

**COMPARISON
OF
HYDROLOGICAL AND GROUNDWATER
DESIGN METHODS**

A Thesis presented to the
DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF CAPE TOWN

In partial* fulfilment of the requirements for
the degree of Master of Science in Engineering.

by

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*This candidate has successfully completed seven postgraduate courses (31 credit points) in partial fulfilment of the requirements for the M.Sc.Eng. degree, and therefore this thesis represents approximately one quarter of the value of an M.Sc. thesis submitted in complete fulfilment of the requirements for the degree. (For details of courses taken, refer to Appendix D.)

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Declaration

The writer hereby declares that except where otherwise stated, the content of this thesis is substantially his own work and has not been submitted to any other university.

Signed by candidate

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PART A

HYDROLOGY

Symbols used in Part A

The following symbols have been used in Part A of this project:

- Q = run-off - m^3/sec
- I = rainfall intensity - mm/hour
- A = catchment area - square kilometres
- C = run-off coefficient
- T_c = time of concentration - hours
- H = height difference between highest and lowest points in catchment - metres
- L = catchment watercourse length - kilometres
- S = slope of watercourse - percent
- t = time of concentration - minutes
- r = total rainfall during time of concentration - millimetres
- R = average annual rainfall - millimetres
- G = geographic coefficient
- K = Hazen's coefficient
- T = period - years
- F = return period factor
- L^1 = design life - years
- V = run-off velocity - m/sec.
- D = duration of storm - minutes

The units are as given above unless otherwise stated.

Preface

One of the objectives in this thesis is to provide a handbook on groundwater hydrology for the practical engineer. A handbook should be a useful and dependable servant to its owner and user. In order to fill this role, handbooks in different fields must differ in accordance with the degree of complexity of the material systems involved and the extent - from qualitative to quantitative - to which fundamental parameters have been recognised, defined, and built into dependable theoretical solutions. A handbook on engineering drainage must firstly, provide an understanding of the approach and reasons for determining the nature of run-off from any particular catchment whether it be a river catchment or a surfaced pavement of relatively small surface area. Then, secondly, the handbook must provide reliable methods for determining the amount of run-off that would result from a particular catchment. There are a number of methods which can be used to produce the desired results and an effort is made herein to present those methods, presently used in Southern Africa, as clearly as possible.

It is accepted that computers are being used more often for drainage calculations, and that a seemingly precise solution appears with relatively little input effort. The work contained herein is a plea to engineers to return to the basics in order to understand the complexity of drainage determinations and to achieve what the author terms "the feel for drainage" - i.e. the knowledge of when to accept a result and when to advocate an amendment to the solution. This results not only in the most practical solution but also possibly the most economical. The final solution is influenced by both the structure involved and the skill of the associated worker. There is a very real danger of the engineer's sense of judgement being lost by the over application of the computer.

The author is aware of the undesirable state that faces every designer when delving into long and complex formulae. The object of this paper is to consolidate the various methods used in Southern Africa, produce easy to use graphs, give simplified procedure steps and guide the user towards the solution without undue difficulty. It must at this stage be pointed out that the various methods dealt with, will produce different solutions which depend on the constants used in these methods. The accuracy of the solutions depend on the suitability of the methods for the particular catchment or material involved. The reader will be disappointed if he expects to find this project of theoretical value as the practical aspect is the goal, with only some indication of theoretical content sufficient to justify the approaches adopted. If the theoretical derivations are required it is recommended that the reader should refer to the original works of the various investigators of these methods.

All the hydrological methods have been based either on the rational or empirical approaches. The rational approach is based on deriving an exact formula which fits the quantity expected. The empirical approach is established through experiment and observation. The various formulae used are, by their very structure limited in accuracy depending on the dominant variable. As catchment area is the major contributor in the different formulae the accuracy of the results is dependant on area. The result of the study of the acceptable limits of accuracy of the different methods is contained in this project and forms one of the major aims of this investigation.

This project is divided into two parts, the first dealing with hydrology and the second with sub-surface drainage. Under the hydrology section, nine different rational and empirical methods of calculating run-off in Southern Africa have been analysed and simplified into graphs and nomographs to overcome unnecessary laborious calculations. These methods were mostly still being used in imperial units and it was necessary to convert and update information in order to metricate and obtain uniformity of approach.

The sub-surface drainage section of the project includes descriptions of practical methods for determining the permeability, under field conditions, by using the minimum amount of equipment. Explanations are also given of how these measured permeability values can be used in the design of practical sub-surface drainage systems.

In many cases, close correlations existed between the results of different methods. This has meant that some methods can be used with a certain degree of confidence.

When calculating potential flows from a catchment coefficients of run-off are used to compensate for the loss of water which is absorbed by the soil. This loss to the sub-surface assists in building up the groundwater reserves. Once the rains have subsided or ceased there begins a movement of water from the sub-surface to the surface assisting with the stream flow. This phenomenon is known as the groundwater depletion and, as the logical sequence of surface run-off, sub-surface flow and depletion of sub-surface water exists, this aspect has been included in this project.

An extensive search was made to establish whether any appropriate codes of practice existed for the determination of groundwater flow. Unfortunately no codes of practice were found to cover the particular approach adopted in this project. The only closely related subjects dealt with, by the two codes found, were the drilling and testing of boreholes, and as these bear some relationship to the project these have been included herewith.

Chapter 1

Introduction

There are three aspects of surface drainage design in which the engineer is particularly interested. Firstly, the engineer is concerned that the precipitation falling on the road or paved surface should be removed as quickly as possible so as to minimise the danger to the moving vehicle and inconvenience to the pedestrian. This is achieved by crowning the carriageway or paved surface, inserting with drop inlets to culverts and providing adequate cross drainage structures. Secondly, the engineer becomes involved in spanning rivers and streams. Thirdly, the engineer designing a long stretch of road or a road system is continuously faced with the twin problems of intersecting the water in the myriads of permanent or temporary water courses which are interfered with by the road construction, and then guiding this water into desired paths.

This section deals therefore with the determination of the quantities of water arriving at the inlet to a road or rail structure. This is termed the hydrological study. Hydrology can be described as the science which deals with the operations governing the circulation of moisture in its various forms, i.e. precipitation, run-off, infiltration, evaporation and condensation. The two aspects of the hydrological cycle in which the engineer is most interested are precipitation and run-off.

The engineer must study information concerning the rainfall intensities (I) within the catchment area, in order to estimate the maximum run-off flow rates which might occur through the designed structure. A fundamental feature of rainfall is that the average intensity of rainfall throughout a given storm is inversely proportional to the length of the storm, i.e. as the duration of the rainfall increases, its average intensity decreases. This is perhaps to be expected since the

meteorological forces which cause a heavy rainfall are also continually causing it to move quickly from one area to another. A further point to note is that for a given site, and for a given storm duration, the intensity can vary considerable, with the least heavy rainfalls occurring most often.

Whereas an engineer designing a large bridge or a complete flood control system is mainly interested in storms which cover large areas and last for hours, and perhaps days, the highway engineer engaged in designing a culvert or a drainage ditch is primarily interested in the high intensity, short duration storms, i.e. the storms which result in the peak rate of run-off from the catchment area. The run-off quantities are based on the assumption that the maximum discharge at any point in a drainage system occurs when the entire catchment area above that point contributes. It is also assumed that the rainfall intensity producing this flow is the average rate of rainfall which can be expected to fall in the time required for a raindrop falling at the most remote point of the catchment area to flow to the point under investigation.

Apart from assigning to the design input the same recurrence interval as that of the desired design structure, the input rate must also be selected as that having a duration closely associated with the catchment response time. The response time has been more commonly called the concentration time. It was suggested in former times that this would be the time taken for input at the most remote part of the drainage system to reach the exit and that this time could be computed from an open channel flow formula (e.g. that of Manning). It was subsequently appreciated, however, that the response time is in

fact analogous to an ocean wave travel time, which is invariably shorter than the travel time of the wind flow. Various empirical formulae for determination of catchment response time has been offered. Most of these give strong emphasis to the general slope of the catchment. Perhaps the most widely used is one suggested by the U.S. Bureau of Reclamation:

$$T_c = \left(\frac{0,87 L^3}{S} \right)^{0,385} \quad (1)$$

where T_c = concentration time in hours

L = length of watercourse in kilometres

S = dimensionless slope

Once the time of concentration has been determined the next step involves determining the design intensity of rainfall for a storm duration equal to the time of concentration. Before this can be done, however, a decision must be made as to the storm frequency which is to be catered for in the design of the structure.

There is considerable debate regarding what storm return period should be taken and for this reason this has been fixed by various authorities for different structures, i.e. 50 years for bridges and 25 years for box culverts. In order to standardise the graphs or nomographs in this project, the writer has produced formulae and graphs which correspond to a return period of 10 years, unless otherwise stated. Graphs for the conversion factors are included herein in order to determine any return period using a known run-off, whether a 10 year or otherwise.

While the best practice in selecting the design intensity is to prepare rainfall curves relating intensity and duration, there are occasions when this is not possible. In such instances, recourse is often made to the use of simple empirical formulae which attempt to relate intensity to duration of rainfall. These formulae usually take the form:

$$I = \frac{a}{t + b} \quad (2)$$

where I = intensity of rainfall in mm/hr
 t = duration of storm in minutes
 a & b = constants

Care should be taken using formulae of this type, since there is no apparent reason why intensity and duration should be related by a simple mathematical law, at best they only have a local application, i.e. the constants will probably vary from location to location. In analysing the rainfall figures in Southern Africa the writer suggests that the following equation be accepted:

$$I = \frac{6,65 R}{(t+20)} \quad \text{mm/hr} \quad (3)$$

where R = annual rainfall in millimetres
 t = duration of storm in minutes

The return period in this case would be for 10 years which can be amended for different return periods by using the graph in Figures 5.

Drainage calculations undertaken for any catchment area whether for a large river or for a small surfaced pavement incorporate a number of assumptions to be

made by the designer. These assumptions include such features as catchment size, annual rainfall, period of return, run-off coefficients, topography coefficients, height drop in the catchment area, the actual drainage length over the catchment, and many other interrelated factors. It must be obvious to the reader that each of these factors for a particular catchment would be as variable as the number of designers involved. It is for this reason that variable factors concerning drainage calculations should be reduced to their simplest values hence reducing the choices available.

A number of different methods exist whereby run-off quantities may be determined and each of these methods vary in complexity of approach. It is not surprising therefore that at times very large differences in quantities are obtained when using several of these methods for the same catchment area. One of the main reasons for the variations obtained is the use by the designer of a method for which it was not intended. Each method submitted herein has its limitations and use thereof should be governed by the size of the catchment involved.

An investigation has been made to determine how many methods are used in Southern Africa. It is hoped that the methods listed below cover the spectrum of popular use, although occasionally it was noted that other methods were mentioned but not generally used. It must also be mentioned here that such methods as the Unit Hydrograph have not been included in this investigation as they are based on actual run-off data whereas this project is concerned with the approach of using formulae to determine run-off especially where rainfall data is not available.

The following methods are incorporated in this project:

Lloyd-Davies

Rational

Depth - Duration - Frequency

Empirical

Mc Math

Reich

Holtzhauzen

South West African/Namibia

Roberts

Chapter 2

Lloyd-Davies Method

Many formulae have been developed with an attempt to measure run-off from a storm and one method which gained most favour in Britain was that published by D.E. Lloyd-Davies in 1906 (2). In this method he related run-off to rainfall by means of the following equation:

$$Q = 0,167 ACr \frac{60}{t} \quad (4)$$

where Q = discharge, cubic metres/minute
t = time of concentration, minutes
r = total rainfall during the time of concentration, millimetres
A = catchment area, hectare
C = proportion of total rainfall running off after allowing for soakage and evaporation

Now if I is the rainfall intensity, in millimetres per hour, for a storm of duration t minutes, then

$$r = \frac{t}{60} \times I \quad (5)$$

If this value for r is inserted in the above equation, then

$$Q = 0,167 CIA \quad (6)$$

The Lloyd-Davies Formula for run-off is essentially the same as the one known as the Rational Formula, which was derived in the United States. This is illustrated by converting the area to square kilometres and the run-off Q to cubic metres per second:

$$\begin{aligned} Q &= 0,167 \cdot CIA \cdot \frac{1}{60} \cdot 100 \\ &= 0,278 CIA \text{ cumecs} \end{aligned} \quad (7)$$

The Rational Formula is given by the following as shown later in this section, namely

$$Q = 0,275 \text{ CIA cumecs} \quad (8)$$

Some of the assumptions underlying the use of the Lloyd-Davies formula should be emphasized at this stage so that the reader can realise why, in practice, the results must be qualified by engineering judgement.

1. The rainfall intensity is selected for a storm duration which is equal to the time of concentration. The entry time component of this time is usually selected by measurements based on the assumption that the raindrop moves perpendicularly to the ground contour-lines. If too low a time of concentration is chosen, this will result in the selection of too high a rainfall intensity and the over-estimation of the run-off.
2. The rainfall intensity value used is assumed to be the same throughout the catchment area during the storm. If the area is very large and/or the topography is very variable, this assumption may not be valid.
3. The impermeability factors which are used in the calculations, such as those given in Table 1 in Figure 2, only taken into direct account the type of surface and the slope, and assume that the other influencing factors are covered by these.

4. Finally, both the Lloyd-Davies and the Rational Formulae assume that the maximum run-off occurs with the same frequency at the design storm. In fact, this assumption is only valid when the ground has been well wetted beforehand by a preliminary rainfall, or where the precipitation falls on an impervious surface. Thus, for instance, if in a rural area a design storm with an occurrence interval of 25 years is chosen and the ground is well wetted, before such a storm, but once in every three occasions, then it is likely that the maximum run-off may occur only once in 75 years.

From these and other considerations it can be seen that the design precision of the Lloyd-Davies and Rational Methods is somewhat limited. In general, they can tend to over-estimate the maximum run-off, and the extent to which this occurs increases with the size of the catchment area. As a result it is probable that the use of the formulae should be confined to urban areas and relatively small catchment areas in rural locations.

Chapter 3

Rational Method (small areas)

The Rational Formula combines the relationship between rainfall and peak run-off. Although this formula is based on a number of assumptions which cannot be readily satisfied under actual circumstances ⁽¹⁾, its simplicity has won its popularity. The original of this formula is somewhat obscure but was first mentioned in 1889 by Kruichling ⁽²⁾ for a determination of peak run-off for sewer design in Rochester, New York, during the period 1877 to 1888. Some authors believe that the principles of the formula were explicit in the work of Mulvaney ⁽³⁾ in 1851.

The Rational formula is:

$$Q = CIA \quad (9)$$

The formula is called Rational because the run-off quantity is exactly expressed by numbers incorporating the rainfall intensity and the catchment area.

When using the Rational Formula, one must assume that the maximum rate of flow, owing to a certain rainfall intensity over the drainage area, is produced by that rainfall which is maintained for a time equal to the period of concentration of flow at the point under consideration. Theoretically, this is the time of concentration, which is the time required for the surface run-off from the remotest part of the drainage basin to reach the point being considered. For a uniform rainfall intensity this would be the time of equilibrium at which the rate of run-off is equal to the rate of rainfall supply. For natural drainage and basins of large size and complex drainage pattern, run-off water originating in the most remote portion may arrive at the outlet too late to contribute to the peak flow. Accordingly the time of concentration

is generally greater than the lag time of the peak flow. For small drainage basins with simple drainage patterns, the time of concentration may be very close to the lag time of the peak flow.

For small drainage basins, Kirpich ⁽⁴⁾ submitted an empirical formula for the time of concentration in hours and in its metric form it is:

$$t = 0,00009 \frac{L}{S} \begin{matrix} 0,77 \\ 0,385 \end{matrix} \quad (10)$$

where L = length of catchment in kilometres

S = dimensionless slope ratio between drop from highest point in catchment to outlet, to the greatest length of the catchment.

When analysing the different derivations of the run-off coefficient C it will be noticed that the values thereof may vary for the same type of terrain. In trying to establish the value of the run-off coefficient different researchers have tended to establish their own terrain criteria and adapting the run-off coefficient accordingly. It is therefore important to use only those run-off coefficients which relate to a particular presentation of Rational Formula.

According to Kringold ⁽⁵⁾, the assumptions involved in the Rational Formulae are:

1. The rate of run-off resulting from any rainfall intensity is a maximum when this rainfall intensity lasts as long, or longer, than the time of concentration.

2. The maximum run-off resulting from a rainfall intensity, with a duration equal to or greater than the time of concentration, is a simple fraction of such rainfall intensity; that is, it assumes a straight line relation between run-off Q and intensity I and when $Q = 0$ then $I = 0$.
3. The frequency of peak discharges is the same as that of the rainfall intensity for the given time of concentration.
4. The relationship between peak discharges and the size of drainage area is the same as the relationship between duration and intensity of rainfall.
5. The coefficient of run-off is the same for all storms in a given watershed.
6. The coefficient of run-off is the same for all storms in a given watershed.

It is believed that these assumptions hold satisfactorily for paved areas with gutters and sewers of fixed dimensions and hydraulic characteristics. The formula has thus been rather popular for the design of drainage systems in urban areas and airports.

Two derivations of the Rational Method are given herein, the first in this chapter, the second in Chapter 4. The first method concerns relatively small areas and is particularly useful in calculating run-off for pavements when determining drop and kerb inlet spacings. The method is adapted from two sources, namely Yarnell ⁽⁶⁾ and the Engineering Manual

of the War Department (7). In order to simplify the rainfall intensity-duration graph further, average annual rainfall figures have been used in the graph instead of the time of concentration. Refer to Figure 2. A further modification is the elimination of the need to apply the Rational Formula by the direct reading of the run-off quantities, in cumecs, from the graph. Refer to Figure 2.

- METHOD
1. Assess the particular catchment area, the length of the overland flow, the ground conditions and the percentage slope of the overland watercourse.
 2. Using this information use Figure 1 to obtain the flow time in minutes.
 3. Enter Figure 2 with this duration, extend up to the annual rainfall curves, cross to the coefficient of run-off curves, drop down to the area of catchment curves and lastly extend to read the run-off in cumecs.

PROBLEM

Using the above approach consider a catchment having the following characteristics:

A = 3 sq km

L = 300 m

S = 6%

Rainfall = 1 000mm per annum

Ground = poor grass

SOLUTION 1. From Figure 1 using $L = 300 \text{ m}$

$S = 5\%$

Poor grass $T_c = 19 \text{ min}$

2. From Figure 2 using Table 1 $C = 0,6$ $C,4$

3. From Figure 2 using $T_c = 19 \text{ min}$

Rainfall = 1 000mm

$C = 0,6$

$A = 3 \text{ sq km}$

$Q = 78 \text{ cumecs}$

OVERLAND FLOW TIME

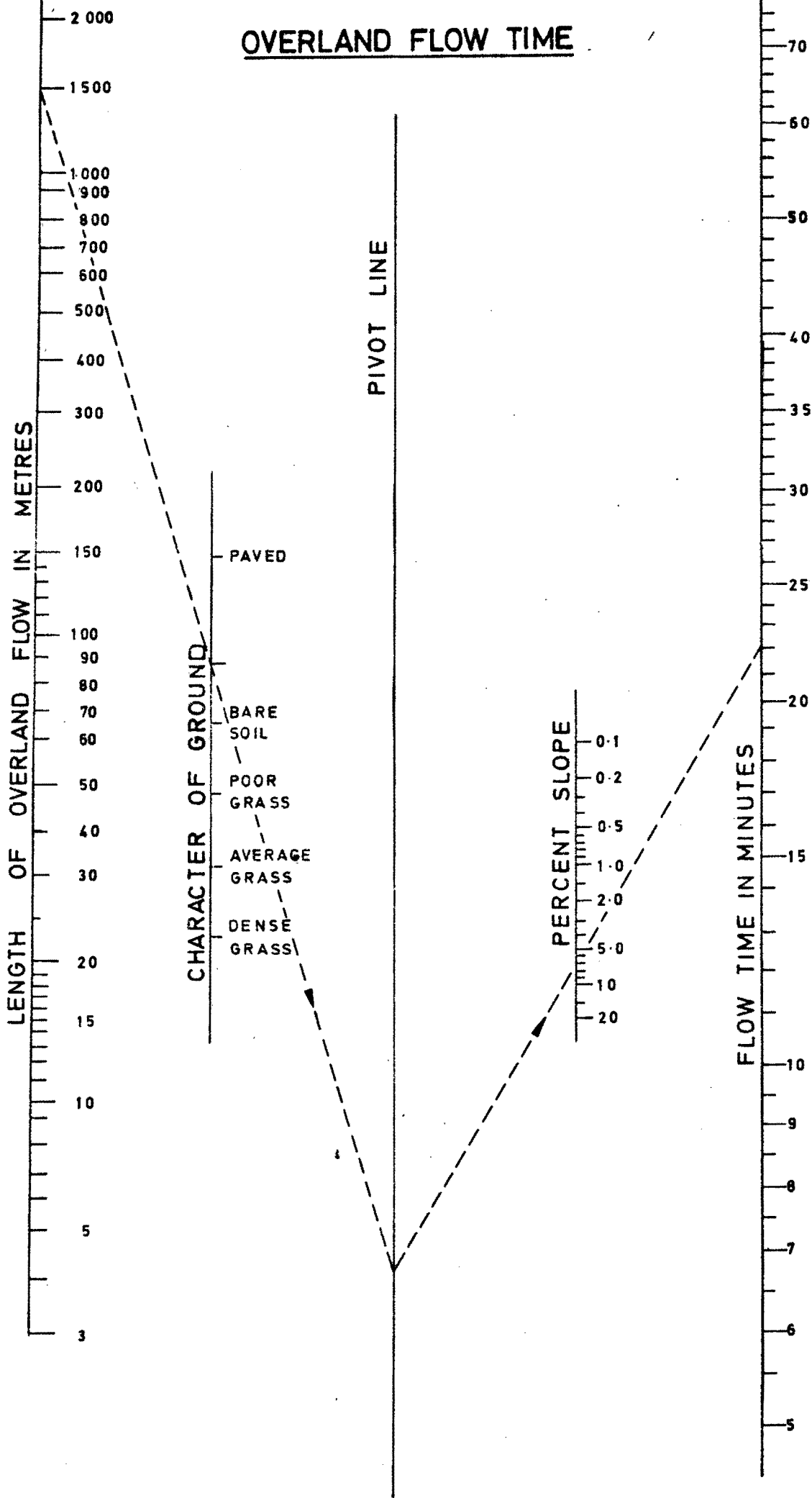


fig 1

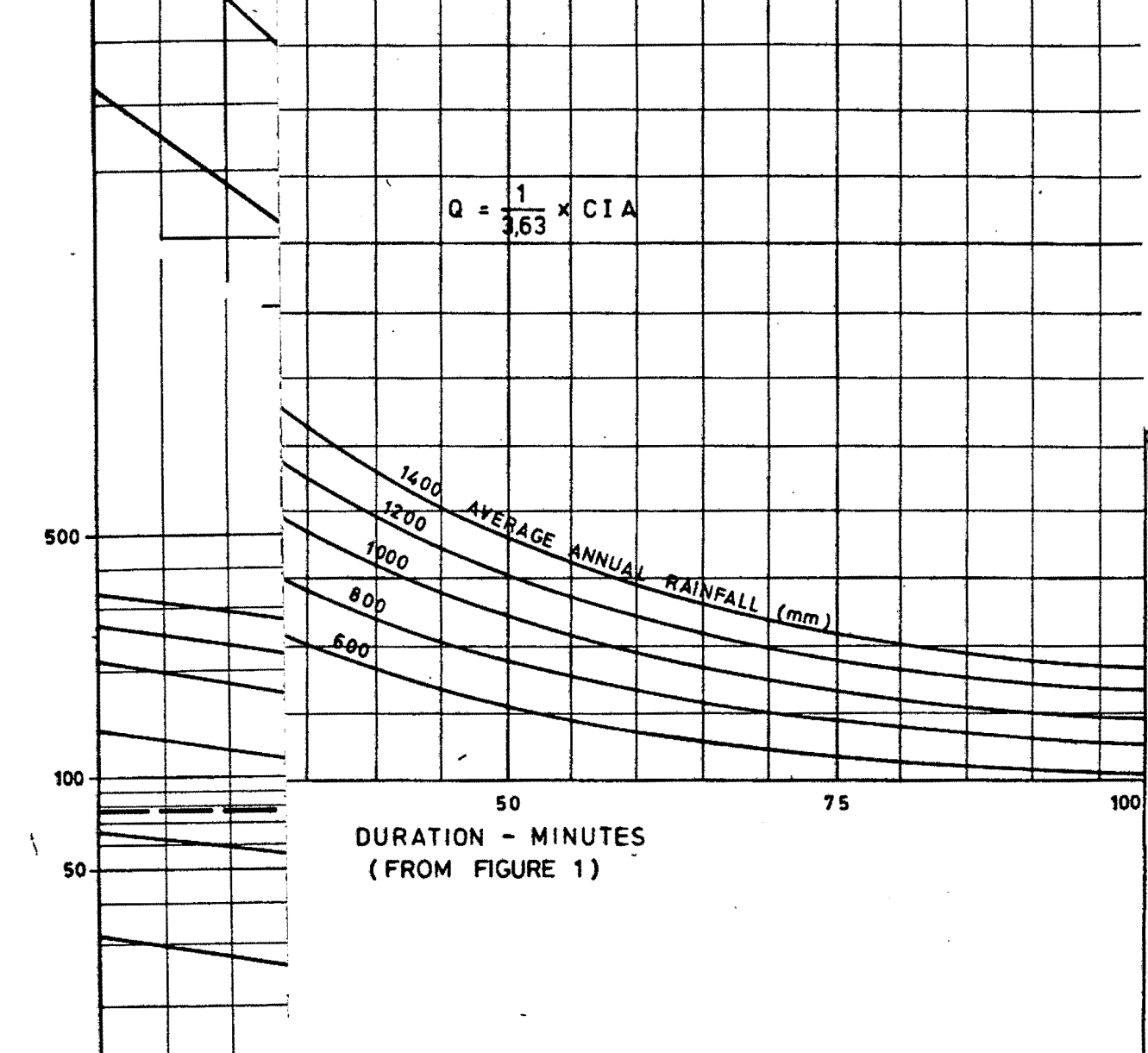


TABLE 1

**RUN-OFF CO-EFFICIENT "C"
FOR WATERSHEDS SMALLER THAN 5km²**

	FLAT	ROLLING	HILLY
	UNDER 5% SLOPE	5% - 10% SLOPE	10% - 30% SLOPE
ON	0.16	0.18	0.21
E	0.31	0.36	0.43
TED	0.50	0.70	0.72
	0.70	0.80	0.85

ADDITIONAL METHOD

BY WRITER

Chapter 4

Rational Method (medium areas)

The second method using the Rational Formula utilises the rainfall intensity map as submitted by Yarnell ⁽⁶⁾ who conducted extensive research in America on the derivation of the one hour duration-intensity lines across the USA. The approach has been adjusted to suit South African conditions and has been further split into four distinct climatic zones. This method is reasonably accurate for catchment areas up to 50 square kilometres.

The coefficient of run-off for different catchment conditions for the second method is given in Table 2. In this table a factor is allocated for the different ground characteristics such as soil cover, ground protection and ground slope.

For determination of run-off quantities for frequencies other than 10 years Figure 5 has been produced. For the Rational Method the intensity has been adjusted in preference to the run-off quantity as it is considered a more accurate approach. The factor corresponding to a particular desired return period is obtained from Figure 5 and multiplied by the intensity for a 10 year frequency will provide the intensity corresponding to the desired return period. Care must be taken in both Figures 2 and 4 that once the 10 year intensity is determined and multiplied by the frequency factor that this should be used before completing the steps towards the final run-off quantity.

It is emphasized that the Rational Method like all deterministic methods is not aimed at endeavouring to reproduce the peak catchment response to a specific storm. It is rather making an estimate of that peak discharge which has an average return period specified by the designer.

- METHOD
1. Determine the characteristics of the catchment, the catchment length, slope, height and area, and the climatic zone.
 2. Enter Table 2 to obtain the coefficient of run-off.
 3. From Figure 3 obtain the zone and the one hour - ten year return intensity.
 4. Using values for catchment height, length, zone, run-off coefficient and area, obtain the run-off Q in cumecs from Figure 4.
 5. If any other return period than 10 years is required refer to Figure 5 and obtain a value for the frequency coefficient which is multiplied by the intensity before completing the procedure in Figure 4.

Table 2**DETERMINATION OF RUN-OFF COEFFICIENT FOR THE RATIONAL METHOD**

CATCHMENT		FACTORS
Type	Description	a:b:c
a) Soil Cover	Deep Permeable Sand	0
	Deep Sandy Soil	1
	Shallow Soil - Clay and Silt	2
	Impermeable Clay, Rock	3
b) Protection	Forest and Dense Bush	0
	Thick Grass and Bush: close seeded crops: contoured and terraced	1
	Close Seeded Crops: imperfectly terraced; open crops contoured and terraced	1½
	Light Grass: close seeded crops not terraced; open crops imperfectly contoured	2
	Light and Sparse Bush: open crops not contoured	2½
	Bare Ground	3
c) General Ground Slope	Flat (0 - 3%)	1
	Moderate (3 - 8%)	2
	Steep (8 - 20%)	3
	Very Steep (over 20%)	4

$$\text{Coefficient of Run-off } C = \frac{1}{10} (a+b+c)$$

PROBLEM

Consider Cape Town as the geographic zone in which a catchment is located having the following characteristics:

Height = 60 m
Length = 10 km
Coefficient = 0,4 (Table 2)
Area = 15 sq km

SOLUTION 1. Enter the map in Figure 3 and establish a zone value for Cape Town in the South Western zone (SW). P=3

2. Enter Figure 4 using H = 60 m
L = 10 km T = 7 hours

join T = 7 hours
SW= 3 I = 10 mm/hr

join I = 10 mm/hr
C = 0,4
A = 15 km²

Run-off Q = 16 cumecs

SOUTH WEST AFRICA

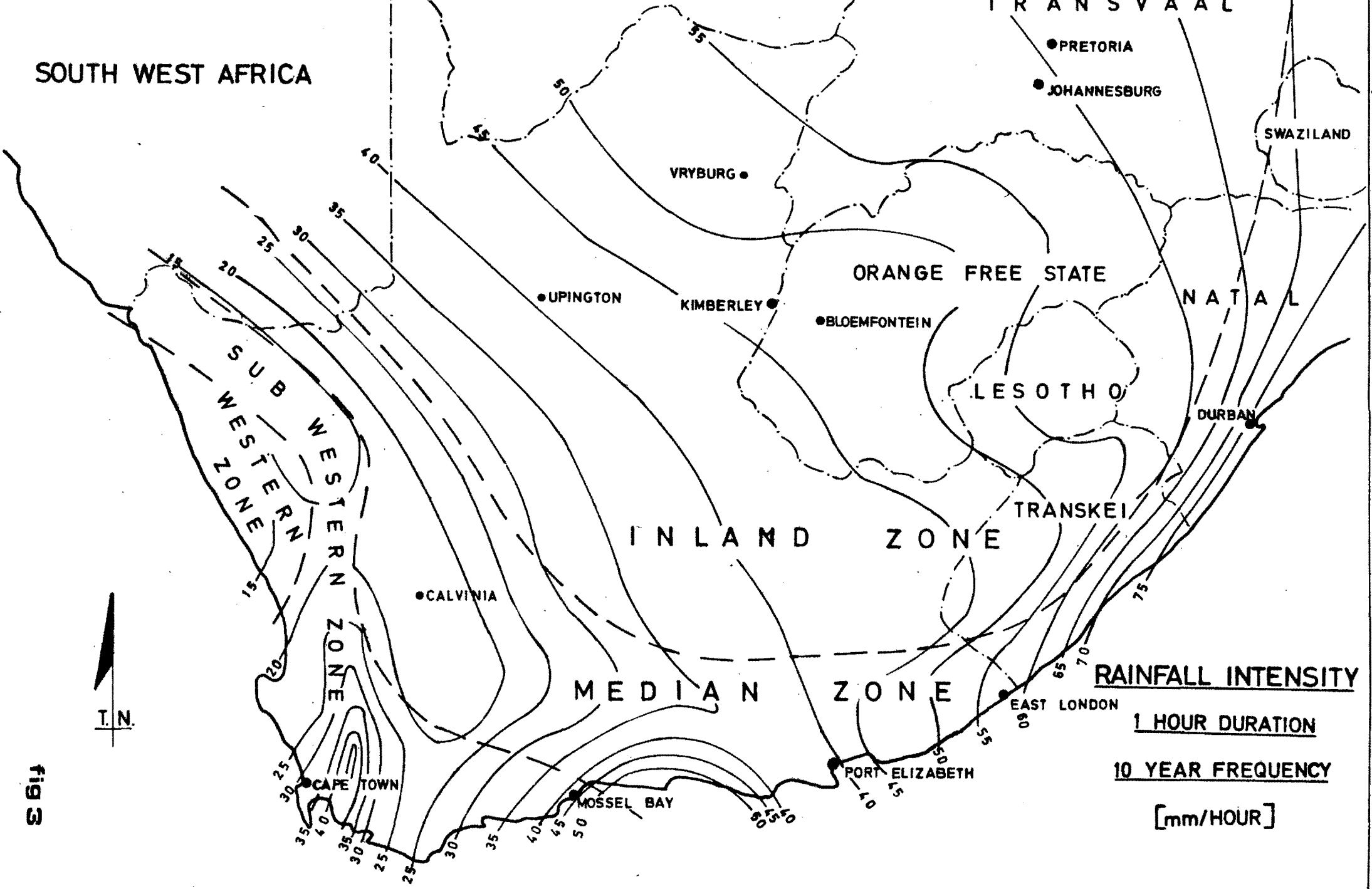


FIG 3



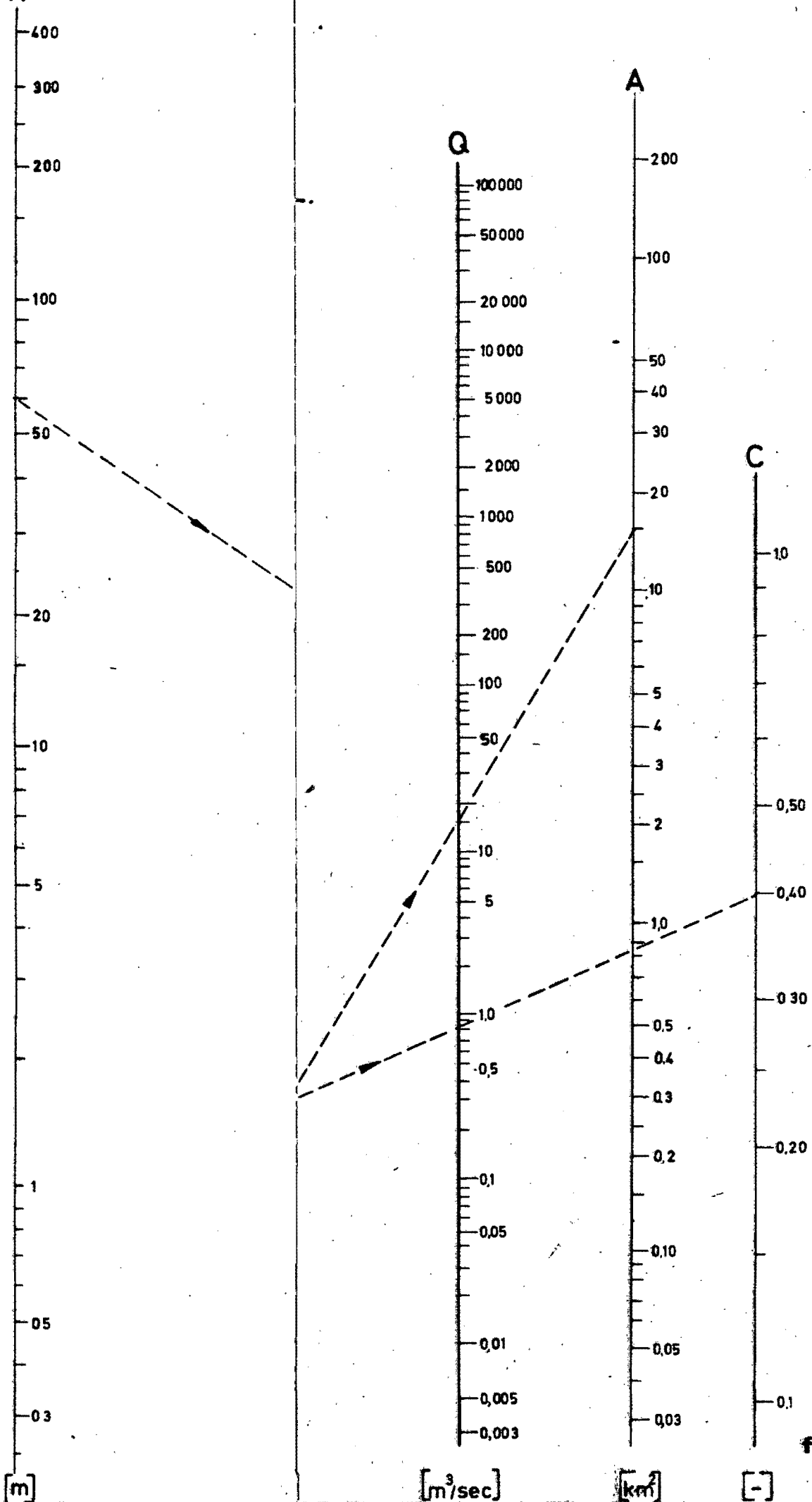
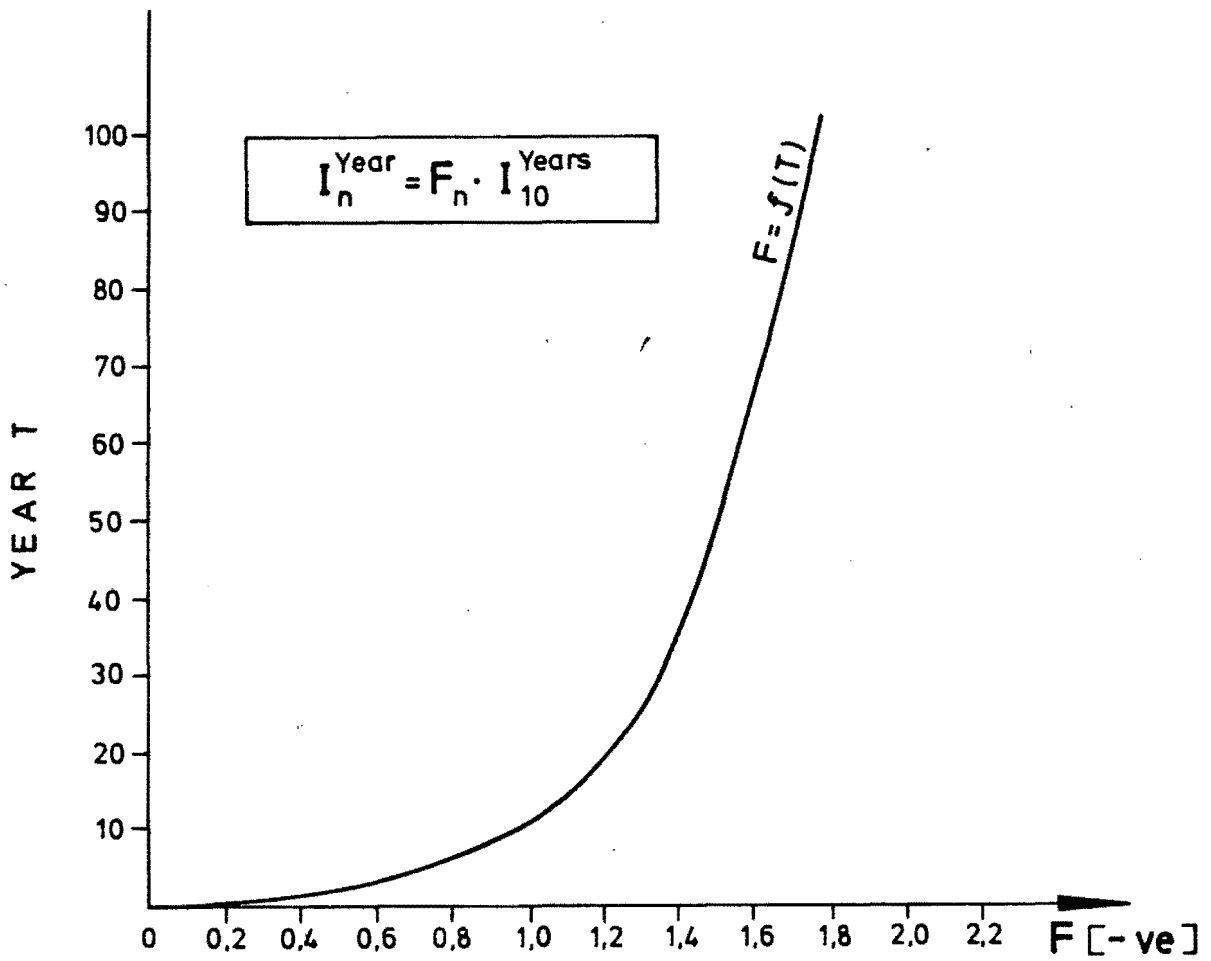


fig 4



THE VALUE OF FREQUENCY COEFFICIENT F
FOR THE RATIONAL METHOD

Chapter 5

Depth-Duration-Frequency Method

In December 1969 the former Hydrological Research Unit, operating under a fund established by the South African Institution of Civil Engineers, issued its Report No. 4/69 "Design Flood Determination in South Africa" - a manual for guidance in the application of hydrometeorological methods to hydraulic design. This was followed in February 1971 by Report No. 1/71 "Amendments to Design Flood Manual HRU 6/69" and this was subsequently metricated in Report No. 1/72. The present organisation which compiled the last two mentioned reports was a joint CSIR-University (Witwatersrand) Council Hydrological Research Unit (10).

