through the permeameter as well as the hydraulic head at the stand pipe positions (1, 2, 3, 4 and 5 on *Figure B3-2*). These measurements are carried out for a period of 400 hours, or until such time as the flow rate and stand pipe levels stabilise.

The permeability of the soil (between stand pipes 2 and 4) is calculated as well as the soil/geotextile interface (the bottom 25 mm of soil and the geotextile, between stand pipes 4 and 5).

Using the flow measured through the system and the hydraulic gradient between the relevant stand pipes, permeability is calculated in the following matter :

$$k = \frac{Q}{iA} \text{ (mm/s)}$$

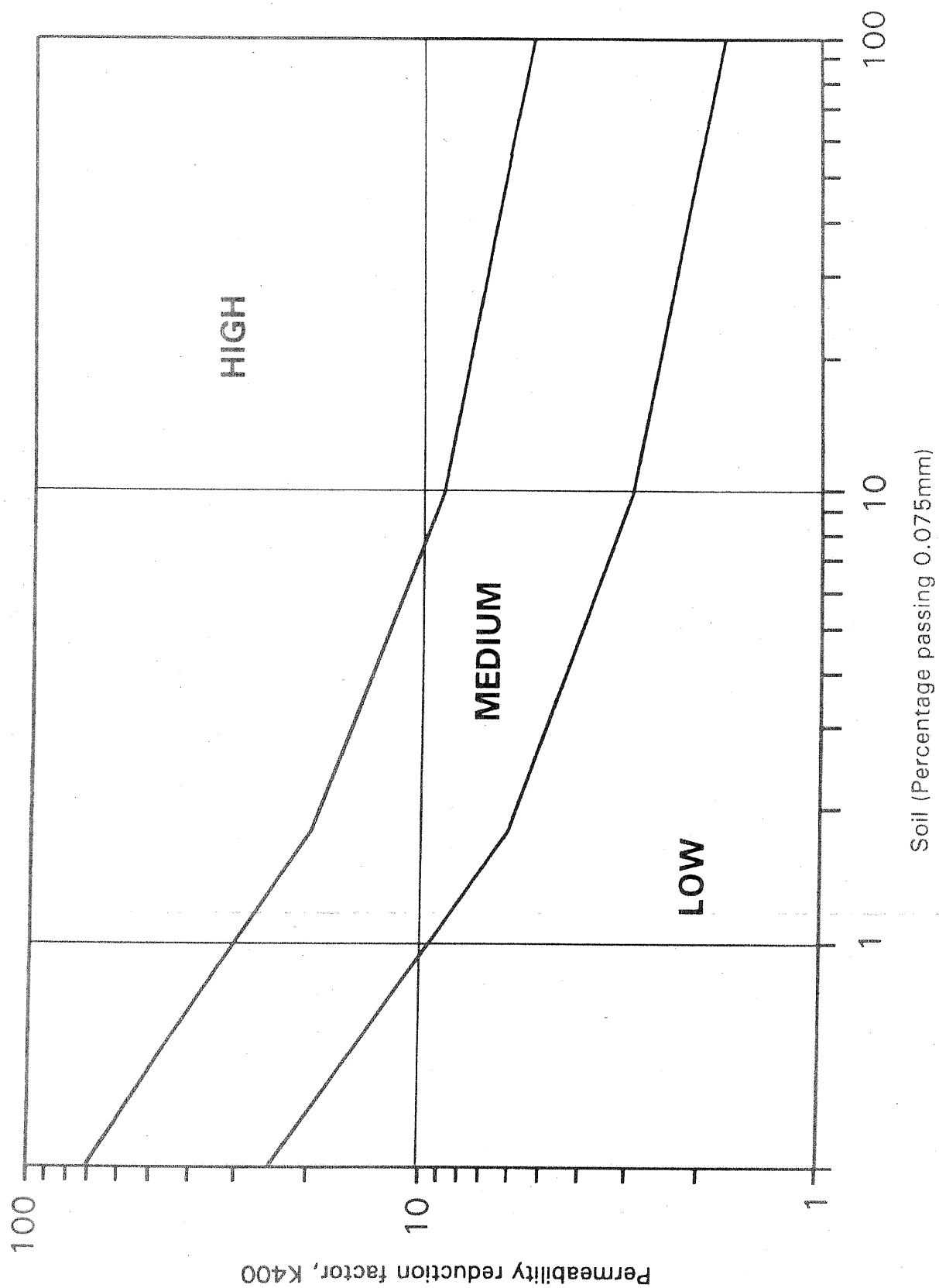


FIGURE B3-3

Permeability reduction criteria

where : k = permeability (mm/s)

i = hydraulic gradient

= $\frac{\text{difference in head between stand pipes (mm)}}{\text{distance between stand pipes (mm)}}$

A = cross-sectional area of permeameter (mm²)

A graph is drawn to compare the permeability of the soil and the soil/geotextile interface, over time. If at the end of testing the permeability of the soil/geotextile interface is greater than that of the soil, then the geotextile is suitable.