The approach used by the Research Unit was basically that of the Rational Method. The particular subdivision chosen for this project is therefore for the smaller areas, up to 15 square kilometres, which forms a major portion of the engineer's drainage design.

The method is theoretically sound for small, relatively impervious areas where the coefficient of run-off tends towards unity and point rainfall values can be assumed. As the size and permeability of a catchment increases, estimates of the values for the coefficient of run-off become more problematic.

The approach is split up into two separate nomographs. The first nomograph on Figure 6, enables the time of concentration to be determined knowing the overall height drop and the total length of the catchment. The derivation of this nomograph is based on the most widely used of the empirical formulae, originally formulated by the U.S. Bureau of Reclamation. When converted to metric units it appears as follows:

$$T_c = \left(\frac{0,87L^3}{S} \right)^{0,385} \quad (11)$$

where S = dimensionless slope.

L = catchment length in kilometres

The second nomograph is in part a derivation from that produced in the Report No. 1/72. The calculation steps resorted to in the Report have been eliminated and the run-off can be read off directly on Figure 7.

- METHOD
1. Using the catchment height and length enter Figure 6 to obtain the time of concentration.
 2. Obtain a value for the coefficient of run-off C from the table in Figure 7. Using the values for C, annual precipitation, return period, rainfall zone and area obtain the run-off in cumecs.

To assist in obtaining a clearer idea of the use of the graphical approach the method is illustrated using the information in the following problem:

PROBLEM Consider a catchment area near Cape Town in cultivated lands with a slope of 3%, a height drop of 5 metres, a watercourse length of 10 kilometres, an area of 5 square kilometres and an annual rainfall of 2 500 mm. Determine the run-off for a 20 year frequency. Summarising the above the following is obtained:

H = 5 m
L = 10 km
A = 5 square km
C = 0,5
R = 2 500 mm/annum
Zone = Winter rainfall region

SOLUTION 1. Enter Figure 6 using H and L and obtain

$$T_c = 8 \text{ hours}$$

Enter Figure 7 starting with R travel vertically until the T_c line of 8 hours is reached. From this point drop vertically to the Winter Rainfall Region line.

Traverse to the right and read off the depth of precipitation.

Use the duration line with a value of 8 hours again and extend to intersect the intensity line.

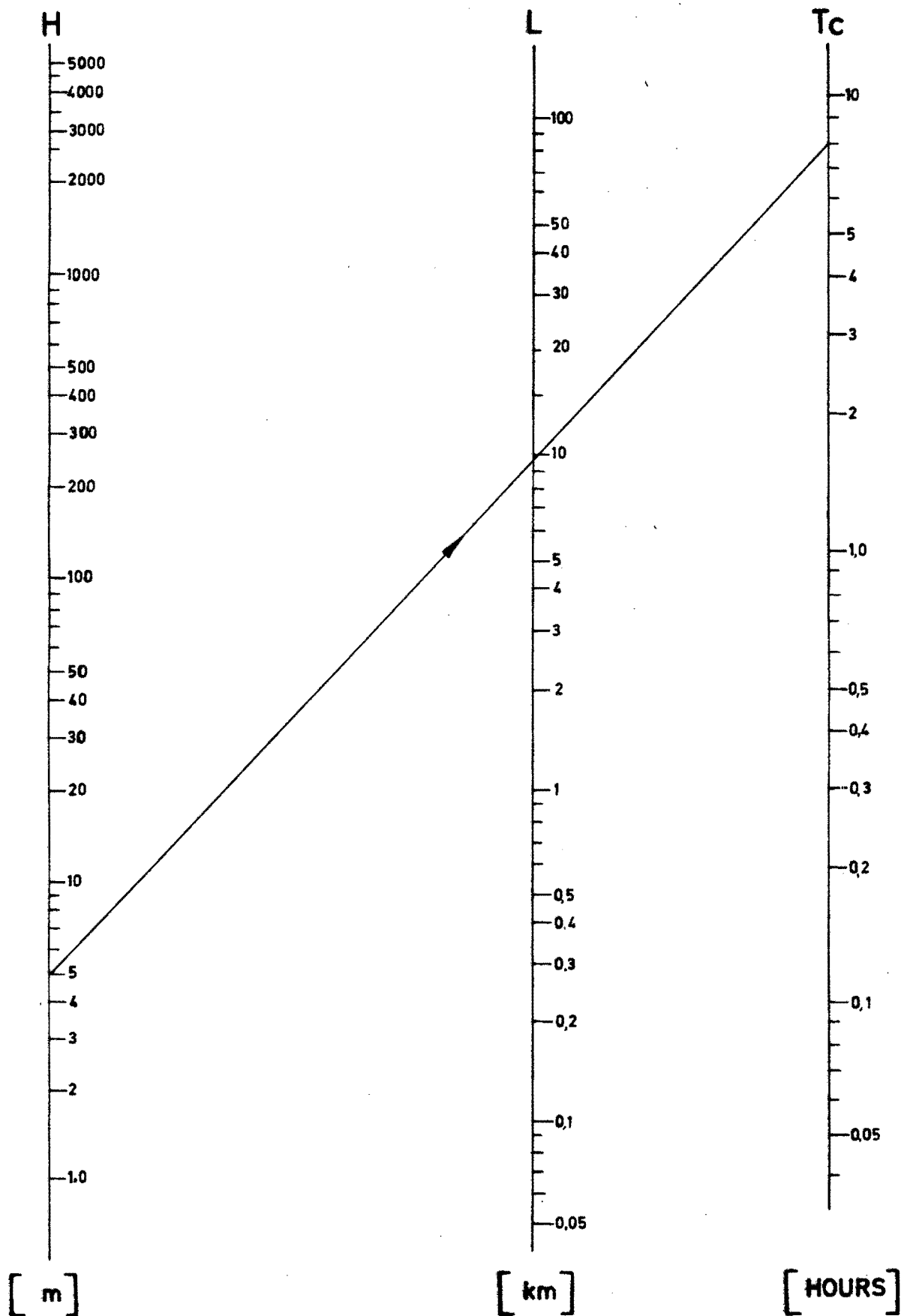
Join the intensity line to the run-off coefficient (C) value of 0,5 and intersect the turning line TL.

From the point on the TL line extend a line to the catchment area A.

The run-off can be read off directly from the Q line.

$$Q = 18 \text{ cumecs}$$

OF THE TIME OF CONCENTRATION EQUATION.



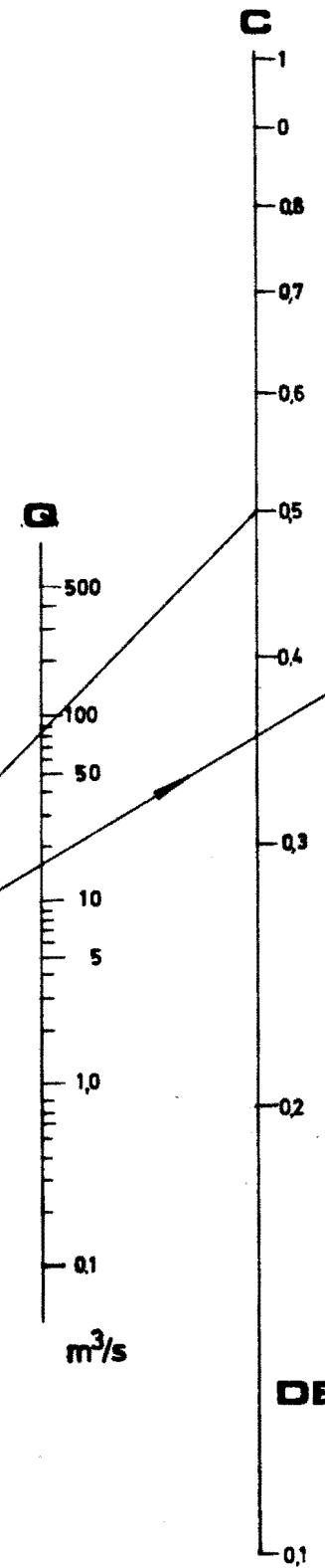
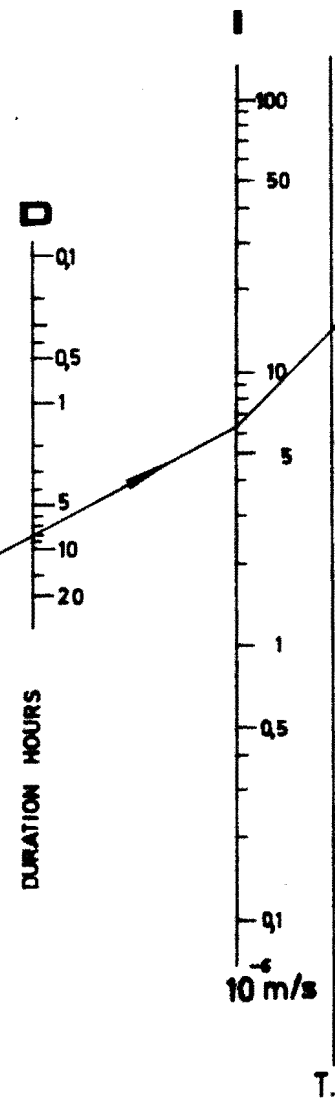
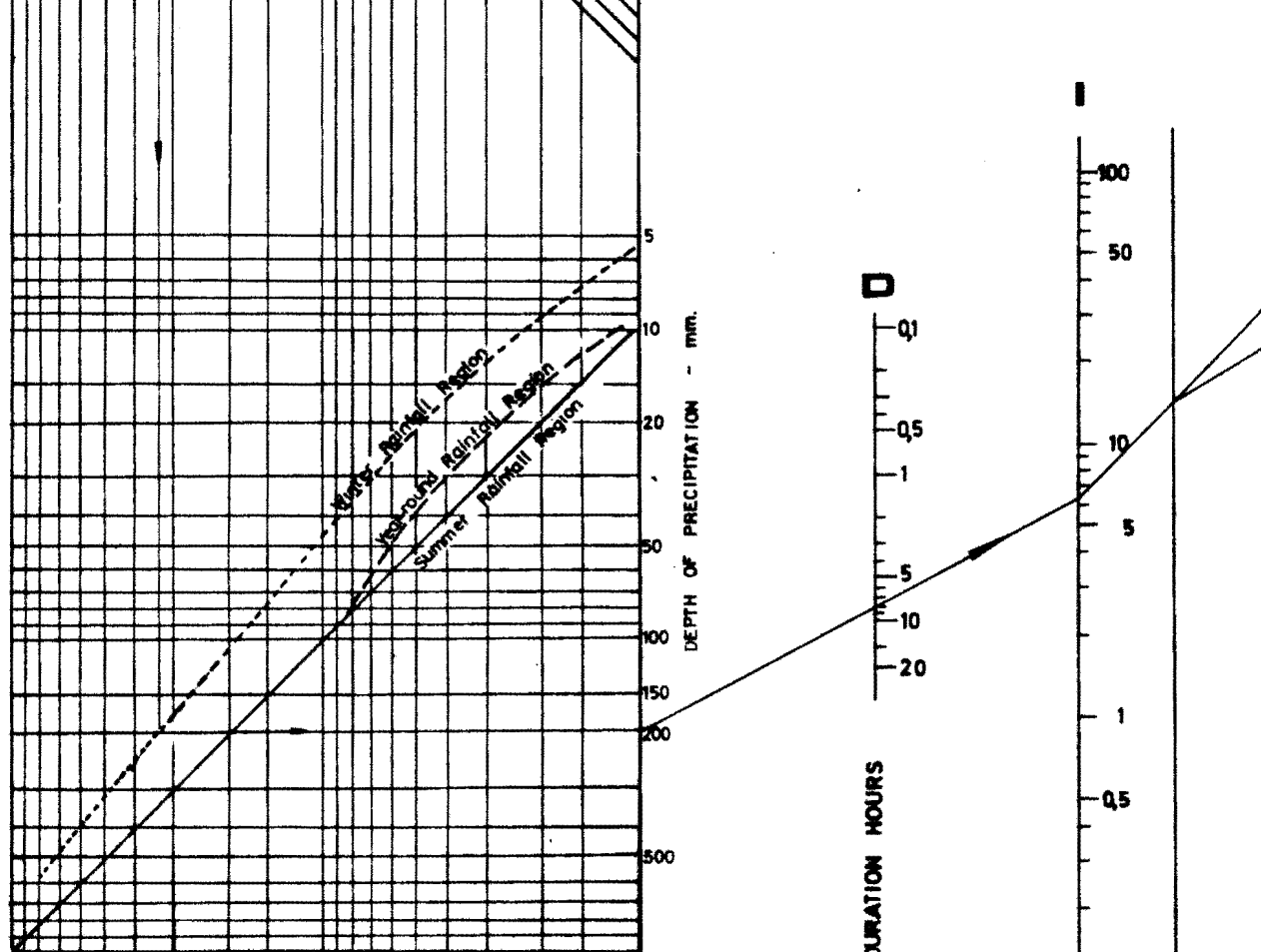
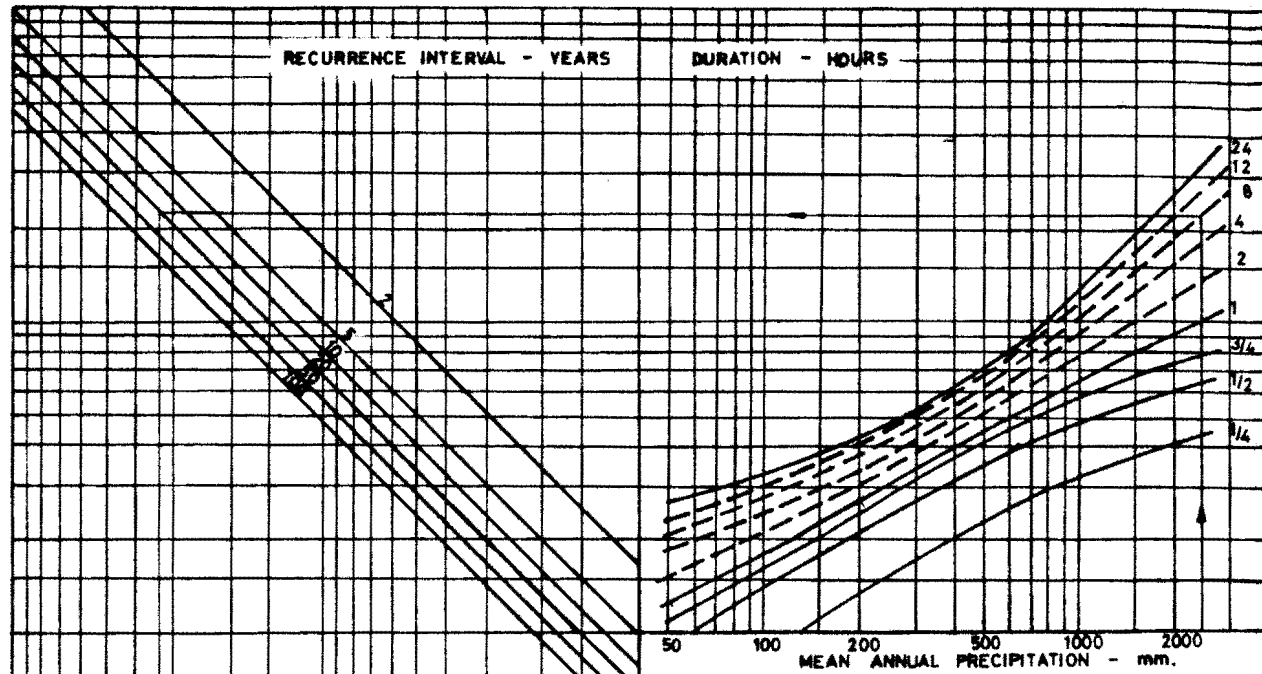
$$T_c = \left[\frac{0.87 L^3}{H} \right]^{0.385}$$

H = ELEVATION DIFFERENCE IS IN METRES.
 L = LENGTH OF MAIN CHANNEL IS IN KILOMETRES.
 Tc = TIME OF CONCENTRATION IS IN HOURS.

fig 6.

$Q = C.I.A / 3.63$

$0 < A < 15 \text{ [km}^2\text{]}$



RUN-OFF CO-EFFICIENT "C"			
OPEN AREAS - LESS THAN 5km ²			
SLOPE	SLOPE		
	0% - 5%	5% - 10%	10% -
LAWN SANDY SOIL	0,08	0,13	0,17
LAWN HEAVY SOIL	0,15	0,20	0,30
CULTIVATED LAND	0,50	0,60	0,70
GRASSLAND (VELD)	0,30	0,35	0,45
TIMBER	0,15	0,18	0,20
AREAS OVER 5km ²			
5 - 13 km ²	0,3 - 0,2		
13 - 130 km ²	0,2 - 0,1		
BUILT UP AREAS			
STREETS, PAVEMENTS	0,80		
PLAYGROUNDS	0,30		
PARKS (SEE TOP)	0,10 - 0,30		
RAILWAY YARDS	0,30		
ROOFS	0,90		
INDUSTRIAL AREAS:			
OPEN DEVELOPMENT	0,70		
UP TO 50% COVERED			
DENSE DEVELOPMENT			
MORE THAN 50% COVERED	0,80		
RESIDENTIAL AREAS:			
ONE HOUSE PER PLOT	0,40		
HIGH DENSITY TOWNSHIPS	0,60		
NEIGHBOURHOOD SHOPPING CENTRES	0,60		
CENTRAL BUSINESS DISTRICT	0,85		

DEPTH-DURATION-FREQUENCY METHOD

MODIFIED BY WRITER

km² fig 7

Chapter 6

Empirical (EMWD) Method

The Empirical Method is based on experiment and not on any theoretical basis. Attempts were made by a number of researchers in the USA to derive a closer relationship between rainfall, ground conditions and run-off.

Although this method could be attributed to a number of authors the main credit should be given to the USA War Department⁽⁷⁾. At present attempts are still being made to formulate more accurate values for the coefficient of run-off which results in most of the variation of results. It is well known that these values will change as many times as there are sets of data from which to work.

The method is very susceptible to the type of data and climatic zone from which the study is made and hence the results must be treated with a certain amount of caution. The results however are reasonably accurate for small and medium sized catchment areas and could be used in conjunction with the Rational Method.

The calculation of the run-off in a catchment is based on the following hypothesised formula which was derived from the "best fit" solution from field data collected:

$$Q = 0,015 GCA^{2/3}$$

where G = value of the geographic coefficient
given for various parts of the country.
C = coefficient of run-off
A = catchment area - square kilometres

The coefficient of run-off (C) for the use in South Africa can be derived from the list of values given in Table 3. The categories into which sections of the coefficient of run-off is subdivided include not only the soil cover, ground protection and ground slope but also factors such as collector slope and catchment shape. According to information assembled it has become evident that catchment shape influences the run-off determination. The explanation for this is that the time of concentration will vary appreciably depending on the catchment shape and this has caused considerable attention to be focussed on the Empirical Method. For this reason the run-off values obtained from this method are preferred by many local authorities.

In order to simplify the formula given two graphs have been prepared from which the run-off can be read off directly. Basically the two graphs are the same but for accuracy the first ranges for areas between 0,002 square kilometres to 10 square kilometres and the second between 2,5 square kilometres and 10 000 square kilometres. It is however expedient not to extend the run-off determination for this method beyond 2000 square kilometres as large variations may occur.

Table 3**COEFFICIENT OF RUN-OFF C FOR THE EMPIRICAL (EMWD) METHOD**

CATCHMENT		FACTORS
Type	Description	a:b:c:d:e
a) Soil Cover	Deep Permeable Sand	0
	Deep Sandy Soil	1
	Shallow Soil - Clay and Silt	2
	Impermeable Clay, Rock	3
b) Protection	Forest and Dense Bush	0
	Thick Grass and Bush: close seeded crops contoured and terraced	1
	Close Seeded Crops: imperfectly terraced; open crops contoured and terraced	1½
	Light Grass: close seeded crops not terraced; open crops imperfectly contoured	2
	Light and Sparse Bush: open crops not contoured	2½
	Bare Ground	3
c) General Ground Slope	Flat (0 - 3%)	1
	Moderate (3 - 8%)	2
	Steep (8 - 20%)	3
	Very Steep (over 20%)	4
d) Collector Slope	Flat (0 - 0,75%)	1
	Moderate (0,75 - 2,28%)	2
	Steep (2,28 - 7,58%)	3
	Very Steep (over 7,58%)	4
e) Catchment Shape	Length 4 or more times breadth	1
	Length 2 to 3 times breadth	2
	Circular, square, diamond shaped or length half breadth	3
	Semi-circular, baloon shaped or length half breadth	4

$$\text{Coefficient of Run-off } C = a+b+c+d+e$$

METHOD

1. From Table 3 derive the value for the coefficient of run-off C.
2. Using Figure 8 decide on the geographical coefficient G and the rainfall zone.
3. With values for G, C and H use either Figures 9 or 10 to determine the value for the run-off.

PROBLEM 1:

Assume a catchment area in Cape Town of 5 square kilometres with the following characteristics :

Shallow Soil	2
Light Grass	2
Steep (15%)	3
Steep Slope (3%)	3
Catchment Shape-Square	3

- SOLUTION 1:
1. The coefficient of run-off C is determined from the above characteristics; refer to Table 3.

$$C = 2 + 2 + 3 + 3 + 3 = 13 \quad C=13$$

2. The geographical coefficient for Cape Town from Figure 8. $G=45$

3. Using Figure 9 join $G = 45$ with $C = 13$ to obtain a point on the turning line TL. Join this point to the area $A = 2$ square kilometres and where the line intersects Q, the run-off can then be read off directly.

$$Q = 41 \text{ cumecs}$$

PROBLEM 2: Assuming that the following control figures have been determined for a particular catchment area :

$$G = 80$$

$$C = 4$$

$$A = 800 \text{ square kilometres}$$

SOLUTION 2: 1. Using the above values it becomes obvious that Figure 10 should be used as the area is fairly large.

As in the above solution join G to C and obtain an intersecting point on the TL.

Join this point with A to obtain an intersection with the Q line.

$$Q = 410 \text{ cumecs}$$

The run-off determination , as obtained from Figures 9 and 10, is for a 10 year return period which in a number of cases is not suitable for engineering problems. In order to convert the run-off from a 10 year return period Figure 11 has been prepared so that the run-off at any desired return period can be determined. Factor F is read off corresponding to the desired return period and multiplied by the 10 year run-off producing the required design run-off.

SOUTH WEST AFRICA

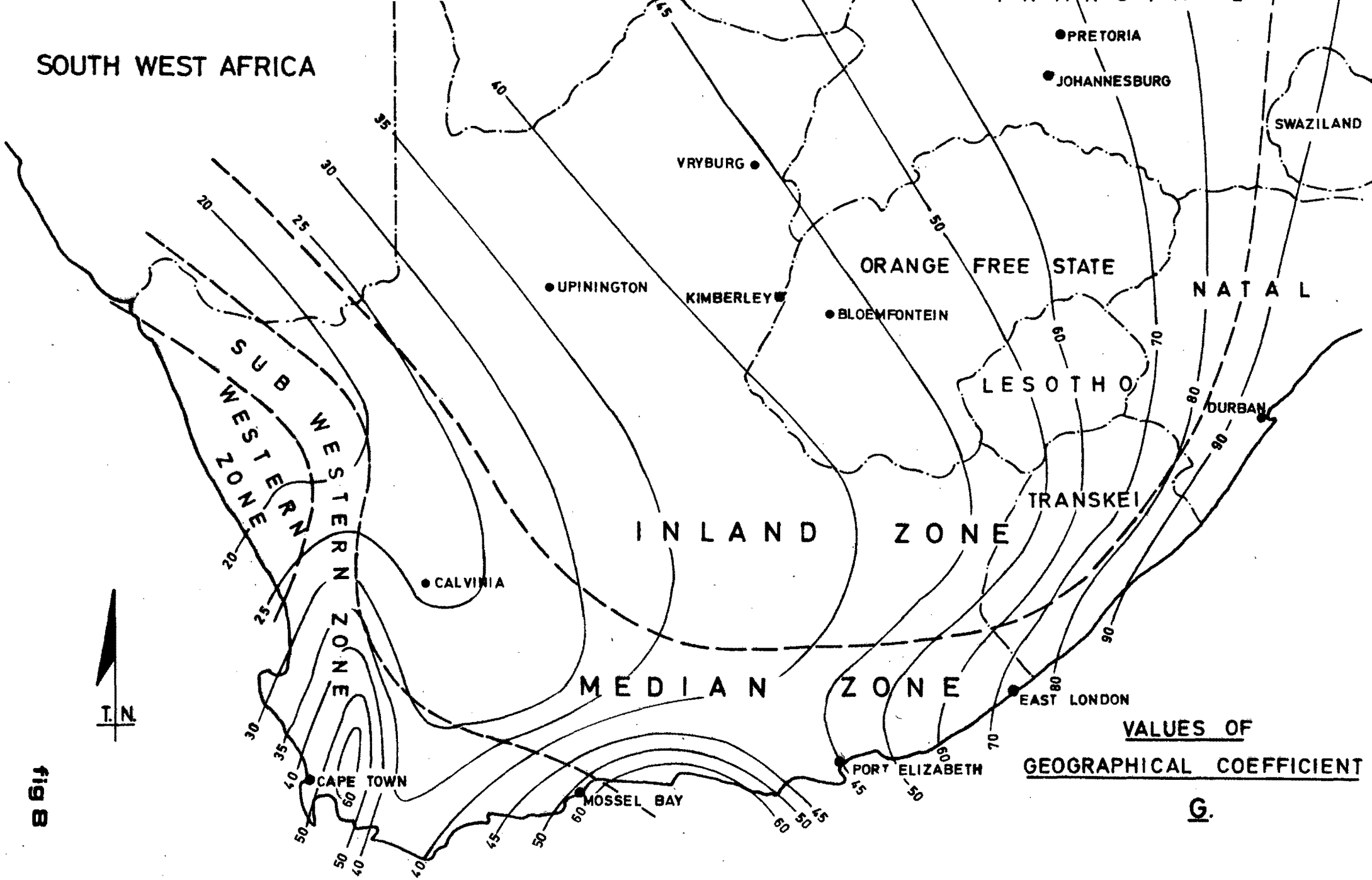


FIG 8

EMPIRICAL EMWD METHOD

G = GEOGRAPHICAL COEFFICIENT
 C = CATCHMENT COEFFICIENT
 A = CATCHMENT AREA

FOR $0,002 < A < 100 \text{ [km}^2\text{]}$

$$Q = 0,015 \cdot G \cdot C \cdot A^{2/3}$$

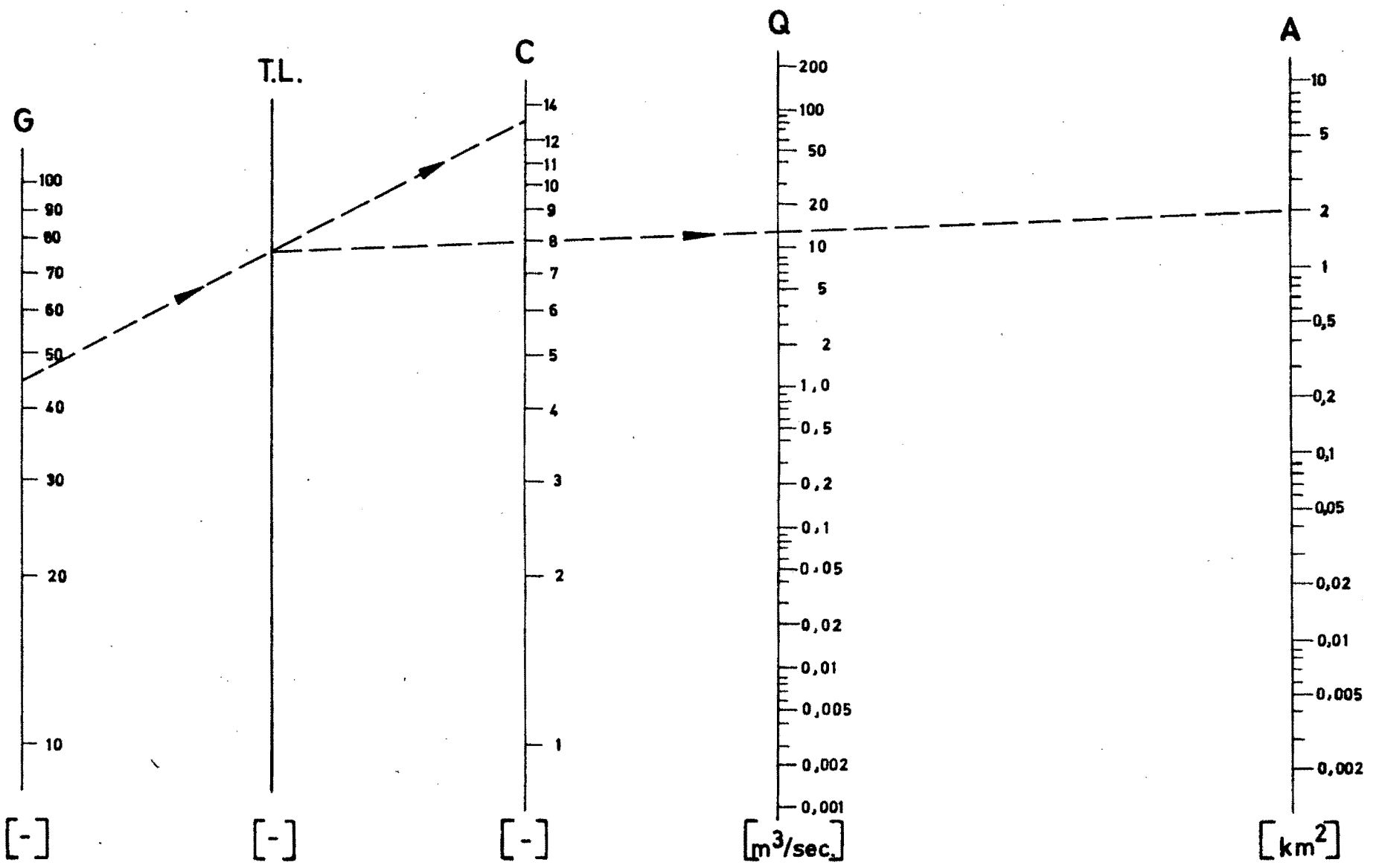


fig 9

EMPIRICAL EMWD METHOD

FOR $2,5 < A < 10\,000$ [km²]

$$Q = 0,015 \cdot G \cdot C \cdot A^{2/3}$$

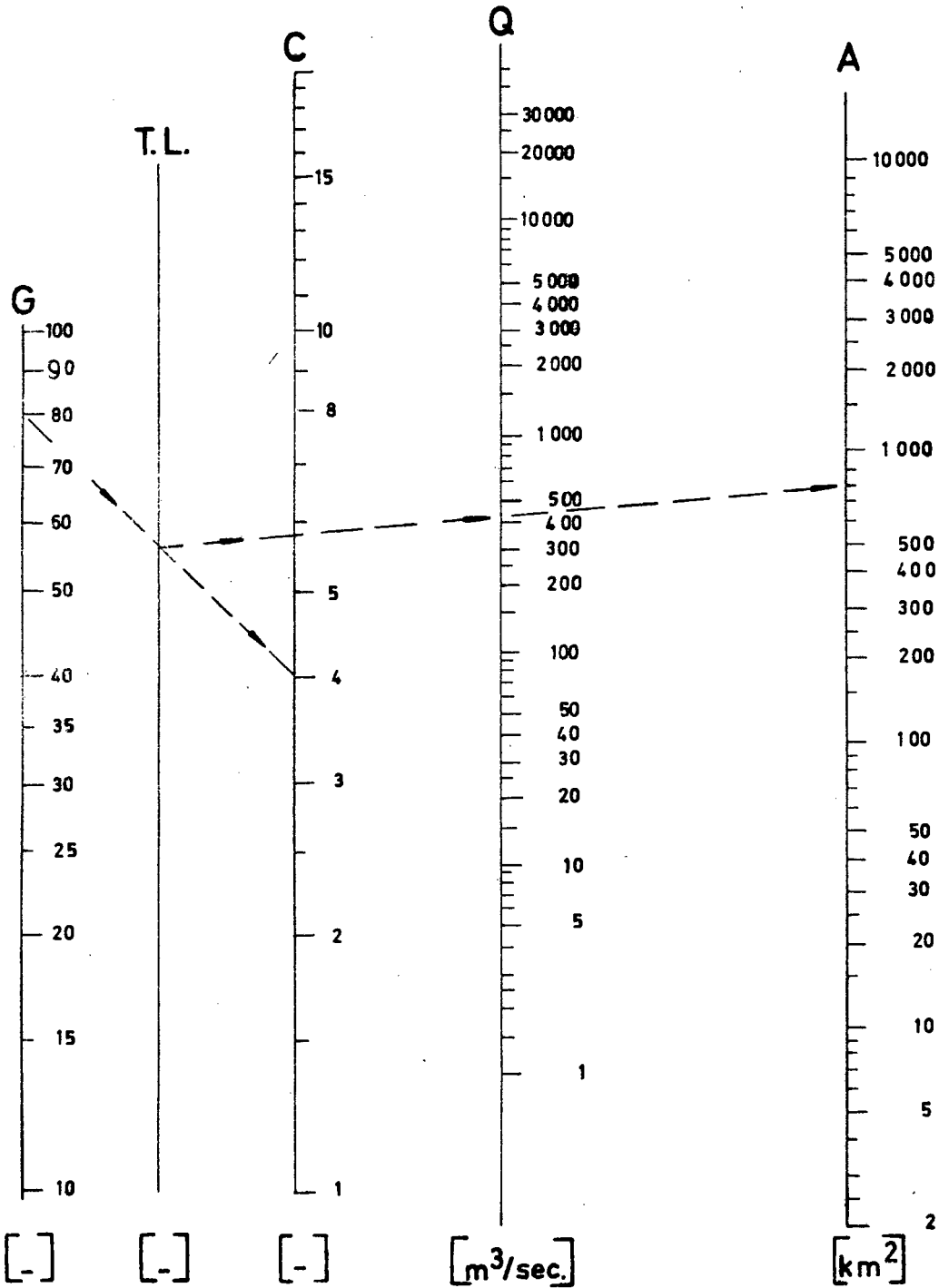
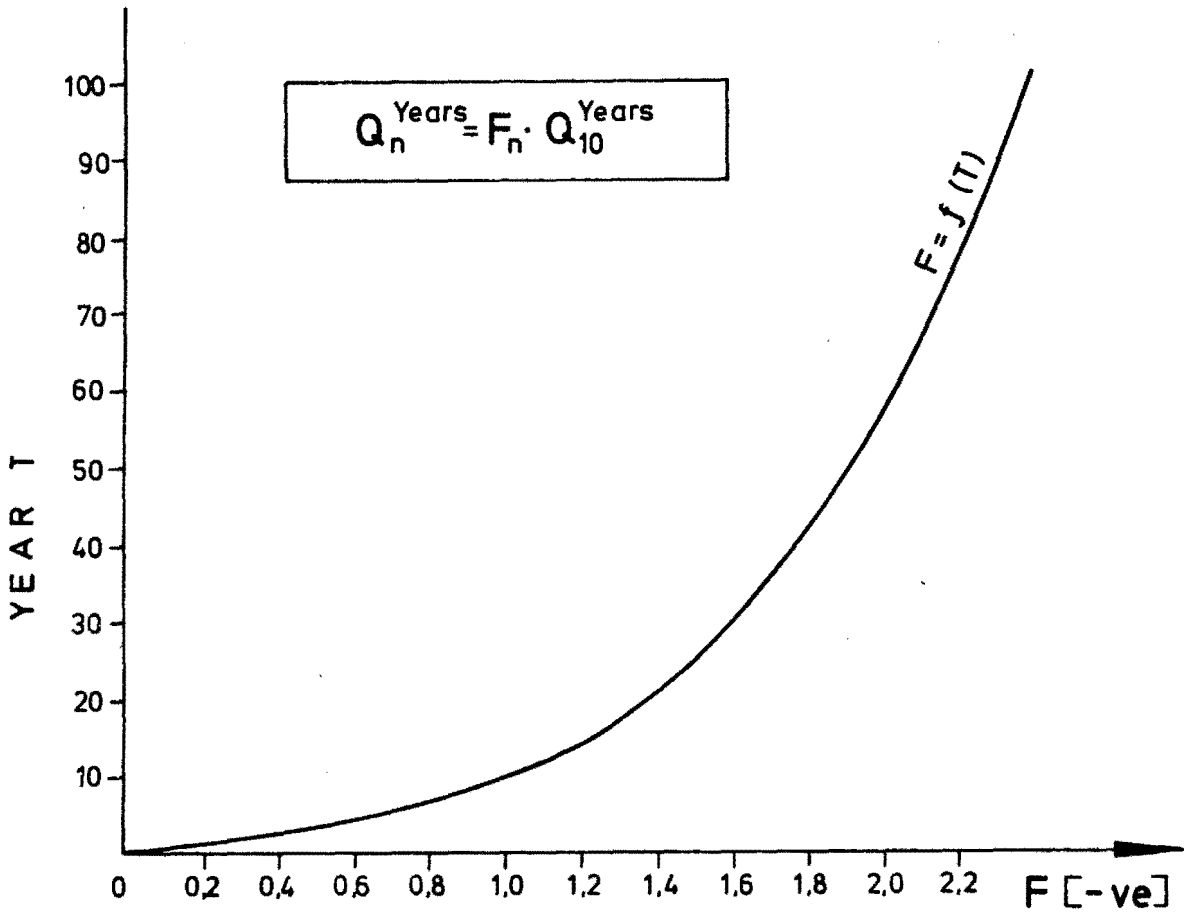


fig 10



THE VALUE OF FREQUENCY COEFFICIENT F
FOR THE EMPIRICAL METHOD

Chapter 7

Mc Math Method

Although a number of empirical approaches has been adopted by designers perhaps one of the more popular is the Mc Math Formula. This method of run-off calculation is favoured particularly by some of the larger municipalities in Southern Africa.

The Mc Math Formula has been quoted in a number of publications such as by Metcalf and Eddy⁽¹¹⁾, and Seelye⁽¹²⁾. The calculation as set out in Seelye uses the USA as a basis. Reich reproduced comparable curves for South Africa in his M.Sc. thesis "Probable Maximum Precipitations for Short Durations"⁽¹³⁾.

In his research into the precipitation Reich compiled Figures 12 and 13 which deal with the expected 2 year maximum precipitation for any one hour period, the ratio of maximum precipitation between 2 and 100 years, respectively.

Mc Math's formula, converted into metric units, is postulated as follows:

$$Q = 0,145 CIA \sqrt[5]{\frac{S}{A}} \quad (13)$$

where S = percent slope of catchment
I = intensity in mm/hour
A = catchment area in square kilometres
C = coefficient of run-off as given in Table 4

Table 4

VALUES FOR THE COEFFICIENT OF RUN-OFF C

SURFACES		C _{min}	C _{max}	
Roofs	Slag To Metal	0,90	1,00	
Road Surfaces	Concrete Or Asphalt	0,90	1,00	
	Bituminous Macadam, Open and Closed Type	0,70	0,90	
	Gravel From Clean And Loose To Clayey And Compact	0,25	0,70	
Railroad Yards		0,10	0,30	
Earth Surfaces	<u>Sand</u> : From Uniform Grain Size With No Fines, To Well Graded With Some Clay Or Silt	Bare	0,15	0,50
		Light Vegetation	0,10	0,40
		Dense Vegetation	0,05	0,30
	<u>Loam</u> : From Sandy Or Gravelly To Clayey	Bare	0,20	0,60
		Light Vegetation	0,10	0,45
		Dense Vegetation	0,05	0,35
	<u>Gravel</u> : From Clean Gravel And Gravel-Sand Mixtures With No Silt Or Clay, To High Silt Or Clay Content	Bare	0,25	0,65
		Light Vegetation	0,15	0,50
		Dense Vegetation	0,10	0,40
	<u>Clay</u> : From Course Sandy Or Silty To Pure Colloidal Clays	Bare	0,30	0,75
		Light Vegetation	0,20	0,60
		Dense Vegetation	0,15	0,50
Composite Areas	City, Business Areas	0,60	0,75	
	City, Dense Residential Areas: Vary As To Soil And Vegetation	0,50	0,65	
	Suburban Residential Areas: Vary As To Soil And Vegetation	0,35	0,55	
	Rural Districts: Vary As To Soil And Vegetation	0,10	0,25	
	Parks, Golf Courses, Etc.: Vary As To Soil And Vegetation	0,10	0,35	

METHOD

1. Determine the factor of run-off from Table 4.
2. From Figure 12 obtain the expected 2 year maximum precipitation for any 60 minute period.
3. From Figure 13 obtain the precipitation ratio for a 100 year /2 year period.
4. Enter Figure 14 using the overland flow, the character of the ground and the catchment slope obtain the time of concentration in minutes.
5. Enter Figure 15 with the above time, traverse up to the line corresponding with the 2 year precipitation obtained in (2) above, travel horizontally to the adjusted 2 year intensity line. Multiply this result with the 100/2 year ratio found in (3) above, plot it on the 100 year line, join the 2 year - 100 year values and then read off the intensity for the desired return period.
6. Using the nomograph in Figure 16 obtain the run-off Q by traversing the scales with the intensity above with the run-off coefficient then to the catchment slope and finally the catchment area.

PROBLEM

Assume a catchment area near Bloemfontein with the following characteristics:

Area 10 square km
Length 4 km
Height 80 m
Catchment Slope (general) 5%
20 year return period
Gravel surface with high clay content

- SOLUTION
1. Table 4 $C = 0,5$
 2. Figure 12 2 year precipitation = 32 mm/hr
 3. Figure 13 100/2 year ratio = 2,4
 4. Figure 14 Duration $D = 28$ minutes
 5. Figure 15 2 year amended intensity $I = 54$ mm/hr
100 year intensity $I = 130$ mm/hr
20 year intensity $I = 100$ mm/hr
 6. Figure 16 use nomograph:

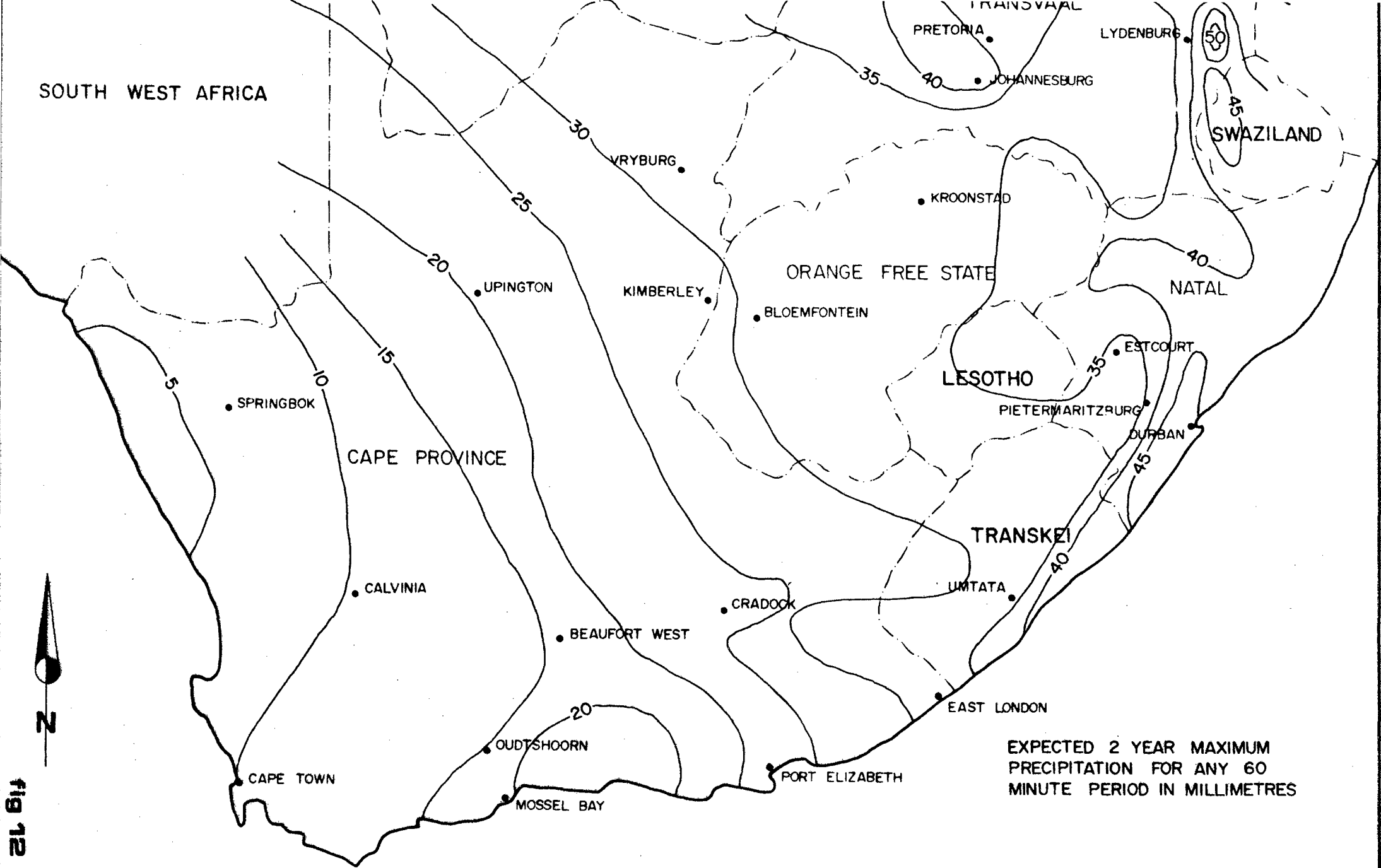
$I = 100$ mm/hr with $C=0,5$
to pivot line 1.

Pivot line 1 with $S=5\%$
to pivot line 2.

Pivot line 2 with $A=10$ km²
to Q.

Run-off $Q = 68$ cumecs

SOUTH WEST AFRICA



EXPECTED 2 YEAR MAXIMUM
PRECIPITATION FOR ANY 60
MINUTE PERIOD IN MILLIMETRES



fig 12

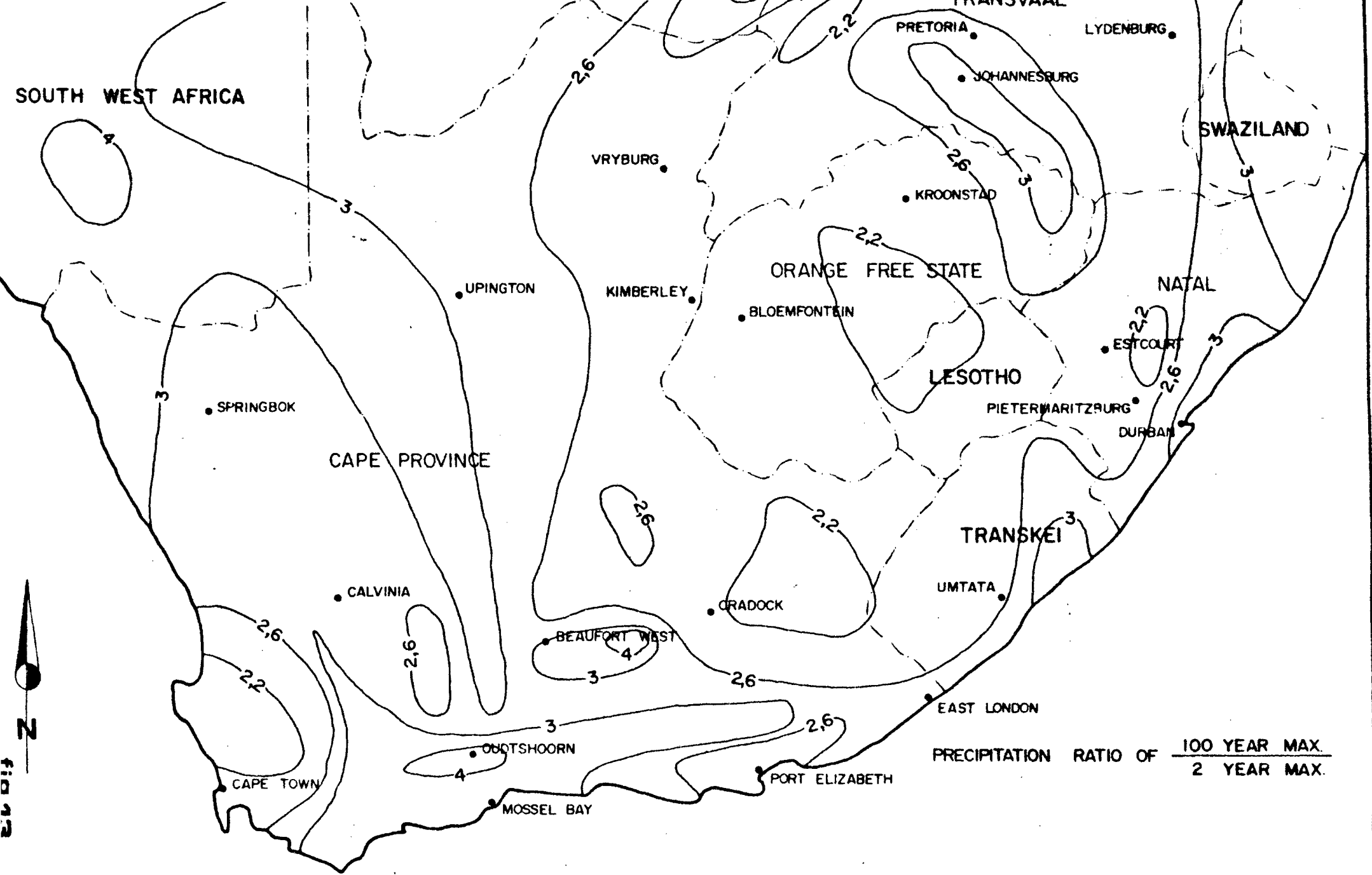
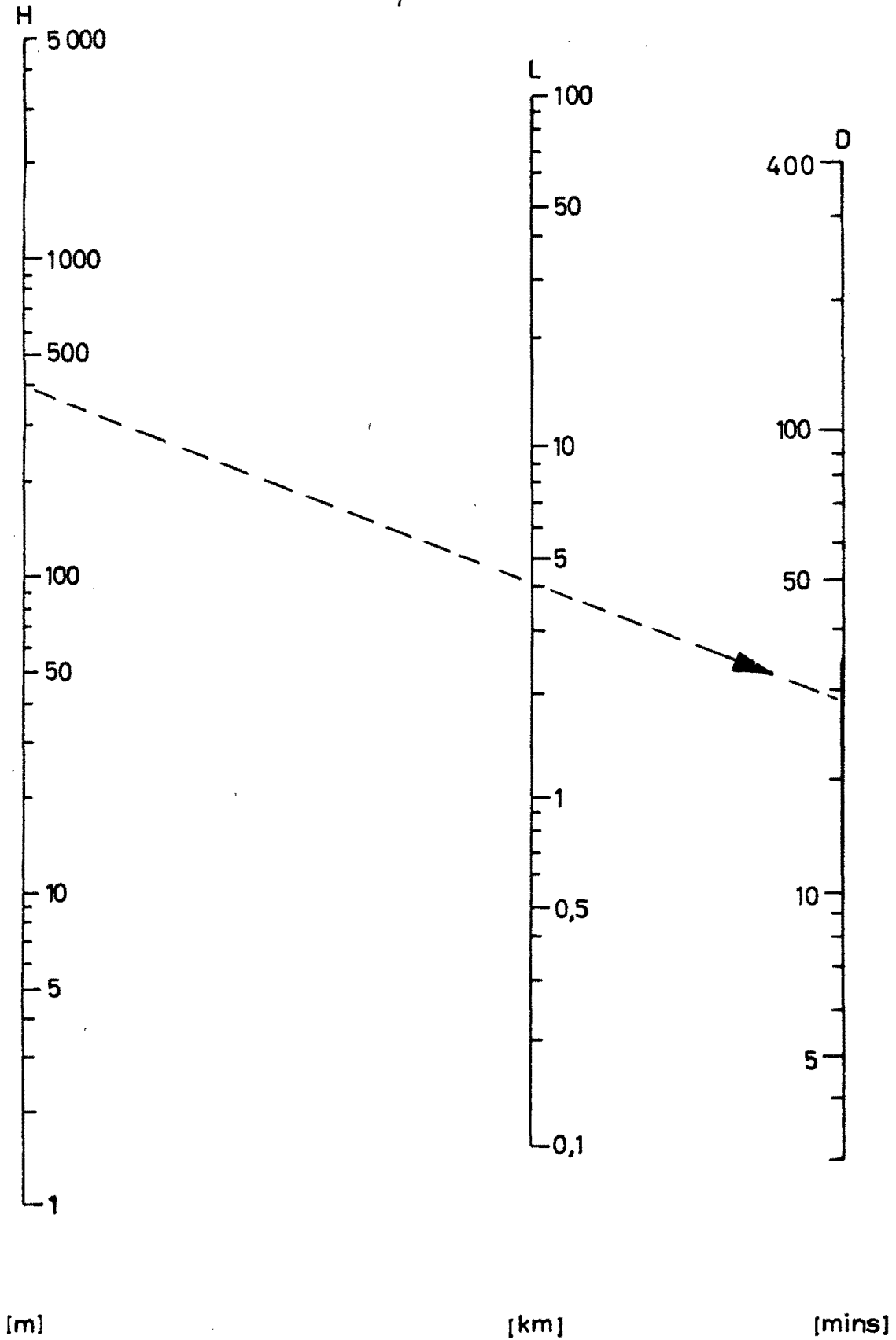


fig 13

PRECIPITATION RATIO OF $\frac{100 \text{ YEAR MAX.}}{2 \text{ YEAR MAX.}}$

Storm Duration



COMPILED BY WRITER

fig 14

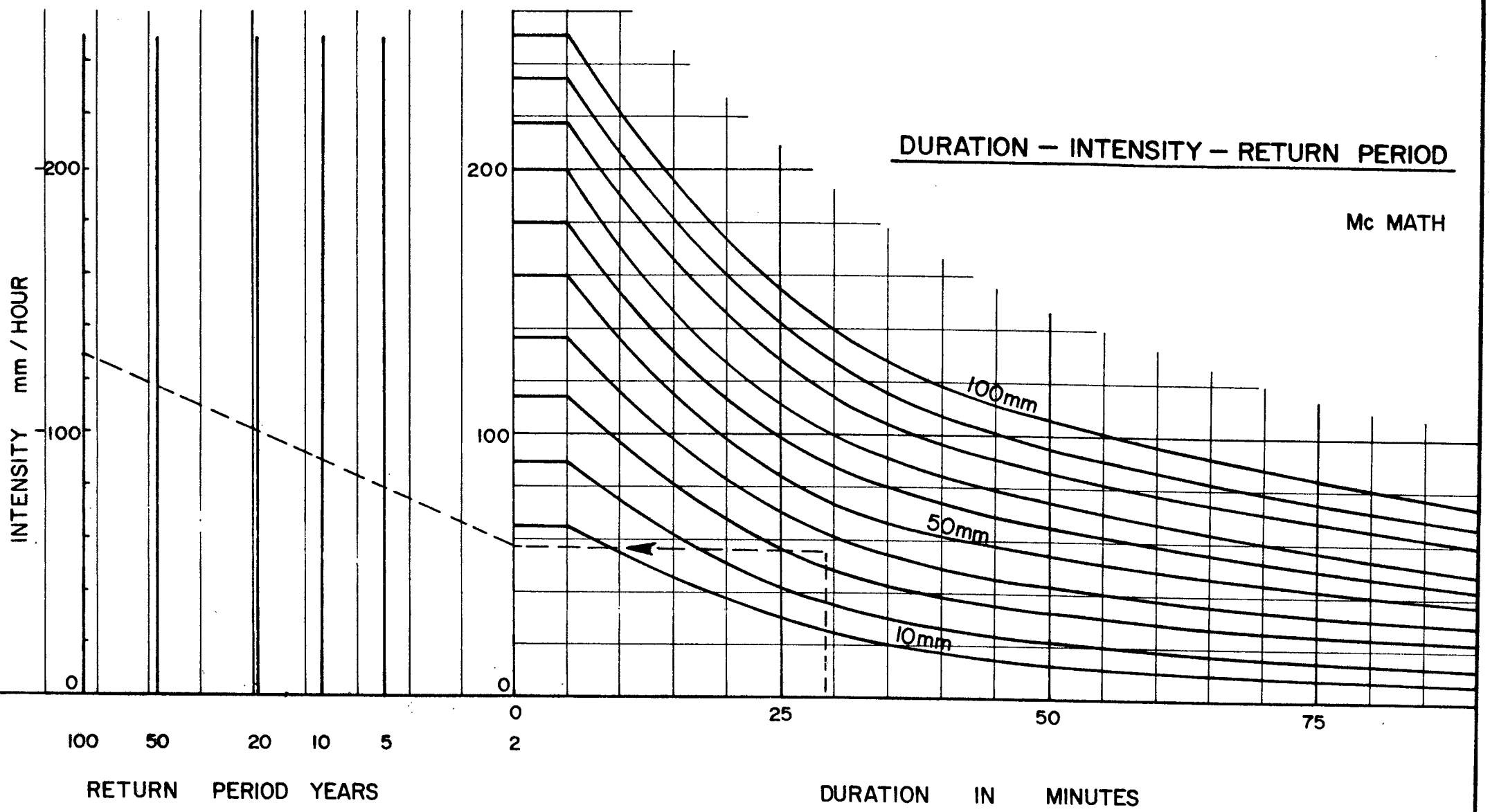
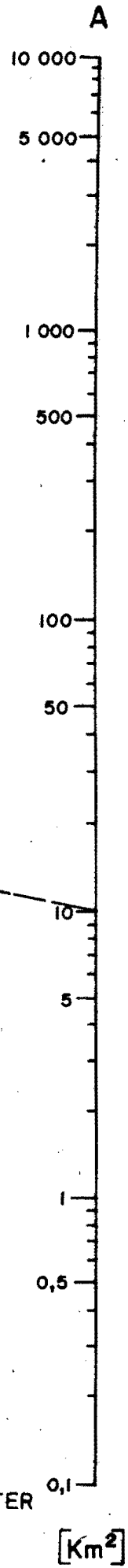
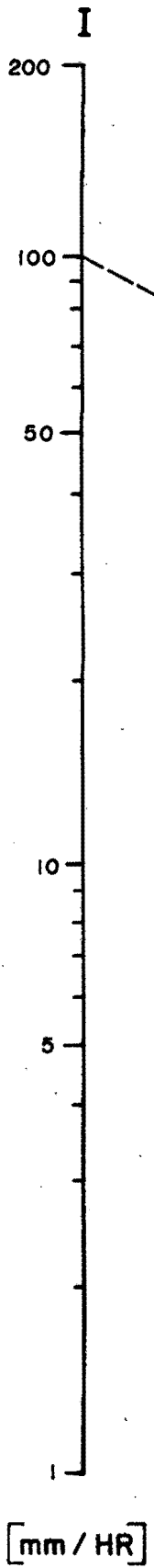


fig 15

COMPILED BY WRITER

TH METHOD



COMPILED BY WRITER

fig 16

The Reich Method of calculating the run-off was originally put forward as the "Soil Conservation Service Design Hydrograph Method" (13) and enjoyed a certain popularity in the early sixties. It was one of the first attempts at undertaking a comprehensive investigation into soil conditions for the determination of run-off in South Africa. It is still considered a fairly reliable means of assessing the run-off but has been somewhat overshadowed by later work undertaken by the Hydrological Research Unit, at the University of the Witwatersrand.

B.M. Reich undertook as a thesis for his M.Sc degree, at the Iowa State University of Science and Technology in 1959, the subject being "Probable Maximum Precipitations for Short Durations in the Union of South Africa". On his return to South Africa he gave a number of talks in connection with his work and a paper was duly published in the S.A.I.C.E. transactions in 1962 (13).

The method attempts to combine the following criteria:

1. Antecedent conditions likely during any design storm.
2. Soil classification according to run-off potential.
3. Correlation between storms for different durations.
4. Correlation between storms for different return periods.
5. Direct run-off based on a hydrograph.
6. Determination of a theoretical storm duration for specific areas.

To simplify the rather complex procedure derived by Reich tables, graphs and nomographs have been compiled in this project by the writer. Reich's formula has also been simplified and is as follows in metric units:

$$Q = 28,6 \frac{q S^{1/5} A^{11/10}}{L D} \text{ cumecs}$$

where q = direct run-off in mm
 S = percent slope of catchment
 A = catchment area in square km
 L = catchment length in km
 D = duration of storm in hours

Also used in the graphs

P = total run-off in mm

The following tables have been derived to categorize the soil into groups and run-off numbers :

Table 5

SOIL CLASSIFICATION ACCORDING TO RUN-OFF POTENTIAL

GROUP A	Includes deep sands with very little silt and deep rapidly permeable loess - lowest run-off potential
GROUP B	Includes sandy soils less deep than GROUP A, and loess less deep or less aggregated than GROUP A, but as a whole has above-average infiltration after thorough wetting - higher run-off potential than GROUP A.
GROUP C	Includes such materials as shallow soils and soils containing considerable clay and colloid, although less than those of GROUP D. It has below-average infiltration after pre-saturation and has a higher run-off potential than GROUP B.
GROUP D	Includes such materials as clays of high swelling property and some shallow soils with nearly impermeable subhorizons near the surface and has the highest run-off potential.

Table 6

RUN-OFF CURVE NUMBERS FOR HYDROLOGIC SOIL COVER COMPLEXES

LAND USE OR COVER	TREATMENT OR PRACTICE	HYDROLOGIC CONDITION	HYDROLOGIC SOIL GROUP			
			A	B	C	D
Fallow	Straight row		77	86	91	94
Row crops	Straight row	Poor	72	81	88	91
		Good	67	78	85	89
	Contoured	Poor	70	79	84	88
		Good	65	75	82	86
	Contoured and terraced	Poor	66	74	80	82
		Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Contoured	Poor	63	74	82	85
		Good	64	73	81	84
	Contoured and terraced	Poor	64	72	79	82
		Good	59	70	78	81
Close- seeded legumes or rotation meadows	Straight row	Poor	66	77	85	89
		Good	58	72	81	85
	Contoured	Poor	64	75	83	85
		Good	55	69	78	83
	Contoured and terraced	Poor	63	73	80	83
		Good	54	67	76	80
Pasture or range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
		Fair	25	59	76	83
		Good	6	35	70	79
Meadow permanent		Good	30	58	71	78
Woods and forests		Very poor	56	76	86	91
		Fair	36	60	73	79
		Very good	15	44	54	64
Farmsteads Roads dirt hard			59	74	82	86
			72	82	87	89
			74	84	90	92

Table 7
WATERSHED CONDITION FOR 5 DAY ANTECEDENT RAINFALL

CONDITION	GROWING SEASON	DORMANT SEASON
CONDITION I	Less than 35 mm	Less than 15 mm
CONDITION II	35 mm to 55 mm	15 mm to 30 mm
CONDITION III	Greater than 55 mm	Greater than 30 mm

METHOD

The calculation procedure is as follows :

1. Determine the average annual precipitation from the rainfall charts.
2. Determine the soil classification group number according to run-off potential from Table 5.
3. Obtain a curve number for the hydrological soil cover from Table 6 using the group number from Table 5.
4. Decide on the type of condition given in Table 7.
5. Using the curve number and watershed condition enter Figure 17 to ascertain the revised curve number based on the watershed conditions. Retain this for entry into Figure 23 used later in the calculation cycle.

6. Obtain
 - (a) the expected 2 year maximum precipitation for any 24 hour period from Figure 18.
 - (b) the expected 2 year maximum precipitation for any 60 minute period from Figure 19.
 - (c) the precipitation ratio of 100 year to 2 year return periods from Figure 20.

7. Enter Figure 21 with the average annual precipitation and obtain the effective average storm duration. Based on the duration use the 24 hour and 60 minute precipitations from Figures 18 and 19 obtain the precipitation for this specific storm.

8. Using the precipitation derived above and the 100/2 year ratio, obtained from Figure 20, scale off the precipitation for the desired design return period from Figure 22. Combine the results from the graph of the area - area/point rainfall ratio and derive the value for the total rainfall P by means of the nomograph.

9. Enter Figure 23 using P and the curve number from Figure 17 obtain the direct run-off q for the catchment.

10. The nomograph in Figure 24 then solves for the catchment run-off.

PROBLEM

Assume a catchment area near Bloemfontein with the following characteristics :

- Area 5 square km
- Length 7 km
- Slope 0,5%
- Annual rainfall 500 mm
- Shallow soils containing considerable clay.
- Fair pasture lands.
- 5 day precipitation in growing season less than 35 mm.

SOLUTION

- 1. Table 5 Group C
- 2. Table 6 79
- 3. Table 7 CONDITION I
- 4. Figure 17 using Condition I Curve No = 62
- 5. Figure 18 24 hour precipitation = 51 mm
- 6. Figure 19 60 minute precipitation = 31 mm
- 7. Figure 20 100/2 year ratio = 2,35
- 8. Figure 21 Summer rainfall D = 3 hours
- Precipitation per 60 minutes = 32 mm
- 9. Figure 22 using 100/2 year ratio
32 mm x 2,35 = 75 mm P = 50 mm
- 10. Figure 23 Curve No. 62 P = 50 mm q = 5 mm
- 11. Figure 24 use nomograph

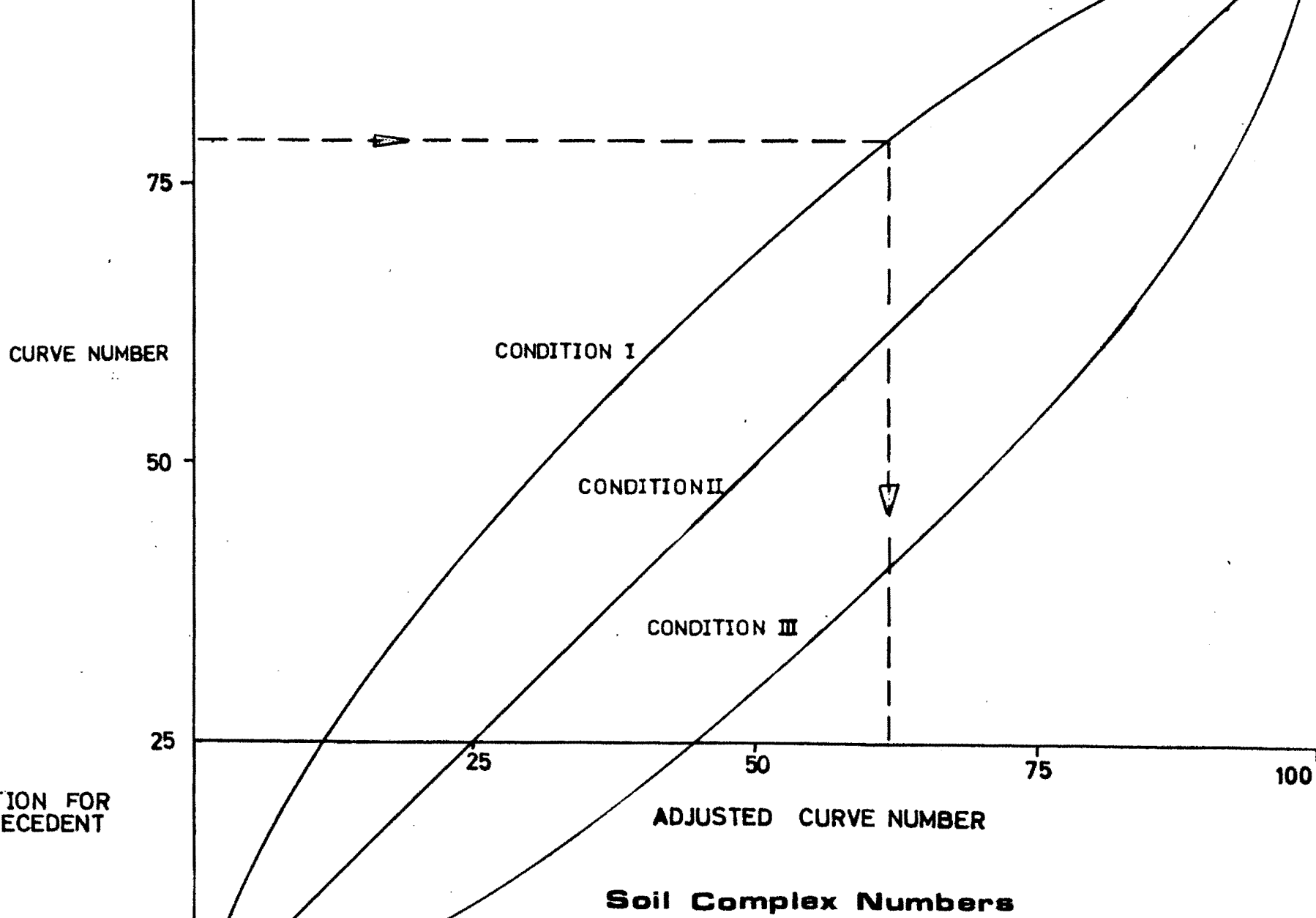
q = 5 mm with D = 3 hours to pivot line 1.

Pivot line 1 with S = 10% to pivot line 2.

Pivot line 2 with S = 10 km to pivot line 3.

Pivot line 3 with A = 50 km² to Run-off Q.