In a open system stand pipe 5 acts as a breather hole below the geotextile so there is air below and water above the geotextile. The hydraulic head at the inlet (stand pipe 1) is set at 1,0 m. In this situation the hydraulic head below the geotextile is zero. The permeability of the soil, based on head loss between stand pipes 2 and 4, is calculated as above, and compared with that of the soil/geotextile interface (using a hydraulic gradient equal to the head measurement at stand pipe 4).

METHOD B4 : MEASURING THE SOIL-GEOTEXTILE SYSTEM CLOGGING POTENTIAL BY THE GRADIENT RATIO TEST (ASTM D 5101 - 90)

B4.1 SCOPE

This test is a performance test for determining the soil-geotextile system permeability and clogging behaviour under unidirectional flow conditions.

B4.2 TEST REFERENCE

ASTM D 5101 - 90.

B 4.3 DEVIATIONS FROM TEST METHOD

Notes 2, 3, and 4 that are given in ASTM D 5101 - 90 allow certain deviations. These deviations are generally acceptable, provided that they are reported with the results.

Locally manufactured equipment can also be used, provided that all the main characteristics described in the ASTM test method are maintained. A description of the apparatus should also be given when results are reported.

B 4.4 SUMMARY OF TEST METHOD AND INTERPRETATION OF RESULTS

The method uses apparatus similar to that described in Method B3, but with manometers placed at specific heights for measuring hydraulic gradients. The gradient ratio (GR) is defined as follows :

$$GR = (\Delta h_{sf}/L_{sf})/(\Delta h_s/L_s)$$

where :

Δh_{sf} = average head loss (in cm) over the geotextile plus 2,5 cm of soil directly above the geotextile

Δh_s = average head loss (in cm) over 5,1 cm of soil immediately above the 2,5 cm of soil over which Δh_{sf} is determined

L_{sf} = 2,5 cm + the geotextile thickness

L_s = 5,1 cm

If the value of GR is less than 3,0 the geotextile is considered acceptable for the soil and hydraulic conditions that were used for the test. If GR is greater than 3,0 the geotextile is likely to clog in an unacceptable manner⁴⁴.

METHOD B5 : THE INTERFACE FLOW CAPACITY (IFC) TEST⁴⁵

B 5.1 SCOPE

This test is used to evaluate the compatibility of a geotextile with a specific soil. Clogging potential as well as the risk of piping are evaluated. Only a summary of the test is given here and designers are referred to the reference for the test method.

B 5.2 SUMMARY

The test evaluates the effect of the fine fraction of a soil on the permeability of the soil-geotextile system with respect to the medium and coarse fractions.

A geotextile is submerged in the flow path of a column of water. The through-flow capacity or interface flow capacity (IFC) of the geotextile is determined. The fine fraction of soil is introduced into the flow path in small quantities and the reduction in flow recorded until the amount of soil introduced is equal to a single layer of grains on the face of the geotextile. The total reduction in flow is termed ΔIFC_{fine} . This is repeated with the medium and coarse fractions. If ΔIFC_{fine} is higher than $\Delta IFC_{\text{medium}}$, then the fine fraction of soil (which controls permeability) is being retained at the interface zone and is reducing the through-flow capacity of the system. This indicates the potential for clogging.

METHOD B6 : GEOTEXTILE FILTER DESIGN BASED ON OPENING SIZES⁴⁶

B 6.1 SCOPE

This method gives a filter design procedure to design a filter that is based mainly on geotextile opening sizes. Criteria are given for piping (retention), permeability and permeability reduction (anti-clogging).

B 6.2 SUMMARY

The following gives a summary of some aspects of the design method. It is recommended that the reference document be used if the test method is applied.

B6.2.1 Piping criteria

Figure B6-1 shows the soil retention or piping criteria for steady state flow conditions. A similar figure is given in the reference document for dynamic flow conditions.