Run-off Q = 150 cumecs

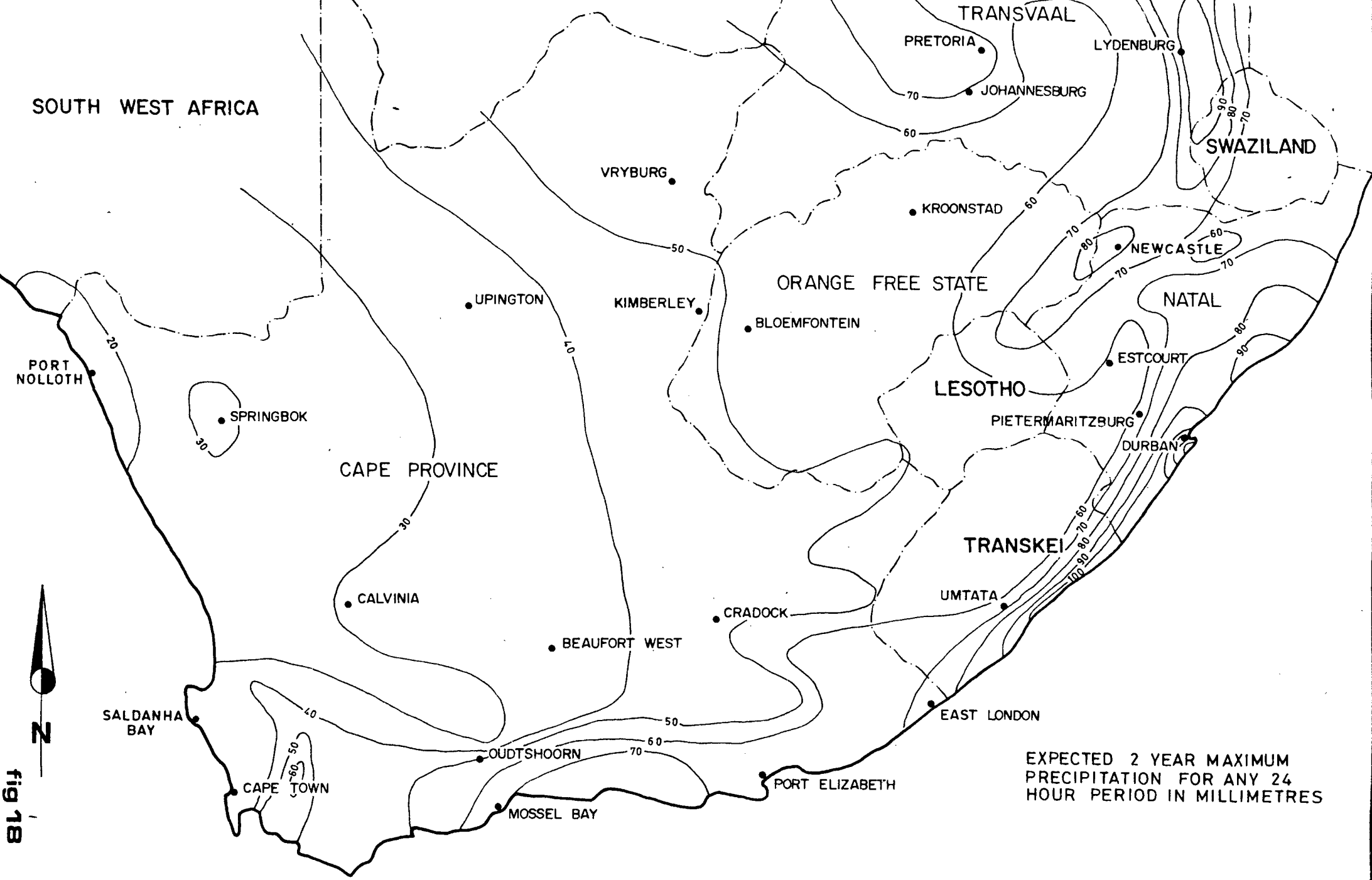


NOTE: PRECIPITATION FOR
5 DAY ANTECEDENT
RAINFALL

Soil Complex Numbers

COMPILED BY WRITER

FIG 17



SOUTH WEST AFRICA

TRANSVAAL

PRETORIA

LYDENBURG

JOHANNESBURG

SWAZILAND

VRYBURG

KROONSTAD

NEWCASTLE

ORANGE FREE STATE

NATAL

UPINGTON

KIMBERLEY

BLOEMFONTEIN

ESTCOURT

LESOTHO

PIETERMARITZBURG

DURBAN

CAPE PROVINCE

TRANSKEI

UMTATA

CRADOCK

BEAUFORT WEST

EAST LONDON

SALDANHA BAY

CAPE TOWN

MOSSEL BAY

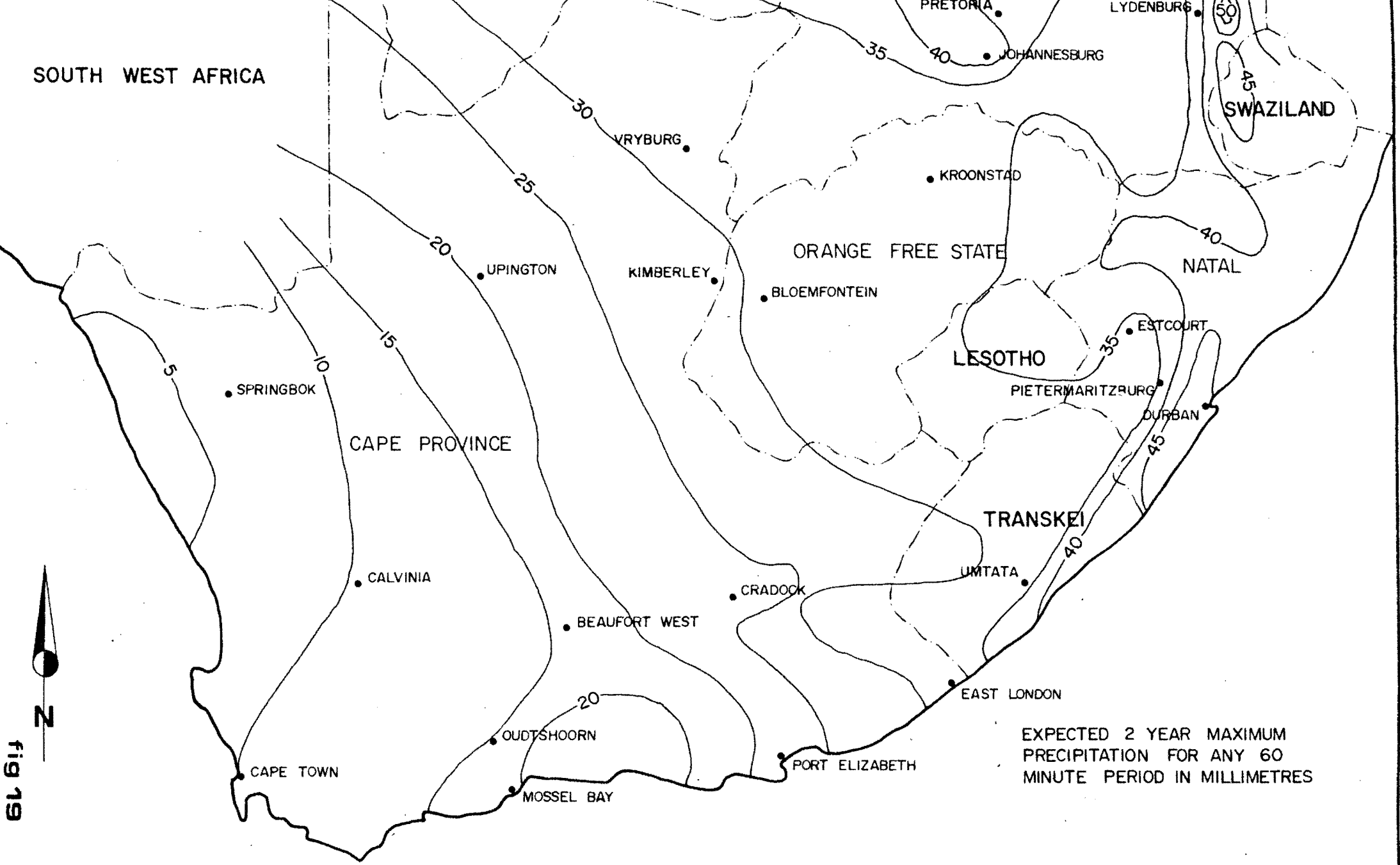
PORT ELIZABETH

EXPECTED 2 YEAR MAXIMUM
PRECIPITATION FOR ANY 24
HOUR PERIOD IN MILLIMETRES



Fig 18

SOUTH WEST AFRICA

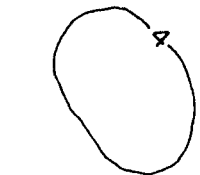


EXPECTED 2 YEAR MAXIMUM
PRECIPITATION FOR ANY 60
MINUTE PERIOD IN MILLIMETRES

fig 19

SOUTH WEST AFRICA

SWAZILAND



2,6

2,2

PRETORIA

LYDENBURG

JOHANNESBURG

VRYBURG

KROONSTAD

2,6

3

3

UPINGTON

KIMBERLEY

BLOEMFONTEIN

2,2

ORANGE FREE STATE

NATAL

ESTCOURT

2,2

3

SPRINGBOK

LESOTHO

PIETERMARITZBURG

DURBAN

2,6

CAPE PROVINCE

CALVINIA

2,6

2,2

TRANSKEI

UMTATA

3



BEAUFORT WEST

4

GRADOCK

2,6

2,6

2,2

2,6

3

ODOTSHOORN

4

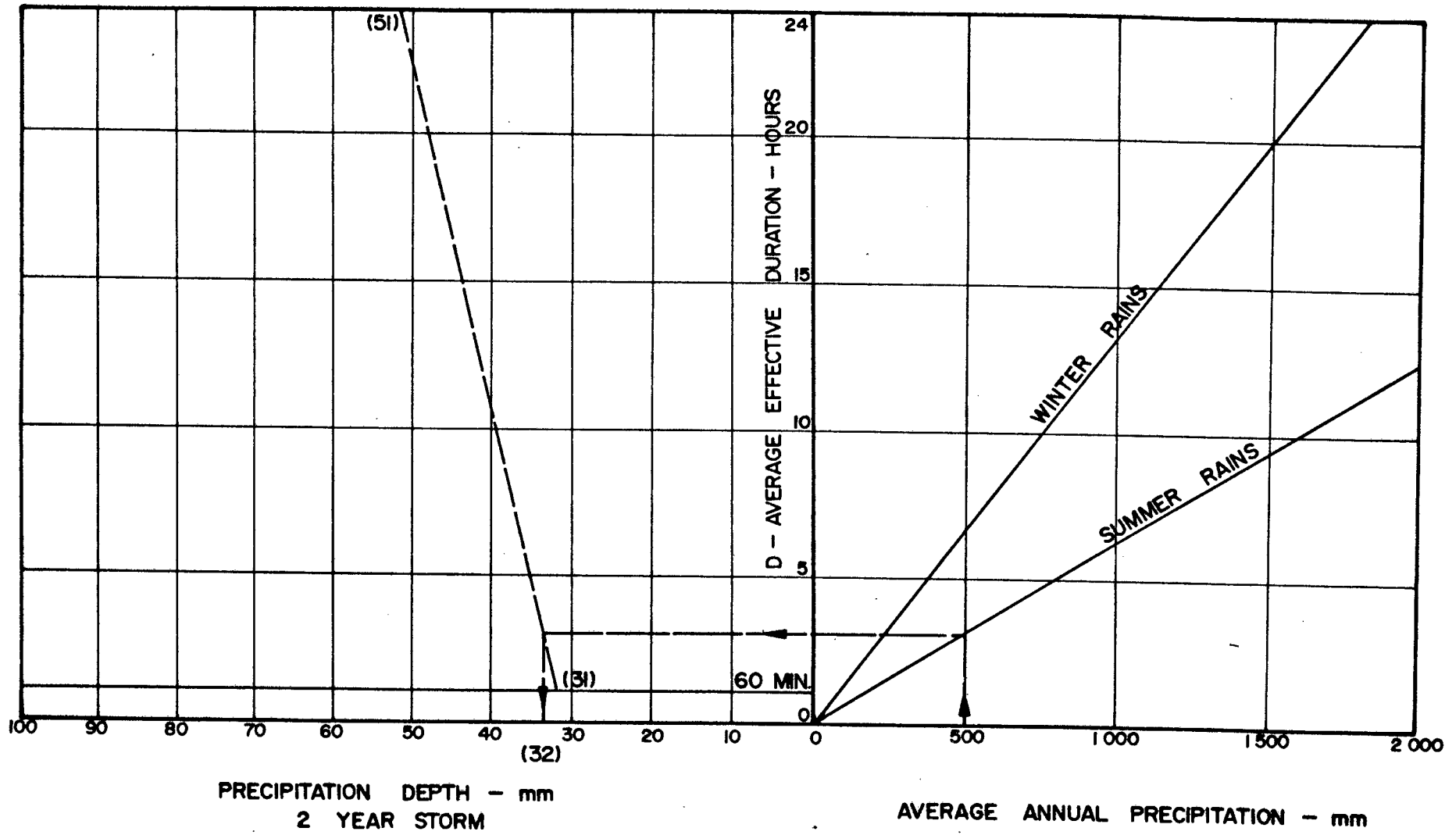
CAPE TOWN

MOSSSEL BAY

PORT ELIZABETH

EAST LONDON

PRECIPITATION RATIO OF $\frac{100 \text{ YEAR MAX.}}{2 \text{ YEAR MAX.}}$



PRECIPITATION FOR EFFECTIVE DURATION

COMPILED BY WRITER

FIG 21

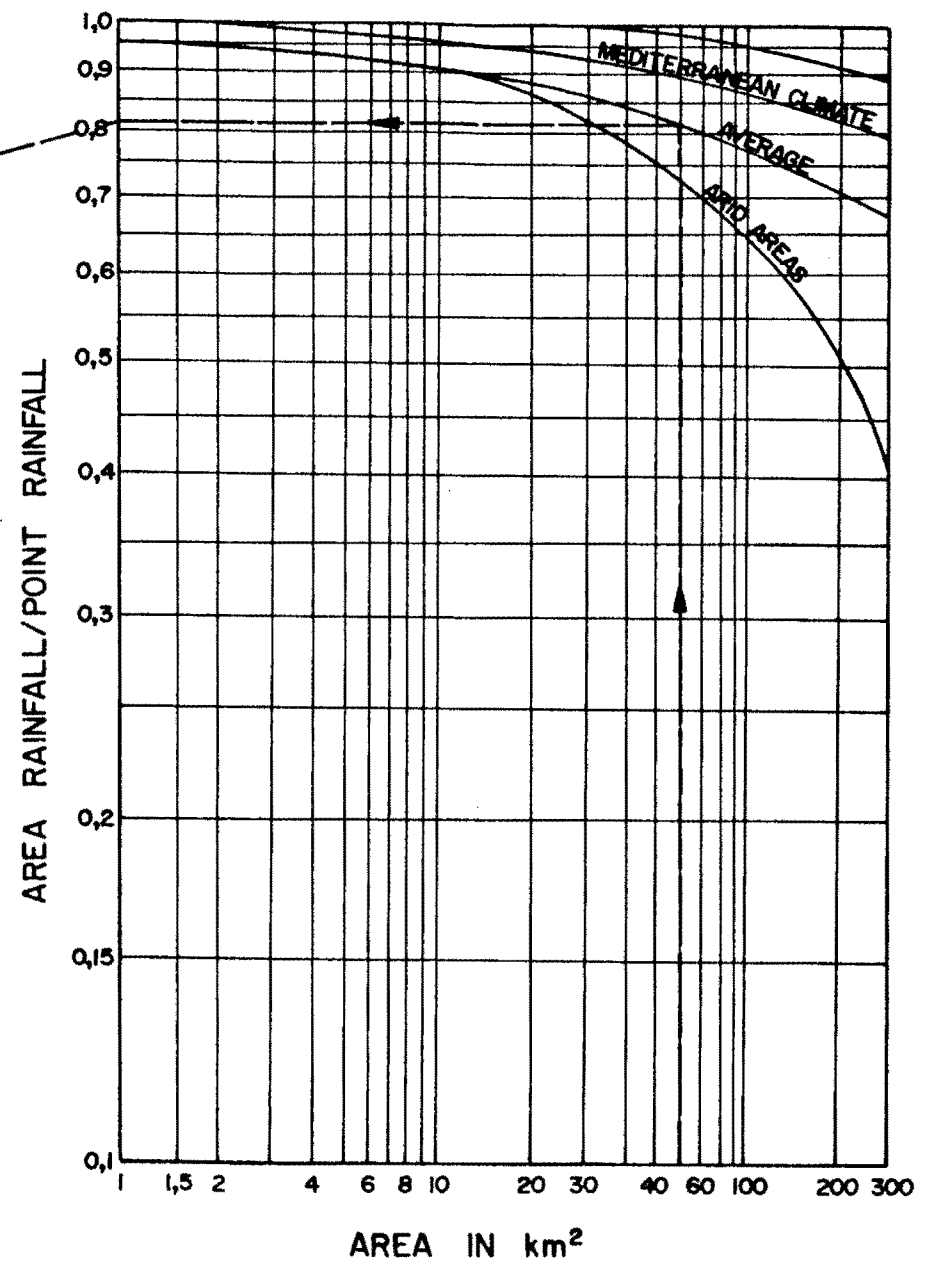
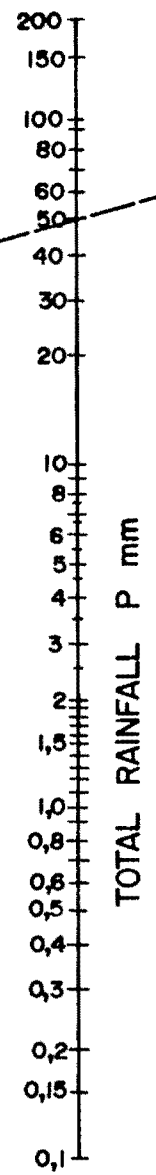
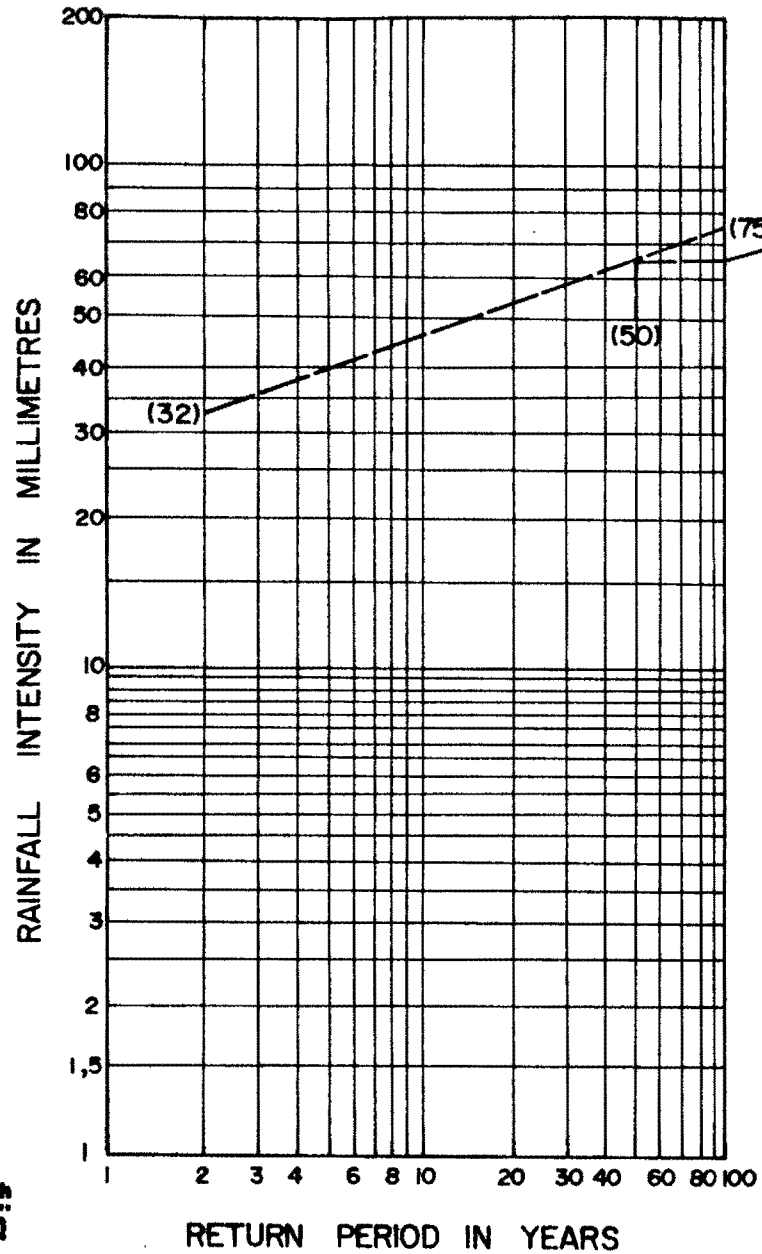
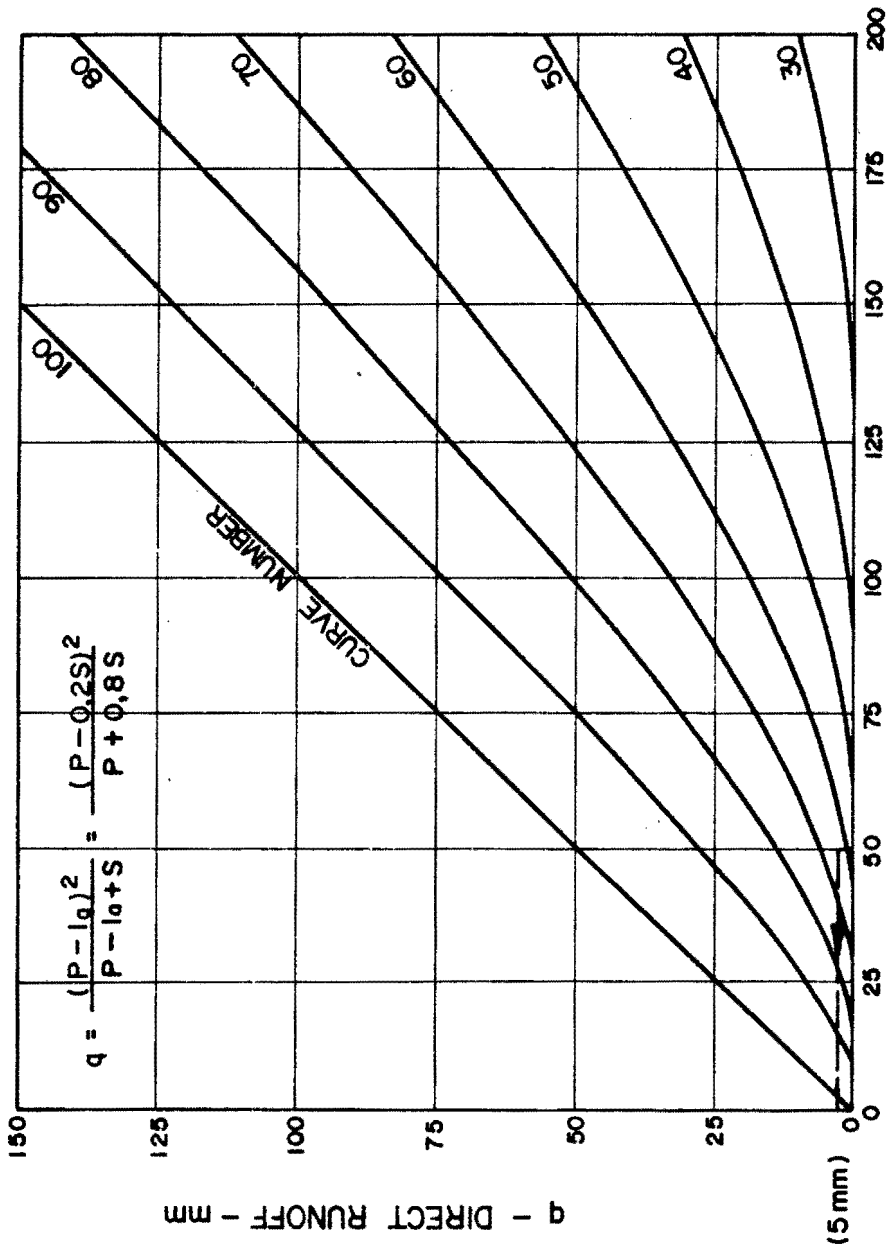


fig 22

COMPILED BY WRITER



P - TOTAL RAINFALL - mm

Reich Method

$$Q = \frac{28,6 q S^{1/5} A^{1/10}}{DL} \text{ cumecs}$$

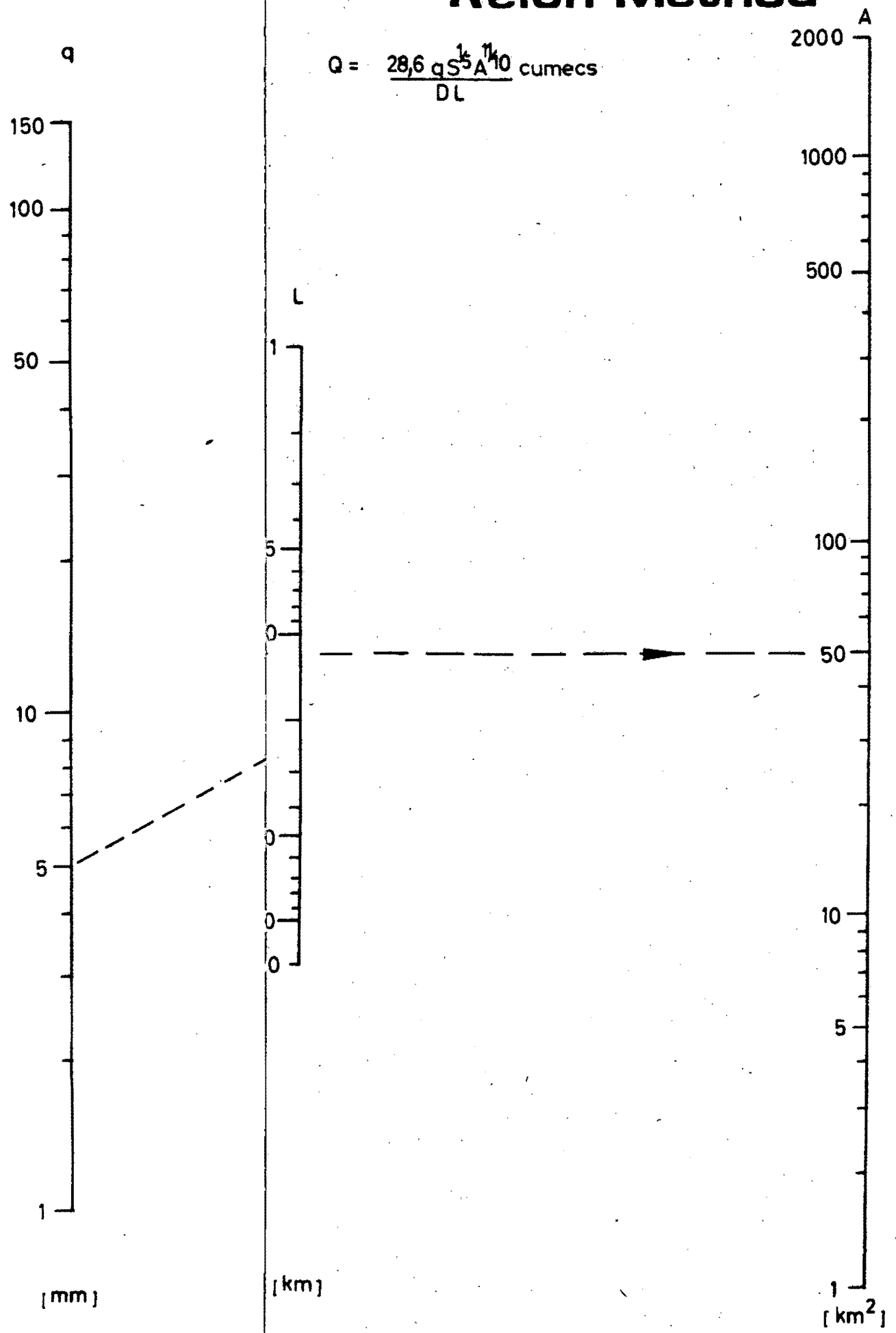


fig 24

Chapter 9

Holtzhausen Method

Certain inaccuracies existed when applying the standard Rational Method to South West Africa and it was for this reason that D. Holtzhausen, the then Director of Water Affairs in South West Africa, developed his method. The method is based on the Rational Method with emphasis on a different approach to obtaining a more realistic run-off factor. Holtzhausen derived catchment categories and flood time/intensity curves in keeping with the terrain and weather characteristics in South West Africa. Holtzhausen's work is at present used to some degree by the Provincial Administration as well as by the Department of Water Affairs (14).

Holtzhausen decided to group the terrain into five separate categories:

Table 8

COEFFICIENT OF RUN-OFF

CATCHMENT AREA	CATEGORY	RUN-OFF FACTOR C
Mountainous, steep slopes	A	C = 0,85 (1-0,002L)
Steep hilly country	B	C = 0,75 (1-0,003L)
Undulating country : slopes <1:20	C	C = 0,65 (1-0,003L)
Moderately sloping : slopes >1:20 <1:40	D	C = 0,55 (1-0,004L)
Gently sloping : slopes >1:50	E	C = 0,45 (1-0,004L)

This table has been simplified by the writer into graph form for the purposes of this project. Refer to Figure 25.

The time of flood concentration t can also be considered as the time taken to build up to the flood peak. Based on the assumption of no data being available the time t is derived from the following formula

$$t = 16,7 \frac{L}{V} \text{ minutes} \quad (15)$$

where V the water flow in metres per second is given by the following table:

Table 9
PEAK FLOOD BUILD-UP

CATCHMENT	VELOCITY m/sec
A	7
B	6
C	5
D	4
E	3

In order to assess the value of the rainfall intensity I Holtzhausen used the following formula which he derived:

$$I = \frac{425}{t+6\sqrt{t} + 30} \text{ mm/hr} \quad (16)$$

This gives a good approximation over a 24 hour period and will probably be acceptable in the Transvaal, Orange Free State and Northern Cape. The above has been simplified in graph form in Figure 26 by the writer.

Holtzhauzen advocated that a reduction be made for the value of the intensity I, based on area and on his calculation. For this reduction Holtzhauzen postulated the following formula:

$$F = (1 - 0,0003 A).$$

This has been presented in graph form by the writer in Figure 26.

Holtzhauzen's method gives run-off figures for an eighty (80) year return period. In order to enable run-off figures for other return periods to be derived a graph has been compiled in Figure 26.

The basic limitation outside South West Africa is that the method is not based on rainfall and hence wetter areas in the Republic of South Africa and elsewhere would tend to be underestimated insofar as run-off is concerned. Possible areas of use would be the Karroo, northern and western Cape, western Orange Free State and western Transvaal.

METHOD The calculation procedure is as follows:

1. Determine the area, length and slope of the catchment.
2. Decide on the catchment category from Table 8.
3. Using Figure 25 obtain the run-off factor C.

4. From Figure 26
 - (a) using the catchment length obtain the rainfall intensity I .
 - (b) obtain the reduction factor R to be applied to the intensity I using the catchment area.
 - (c) obtain the frequency factor F for the required return period which is to be applied to the rainfall intensity.

5. From Figure 27 obtain the adjusted rainfall intensity.

6. From Figure 28 with values for the run-off factor, intensity and catchment area obtain the run-off for the catchment.

PROBLEM Assume a catchment area with the following characteristics:

Undulating country with
an average slope of 1:10

Catchment length	7 km
Catchment area	60 km ²
Return period	20 years

- SOLUTION
1. Table category C

 2. Figure 25 C = 0,64

 3. Figure 26 using $L = 7$ km $I^1 = 70$ mm/hr
return period of 20 years F = 0,7
 $A = 60$ km² R = 0,99

 4. Figure 27 using
 $I^1 = 70$ mm/hr
R = 0,99
F = 0,7 I = 42 mm/hr

 5. Figure 28 using
I = 42 mm/hr
C = 0,64
A = 60 km²

Run-off Q = 420 cumecs

CATCHMENT

Mountainous, steep slopes

Steep hilly country

Undulating country - slope > 1:20

Moderate slopes 1:20 to 1:40

Gentle slopes < 1:40

CATEGORY

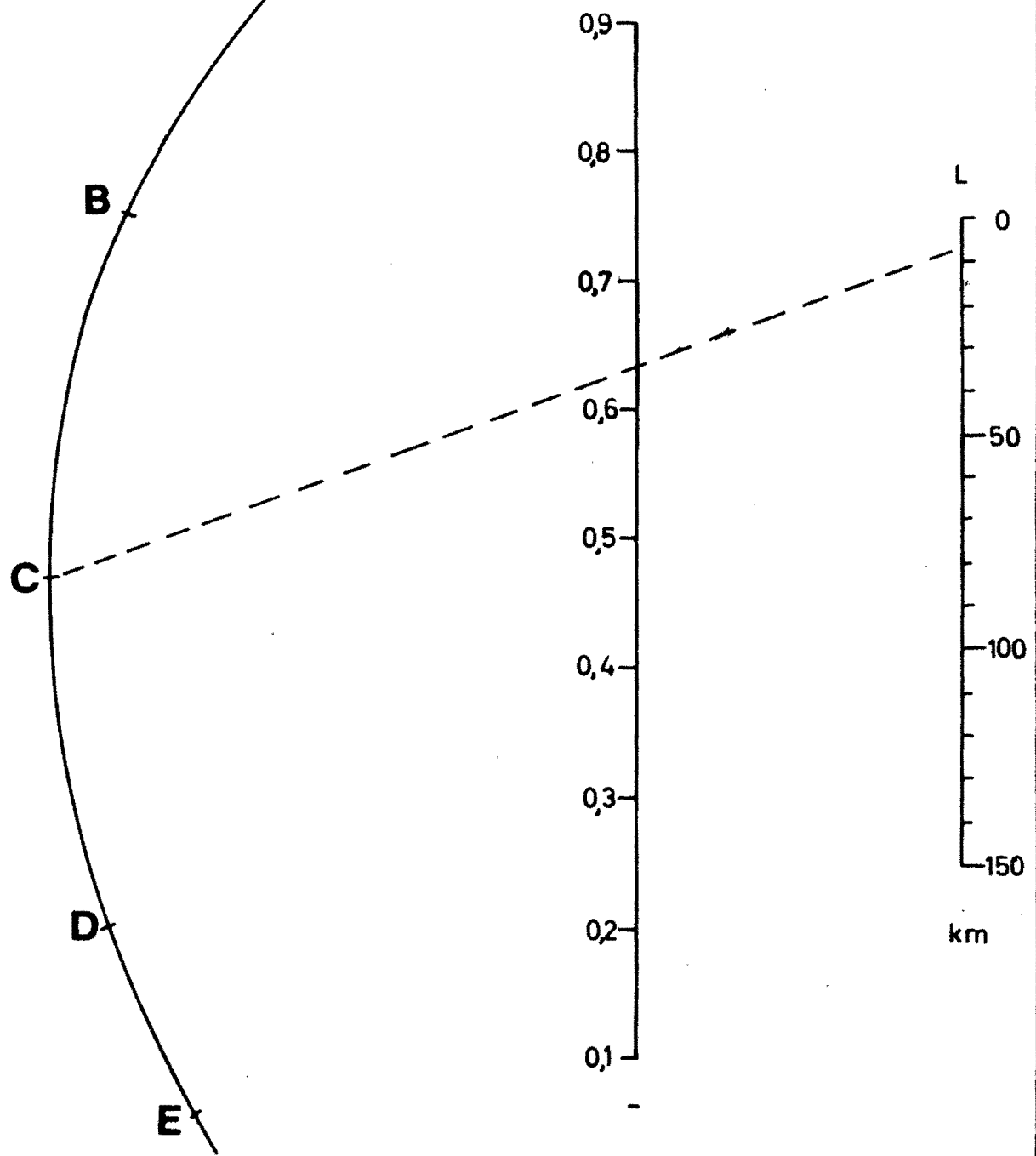
A

B

C

D

E



Catchment Length-Runoff Factor

COMPILED BY WRITER

fig 25

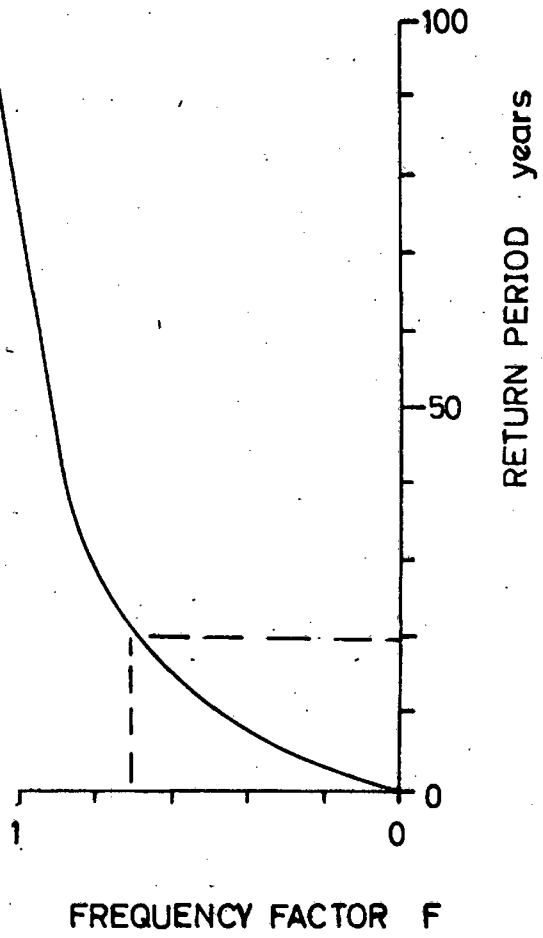
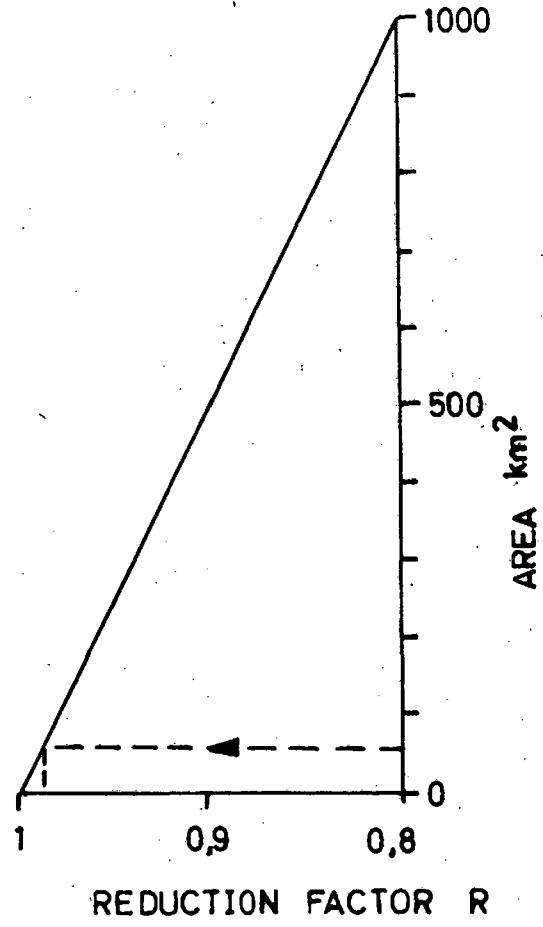
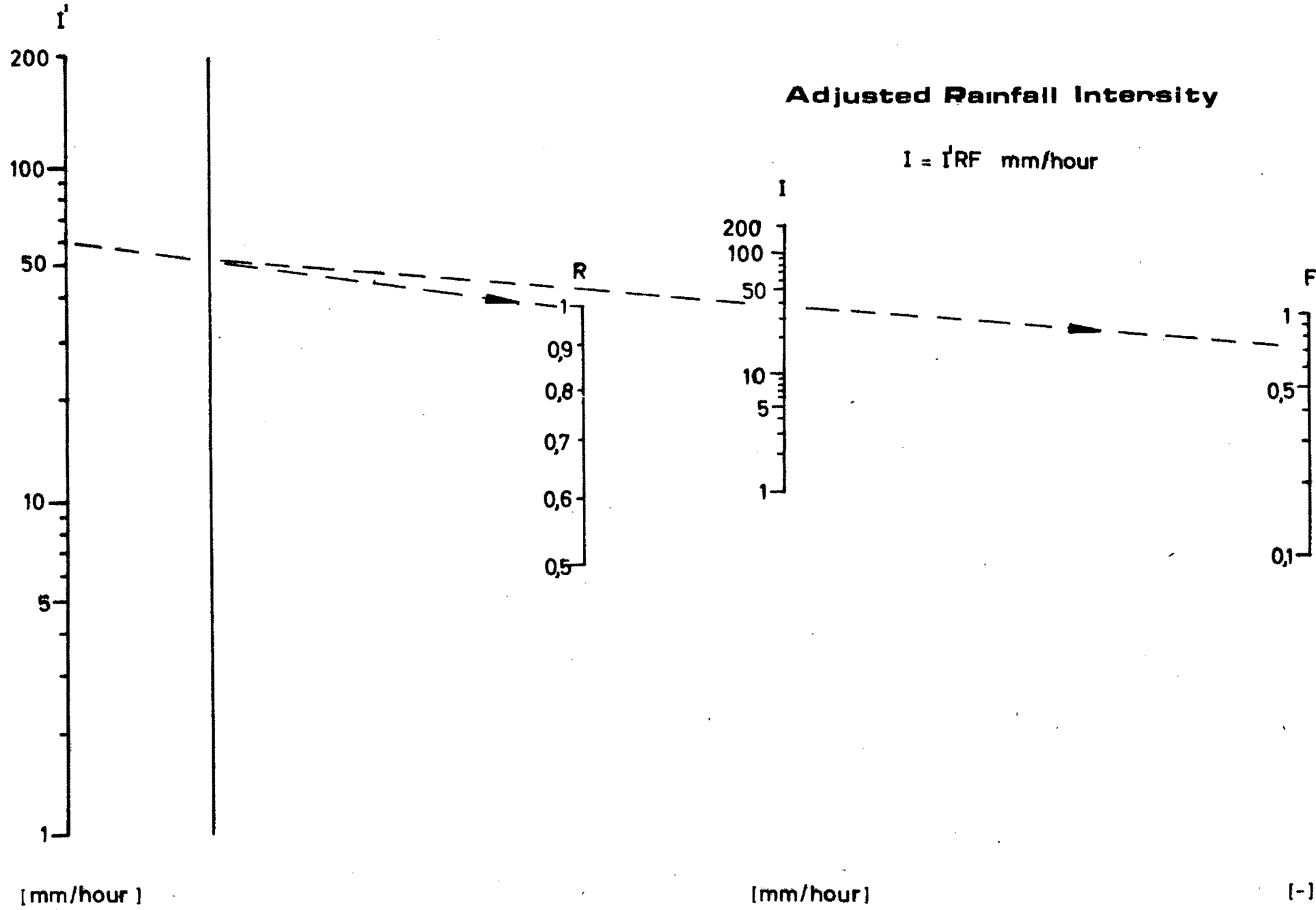


fig 26

fig 27



COMPILED BY WRITER

HOLTZHAUZEN METHI

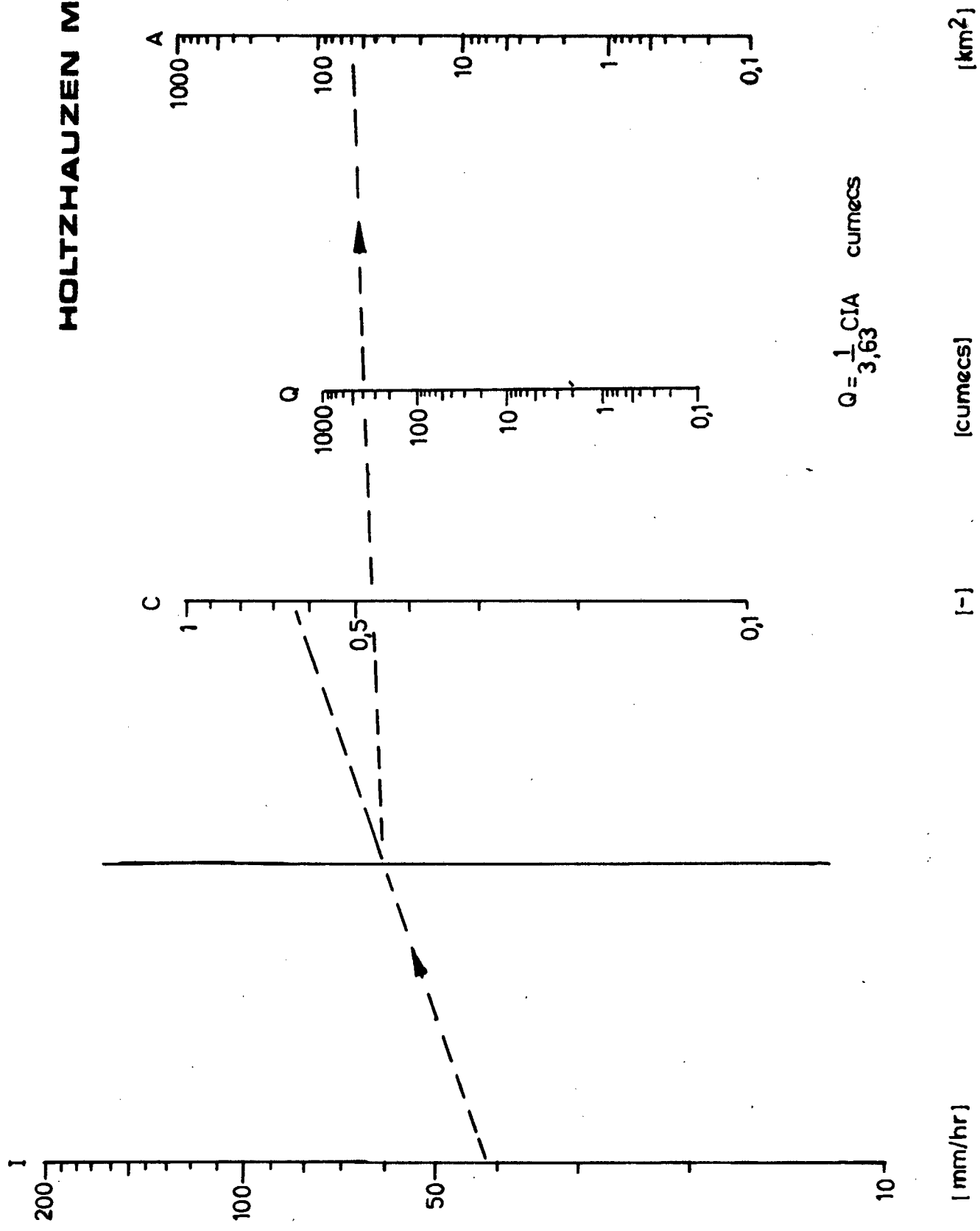


fig 28

COMPILED BY WRITER

Chapter 10

SWA - Namibian Method

This approach is based on the Rational Method with adjustments to suit the climatic and geographic characteristics of South West (Namibia). This method can equally suit South Africa particularly in the drier zones of the country.

The quest by the Department of Roads in South West Africa to obtain a reliable means of calculating the run-off for any catchment in the country led to the appointment of a consulting engineer, in 1962, to formulate a standard method which could be accepted. This assignment became particularly important following heavy January rains in 1963. It was decided as a result of these rains to amend the return period criteria for various structures throughout the territory placing a greater importance to a reliable means of calculating the run-off for a particular catchment.

The resultant report (15) submitted that the Rational Method could be used but that the country should be subdivided into zones, as shown in Figure 30, and that amended values of C, the coefficient of run-off as given in Table 8, should be adopted.

The coefficient of run-off can be obtained by using the following:

$$C = C1 + C2 + C3 \quad (18)$$

The values for the individual components are derived from the following table:

Table 10
COEFFICIENT OF RUN-OFF

DESCRIPTION	VALUE OF C_N
<u>Catchment Characteristics</u>	
High degree of infiltration e.g. sand	$C1 = 0$
Above average degree of infiltration with some clay and silt	$C1 = 0,1$
Normal infiltration reasonable quantities of clay	$C1 = 0,2$
Shallow soil-rockey with hard clay	$C1 = 0,4$
Extensive rock on surface	$C1 = 0,4$
<u>General Catchment Slope</u>	
Very flat 0 - 2%	$C2 = 0,1$
Reasonably flat 2% - 5%	$C2 = 0,15$
Undulating 5% - 10%	$C2 = 0,2$
Rolling hills 10% - 15%	$C2 = 0,3$
Mountainous > 15%	$C2 = 0,4$
<u>General Vegetation</u>	
Thick bush	$C3 = 0,1$
Bush, veld	$C3 = 0,15$
Scarce vegetation	$C3 = 0,3$

The time of concentration has not been accurately determined for all catchments throughout South West Africa and hence the imperial formula $T_c = 88 \frac{L}{V}$ was used. This converted to metric units gives:

$$T_c = 16,66 \frac{L}{V} \text{ minutes}$$

The relationship between the time of concentration and the rainfall intensity for each zone was derived from available rainfall data and these are shown in Figure 31 based on a 10 year return period.

METHOD The calculation procedure is as follows:

1. Determine the catchment length, slope, area, general characteristics, desired return period necessary for the structure envisaged.
2. Using the catchment length and the stream velocity, via the terrain type, determine the time of concentration from Figure 29.
3. Determine the zone in which the major portion of the catchment falls as indicated in Figure 30.
4. Using the zone and the time of concentration determine the ten year rainfall intensity from Figure 31.
5. Determine the return period factor for any other duration than the ten year from Figure 32.

Time of Concentration

$$T_c = 16,66 \frac{L}{V} \text{ mins}$$

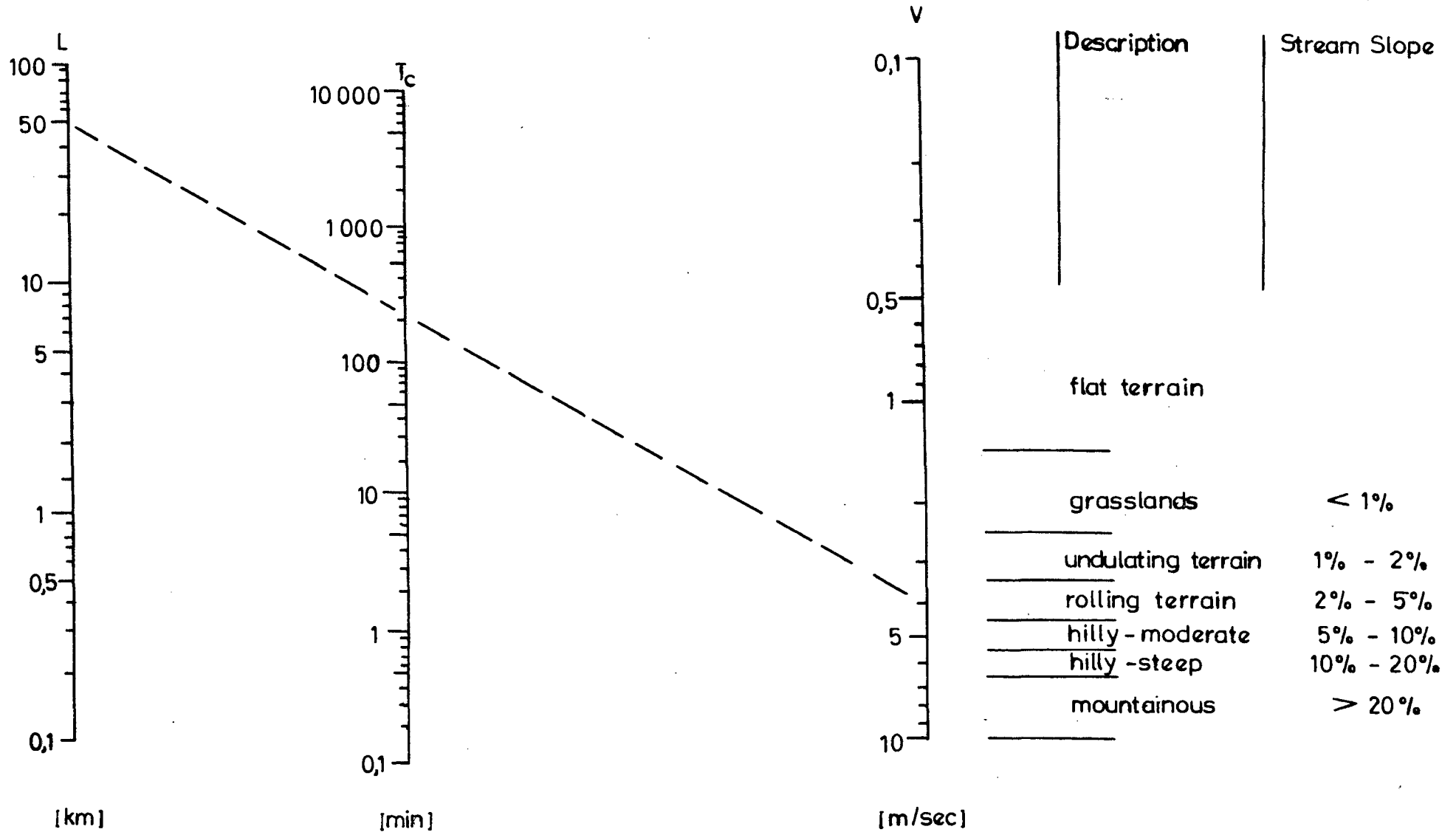


fig 29

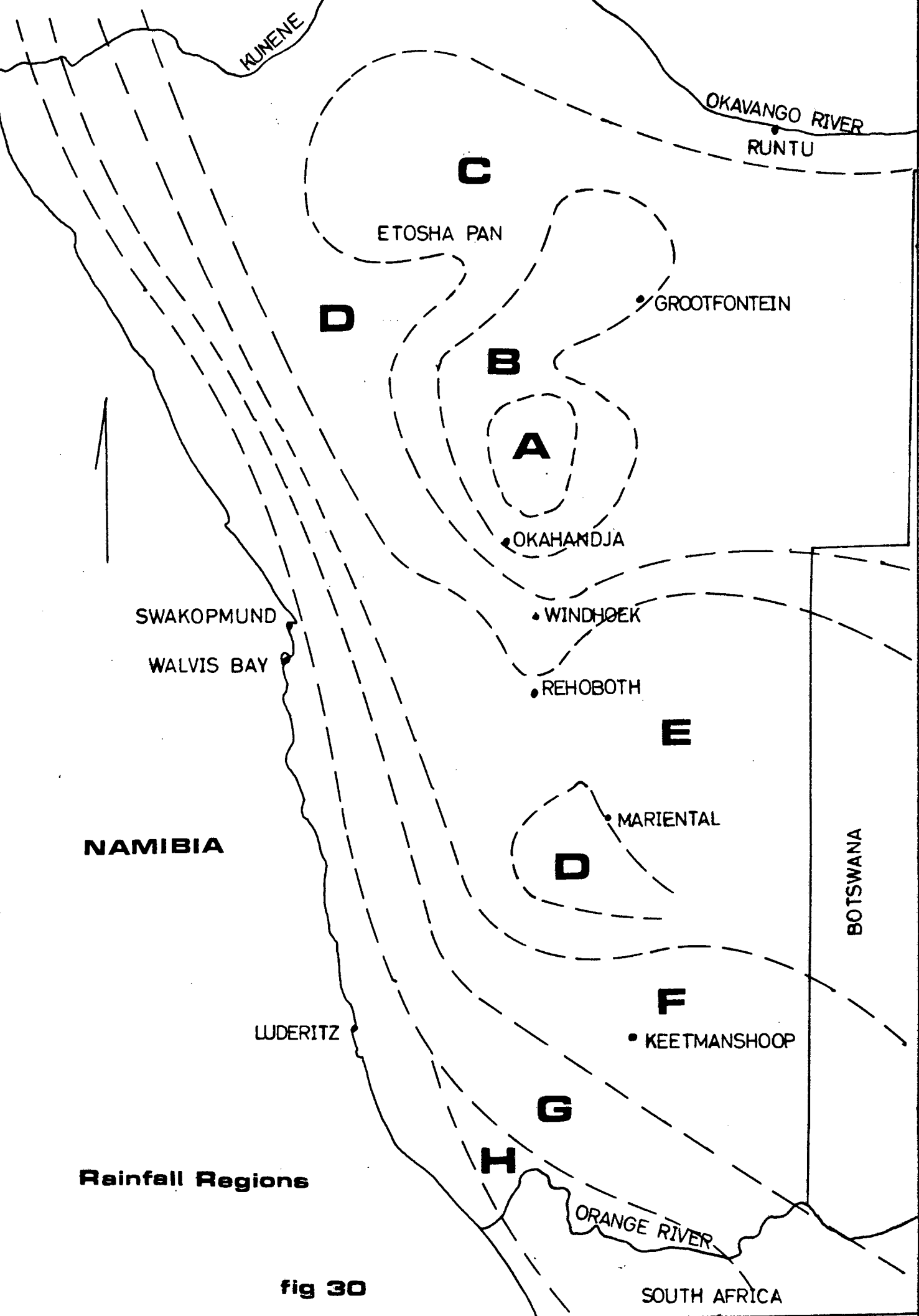
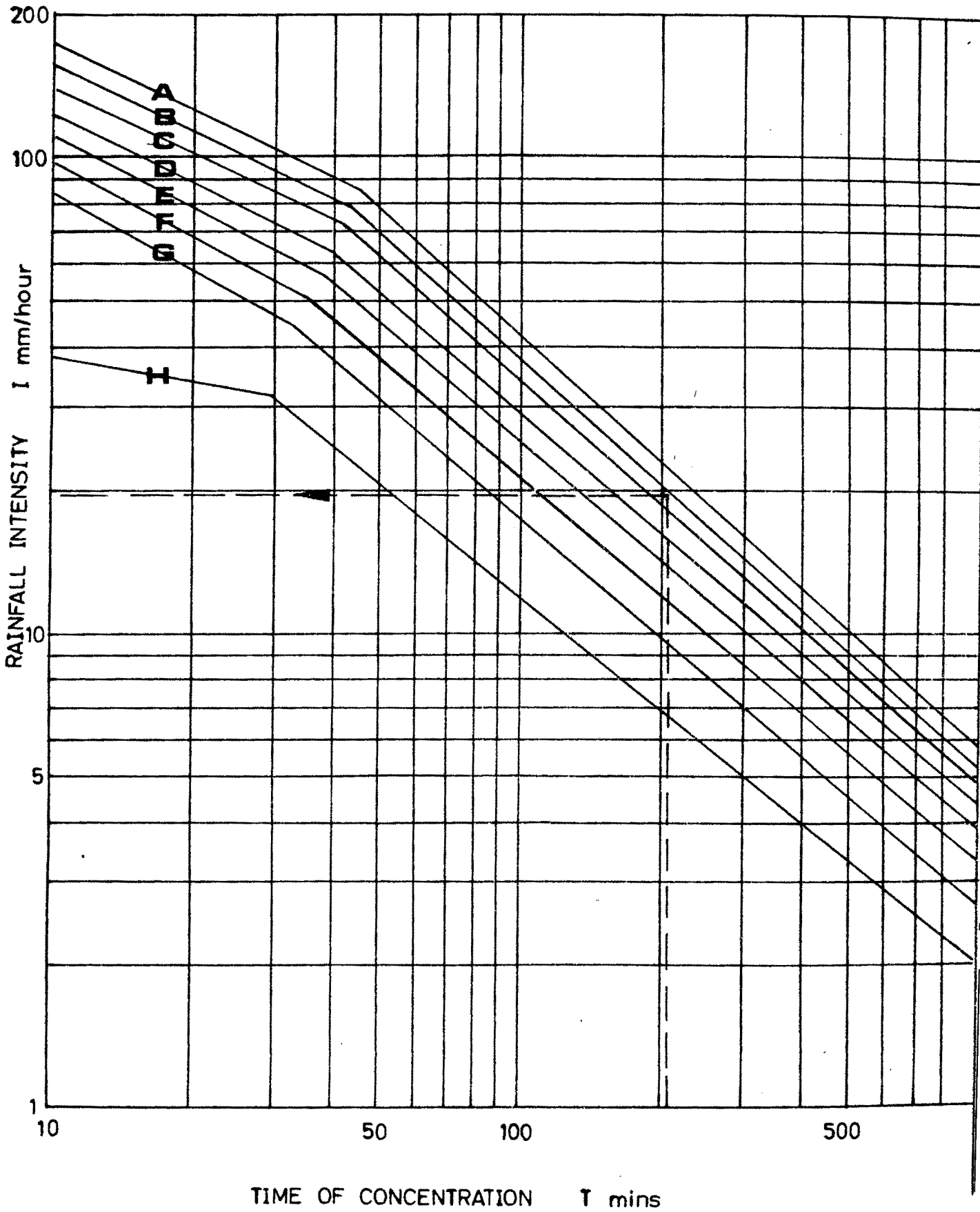
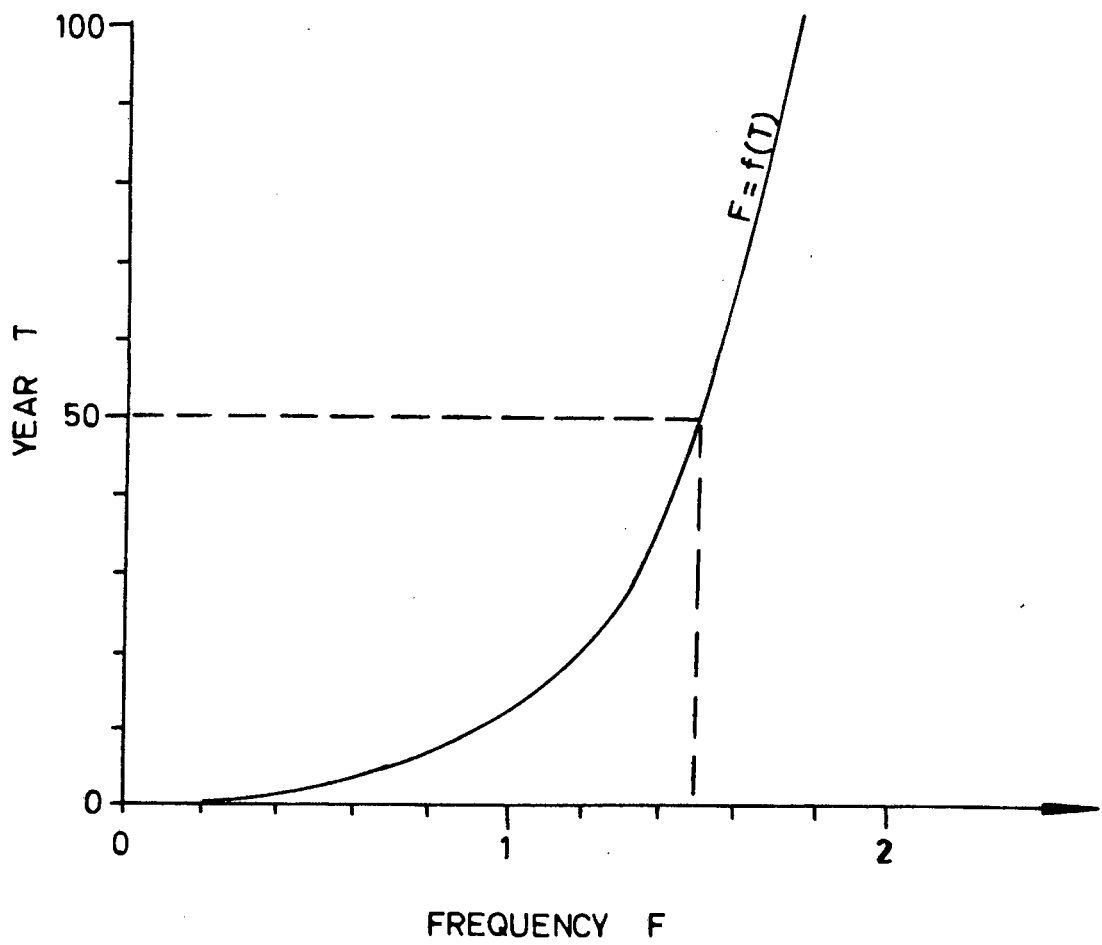


fig 30



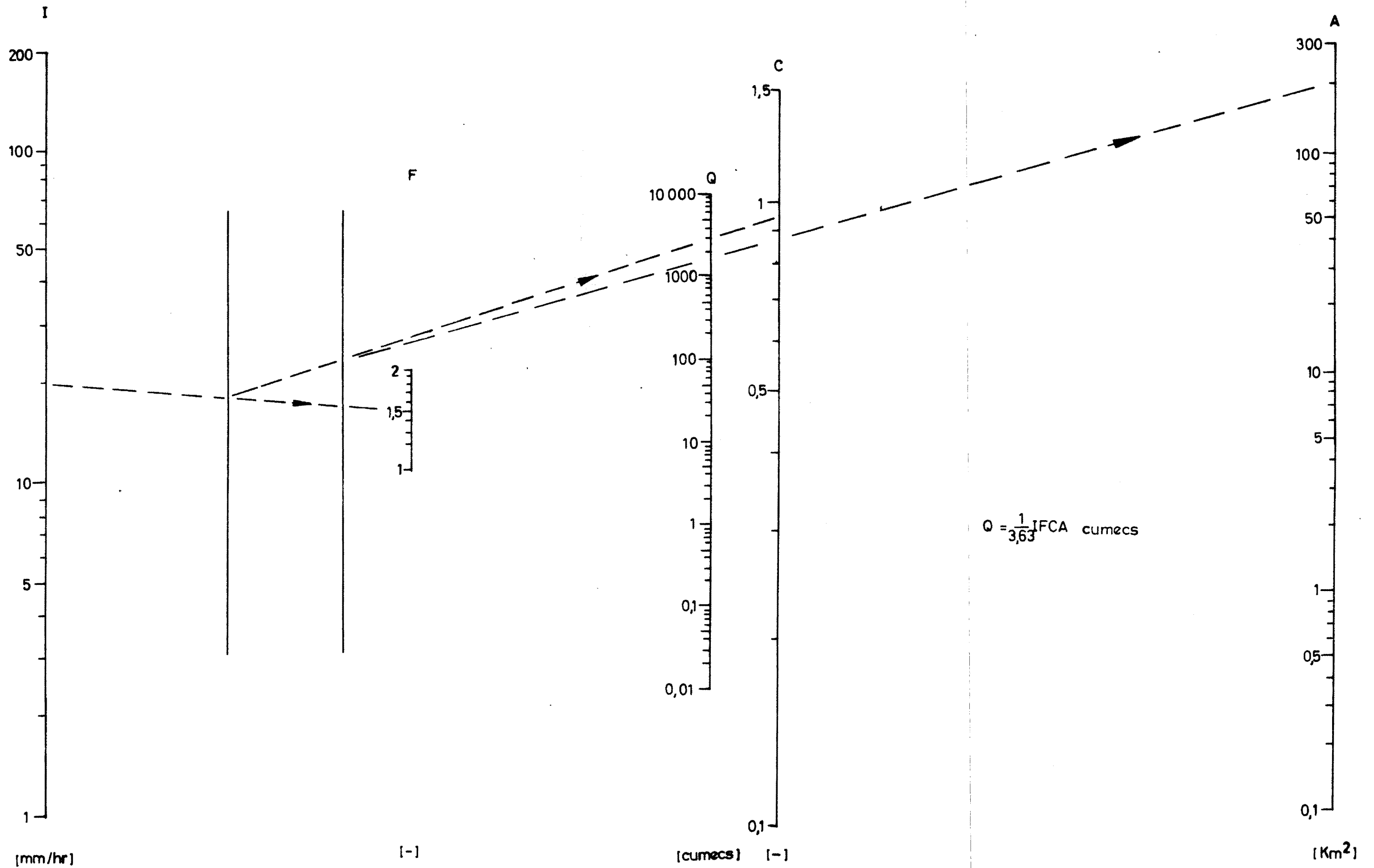
Time of Concentration - 10 Year Intensity

$$I_n^{\text{year}} = F_n \cdot I_{10}^{\text{year}}$$



Value of Frequency Coefficient F

SWA - NAMIBIA METHOD



Chapter 11

Roberts Method

The method proposed by D.F. Roberts (16) has been widely used by the Department of Water Affairs and other organisations.

This method is based on assessing large quantities of data from similar streams and rivers in Southern Africa and deriving a best fit formula.

The major objection to the Roberts Method is that the run-off factor C shows very wide variations from stream to stream and cannot be related to any region or measured variables. Another weakness is the assumption of single values for the variance and the skewness of annual flood peaks for all Southern African rivers. The average record length of data used by Roberts has almost doubled since he carried out his analysis and it is no longer necessary to assume constant variance or skewness.

The formula is:

$$Q = C.K.A^{\frac{1}{2}} \text{ cumecs}$$

where Q = flood peak for the chosen return period

C = catchment coefficient of run-off

K = coefficient derived from the Hazen frequency distribution function

The attached graph in Figure 34 simplifies the derivation of the flood peak run-off in cumecs for a given size of catchment in square kilometres and a given return period. This value is the average for all Southern African rivers.

The peak obtained from the graph should be multiplied by a factor which varies between 0,1 and 2,4. No information is available on the regional distribution of this factor. In the absence of recorded information, the following values could be used which provide a rather subjective estimate of the upper confidence limit.

Table II
REGIONAL DISTRIBUTION FACTOR

RETURN PERIOD	FACTOR
Up to 20 years	1,0
20 years to 200 years	1,5
200 years and longer	2,0

METHOD The calculation procedure is as follows:

1. Determine the catchment area and the desired return period.
2. Using the graph in Figure 34 determine the run-off for the desired return period.
3. Multiply the run-off by the factor from Table 11 to obtain the adjusted run-off quantity.

PROBLEM Assume the catchment area is 450 square kilometres and a 10 year return period is desired.

- SOLUTION
1. Enter Figure 34 using $A = 450$ square km.
(10 year return period) $Q = 270$ cumecs
 2. To adjust multiply by factor = 1,0

$$Q = 270 \text{ cumecs}$$

ROBERTS METHOD

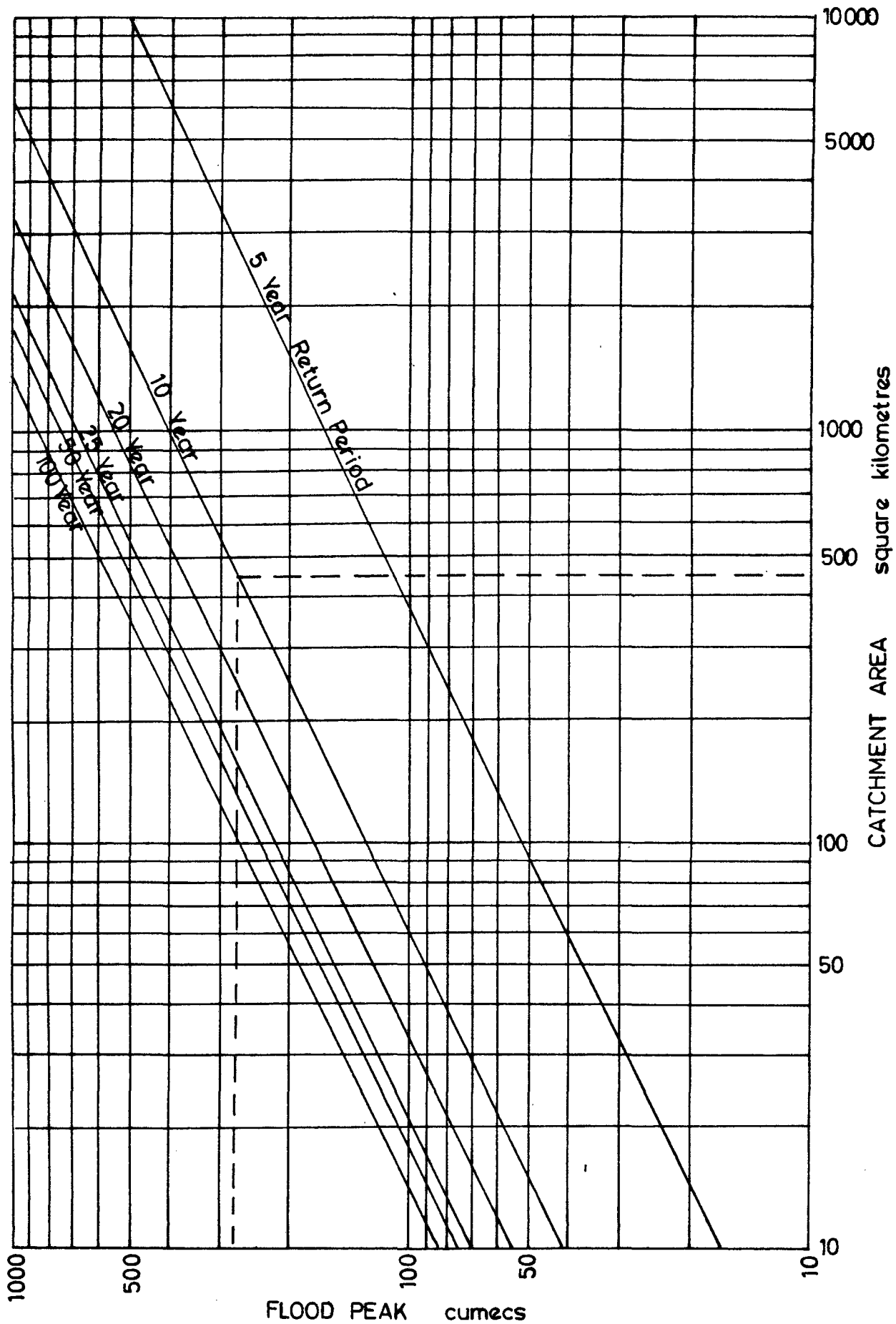


fig 34

Chapter 12

Analysis of Methods & Recommendations

The main concern of engineers is to decide with what confidence the results of the calculated empirical methods be accepted. Each method dealt with herein has been researched in order to produce an acceptable hypothesis. There must be limitations attached to the various methods whether by area or climatic zones. This is especially noticeable for the South West African Method which produces much larger run-off values than comparative catchment areas in South Africa.

A comparison of the run-off quantities using the calculation methods, contained in this project, have been presented in Figure 35. As an area is a common variable this was used as a basis of comparison.

DATA COMPILATION

In examining the methods mentioned in this project data from the following catchment areas were used:

<u>Name of River</u>	<u>Catchment Area - km²</u>
Bree	4650
Olifants	4334
Buffels	4000
Swart - Mfolozi	3470
Traka	3040
Mgeni	2527
Mtamvuma	2100
Keiskamma	1516
Mooi River	933
Mzimkulu	534
Suid-Kaap	280
Hartenbos	143
Quenewe	60
Mfuluzone	44
Inkwaleni	10

The index used in Figure 35 is as follows :

1. Lloyd-Davies Method
2. Rational Method - small areas
3. Rational Method - Medium areas
4. Depth - Duration - Frequency Method
5. Empirical EMWD Method
6. Mc Math Method
7. Reich Method
8. Holtzhauzen Method
9. South West African/Namibia Method
10. Roberts Method

Data was assembled from information available from the Department of Water Affairs. Areas, lengths of catchment and catchment heights were obtained from topographical plans at the Government printer. The writer has some personal knowledge of the areas chosen and hence a reasonably satisfactory assessment of the controlling variables, used in each hydrological method, should have been achieved.

RESULTS OF ANALYSIS

It is not possible to show the scatter of results from the different methods and hence only the "best fit" solution for each method has been submitted in Figure 35. The individual results in each method, for the abovementioned catchments varied up to 22 percent from the mean line. It was observed that the catchment areas having an annual rainfall of less than 400 millimetres tended towards the lower scatter range. Catchments with an annual rainfall of greater than 900 millimetres usually plot above the mean line.

The South West African/Namibia Method shows an almost four times larger estimate than the other methods dealt with in this project. The remaining methods exhibit reasonable correlation up to a catchment area of 50 square kilometres. After 50 square kilometres the first to show increasing inaccuracies is the Depth-Duration-Frequency Method. The Rational Method appears to be fairly reliable up to 200 square kilometres which is the point at which the nomograph for the solution of this method stops. It is felt that the Rational Method could be used up to 500 square kilometres with reasonable accuracy. The Holtzhausen Method exhibits slightly higher results in the catchment areas less than 100 square kilometres, this is perhaps acceptable in the drier areas where an almost total run-off can be expected. The Holtzhausen Method should not be used for a catchment over 500 square kilometres as the run-off quantities become grossly underestimated. The Roberts Method increasingly tends to underestimate the actual run-off quantities the larger the catchment area. The Empirical EMWD and the Mc Math Methods both increasingly overestimate the run-off with increased catchment area. The last method which is that compiled by Reich is perhaps the most accurate and can be used with a certain confidence up to 10 000 square kilometres.

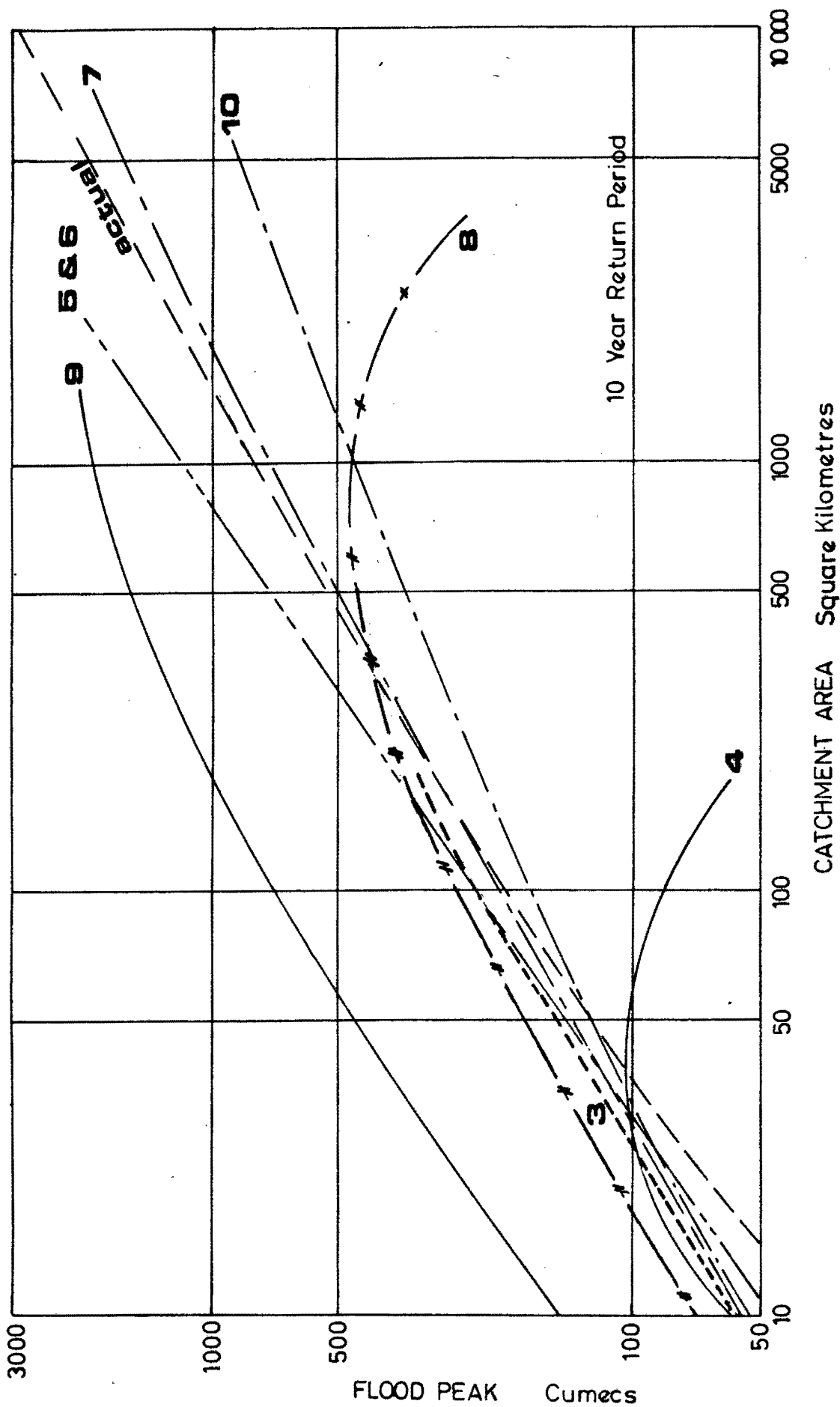
RECOMMENDATIONS

Some limitation must be placed on the methods contained herein and the following is proposed:

- Rational Method (3) - Can be used with confidence up to 500 square kilometres without any adjustment.
- Depth-Duration-Frequency Method (4) - Use up to 40 square kilometres without adjustment. Increase the run-off figures between 40 to 100 square kilometres by multiplying with factors ranging between 1,0 and 2,5 respectively. Do not use for areas greater than 100 square kilometres.
- Empirical Method (5) - Use up to 400 square kilometres without adjustment. Decrease the run-off values between 400 - 4000 square kilometres by dividing with factors ranging between 1,0 and 1,5 respectively.
- Mc Math Method (6) - The same will apply to the Empirical Method above.
- Reich Method (7) - The values obtained from this method can be used with confidence up to 10 000 square kilometres.

- Holtzhauzen Method (8) - These values can be used with confidence up to 500 square kilometres. This method should not be used for larger catchments.
- S.W.A./Namibia Method (9) - These values are very high and even for S.W.A. may be considered conservative. If used in South Africa the values should be reduced by multiplying all values for catchment up to 1000 square kilometres by 0,4.
- Roberts Method (10) - The values obtained from this method appears to decrease with increased catchment area. It is suggested that an adjustment be made by multiplying the run-off values between catchment areas zero to 5000 square kilometres by 1,0 to 2,0 respectively.

All comparisons above were based on a ten year return period. Checks however were conducted on fifty and one hundred year return periods and it was found that the above recommendations did not alter appreciably.



COMPARISON OF METHODS

fig 35

Chapter 13

Safety & Risk

FACTOR OF SAFETY

It is common practice for engineering structures to incorporate a factor of safety in the design assumptions to allow for unforeseeable circumstances. It is not usual to apply an additional factor of safety to flood estimates and the unstated assumption is that these are incorporated in the design tolerances.

If the flood estimate and structural design are not determined by the same person, it is important that the results of the hydrological analysis should include a clear indication of the magnitude of the uncertainties in the assessment. For example, the calculated upper confidence band is not a factor of safety, but an estimate of the range within which the actual flood peaks for the specified return period may lie. It takes no account of the possible inaccuracies in the data, or effect of the assumption of a single population or probability distribution.

CONCEPT OF RISK

The designer has to select his design flood from a whole series of values, each associated with a probable risk of occurrence during the effective life of the structure.

The relationship between the severity of the event and the probability of the event occurring during the design lifetime is given by

$$r = 1 - \left(1 - \frac{1}{T}\right)^{L'} \quad (21)$$

where r = risk event
 T = return period in years
 L' = design life.

To simplify this the relationship is given in Table 12.

Table 12

PROBABILITY OF RISK

RETURN PERIOD IN YEARS T.	DESIGN LIFE IN YEARS - L'								
	1	2	5	10	15	20	25	50	100
10	10	19	41	65	79	88	93	99,5	99,9
20	5	10	23	40	54	64	72	92	99,4
50	2	4	10	18	26	33	40	64	87
100	1	2	5	10	14	18	22	40	63
200	0,5	1	2	5	7	10	12	22	39
500	0,2	0,4	1,0	2	3	4	5	10	18
1000	0,1	0,2	0,5	1,0	1,5	2	2	5	10

Alternatively this can be reconstructed to show the required return period for a specified design life and permissible risk.

Table 13
RETURN PERIOD

PERMISSIBLE RISK OF FAILURE	DESIGN LIFE IN YEARS - L'							
	1	2	5	10	20	25	50	100
0,99	1,0	1,1	1,7	2,7	4,9	6,0	11,4	22,2
0,95	1,1	1,3	2,2	3,9	7,2	8,9	17,2	33,9
0,90	1,1	1,5	2,7	4,9	9,2	11,4	22,2	43,9
0,75	1,3	2,0	4,1	7,7	14,2	18,6	36,6	72,6
0,50	2,0	3,4	7,7	14,9	29,4	36,6	72,6	145
0,33	3,0	5,5	12,9	25,2	49,9	62,1	124	247
0,25	4,0	7,5	17,9	35,2	70	87,3	174	348
0,20	5,0	9,5	22,9	45,3	90	113	225	449
0,10	10,0	19,5	48,0	95,4	190	238	475	950
0,05	20,0	39,5	98,0	195	390	488	975	1950
0,02	50,0	99,0	248	495	990	1238	2476	4951
0,01	100	199,5	498	995	1990	2488	4977	9953

It must be noted that if a design life of a structure is 20 years with 10% risk of failure then a return period of 190 years will be required.

It is thus important that a client should specify not only the design life for a structure but also the percent risk of acceptable failure.

PART B

**IN-SITU PERMEABILITY TESTS
&
SUB-SURFACE DRAINAGE**

Symbols used in Part B

The following symbols have been used in Part B of this project:

- k = coefficient of permeability - cm/sec/cm^2
- k_s = standard coefficient of permeability - cm/sec/cm^2
- d = depth of soil layer - centimetres
- n = kinetic viscosity - poise
- i = hydraulic gradient
- γ = unit weight of water - grams/cm^3
- l = length of sample - centimetre
- V = velocity of water - cm/sec
- C = Hazen's sand size coefficient - sec/cm
- D = diameter of soil particle
- q = water flow - litres/sec
- L = length of hole tested - centimetres
- r = radius of hole - centimetres
- H = pressure head - centimetres
- F = shape factor
- H_c = water level in constant height test above the water table - centimetres
- m = coefficient used in shape factor
- t = time recorded - second
- e = void ratio
- S = spacing between sub-surface drains - metres
- a = distance between impermeable layer and sub-surface drain - centimetres
- b = distance between impermeable layer and crown of draw-down point - centimetres
- A = area of tube used in permeability determinations - cm^2
- E = Hooghoudt constant
- p = radius of auger hole - centimetres
- f = depth of auger hole below water table - metres
- y = varying depth of water in auger hole below water table - centimetres

The units are as above unless otherwise stated.