The parameters shown on *Figure B6-1* are defined as follows :

- d_x = Soil particle size of which x percent is smaller, i.e. the size of the sieve that allows x percent of the material to pass through
- O_{95} = Geotextile opening size of which 95 percent is smaller
- DHR = Double hydrometer ratio from ASTM D 4221 test method to determine dispersion potential
- PI = Plasticity index (from Atterberg limits)
- C_c = Soil coefficient of curvature
= $(d_{30})^2 / (d_{60} \times d_{10})$
- C_u = Soil coefficient of uniformity
= d_{60} / d_{10}
- C'_u = Soil linear coefficient of uniformity
= $\sqrt{d'_{100} / d'_0}$

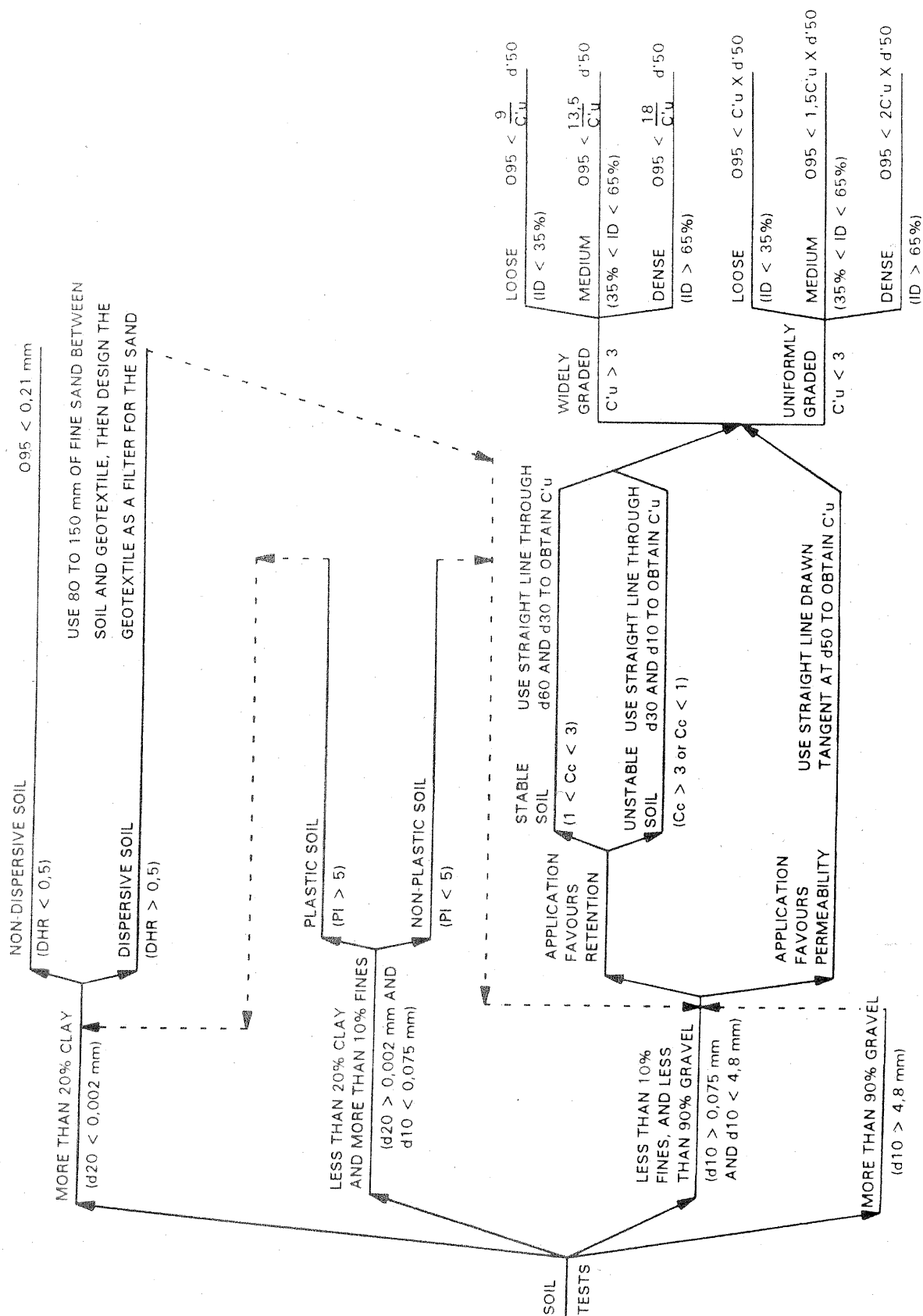


FIGURE B6-1

Soil retention criteria for steady-state flow conditions

- d'_x = Soil particle size of which x percent is smaller; obtained from a straight line approximation to the soil particle size distribution
- ID = Soil relative density from ASTM D 4254

B 6.2.2 Permeability criteria

Minimum geotextile permeability must be determined from :

$$K_g > I_s K_s$$

where :

K_g = Geotextile permeability

I_s = Hydraulic gradient

K_s = Soil permeability

A value of 1,0 is recommended for I_s for typical side drains, but pavement edge drains may require a higher value.

B 6.2.3 Permeability reduction (anti-clogging)

The following criteria are recommended to minimise the risk of clogging :

- Use the largest opening size (O_{95}) that satisfies the retention criteria.
- For nonwoven geotextiles, use the largest porosity (n) available, but not less than 30%.
- For woven geotextiles, use the largest percent open area (POA) available, but not less than 4%.