Chapter 14

Introduction

The design of sub-surface drains has been based largely on personal choice rather than on sound engineering investigation and calculation. In order to propose any system of sub-surface drainage the properties of the particular material or materials in which the water table is to be lowered must be known. In particular the permeability of the various strata must be determined and the means by which this is obtained is of vital interest to the engineer. It has often fallen on the site staff, during the construction of a project, to determine the position and depth of the sub-surface drains and hence this project describes the procedures to be followed to derive the in-situ permeability by the most convenient and economical methods.

The object of this project is to conduct tests to determine the in-situ permeability of soils and the application to which it can be directed. In order to appreciate the basis of the tests discussed herein a brief background to permeability and sub-surface water flow must be given. For this background refer to Appendix A at the end of this project.

Five practical test methods have been investigated in this project. Four methods are intended for soils and sands and these can be used with little or no equipment. These four methods include the constant head, falling head, rising head and the sieve analysis approaches. The fifth method is more complex as it utilises pressure to determine the permeability of rock. Whilst field experiments were conducted on the first four methods mentioned the fifth has only been discussed as the main emphasis is on water movement in soil and not in rock.

The derivation of the spacing and depth of the sub-surface drains can be determined by using an empirical formula which has been satisfactorily tested by various authors on the subject. Personal knowledge of the terrain must supplement the results obtained from calculation to ensure the most effective and economical solution to a drainage problem.

The capacity for storage of the sub-surface soils and rocks is fixed according to their geological and physical properties. During the wet season a certain percentage is absorbed and is slowly released between rains and in the drier periods. This depletion or seepage results in streams and rivers flowing long after a rainfall has ceased. This depletion of the sub-surface water causes movements of the water which is of particular interest to the engineer. This project includes in the normal assessment of sub-surface water the tendency for depletion of the groundwater during the drier seasons.

Two Codes of Practice have been included in this report which deal with boreholes and their associated observation advantages. The German Code is of particular interest as it deals more closely to the subject matter of this report.

Chapter 15

Pressure Tests

Information on the permeability of rock in-situ as well as some indication of the soundness of the rock may be obtained from pressure tests in drill holes in rock. A schematic diagram of a pressure-test apparatus is shown in Figure 36. The apparatus seals off a section of the drill hole by means of inflatable rubber packers. The water pressure in the sealed-off section is then raised above the piezometric level normally obtaining in the rock at this section. The leakage into the rock from the sealed off section, under various pressures above the natural piezometric pressure, is observed. A linear variation of leakage with pressure indicates stable passages for flow of water in the rock. If the leakage increase is less than the linear increase the passageways are plugged, if greater than the linear passageways are unplugged or perhaps enlarged by spreading apart under the increase in pressure.

The Packer Test formula is frequently used for computing the permeability of rock and is as follows:

$$k = \frac{q}{2 \pi L H} \log_e \left(\frac{L}{r} \right) \quad \text{if } L \geq 10r \quad (22)$$

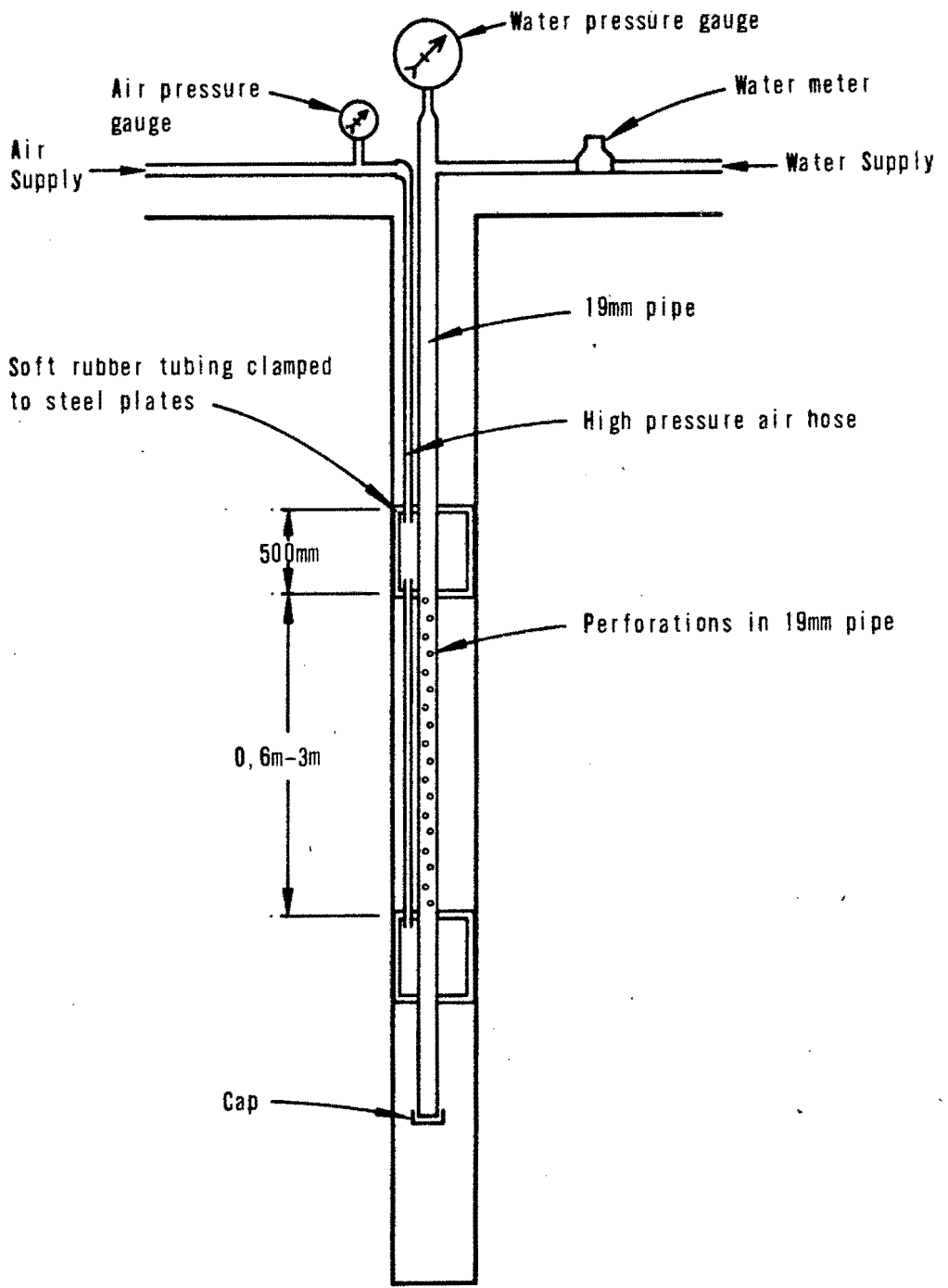
where K = coefficient of permeability
 q = constant rate of flow into the hole
 L = length of the portion of the hole tested
 H = pressure head
 r = radius of the hole

This formula gives an approximate value of k only because it does not take into account the effect of back flow to the borehole around the packer and includes several simplifying assumptions. These, however, do not change the order of the magnitude of the coefficient of permeability obtained, therefore, the formula is satisfactorily accurate for practical purposes. Figure 37 gives the coefficient of permeability k in centimetres per second based on equation (22) above.

An advantage of this type of test is that the permeability of the rock can be determined for different layers and hence the zone of greatest permeability can be located. This could assist in deciding on the depth of the subsurface drain to produce the maximum desired drawdown. Equations A1 and A2 can be used to determine the maximum and minimum coefficients of permeability for the rock strata as a whole.

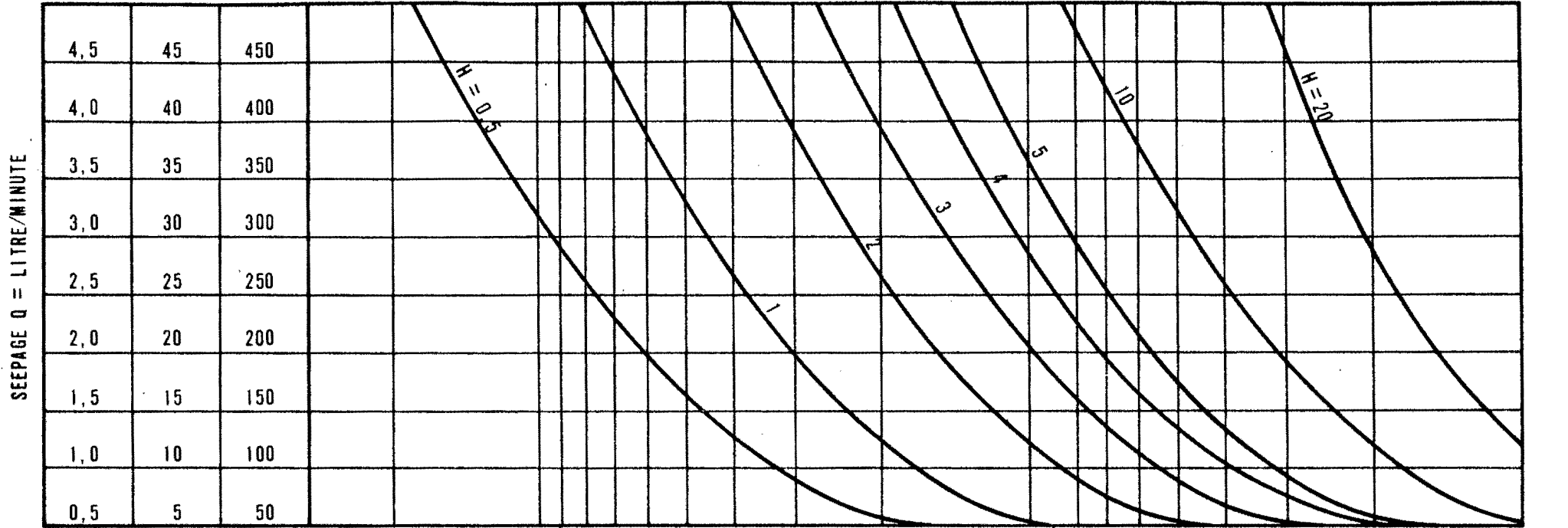
Care must be exercised when conducting this test that the rock is not too highly fissured which would probably result in the water from the sealed zone flowing freely into the unsealed sections hence producing an erroneous result. It is to be expected that the instrument operator would be experienced as selection of the zones would necessitate some correlation with the drill hole cores.

A comprehensive code of practice on the determination of the permeability in rock strata has been included with this project. The code, found in Appendix C, is in German (DIN 4021 Seil 3) and has been translated.



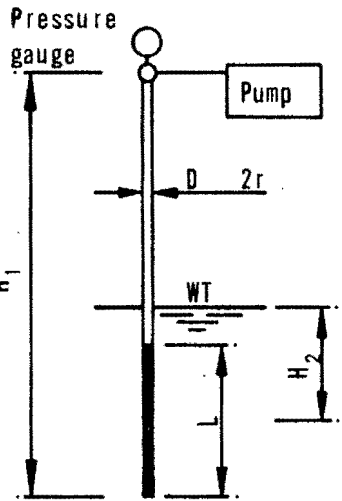
PRESSURE-TESTING APPARATUS

500



(IF $1 \leq Q \leq 10$) 10^{-1} 10^{-2} 10^{-3} 10^{-4} 10^{-5}
 (IF $0.1 \leq Q \leq 10$) 10^{-2} 10^{-3} 10^{-4} 10^{-5}
 (IF $0.01 \leq Q \leq 0.09$) 10^{-3} 10^{-4} 10^{-5}

GRAPH A - COEFFICIENT OF PERMEABILITY - K_T cm/sec.



$H = H_1 - H_2 + H_3 = \text{TOTAL HEAD}$

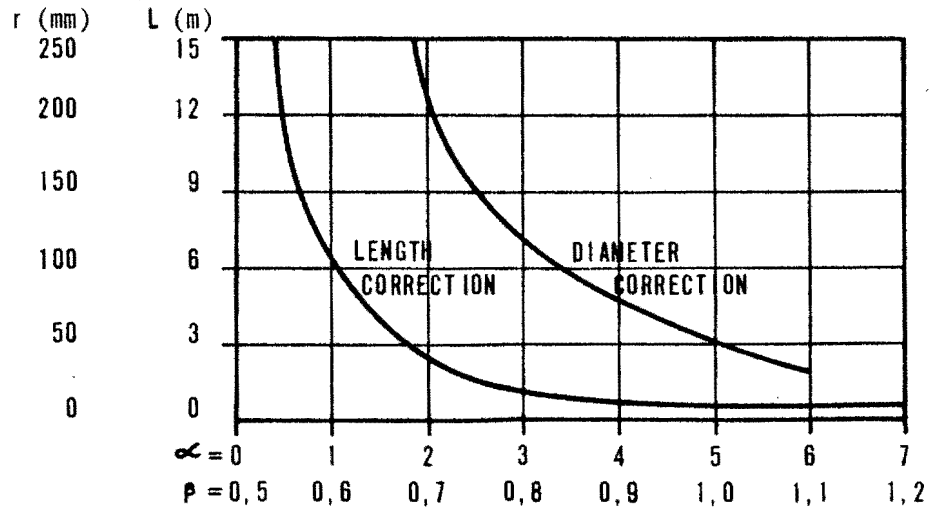
$H_3 = 0, 1P = \text{PRESSURE HEAD}$

$P = \text{PRESSURE} - \text{kg/cm}^2$

NOTE:

GRAPH A GIVES COEFFICIENT OF PERMEABILITY DIRECT IF $r = 50\text{mm}$ AND $L = 6$ metres. IF NOT USE

GRAPH B WITH EQUATION $k = \infty P K_T$



GRAPH B

INSTRUMENT

Chapter 16

Constant Head Test

The concept of the constant head test is derived from the permeameters used in the laboratory. The principle of analysis and approach is basically the same but the variation suggested by Hoek and Bray (28) is preferred and has been adopted here. The variation in approach is due primarily to the disadvantages experienced when obtaining the coefficient of permeability in the laboratory. In the laboratory the sample is small and is possibly not representative of the true conditions in the aquifer. To avoid wall effect the permeameter diameter should be at least 40 times the mean particle diameter, otherwise the measure coefficient of permeability will be inaccurate. Care must also be taken to ensure that a quick sand condition does not occur at the bottom of the hole or that a film of fine material does not coat the sides of the hole.

There are many ways of producing a constant-head permeameter which can be used in the field and this is left mainly to the operator to construct. The principle remains the same however and a very basic diagram is shown in Figure 38. A tube is inserted into a drill or an auger hole to a point well below the water table. If the tube does not fit tightly into the hole then a watertight packing must be inserted between the tube and the sides of the hole at the water table level. The tube should extend about 1,5 metres above the ground with a hole at about 0,5 metres from the top of the tube.

A steady flow of water is then introduced at the top of the tube, at a known rate, sufficiently to compensate for the seepage into the soil with a minimal steady flow through the hole which is measured in a container over a predetermined period.

This would give the flow rate in excess which cannot be absorbed into the soil. The quantity of water seeping into the soil is therefore determined by :

Seepage flow = water introduced into the tube
less the water measured out of
the hole.

$$q = q_1 - q_2 \quad (23)$$

Before measurements of the flow of water is undertaken, it is recommended that the water be allowed to flow into the soil for an initial period in order to allow the soil to become saturated before testing. It is also recommended that a series of tests be done in order to establish at which level of flow consistency exists. It should be pointed out that the flow could be slightly incorrect due to trapped air in the soil voids, where possible, to avoid this difficulty water used should be de-aerated. It is felt however if this is not possible and normal clean water is used the results should still be acceptable. Another factor to bear in mind is the fact that even if there is a stratification of the soil components the permeability determined is considered the resultant of the depth tested below the water table. If a clearer picture is required the packing around the tube should be moved down in stages and a new set of flows measured. This would clarify the possibility as to the permeabilities at different depths.

It can be reasoned that the permeability will be influenced by the location of the hole in the soil strata and also the relationship of the tube to the hole. In Figure 38 the shape factor F can be determined from the end conditions of the hole which is then incorporated into the following formula to obtain the coefficient of permeability:

$$k = \frac{q}{F H_c} \text{ cm/sec} \quad (24)$$

where q = flow rate determined

F = shape factor

H_c = is the water level, measured from the rest water level maintained during the constant-head test.

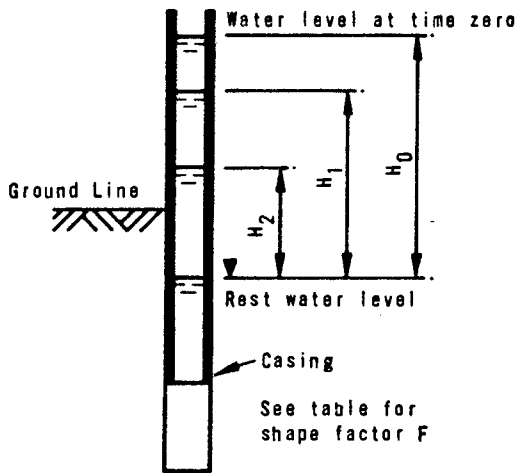
Natural soils almost invariably occur in a stratified condition, with the result that the permeability of the soil varies from a maximum in the direction of the strata to a minimum perpendicular to the strata. Even a compacted fill of selected materials will have some stratification because of the inevitable variation in density and composition from layer to layer. Casagrande (23) suggests that the ratio of maximum and minimum permeabilities for a compacted fill is probably at least 9:1. As a postulated figure for m value used in the shape factor in Figure 8 the following can be used:

$$m = (k_h/k_v)^{\frac{1}{2}} = 3 \quad (25)$$

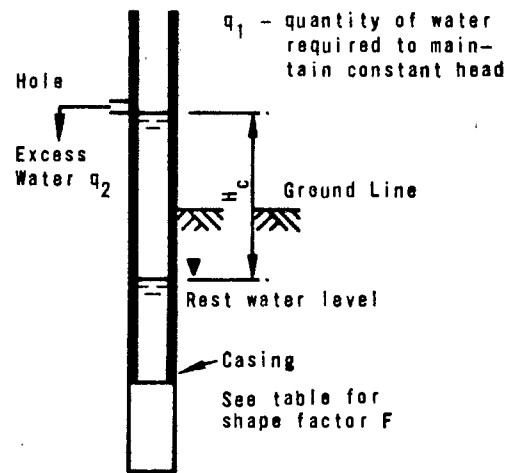
where k_h = horizontal coefficient of permeability

k_v = vertical coefficient of permeability

In order to maintain the same units all measurements should be undertaken in centimetres.



Falling Head Test



Constant Head Test

Diagram	End Conditions	Shape Factor F
	Casing flush with end of borehole in soil or rock of uniform permeability. Inside diameter of casing is d cms.	$F = 2,75d$
	Casing flush with boundary between impermeable and permeable strata. Inside diameter of casing is d cms.	$F = 2,0d$
	Borehole extended a distance L beyond the end of the casing. D is borehole diameter	$F = \frac{0,9\pi L}{\text{Log}(2L/D)}$ For $L > 4D$.
	Borehole extended a distance L beyond the end of the casing in a stratified soil or rock mass with horizontal and vertical permeabilities.	For determination of k_h : $F = \frac{0,9\pi L}{\text{Log}(2mL/D)}$ where $m = (k_h/k_v)^{1/2}$ $L > 4D$
	Borehole extended a distance L beyond the end of the casing which is flush with an impermeable boundary	$F = \frac{0,9\pi L}{\text{Log}(4L/D)}$ For $L > 4D$.

Details of falling head and constant head tests for permeability measurement in soil or rock masses with shape factors for borehole end conditions.

Chapter 17

Falling Head Test

The falling-head test is also derived from the laboratory permeameters and has been adapted for field tests. This test is much easier to perform than the constant-head test as the apparatus required is much simpler. A simplified diagrammetrical sketch is shown of the falling-head apparatus in Figure 38 and consists of a tube inserted into a drill or auger hole. Again if a tight fit between the tube and the hole does not exist then packing between the tube and the hole will be necessary at approximately the water table level. The portion of the tube above ground level should be constructed of glass in order that the fall in water level can be clearly noted during the test.

The same applies to the falling-head test regarding size of tube to soil grain size, quick sand condition and lining of the hole with finer material as does to the constant-head test and is not repeated here. The advantage in using this test is the use of only limited amounts of water. In this test it would be easier to use de-aerated water as only a limited quantity would be required. A pre-saturation of the soil in the immediate vicinity of the hole is important otherwise high coefficients of permeability would be realised giving an incorrect interpretation of the soil properties.

A certain amount of water is introduced into the tube and the level on the tube noted. The water is then allowed to drop and the times monitored when the water level falls past certain predetermined heights. The heights of the top of the water in the tube are related to the top of the water table as a datum.

The coefficient of permeability is obtained from the following equation:

$$k = \frac{0,3026 A}{F(t_2 - t_1)} \log \frac{H_1}{H_2} \quad (26)$$

where A = the cross sectional area of the water column
 $= 0,25 \pi d^2$ where d is the inside diameter of the tube in a vertical borehole. For an inclined hole, A must be corrected to account for the elliptical shape of the horizontal water surface in the tube.

F = shape factor which depends upon the conditions at the bottom of the hole. Refer to Figure 38.

H_1 & H_2 = water levels in the borehole, measured from the water table, at predetermined positions

t_1 & t_2 = times when the water level drops below the predetermined heights H_1 and H_2 .

It must be remembered that if the water temperature is very different from the 20°C accepted for the standard for the permeability tests then the viscosities should be used to convert to the standard. Refer to Appendix A.

Chapter 18

Rising Head Test

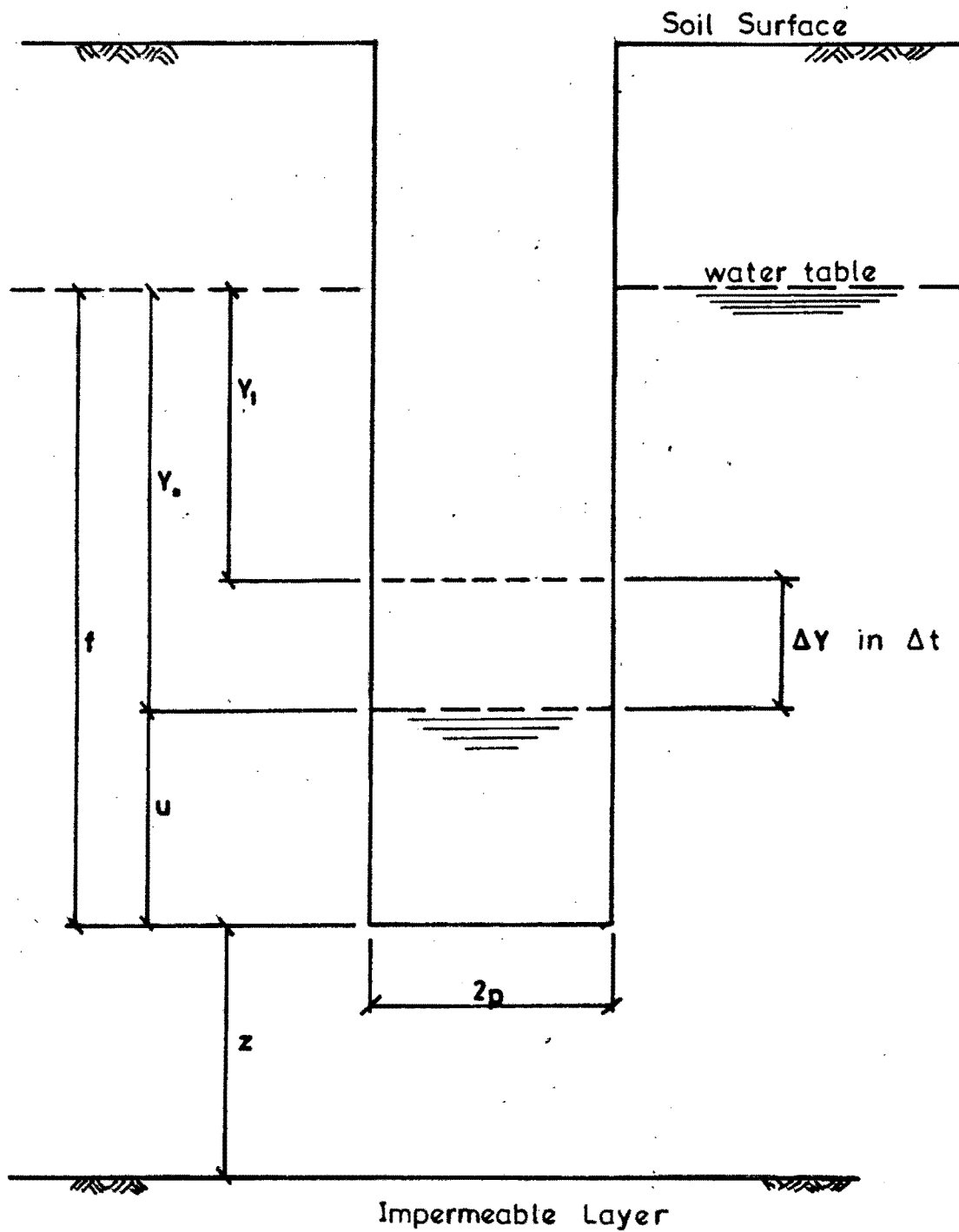
The method used is very simple and can be done with a small amount of equipment. The procedure is to dig a hole in the soil beneath a water table. After allowing the water level in the hole to come into equilibrium with the water table in the soil, the water is pumped out of the hole and measurements are made of the rate of rise of water in the hole.

These measurements are then used to calculate the hydraulic conductivity. Hooghoudt derived an equation to utilise the results of the Rising Head test to obtain permeability. It should be noted that amendments have been made on the original Hooghoudt equation in order to simplify it to units used in this paper, i.e. centimetres/second. It is worthwhile considering Hooghoudt's derivation in some detail since his methods of analysis can be applied to other groundwater flow problems.

Hooghoudt developed two formulae, one for use where the auger hole reaches an impervious layer and one for use where the impervious layer is at great depths below the end of the auger hole.

One of the assumptions made in his derivation is that the water table is not lowered around the auger hole when water is pumped out of it. The condition is satisfied approximately for a short period of time after the auger hole has been pumped. If, however, the auger hole is pumped repeatedly in succession, this condition will not be met.

Another assumption made by Hooghoudt is that water flows horizontally into the sides of the auger hole and vertically up through the bottom of the hole.



AUGER HOLE

fig 39

Hooghoudt's equation is developed in the following manner. If we assume that the water flows horizontally through the sides of the auger hole, the rate at which the hole fills with water is proportional to the circumference of the hole and is inversely proportional to the cross-sectional area of the hole. The rate of rise of water in the hole due to circumferential flow at time t is thus assumed by Hooghoudt to be

$$\frac{dy}{dt} = \frac{-k2\pi pf}{\pi p^2} \cdot \frac{y}{E} = \frac{-2kfy}{pE} \quad (27)$$

The constant E should depend on p , f and Z and also on the height of water in the hole at the time of measurement. Hooghoudt determined E with the aid of a controlled experiment in a sand tank and found that E in his assumed equation depended on p , the radius of the auger hole, and f , the distance from the bottom of the hole to the water table, according to the following empirical relation (in which Z and the height of water in the hole do not occur):

$$E = \frac{pf}{0,19} \quad (28)$$

where E has the dimension of a length. The experimental conditions under which Hooghoudt determined E only approximated field conditions because of the finite size of the sand tank used, and Hooghoudt suggests that the coefficient 0,19 is accurate to within about 27% of the true value; an accuracy which he considers adequate for the determination for the hydraulic conductivity, which may vary in the field from 0.001 to more than 10 m/day. The numerical coefficient 0,19 is valid only for measurement in metres.

Water also flows upward through the bottom of the hole and an approximate expression for rate of rise of water in the hole due to this flow is assumed to be

$$\frac{dy}{dt} = - \frac{k \pi p^2 y}{\pi p^2 E} = - \frac{ky}{E} \quad (29)$$

where E is the same constant as before.

If we add equations

$$\frac{dy}{dt} = \frac{2kfy}{pe} - \frac{ky}{E} = - \frac{k(2f + p)}{pE} y \quad (30)$$

which is the rate of rise of water in the hole due to water entering both circumference and the bottom of the hole. Integrating equation (30) between the limits $y = y_0$ to $y = y_1$ and $t = 0$ to $t = t$, we have

$$\ln \frac{y_0}{y_1} = k(2f+p) \frac{\Delta t}{pE} \quad (31)$$

or introducing logarithms to the base 10,

$$k = \frac{2,3 pE}{(2f + p) \Delta t} \log_{10} \frac{y_0}{y_1} \quad (32)$$

When the auger hole terminates on an impermeable layer the vertical flow of water through the bottom equals zero and equation (32) becomes:

$$k = \frac{2,3 pE}{2f \Delta t} \log_{10} \frac{Y_0}{Y_1} \quad (33)$$

Since $E = \frac{pf}{0,19}$ we can introduce it in the above equations. Then, for an auger hole on an impervious layer we have:

$$k = 523000 \frac{p^2}{\Delta t} \log_{10} \frac{Y_0}{Y_1} \quad (34)$$

where p the radius of the hole is measured in metres and t in seconds. The value k is now given in metres per day. To simplify this to centimetres per second we obtain the following:

$$k = 0,0605 \frac{p^2}{\Delta t} \log \frac{Y_0}{Y_1} \text{ cm/sec} \quad (35)$$

Chapter 19

Sieve Analysis Test

Laboratory tests of a fine sand and a coarse sand show clearly the effect of particle size on permeability. By experiment Johnson (29) found that for a coarse sand, with an average grain size of 0,46 millimetres, a coefficient of permeability of 7,27 litres per day per square centimetre could be obtained. For a fine sand with an average grain size of 0,23 millimetres a corresponding coefficient of permeability of only 1,45 litres per day per square centimetre was observed. The porosity of both samples was the same, each containing 35 percent voids. Even though the total volume of voids is the same, the smaller pores in the finer sand will cause a greater increase in resistance to flow, as is indicated by a lower value of the permeability.

The grading of a sand has a distinct effect on the permeability, a uniformly graded sand has a higher porosity than a less uniform fine and coarse mixture. In the mixture of coarse and fine, the porosity is reduced because the finer sand particles occupy openings between the coarser particles. The result is a more compact arrangement, reducing the volume of the voids. The permeability of the denser mixture is usually lower than that of the finer fraction with the coarser particles removed. It has been found however that most natural sand deposits tend to be fairly well graded and permit reasonable percolation of water. Permeability does vary with such factors as the size, distribution and continuity of the pores as well as with the total porosity. Silt and clay or other cementing materials may partially or completely fill the voids in a sand and make it almost impervious.

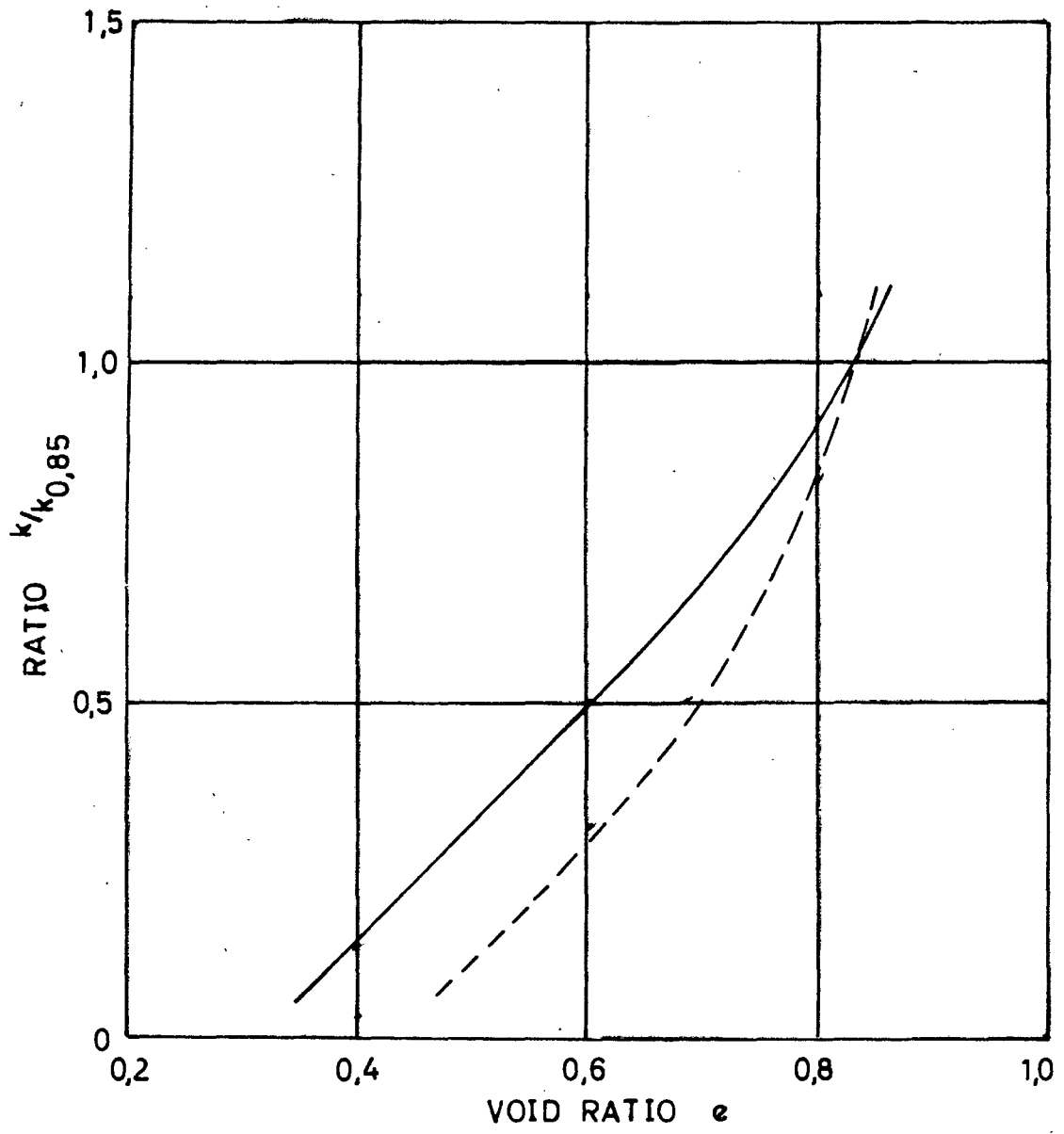


fig 40

Many attempts have been made to calculate the permeability from grain sizes of sand samples, that is, from the sand analysis curves. Some success has been obtained when dealing with uniform or well-sorted materials and when dealing with formation materials from a given locality. Important disadvantages associated with this method is that the sand can be compacted to different densities which means that the porosity of the sand may vary and also the distribution of grain size may be non-uniform, as discussed above. The method of calculating the permeability from the sieve analysis holds definite advantages in that it is easy to obtain in relatively clean, well-graded sands.

The channels through which the water particles travel in a mass of soil have a variable and irregular cross section. However, the average rate of flow through such channels is governed by the same laws that determine the rate of flow through straight capillary tubes having a uniform cross-section. If the cross-section of the tube is circular, the velocity flow increases, according to Poiseville's law, with the square of the diameter of the tube. Since the average diameter of the voids in soil at a given porosity increases practically in proportion to the grain size D , it is possible to express the coefficient of permeability on the basis of Poiseville's law as

$$k = \text{constant} \times D^2 \quad (36)$$

From his experiments with loose filter sands of high uniformity (uniformity coefficient not greater than about 2), Hazen obtained the empirical equation.

$$k = C D_{10}^2 \text{ cm/sec} \quad (37)$$

where $c =$ varies from about 100 to 150

D_{10} = the grain size diameter at which 10 percent of the material is finer by weight and measured in centimetres.

Recent tests by Burmister (30) indicate a wider range of values for C from 20 to 600, using a wider range of cohesionless soil types.

In both cases a mean value of about 100 was obtained for C .

A further factor to be considered is the relationship of void ratio and permeability. The void ratio e is the ratio of the volume of voids to the volume of the solid substance. When a soil is in compression or vibrated, the volume occupied by its solid constituents remains practically unchanged, but the volume of the voids decreases. As a consequence, the permeability of the soil also decreases. The influence of the void ratio on the permeability is illustrated by Figure 40. In this figure the abscissas represent the void ratio. The ordinates represent the ratio $k/k_{0,85}$ between the coefficient of permeability k of the soil at any given void ratio e and that of the same soil at a void ratio of 0,85.

The plain curve shows the relation between e and $k/k_{0,85}$ for fine to medium clean sands with bulky grains. This relation can be expressed fairly accurately by various simple equations, such as by Casagrande's equation

$$k = 1,4 k_{0,85} e^2 \quad (38)$$

In connection with foundation problems, clean sands are seldom encountered. If a sand contains a high percentage of scale-like particles such as mica flakes, the relationship between e and $k/k_{0,85}$ resembles that indicated by the dash curve below the plain curve in Figure 40. Fine grained soils always contain flaky constituents but since the p portion is different for different soils, the corresponding $e-k/k_{0,85}$ curves are different.