METHOD B7 : SOIL CLASSIFICATION FOR A GEOTEXTILE FILTER DESIGN METHOD BASED ON OPENING SIZES⁵⁴

B 7.1 SCOPE

This is an extract from the Swiss Standard for filter materials that is based on geotextile opening sizes. The design method is not given, but merely the soil classification system that is used in that method.

B 7.2 SUMMARY

Figure B7-1 shows three soil zones. Soils in Zone 1 are soils that are classified as having high cohesion and low permeability so that piping and permeability are not a problem. Careful design of a filter is therefore not necessary. The soils in Zone 3 are coarse and are coarse grained and can be drained without a filter according to this method. The soils in Zone 2 are classified as problem soils in which the migration of fine sands and silts can result in both piping and clogging. Careful filter design is therefore required for these soils.

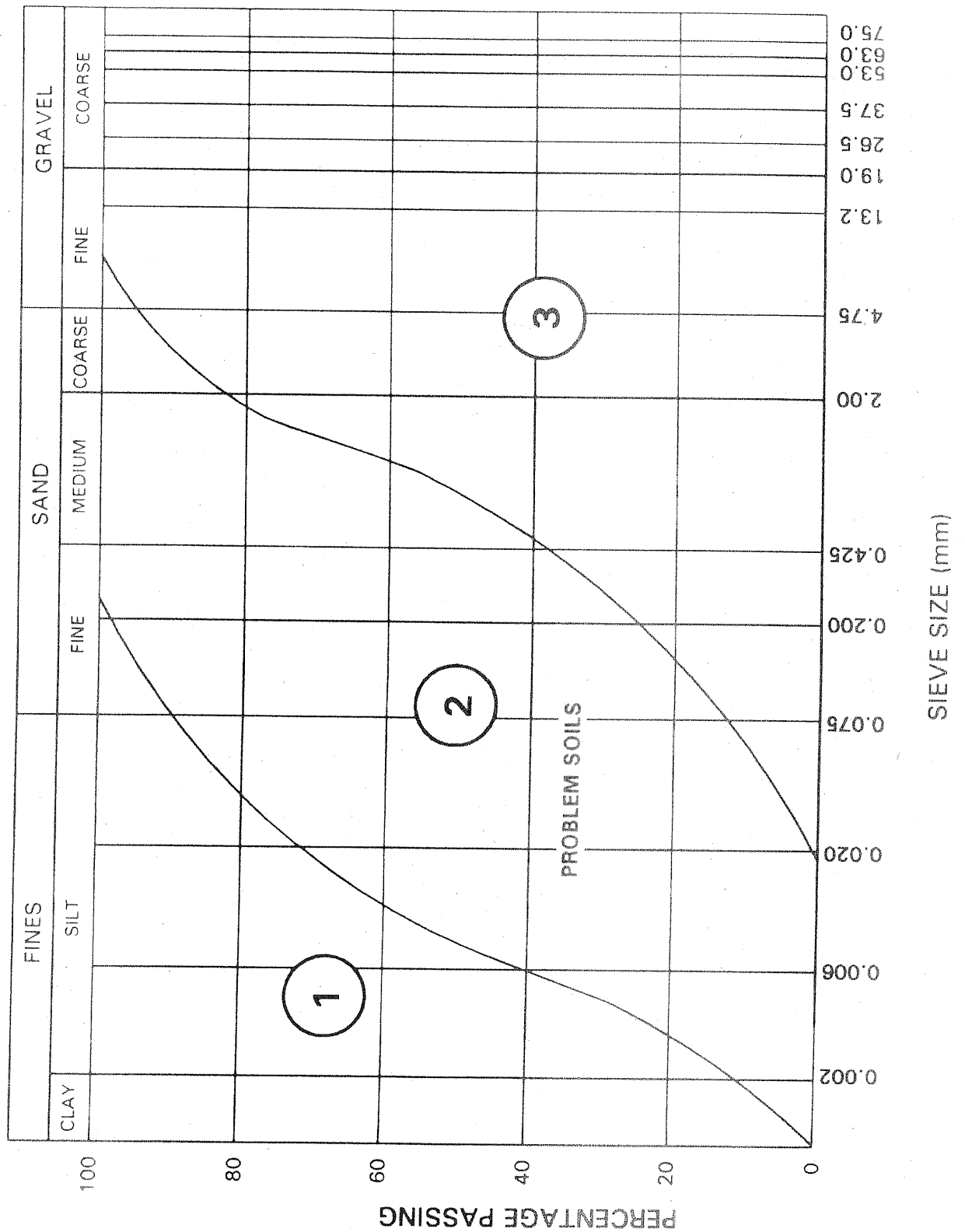


FIGURE B7-1

Soil zones for filter design according to the Swiss standard