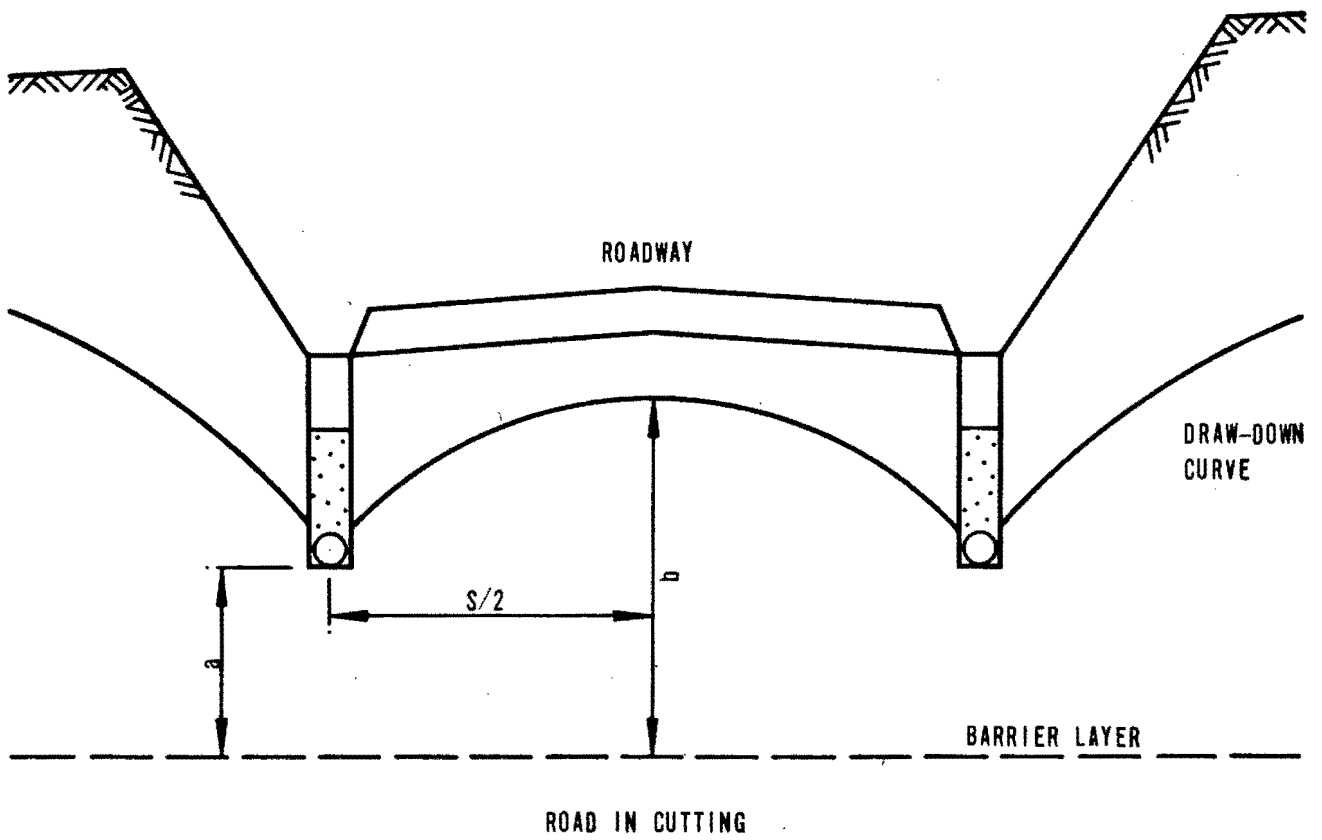
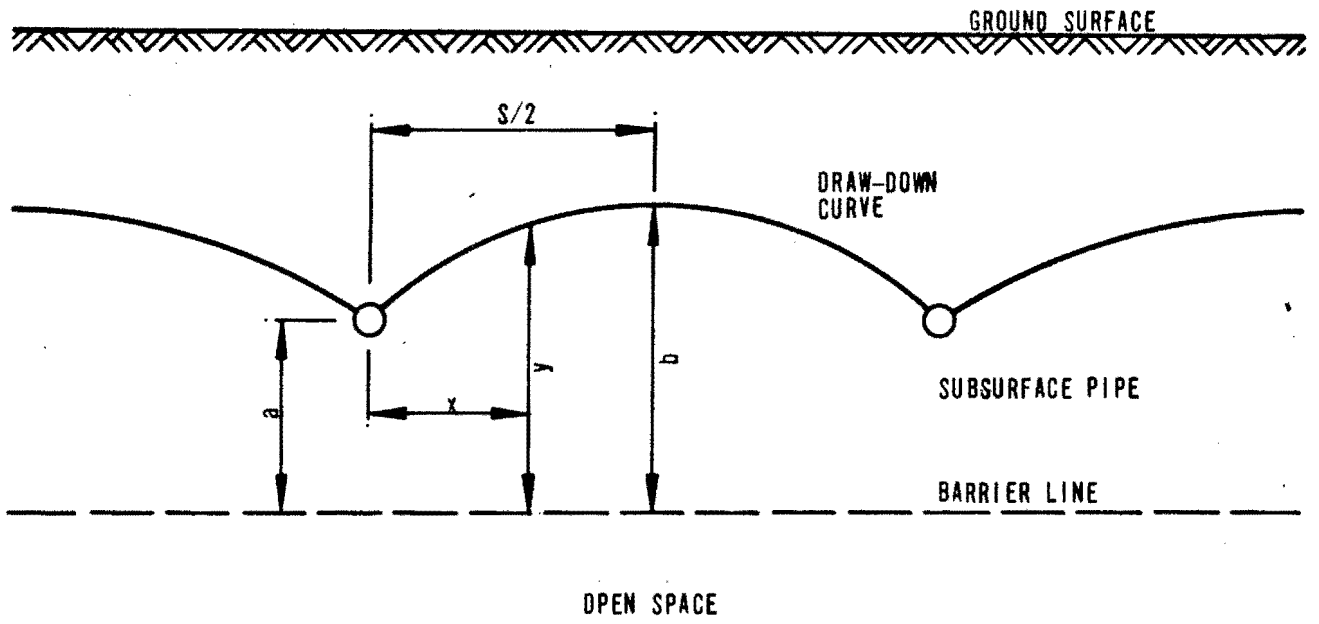
Chapter 20

Design of Sub-surface Drains

The flow of groundwater to a subsurface perforated pipe or ditches is governed by the same factors controlling the flow to a well. Both ditches and drains create a water table like that shown in Figure 41. The cone of depression about a well becomes a trough along the line of the drain. The water table should be lowered to below the selected layers in a road formation or below the root zone where vegetation is contemplated to permit aeration of the soil. In general, the water table should be lowered more in heavy clay soils than in light sandy soils. It is not desirable, however to lower the water table far below the recommended minimum depth. for this deprives the local vegetation of capillary moisture needed, during the dry season.

The same approach regarding the subsurface drainage system will apply to a ditch. The depth of the ditch being determined by the draw-down requirements. Care must be taken to ensure that the sides of the ditch does not erode due to the water pressure caused by the hydraulic gradient towards the ditch.

In homogeneous soil the spacing can be determined by an analysis similar to that for a well in a homogeneous aquifer. In Figure 41 the hydraulic gradient at distance x from the drain is dy/dx , and for a unit length of drain the cross-sectional area of flow is y . Assuming that the flow q towards the drain is inversely proportional to distance from the drain, i.e. when $x = s/2$, $q = 0$ and when



- DEVELOPMENT OF SPACING FORMULA

$x = 0$ then $q = \frac{1}{2}q_d$, where q_d is the design flow per metre of drain, the Darcy equation may be extended as follows:

$$\begin{aligned} q &= kAi \\ Q &= \frac{2}{S} \left(\frac{S}{2} - x \right) \frac{q_d}{2} \\ &= k y \frac{dy}{dx} \end{aligned} \quad (39)$$

where S = spacing between the pipes metres

A = gross cross-sectional area of the aquifer - square centimetres

i = hydraulic gradient

transforming this equation,

$$\frac{q_d}{2kS} (S - sx) dx = y dy \quad (40)$$

and integrating,

$$\frac{q_d}{2kS} \left(Sx - 2\frac{x^2}{2} \right) = \frac{y^2}{2} + c \quad (41)$$

when $x = 0$, $y = a$ and hence

$$c = -a^2/2 \quad (42)$$

substituting this value for c in equation and solving for k ,

$$k = q_d \frac{(Sx - x^2)}{S(y^2 - a^2)} \quad (43)$$

when $x = S/2$, $y = b$ hence,

$$S = 4k \frac{(b^2 - a^2)}{q_d} \quad (44)$$

(a and b in centimetres and q in cubic centimetres/sec).

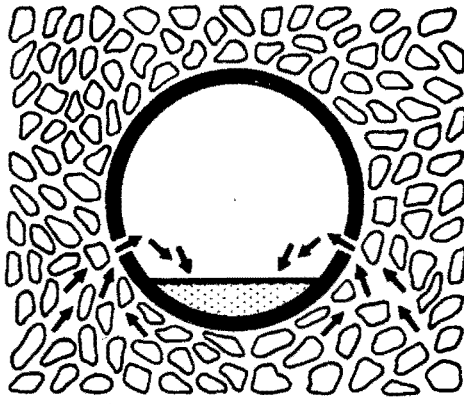
The presence of an impermeable layer near the drain reduces the flow to the drain considerably. In this case a and b are small, and the difference of their squares is accordingly small.

The most important single factor in determining drain spacing is the soil permeability. Since drainage normally involves only a shallow depth of soil any of the last four methods dealt with in this project would prove quite satisfactory.

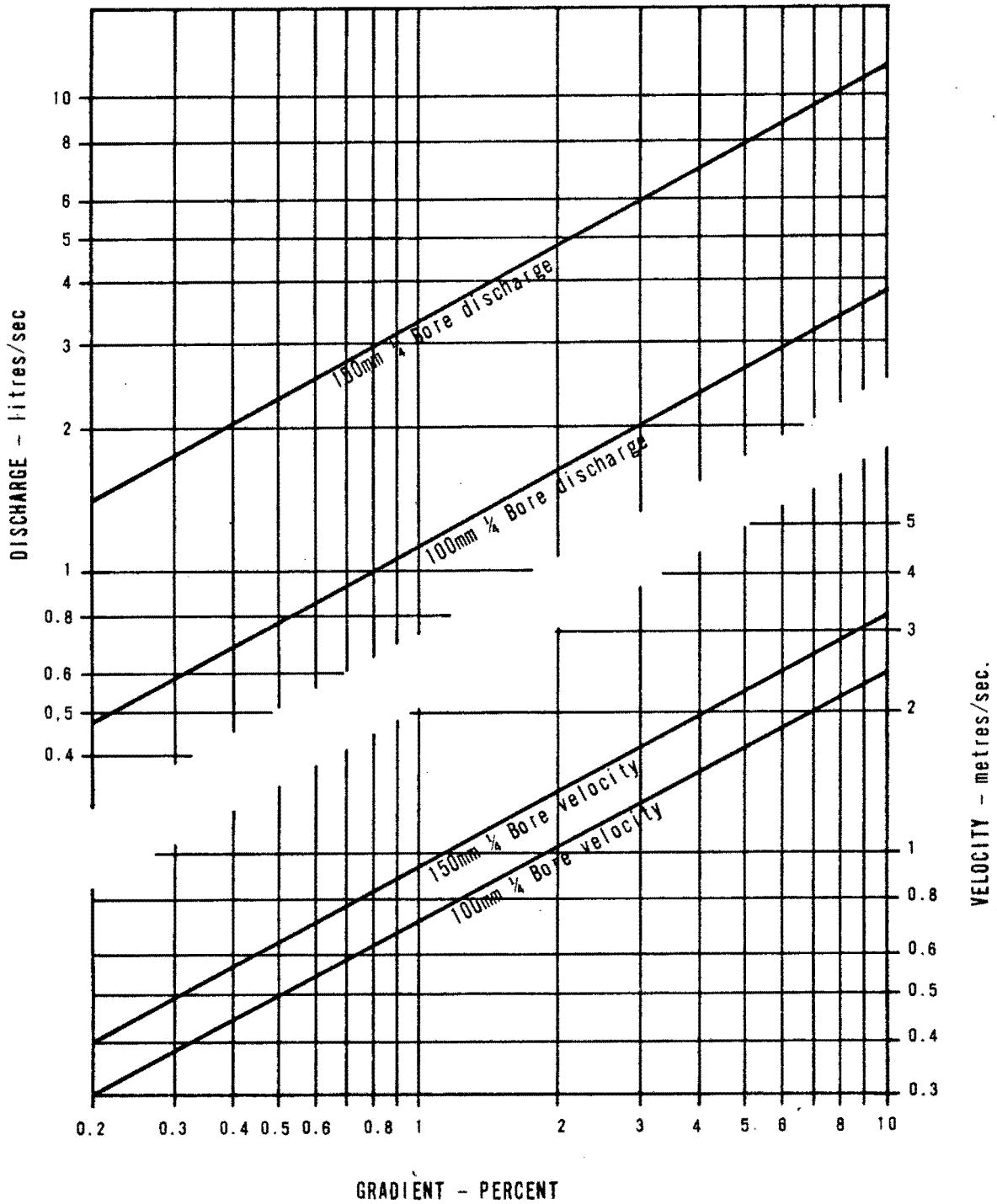
Normally open drains, because of their larger dimensions are more effective in removing soil moisture. Although these are not frequently used in preference to the sub-surface drains due to the physical barrier created, it may still prove to be the better solution in road cuttings where the longitudinal grade is flat or there is a very real danger that the sub-surface pipes may become ineffective due to the root growth, within the pipe, from surface vegetation.

On flat terrain a minimum grade for the sub-surface pipes should be established based on flow conditions. From experience it is felt that a minimum grade of 0,5 percent should be maintained because of two reasons. Firstly although a drain is accurately levelled if the grade is too flat then sags may develop during installation. The second reason is that if the grade is too flat the pipe may accept water from a wet area and deposit it in a drier zone hence shifting the problem. Flatter grades may be acceptable where the soil is cohesive and where the quality of installation is acceptable.

The average flow from subsurface drains will vary depending on spacing, soil type, rainfall intensity and many other interrelated factors. The normal type of subsurface pipe used is the pitch-fibre pipe made in 100 and 150 millimetre diameters with two parallel rows of 8 millimetre holes spaced at 75 millimetre centres. There are two rows in the 100 millimetre diameter pipes at 120° and 4 rows in the 150 millimetre pipes at 100° and 150° . The pipes are laid with the holes facing down enabling soil-free water to enter the pipe, thus ensuring even drainage and preventing silting up and subsequent blockage of the drain. Refer to Figure 42 for a diagram of the perforated pipe as also the resultant discharge and velocity, the 100 and 150 millimetre pipes, with different longitudinal grades.



PIPE CROSS-SECTION



HYDRAULIC FLOW CHART

fig 42

Chapter 21

Design of Filters

In most cases, the drainage of engineering works is accomplished by artificial devices, such as blanket drains over seepage areas, intermittent line drains or trench drains, vertical relief wells and horizontal drains with small diameter perforated or slotted pipes. Most drainage systems make use of porous filter aggregates to collect the water and conduct it to outlets, often with the aid of perforated pipes. If the formations being drained are firm nonerodable rocks, drainage may be obtained simply by drilling small diameter drain wells which feed the water to exits. However, if the water bearing materials are soft, erodable formations of soil or rock the porous aggregate drainage materials must hold the erodable materials firmly in place, while freely allowing water to escape. This is in keeping with the basic purpose of any filter, to allow the passage of a fluid or gas while separating out the solid matter.

To ensure complete filter protection to erodable materials, porous aggregate drain layers in contact with the soil must not have any continuous openings large enough for the passage of the soil particles. These filters must generally be relatively fine grained. In addition, any drainage system must be capable of freely discharging all of the groundwater and seepage that reaches it, under relatively small hydraulic gradients and small excess hydraulic head. Therefore, whenever appreciable quantities of water must be removed, drainage systems must contain porous aggregates of relatively high permeabilities. Thus, filters generally must be relatively coarse grained. Here are two basic, but conflicting requirements of porous aggregate filters and drains, they must be fine enough to hold erodable materials in place, but they must also be coarse enough to discharge all of the water that reaches them.

With the development of the rational and experimental approach to soil mechanics, earthwork design has become more of a science than an art. Bertram (34), with the advice of Terzaghi and Casagrande conducted laboratory filter experiments to test filter criteria that had been suggested by Terzaghi. Bertram's work led to the following widely used criteria for designing filters:

$$\frac{D_{15} \text{ (of filter)}}{D_{85} \text{ (of soil)}} < 4 \text{ to } 5 < \frac{D_{15} \text{ (of filter)}}{D_{15} \text{ (of soil)}} \quad (45)$$

The left half of the above equation, a fundamental criterion for the prevention of piping through filters, may be stated as follows:

Piping criterion: That 15 percent size (D_{15}) of a filter material must be not more than four or five times the 85 percent size (D_{85}) of a protected soil. The ratio of D_{15} of a filter to D_{85} of a soil is called the piping ratio.

The right half of the equation (45) may be stated as follows:

Permeability criterion: The 15 percent size (D_{15}) of a filter must be at least 4 or 5 times the 15 percent size (D_{15}) of a protected soil. This requirement will generally ensure that filter layers will be several times more permeable than the adjacent soils, but does not always guarantee adequate hydraulic conductivity in drains.

Some design organisations place additional restrictions on filter materials. For example, the U.S. Bureau of Reclamation limits the maximum size of filter aggregates to 75 millimetres in order to minimise segregation and bridging of large particles during placement. The U.S. Army Engineers normally also limits the piping ratio to 5 and uses the following criterion:

$$\frac{\text{50 percent size of the filter material}}{\text{50 percent size of the protected soil}} \leq 25 \quad (46)$$

If crushed stone is used the U.S. Army Engineers recommends limiting the piping ratio (D_{15} of filter to D_{85} of soil) to less than 5.

In the case where the protected soil is a plastic clay without sand or silt partings, the D_{15} size of the filter may be as great as 0,4 millimetres and the above D_{50} criteria can be disregarded. This relaxation in criteria for protecting medium to high plastic clays will allow the use of one-stage filter material. However, the filter must be well graded, and to ensure nonsegregation of the filter material, a coefficient of uniformity, D_{60} to D_{10} , of not greater than 20 is necessary.

It is generally agreed that the sieve analysis of the filter material and the soil be almost parallel to each other. Refer to Figure 43 in this connection.

In cases where the in-situ material is of such a size as to necessitate the installation of more than one filter material layer the same procedure regarding soil and filter discussed above must be adopted. For example, the following should apply:

$$\frac{D_{15} \text{ (second filter)}}{D_{85} \text{ (first filter)}} < 5 < \frac{D_{15} \text{ (second filter)}}{D_{85} \text{ (first filter)}} \quad (47)$$

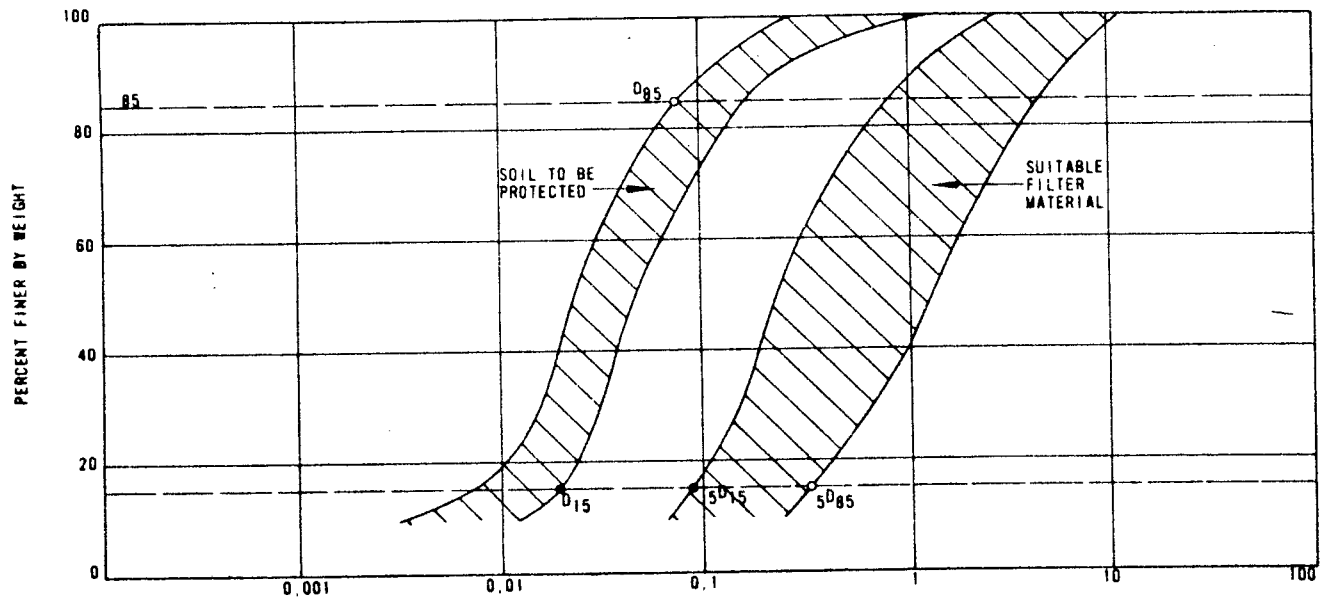
The above procedure should be continued for as many filter layers as is considered necessary. It must be remembered however that it is difficult and expensive to construct multiple filter layers successfully and hence moderation should be practiced when designing filters.

A major step in sub-surface drainage design is the introduction of filter fabrics. The normal fabric can contain a particle of 0,08 millimetres and can contain relatively large stone as a filter. This permits the use of only one filter material within the filter fabric, the only precaution being that the filter material does not syphon through the perforations in the pipe.

Where perforated pipes are used the filter material surrounding the pipe must have gradations which are compatible with the size of the holes. The following criteria is used:

$$\frac{D_{85} \text{ size of filter material}}{\text{hole diameter}} > 1 \text{ or } 2 \quad (48)$$

When the criteria above is satisfied in every part of a filter or drain, piping cannot occur under even extremely large hydraulic gradients.



FILTER CRITERIA FOR DRAINAGE LAYER.

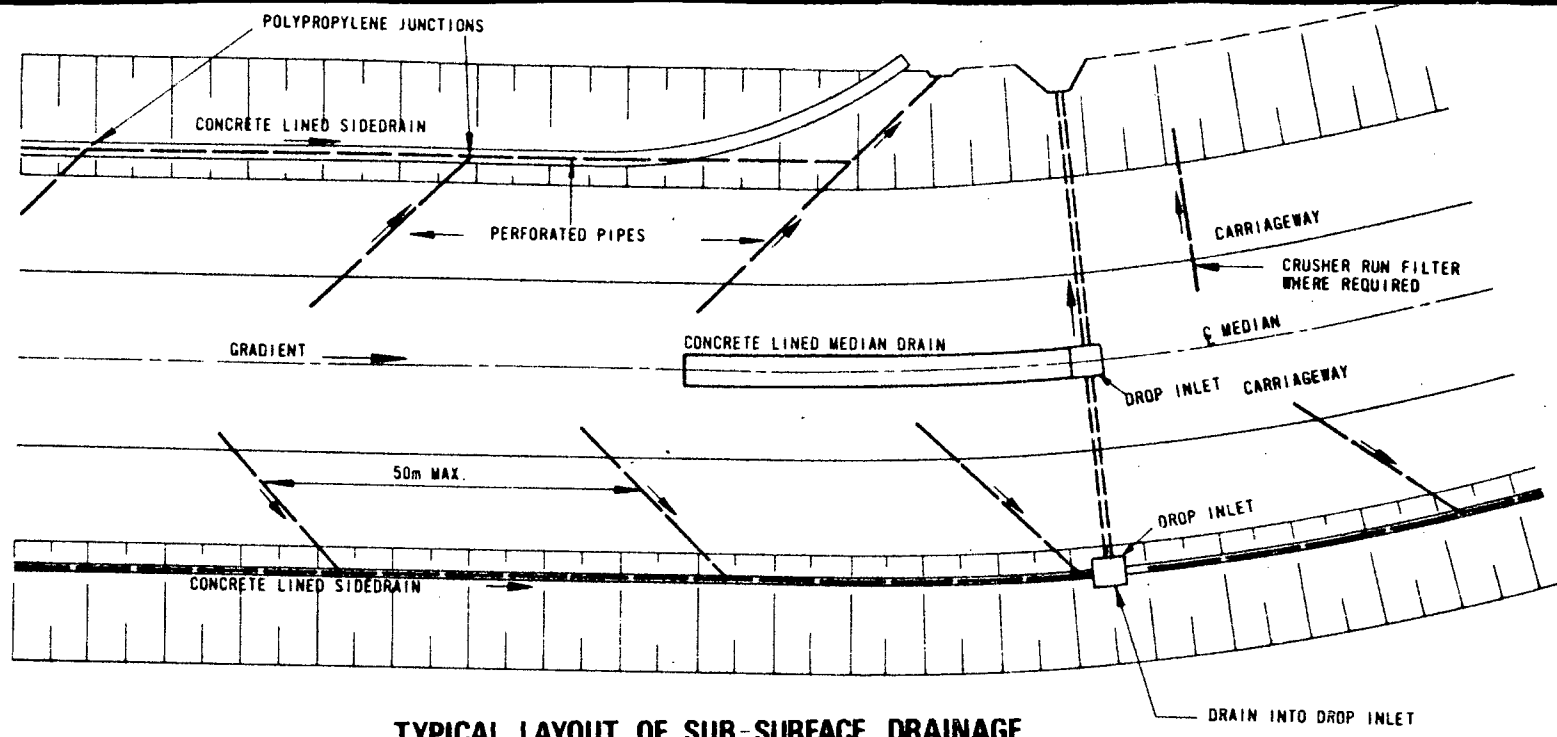
PIPING RATIO $\frac{D_{15} \text{ FILTER}}{D_{85} \text{ SOIL}} < 5$

PERMEABILITY RATIO $\frac{D_{15} \text{ FILTER}}{D_{15} \text{ SOIL}} > 5$

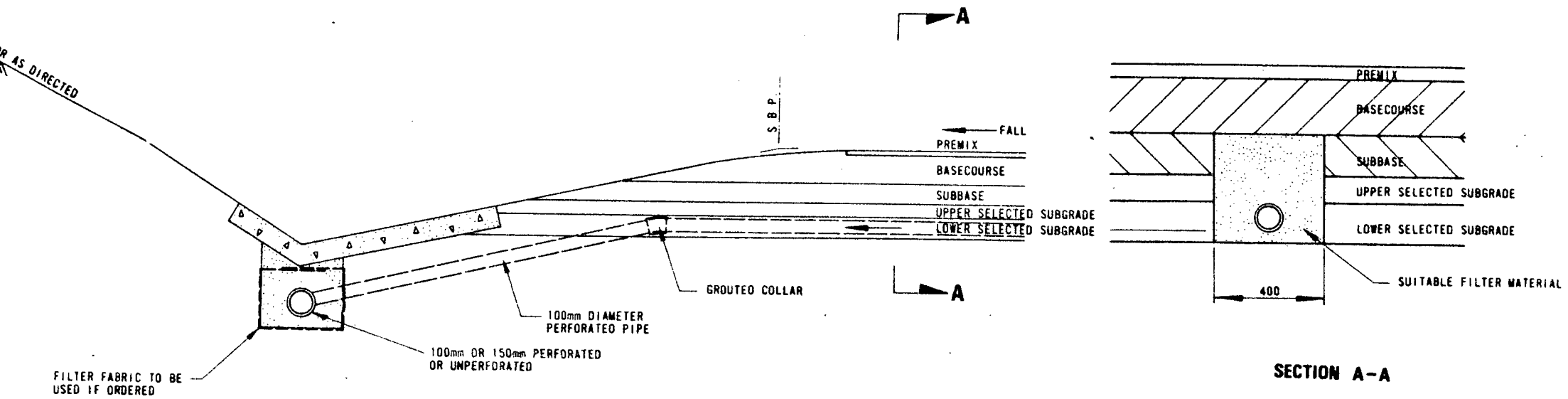
- NOTE ON FILTER FABRIC
1. THE FILTER FABRIC USED ON THE CONTRACT SHALL RETAIN A SOIL WITH A D_{85} PARTICLE $> 0.08\text{mm}$.
 2. THE THROUGH FLOW CAPACITY OF THE FABRIC SHALL BE SUCH THAT A FLOW OF GREATER THAN $0.5\text{m}^3/\text{sec}/\text{m}^2$ CAN BE OBTAINED THROUGH THE MATERIAL UNDER A ONE METRE HEAD OF WATER.

PARTICLE SIZE - mm							
CLAY	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	GRAVEL
	SILT			SAND			

EXAMPLE OF FILTER CRITERIA FOR DRAINAGE LAYERS



TYPICAL LAYOUT OF SUB-SURFACE DRAINAGE



LONGITUDINAL SUB-SURFACE DRAIN

SECTION A-A

Chapter 22

Specimen Solution & Field Application

SPECIMEN SOLUTION

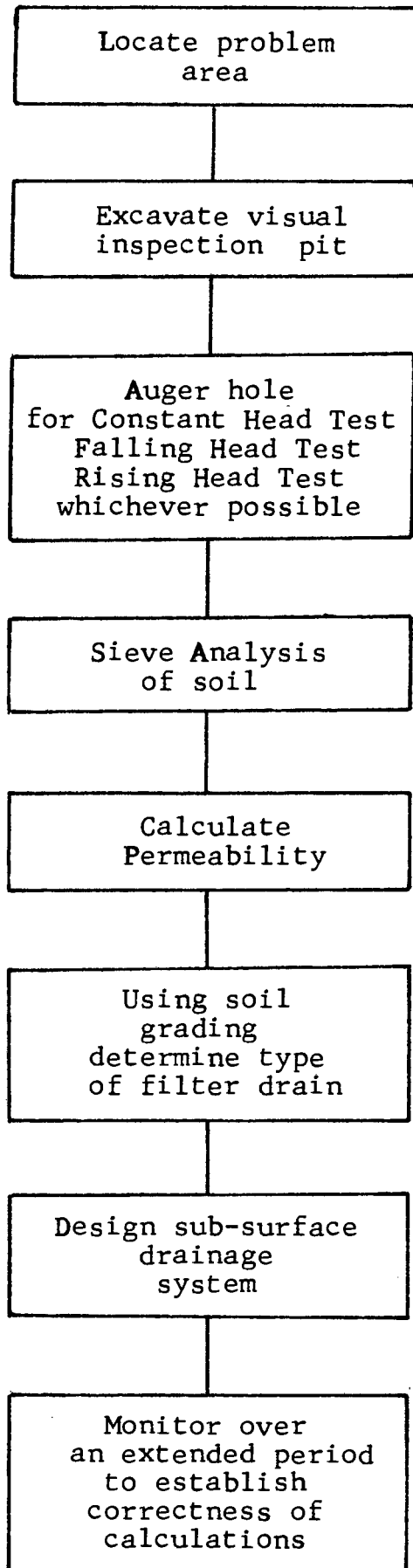
In order to illustrate the above discussion on the testing of the in-situ permeability the following problem is used adopting the Falling Head, the Rising Head tests and the Sieve Analysis method. From the resultant coefficient of permeability the depth of the sub-surface drains is calculated.

The following information is available:

For the Falling Head Test a 20 cm diameter tube is fitted tightly into a borehole in fine cohesionless sand, the end of the tube being 100 cm above the bottom of the borehole. Two marks were made 350 cms and 450 cms above the water table. Water was inserted into the tube and a time of 7,5 seconds was recorded during the fall of the top meniscus of the water past the two marks.

For the Sieve Analysis Test a sieve grading was done on the sand and the diameter of the particle which is 10 percent coarser than the finest by weight was found to be 0,075 millimetres.

For the Rising Head Test an auger hole was sunk in the soil. After allowing the water level in the auger hole to stabilise the level is recorded. The water was pumped out to 80 cm below the water table level. The water rose 10 cm in 5,8 seconds. The diameter of the auger hole was 3 cms.



FLOW CHART

The site is a road cutting in fine cohesionless sand four metres thick. The cutting is one metre deep and it is desired that the water table be lowered to 0,5 metres below this level.

It is required to establish how deep the 150 millimetre diameter sub-surface drains would be installed, at a longitudinal grade of 0,8% and 15 metres apart.

a) FALLING HEAD TEST:

$$\text{Area of tube} = A = \pi \left(\frac{20}{4}\right)^2 = 314,2 \text{ cm}^2$$

From Figure 8, the shape factor

$$\begin{aligned} F &= \frac{0,9 \pi L}{\log (2L/D)} \\ &= \frac{0,9 \pi 100}{\log \frac{2 \times 100}{20}} = 262 \end{aligned}$$

Coefficient of permeability from equation 26

$$\begin{aligned} k &= \frac{0,3026 A}{F (t_2 - t_1)} \log \frac{H_1}{H_2} \\ &= \frac{0,3026 \times 314,2}{262 \times 7,5} \log \frac{450}{350} = 0,00528 \text{ cm/sec.} \end{aligned}$$

b) SIEVE ANALYSIS:

Coefficient of permeability from equation 18

Using $C = 100$ (Hazen)

$$\begin{aligned} k &= CD_{10}^2 \\ &= 100 \times (0,0075)^2 = 0,00563 \text{ cm/sec} \end{aligned}$$

c) RISING HEAD TEST:

From Hooghoudt's equation

$$\begin{aligned}
 k &= 0,0605 \frac{p^2}{t} \log \frac{Y_0}{Y_1} \\
 &= 0,0605 \times \frac{3^2}{5,8} \times \log \frac{80}{70} \\
 &= 0,00655 \text{ cm/sec}
 \end{aligned}$$

d) SUB-SURFACE DRAINAGE DESIGN:

From Figure 42 two 150mm pipes accommodates 6 litres/sec ($6 \times 10^{-3} \text{ m}^3/\text{sec}$)

Using equation 44

$$\begin{aligned}
 S &= 4k \frac{(b^2 - a^2)}{q} \\
 b^2 - a^2 &= \frac{Sq}{4k} \\
 a^2 &= b^2 - \frac{Sq}{4k} \\
 &= (2,5)^2 - \frac{15 \times 6 \times 10^{-3}}{4 \times 5,83 \times 10} - 3 \\
 a &= 1,54 \text{ metres}
 \end{aligned}$$

FIELD APPLICATION

In order to ascertain the practical aspects of using the methods set out above experiments were carried out on different materials using the following four methods:

Constant Head Test

Falling Head Test

Rising Head Test

Sieve Analysis

The in-situ materials used were located in the Bot River lagoon area where the soils vary from mountain wash sand to Table Mountain sandstones and Bokkeveld shales. The area is of particular interest as large flows of sub-surface water occur which cause problems for the road builder. The above tests were conducted and the results used in the construction of sub-surface drains along the road. The drains are being monitored as well as the actual draw-down of the phreatic surface under the road. To date the drains have performed as was expected.

The tests were conducted using a 110 millimetre glass tube which was sunk into the various materials to depths of at least 600 millimetres. The first end condition was used, as shown in Figure 38 for the glass tube in the hole. The depth of the impermeable layer was determined by excavation at each test site. Water was transported in large plastic containers on the back of a truck which gave sufficient head for the experiment.

The results of the tests carried out on various materials on the site, and the mean result of the permeability for each type of material, is given in Table 14, and in Figure 45.

Table 14
COMPARISON OF TESTS

Material	Constant-Head	Falling-Head	Rising-Head	Sieve Analysis
River Sand	$2,6 \times 10^{-3}$	$3,7 \times 10^{-3}$	$2,3 \times 10^{-3}$	$7,0 \times 10^{-3}$
Mountain wash	$9,8 \times 10^{-4}$	$1,5 \times 10^{-3}$	$2,1 \times 10^{-3}$	$2,5 \times 10^{-3}$
Ferricrete Gravel	$2,1 \times 10^{-2}$	$1,9 \times 10^{-2}$	$2,7 \times 10^{-2}$	$2,3 \times 10^{-2}$
Ferruginised Sandstone	$4,2 \times 10^{-2}$	$3,1 \times 10^{-2}$	$4,6 \times 10^{-2}$	$4,0 \times 10^{-2}$
Decomposed Shale	$2,2 \times 10^{-6}$	$9,6 \times 10^{-7}$	$1,2 \times 10^{-6}$	$4,0 \times 10^{-6}$

(Units are in cm/sec.)

The results of the mountain wash was used on the road location as it was through this sand, above the Bokkeveld shales, that the majority of the water flowed. The sand was approximately one and a half metres deep and was relatively clean.

It was established that using the above average permeability of 0,00177 cm/sec two longitudinal drains were required at a minimum depth of 1,5 metres from the shoulder of the road. This would result in the crest of the depressed water table being located below the lowest pavement layer (600 millimetres below the finished road level). Using the pipe capacities for a 150 millimetre perforated pipe it was possible to determine where the discharge points should be along the road particularly with respect to longitudinal gradient. As the drains were 1200 metres long it

was necessary to discharge into concrete pipes, the size of which depended on the calculated sub-surface flow. Generally the perforated pipes could only convey water for 170 metres before requiring a discharge point. This is somewhat longer than would be normally expected as greater capacity was achieved by turning the perforations to the top because of the use of a filter fabric with 19 millimetre stone.

Comparison of Field Results

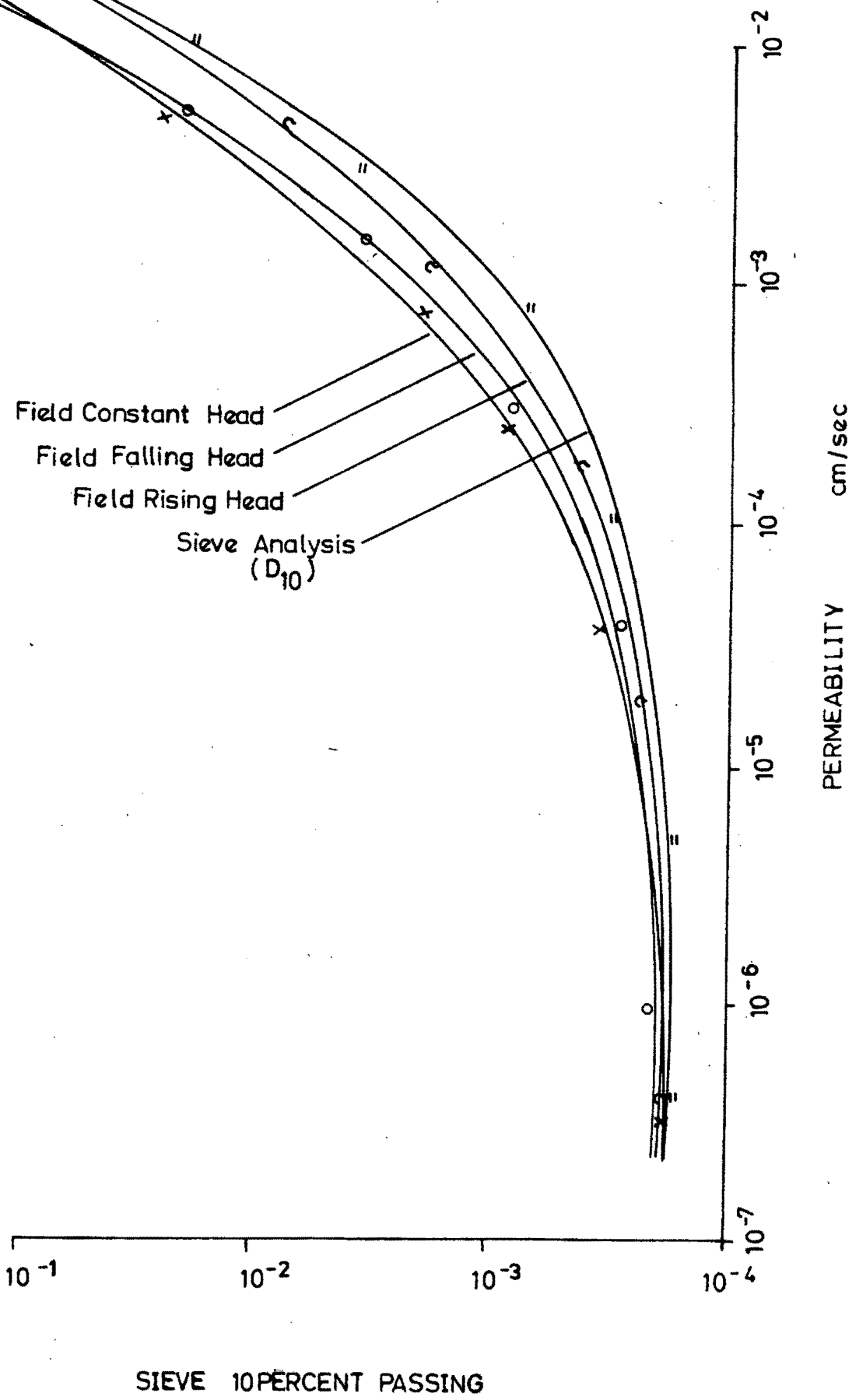


fig 45

Chapter 23

Groundwater Depletion

Although the term groundwater has been considered in this paper as water occurring below the water table it becomes of special interest when considering the recoverable water in wells and boreholes. It must be remembered that it is the water found below the water table which contributes to the recoverable water and not water held in the aeration zone which includes the capillary fringe.

As discussed before the movement of water is dependent on the permeability of the material in the aquifer. The movement is also dependent on the water available in the aquifer which is replenished by rains. After a rainfall a catchment experiences an almost immediate outflow which does not correspond directly to the water reaching the ground from the rainfall. Depending on the position of the water table and the degree of saturation of the material in the aeration zone, the first occurrence of rainwater will be absorbed into the ground. This rate of absorption also depends on the vertical permeability of the material.

After the initial absorption takes place a surface outflow from the catchment area will be observed. This flow will first reach a peak, in the form of a hydrograph, and then a recession will start to take place. At some stage the receding flow from the surface water will become sufficiently low for the hitherto stored groundwater to become a contributing factor to the catchment outflow. Eventually the groundwater will be the only source of water for the catchment outflow. Many factors can control the extent to which groundwater will contribute towards a catchment outflow and these include impermeable layers of material between the storage zones forcing water into adjoining catchments and the shape and groundslope of the catchment.

Studies have been conducted on hydrographs (19) which show that groundwater-depletion curves for a given drainage basin are nearly always the same. Hence the term "normal groundwater-depletion curve" is used by a number of authors. It has been found that this curve, or at least segments of it, follows a simple inverse exponential function of the elapsed time of the form:

$$Q_t = Q_o K^{-t} \quad (49)$$

where Q_o = discharge at any instant
 Q_t = discharge t days later
 K = daily depletion factor

As Q_t is the derivative of storage with respect to time, integration of this equation gives:

$$S_o = \frac{Q_o}{\log_e K} \quad (50)$$

where e = base of natural logarithms
 S_o = groundwater storage at the time of Q_o

From this it can be seen that the discharge at any time is proportional to the water remaining in storage. The value of K can be determined by plotting observed recession curves on semilogarithmic paper taking care to select periods of little or no direct run-off.

Four rivers are shown in Figure 46 namely the Berg, Palmiet, Mooi and Mgeni rivers, where the daily average run-off for a month is shown. From the hydrographs of each the values of K were found to be as follows 1,06; 1,08; 1,03 and 1,02 respectively.

Another ten catchments of varying size were also analysed and from these and the above it appears that the value of K ranges between 1,02 and 1,08. It may therefore be postulated that an average value of 1,05 could be used with reasonable assurance for catchments where full data is not available. A further interesting fact is that there appears to be very little variation in the daily depletion factor for different catchments.

Groundwater Depletion Curves

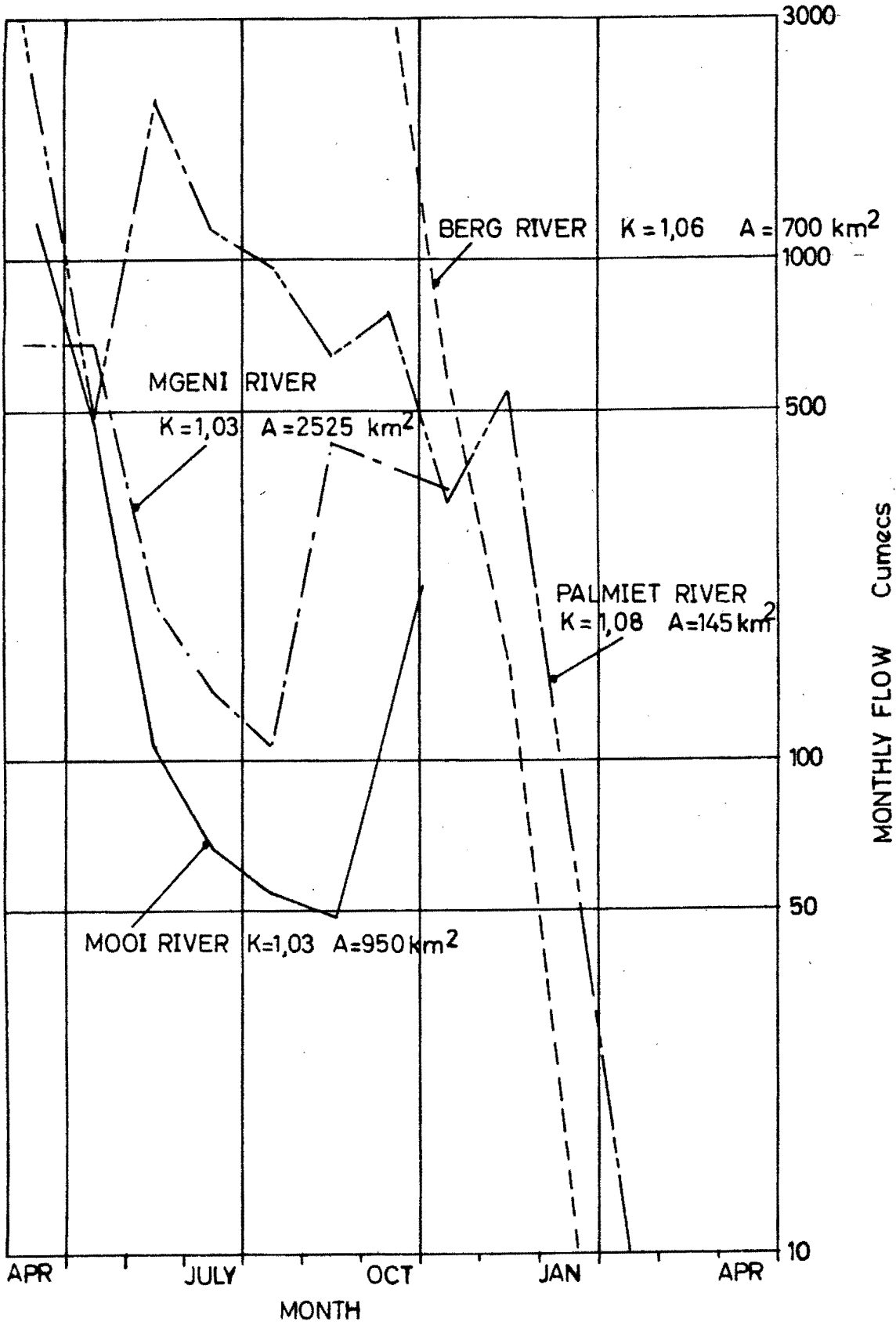


fig 46

Chapter 24

Codes of Practice

There unfortunately does not appear to exist many codes of practice in the world which set out how permeability or sub-surface water yield should be determined. The only comprehensive guide is set out by the West Germans in the code of practice "Erkundung durch Schürfe und Bohrungen sowie Entnahme van Proben " (37). The South African code "Testing Water Boreholes" (38) can be adopted to obtain the permeability via either the pumping returns or the re-establishment of the water level over time (Rising Head Test).

SOUTH AFRICAN CODE OF PRACTICE

The South African code is designed to formulate practical methods, based on scientific considerations, of testing the yield of boreholes. This code was found necessary because many of the more common methods of testing boreholes gave erroneous results.

A strong point mentioned is that the results are dependent on the straightness and the degree from the vertical a hole may be. It is strongly advocated that, at any depth, the centre of the hole should not deviate by more than three millimetres per metre depth.

Another reservation lies in the fact that the results of the tests conducted hold good only under the conditions prevailing at the time of the tests. The quantity of groundwater continually available from boreholes in a particular groundwater compartment is primarily dependent on recharge by direct or indirect infiltration from rainfall.

The code emphasises the need to use a series of boreholes and not rely on the correlation of a single hole. The permeability can be calculated using the yield from the borehole or can be derived by the observation of the water level in the hole. The method used would be as described in the Rising Head Test.

An extract of this code has been copied and can be found in Appendix B.

WEST GERMAN CODE OF PRACTICE

The German code is more in keeping with this paper as it is orientated towards the permeability of materials, with special emphasis on rock formations.

The introduction is very comprehensive covering the different conditions under which water tables and underground water in general are found. The determination of the permeability and the apparatus specified forms the major portion of the code of practice.

A copy of the code (in German) has been included in Appendix C together with an English translation undertaken by Mr. H. Kreft and the writer of this project.

Chapter 25

Recommendations & Conclusions

RECOMMENDATIONS

Pressure tests in rock form the only accurate method of establishing the permeability in rock. Pressure tests can also be used to localise the fracture zones in rock formations which facilitate water movement. This method is expensive as specialised equipment is required and this is not always available to the engineer on site.

Seepage tests using simple tubes, open holes in the ground or boreholes new and existing can produce relatively accurate results. In general, either the Rising Head or the Falling Head Test should be used if the permeability is low enough to permit accurate determinations of water level. In the Falling Head Test the flow from the hole to the surrounding soil may result in clogging of soil pores by sediment in the test water used. This danger does not exist in the Rising Head Test where water flows from the surrounding soil to the test hole although there is a problem that if the hydraulic gradient is too large then the bottom of the hole may become loosened or "quick". If the latter method is used then the test should be accompanied by the sounding of the bottom of the hole with a rod.

It is advisable to conduct a Sieve Analysis Test in all cases where soils and sands are encountered to correlate results obtained using the other methods.

It must be remembered at all times that results must not be blindly accepted and must be tempered with a certain amount of commonsense.

CONCLUSIONS

This project was intended as a guide to the designer and the site staff when designing sub-surface drains. Sub-surface drains are still installed by assumed experience or from some standard drawing produced in a design office. Unfortunately too many sub-surface drains cease to function after a relatively short time and a large number of causes can be related back to insufficient or no investigation. It is felt that as sub-surface drainage is not visible it does not warrant expensive investigation. This lack of investigation is in direct conflict with a client's interest as most pavement failures result from sub-standard drainage provisions during construction.

The problem of sub-surface drainage is becoming a major topic and a number of valuable papers have been written in the last few years. An attempt has been made here to limit the type of tests, to determine the coefficient of permeability, in order that in-situ values may be obtained with the least effort and expense. The types of apparatus necessary for conducting at least the Falling Head and Sieve Analysis Tests should be standard equipment in every laboratory.

The empirical formulae advocated in this project give good correlation with actual laboratory tests undertaken and could be used with a certain amount of confidence.

The sub-surface drainage formula (No. 44), refer to Chapter 20, is a real attempt to overcome guess-work on the part of the engineer where too many assumptions have dictated the size and location of sub-surface drains.

It is sincerely hoped that an awareness for the need for proper design of sub-surface drains will result and that this work may prove to be a contributing factor.

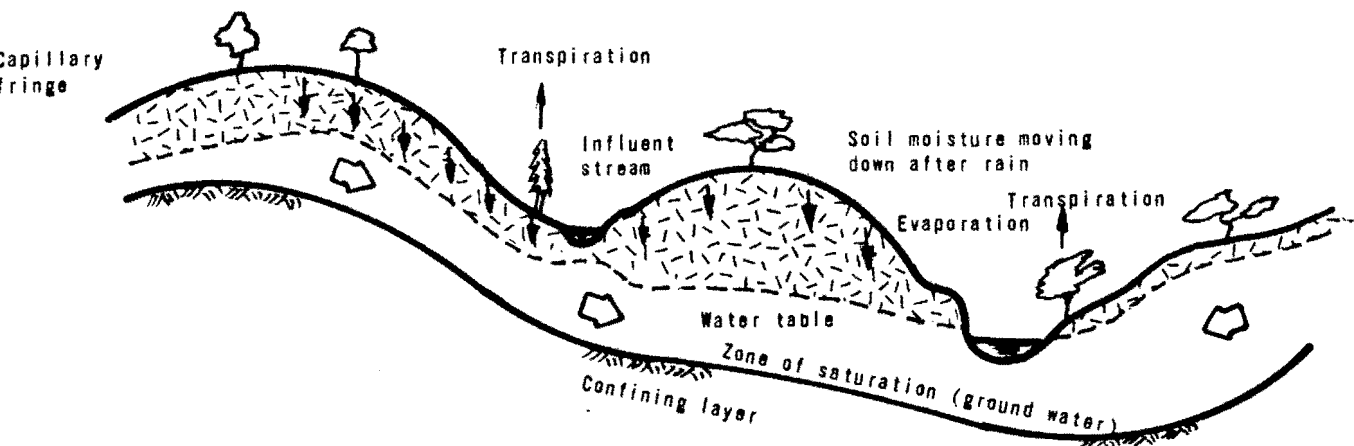
Appendix A

Introduction to Permeability

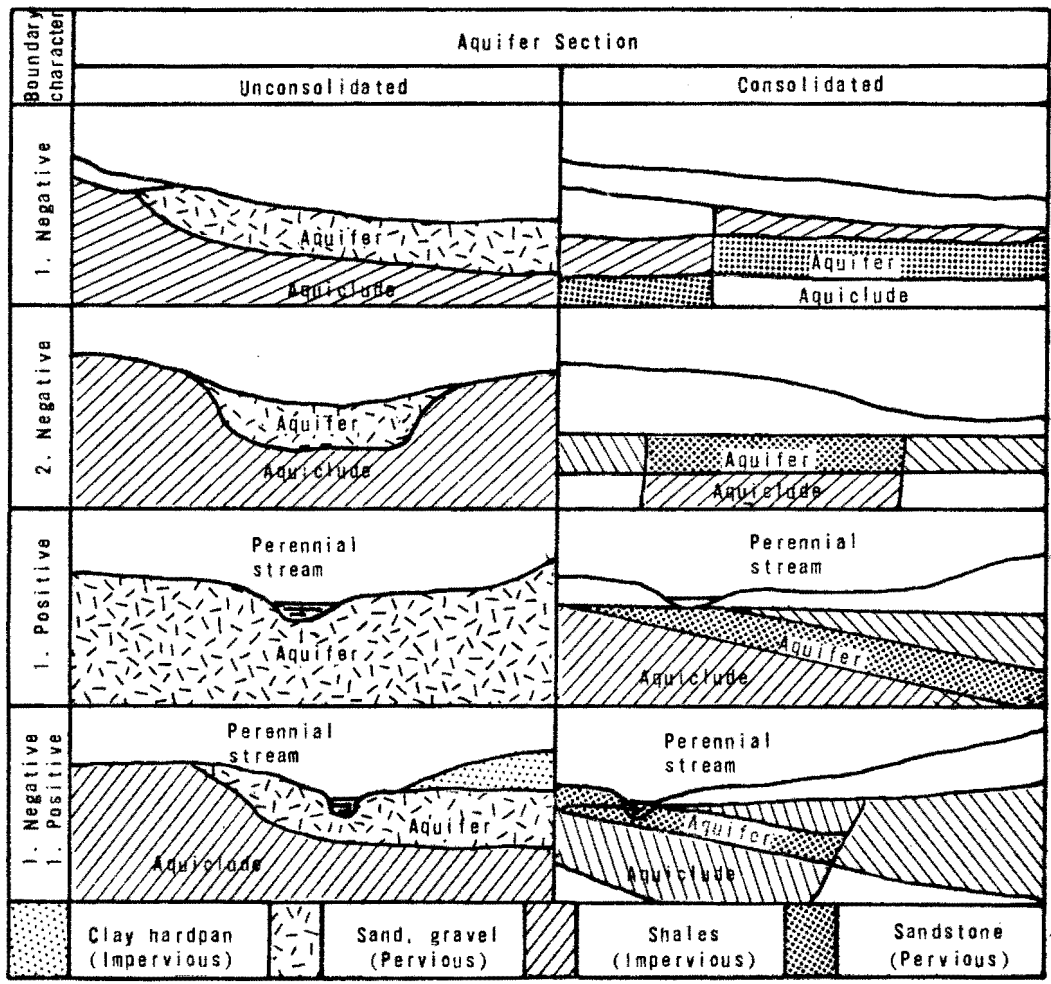
GENERAL

The subject matter of this project is to relate in-situ permeability in groundwater flow to the means by which a water table may be depressed, locally, by means of sub-surface drainage. In order to relate the two subjects a careful inspection of the integral parts and the functions which they play in the overall system must be understood. It is for this reason that a detailed discussion on the various aspects forms an important part of this project.

Permeability is vitally involved with the geology of an area which in turn guides the engineer in his understanding of the sub-surface strata. The earth's crust composed of its myriad and varied hard rocks and unconsolidated overburden, serves as a vast underground reservoir for the storage and transmission of percolating ground waters. The rocks comprising the earth's crust are seldom, if ever, solid throughout. They contain numerous openings called interstices that vary through a wide range of sizes and shapes. Although these interstices are mostly small, they generally are interconnected, permitting movement of the percolating waters. The percolating water moves from the rock formations to the unconsolidated overburden. It is the resultant flow in this material which is of utmost importance to the foundation engineer. However, the fact that the location of rock is important in foundation design cognizance must be taken of the groundwater flow in rock strata. Figure 47 helps to illustrate the sub-surface hydrology and the potential groundwater flow.



SCHEMATIC DIAGRAM ILLUSTRATING THE OCCURRENCE OF GROUNDWATER



IDEALIZED EXAMPLES OF GEOLOGIC STRUCTURES.

A material is said to be permeable if it contains continuous voids. Since such voids are contained in all soils, including the stiffest clays and in all non-metallic construction materials, including granite and neat cement, all these materials are permeable. Furthermore, the flow of water through all of them obeys approximately the same laws. Hence the difference between the flow of water through clean sand and through sound granite is merely one of degree. The permeability of soils has a decisive effect on the cost and the difficulty of many construction contracts, such as the excavation of open cuts in water-bearing sand, or on the rate at which a soft clay stratum consolidates under the influence of the weight of a superimposed fill. The erroneous but widespread conception that stiff clay is impermeable is due to the fact that the entire quantity of water that percolates very slowly through such a material, towards an exposed surface, is likely to evaporate, even in a very humid atmosphere. As a consequence of this evaporation the surface appears to be dry. The absence of visible discharge does not indicate the absence of seepage pressures and it can rather be said that the permeability of the material is low.

The measurements of the permeability of rocks and unconsolidated materials may be made by either field or laboratory methods as described by Muskat (21) and by Wenzel (22). Laboratory determinations of the coefficient of permeability are made by measuring the discharge or the time rate of change of head, for the percolation of measured quantities of water through a known area and volume of soil sample. Devices used for this purpose are termed

permeameters and include a supply reservoir or tank from which water is discharged through a percolation cylinder under either a constant or a variable head.

The use of permeameters to determine the permeability of unconsolidated material is invalidated to a large degree because of the great errors introduced in repacking a disturbed sample. Inasmuch as the packing arrangement is a critical factor in determining the permeability of an incoherent material, it would seem advisable to apply laboratory methods only to consolidated materials or cores of unconsolidated material. Further caution should be exercised because the volume of material used in permeameters tests represents only an infinitesimal sample of a formation that is generally quite heterogenous. Accordingly, to be of value permeameter programs should include many samples collected at frequent depth intervals and at numerous locations within the area. Natural soils almost invariably occur in a stratified condition, with the result that the permeability of the soil varies from a maximum in the direction of the strata to a minimum perpendicular to the strata. Even compacted fill of selected material will have some stratification because of the inevitable variation in density and composition of material from layer to layer. Casagrande (23) suggests that the ratio k_{max} to k_{min} for compacted fills of selected material is

probably at least 9:1. The maximum and minimum permeabilities for a stratified soil may be computed by using the following formulae:

$$k_{\max} = \frac{k_1 d_1 + k_2 d_2 + \dots + k_n d_n}{d_1 + d_2 + \dots + d_n} \quad (51)$$

$$k_{\min} = \frac{d_1 + d_2 + \dots + d_n}{d_1/k_1 + d_2/k_2 + \dots + d_n/k_n} \quad (52)$$

where k and d are the corresponding coefficient of permeabilities and thicknesses of the strata or layers comprising the soil.

The intention of this project is rather to guide the reader towards the concept of in-situ permeability tests which is aimed at overcoming some of the inconsistencies resulting from laboratory experiments. There are a number of ways of determining the permeability by in-situ tests but a large number of these are too cumbersome or too costly to undertake in order to determine the properties of a material which is relatively close to the surface. These more elaborate field determinations are made by either the velocity or the potential method. In the velocity method one well is used for injection of salt, dye or an electrolyte. Two or more wells are used as observation stations to determine the time rate of travel of the injected substance through the water-bearing material. Fluorescein is generally used for the dye method and can be detected by eye or in more dilute form by a colourimeter. The chemical or salt method requires periodic sampling and analysis of water from each

observation well to determine the time of arrival of the salted solution. The electrolyte method requires periodic readings of the electric conductivity of the water in each observation well. Measurements of the water-table gradient, the distance between observation wells and the time of travel of the injected material provide the basis for determining the permeability of the material over the path of travel. In order to secure the results in a reasonable time the observations should be confined to relatively small areas.

Potential methods of determining permeability are based on measurements of the amount and rate of drawdown or recovery of water level in observation wells at different distances from the well that is either pumping or recovering from pumping, respectively. A distinct advantage of the potential method is its ability to sample large areas of the undisturbed aquifer within a limited time at a reasonable expense.

The permeability of coarse-grained soils is determined generally by Constant Head permeability tests and the permeability of fine-grained soils by the Falling Head Test. In the Constant Head Test the difference in head between the existing water level and a specified water level is maintained and the quantity of water introduced to maintain this level is measured. This method can be easily installed but requires a supply of readily available water to maintain the head difference especially if the in-situ material is very porous. Wilkinson (24) suggests an alternative method for determining

PERMEABILITY AND DRAINAGE CHARACTERISTICS OF SOILS

Coefficient of Permeability k in cm per sec (log scale).

	10 ²	10 ¹	1.0	10 ⁻¹	10 ⁻²	10 ⁻³	10 ⁻⁴	10 ⁻⁵	10 ⁻⁶	10 ⁻⁷	10 ⁻⁸	10 ⁻⁹
Drainage	Good						Poor		Practically Impervious			
Soil Types	Clean gravel	Clean sands, clean sand and gravel mixtures			Very fine sands, organic and inorganic silts, mixtures of sand silt and clay, glacial till, stratified clay deposits, etc.				"Impervious" soils, e.g., homogeneous clays below zone of weathering.			
		"Impervious" soils modified by effects of vegetation and weathering										
Direct determination of k	Direct testing of soil in its original position - pumping tests. Reliable if properly conducted. Considerable experience required											
	Constant-head permeameter. Little experience required											
Indirect determination of k				Falling-head permeameter. Reliable. Little experience required.			Falling-head permeameter. Unreliable. Much experience required.			Falling-head permeameter. Fairly reliable. Considerable experience necessary.		
	Computation from grain-size distribution. Applicable only to clean cohesionless sands and gravels										Computation base on results of consolidation tests. Reliable. Considerable experience required	

the in-situ permeability by the use of piezometers. The Piezometers can be installed at different levels in a strata in order to obtain results for various formations. The method of analysis of results suggested is the Gibson Solution (1963). Unfortunately this method is costly and will not be dealt with here. Perhaps the easiest and most economical method of testing the material to determine the permeability is the Falling Head Test. In this method the tube is inserted into the hole, the hole plugged at approximately the water table. Water is inserted into the tube and the height drop of the top of the water in the tube is related to time from whence the permeability can be determined. Although the first two methods will be discussed in this project the last mentioned test together with the sieve analysis method is recommended for quick results and are easily determined by the normal site staff. The last method, namely the determination of the permeability of sands on the basis of grain size, makes use of the Hagen's formula. The standard sieves can be used for this test and are usually normal equipment in any site laboratory.

The last stage of the sequence leading to the drainage of the particular strata concerned or the lowering of the existing water table in road excavations is to design an effective sub-surface drainage system. There are various types of pipes which can be used for sub-surface drainage such as concrete, asbestos cement, PVC and corrugated metal pipes. The design criteria suggested here should apply to all types of under-drainage. The controlling factor regarding the effectiveness of the pipes themselves depend on the design of the

filter material which surrounds the horizontal sub-surface pipe. Davis and Sorenson (19) tend to adopt the Donnan spacing formula (No. 44), refer to Chapter 20, whilst valuable theoretical and laboratory work has been done by Kirkham (25). When designing the drainage system it should be borne in mind that although the main aim is to lower the water table below say the road layer cognizance should be taken of the economics involved in sub-surface installation as also the effect of lowering the water table on the indigenous vegetation.

PERMEABILITY

The vertical percolation of groundwater through capillary interstices results in the build-up of a hydraulic gradient with consequent lateral percolation of water through interconnecting interstices. The capacity of a formation for transmitting water is measured by its coefficient of permeability, which is defined as the flow per unit time across a unit cross-sectional area.

The permeability of granular material varies with the diameter and degree of assortment of the individual particles. A well-sorted gravel has a much higher permeability than a well-sorted coarse sand. However, gravel with a moderate percentage of medium and fine grained material must be considerably less permeable than a uniformly sized coarse sand. In graded material, the particles of moderate size fill the pore spaces between the larger particles, thus forming a compactly knit and impervious mass such as is obtained in good concrete. A material is said to be permeable therefore, if it contains continuous voids. Since such voids are contained in all soils including the stiffest clays, and in all non-metallic construction materials including sound granite and neat cement, all these materials are permeable. Although the size of the sample does not affect the permeability of the soil only the field tests would provide more realistic results because no sample submitted for laboratory tests would remain undisturbed.

As water percolates through a permeable material the individual water particles move along paths which deviate erratically, but only slightly, from smooth curves known as flow lines. If adjacent flow lines are straight and parallel, the flow is said to be linear. As the flow of a fluid is affected by the temperature of the fluid it is necessary to control the coefficient of permeability at some specified temperature. The temperature chosen is 20°C by most of the authors on the subject. A standard coefficient of permeability k_s therefore relates to this temperature and hence as viscosity plays an important part in laminar flow permeability at temperatures other than 20°C vary inversely as the respective kinematic viscosities.

$$k_s = k_{20} \frac{n_{20}}{n_t} \quad (53)$$

If however the field temperature is reasonably close to the standard temperature the discrepancy in the coefficient of permeability may be ignored. Refer to Figure 49.

The hydraulic principles involved in linear flow are illustrated by Figure 49. In this figure, the points a and b represent the extremities of a flow line. At each extremity a standpipe, known as a piezometer tube, has been installed to indicate the level to which the water rises at these points. The water level in the tube at b and the vertical distance between this level to point b is the piezometric head at b. The vertical distance between a and b represents the position head ΔH . If the water in

the hydraulic system stands at the same elevation in the piezometric tubes at a and b, the system is in a state of rest, regardless of the magnitude of the position head. Flow can only occur if the piezometric levels at a and b differ by a distance h known as the hydraulic head at a with respect to b and is termed the difference in piezometric level between a and b.

In Figure 49 a_1 and b_1 represent any two points at the same elevation in the piezometric tubes rising from a and b respectively. Since the unit of weight of the water is γ (grams per cubic centimetre), the hydrostatic pressure at a_1 exceeds that at b_1 by the amount of γh . The difference γh between the hydrostatic pressure at two points located at the same elevation is referred to as excess hydrostatic pressure. It is this pressure that drives the water through the soil between a and b.

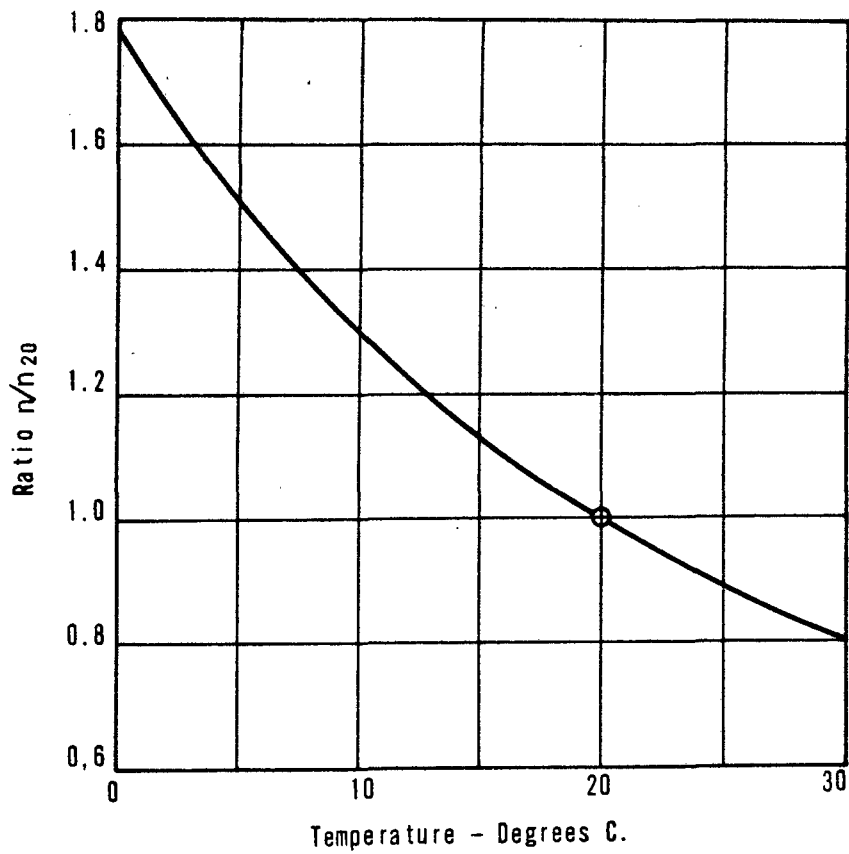
The ratio

$$i_p = \frac{\gamma h}{l} = \frac{u}{l} \quad (54)$$

in which u is the excess hydrostatic pressure, represents the pressure gradient (grams per cubic centimetre) from a to b. The ratio

$$i = \frac{i_p}{\gamma} = \frac{1}{\gamma} \frac{u}{l} = \frac{h}{l} \quad (55)$$

is known as the hydraulic gradient which is a pure number.



RELATION BETWEEN TEMPERATURE AND VISCOSITY OF WATER.

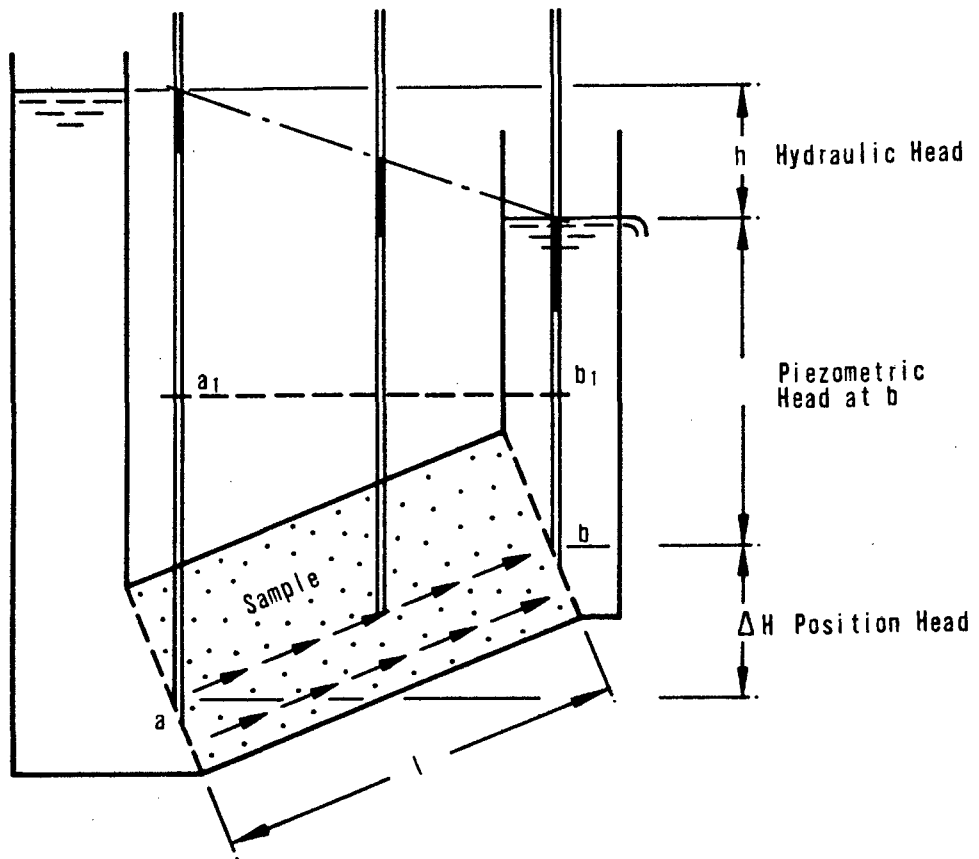


DIAGRAM ILLUSTRATING MEANING OF HYDRAULIC HEAD ASSOCIATED WITH LINEAR FLOW OF WATER THROUGH SOIL SAMPLE.

The discharge velocity V is defined as the quantity of water that percolates in a unit time across a unit area of a section orientated at right angles to the flow lines. In a statistically isotropic porous material the porosity of a plane section is equal to the volume porosity n . Hence the average velocity at which the water percolates through the voids of the material is equal to the discharge velocity divided by the porosity. The average velocity represents the seepage velocity. If the term velocity is used without qualification in connection with permeability, it always indicates the discharge and not the seepage velocity.

If water percolates through fine saturated sand or other fine-grained completely saturated soils without affecting the soil structure, the discharge velocity is almost exactly determined by the equation:

$$V = \frac{k}{n} i_p \quad (56)$$

in which n (gram-second per square centimetre) is the viscosity of the water and K is an empirical constant referred to as the permeability. The viscosity of water decreases with increasing temperature as discussed above, refer to equation (53) and Figure 49. The value of K (square centimetres) is a constant for any permeable material with given porosity characteristics and it is independent of the physical properties of the percolating liquid. From equation (55) and (56) we obtain for the discharge velocity

$$V = \frac{K}{n} \gamma i \quad (57)$$

Seepage problems encountered in civil engineering deal almost exclusively with the flow of groundwater at moderate depths below the surface. As the temperature of the percolating water varies so little the unit weight is practically constant. In addition, the viscosity varies within fairly narrow limits. Therefore, it is customary to substitute in equation (54)

$$k = K \frac{\gamma}{n} \quad (58)$$

whence

$$V = ki \quad (59)$$

This therefore is the coefficient of permeability and is commonly known as Darcy's Law.

The channels through which the water particles travel in a mass of soil have a variable and irregular cross-section. As a consequence, the real velocity of flow is extremely variable. However, the average rate of flow through such channels is governed by the same laws that determine the rate of flow through straight capillary tubes having a uniform cross-section. If the cross-section of the tube is circular, the velocity of the flow increases, according to Poiseuille's law, with the square of the diameter of the tube. Since the average diameter of the voids in soil at a given porosity increases practically in proportion to the grain size D , it is possible to express k on the basis of Poiseuille's law as

$$k = \text{constant} \times D^2 \quad (60)$$

From his experiments with loose filter sands of high uniformity (uniformity coefficient not greater than about 2) Hazen obtained the empirical equation

$$k = CD^2 \quad (61)$$

in which D is the effective size in centimetres, and C varies from about 100 to 150. It should be noted that equation (61) is applicable only to fairly uniform sands in a loose state.

Appendix B

SABS 045-1974

SOUTH AFRICAN BUREAU OF STANDARDS

CODE OF PRACTICE

for

TESTING WATER BOREHOLES

The contents of this Appendix has been extracted from SABS 045-1974. This extract forms part of an explanation necessary for this project and is used for academic purposes only.

PREFACE

The original code of practice was prepared in 1952 in response to the general need for standard methods of testing the yield of water boreholes and of determining their straightness and verticality.

In preparing this revision it has been the object of the responsible committee to formulate practical methods, based on scientific considerations, of testing the yield of boreholes, determining their deviations from the vertical, and checking straightness. The committee has borne in mind that such test work must frequently be done in remote localities, and that the nature of general water-boring operations precludes the use of cumbersome and expensive apparatus. For this reason the committee has not tried to include in the code of practice all test methods and all available apparatuses. The use of other methods is however not excluded, provided the results obtained from these are not less accurate than those obtainable with the methods and apparatuses described in this code.

As the value of a borehole depends on its straightness and verticality, the minimum requirements for these two factors have been included. Disputes in connection with boreholes arise most frequently over these two factors, and usually as a result of the fact that the limits of deviation from straight and vertical were not specified.

Since many of the existing test methods give erroneous results, the committee has considered it imperative to formulate a standard test method and to prescribe a procedure to be adopted in determining the yield of a borehole. Over-estimation of the yield of a borehole leads to the installation of pumping equipment of excessive capacity which pumps a hole dry in a matter of hours, with possible detrimental results.

It should be emphasized that the estimate of yield obtained by testing as prescribed in this code of practice holds good only under the conditions prevailing at the time of the test.

Should information regarding the quantity of ground-water available from a particular borehole or boreholes over an indefinite period be desired, pumping tests and water-level observations more elaborate than those laid down in the code may be required. Such tests and observations are considered essential when supplies of ground-water are to be developed by municipalities and local authorities.

Other large users may also find it advantageous to establish more accurately the supplies available to them. The decision as to whether such elaborate tests and observations should be made depends on the capital expenditure to be incurred in developing and exploiting the supply.

The pumping rate of 60% of the tested (or immediate) yield recommended in Section 5 of the code must be viewed as an arbitrary safety factor which will not necessarily ensure that, on the one hand, the supply is not being exhausted and, on the other, that it is being fully developed.

The quantity of ground-water continually available from boreholes in a particular ground-water compartment is primarily dependent on recharge by direct or indirect infiltration of rainfall. The storage capacity of the formation is of critical importance when ground-water is stored in joints and fractures, i.e. in aquifers of very limited storage capacity which may be rapidly depleted. In such cases the storage capacity determines the quantity of water available for extraction during periods in which no recharge takes place. Furthermore, the immediate yields of boreholes decrease as the stored ground-water is being depleted. The magnitude of this decrease is therefore also of importance.

An investigation for estimating the available ground-water supply usually entails water-level measurements in, and pumping tests on, more than one borehole. Although the manner of testing is that described in the code, it is not possible to lay down hard and fast rules on aspects such as the number of boreholes to be pumped simultaneously and those in which water-level measurements should be made. Local conditions have to be taken into account and for this reason it is in most cases desirable that an experienced ground-water hydrologist should visit the area to make a detailed survey and control the pumping test(s).

The value of long-term records of quantities pumped and water-level fluctuations cannot be over-estimated. Municipalities and large users of ground-water are urged, in their own interest, to keep such records.

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1. SCOPE

1.1 This code of practice covers

- a) the straightness and verticality of boreholes, and
- b) the determination of the yield from a borehole by means of a direct pumping test.

NOTE

- a) Apparatus for the measuring of water-levels is described in Annexure A.
- b) Apparatus for the measuring of discharge is described in Annexure B.
- c) The determination of the deviation of a borehole from the vertical is described in Annexure C.
- d) A copy of the test certificate to be used is given in Annexure D.

2. GENERAL

2.1 The drawing off of water through a borehole results in a lowering of the water-level. This "draw-down" establishes a hydraulic gradient in the water-bearing material resulting in an underground flow into the borehole.

The rate of inflow depends upon the hydraulic gradient, the permeability and saturated thickness of the water-bearing material, and the features of borehole construction. The rate of inflow ranges from rapid to exceedingly slow.

- 2.2 In order to obtain the maximum inflow of ground-water into the borehole, the water-level should be drawn down below the deepest aquifer (water-bearing fissure) in the hole. As this depth is generally not known, the water-level should be drawn down as near as is practically impossible to the bottom of the borehole. The pumping rate required to do this should be determined by measurement. It represents the rate of supply (influx) under existing conditions.
- 2.3 In order to determine whether a particular borehole can be utilized economically and practically, readings are taken of the water-level for a series of constant discharges and the period of time required for the water-level to revert to the original static conditions is noted in each case. These results are tabulated and used as an aid to determine the most economical and practical equipment.
- 2.4 Where information is required regarding the drainage of water-feeding properties and the specific supply or storage coefficient and the potential of a water carrier or ground-water compartment (as in water supply to towns and irrigation schemes), it is necessary to conduct special supply tests not covered in this code.

3. STRAIGHTNESS AND VERTICALITY OF BOREHOLES
- 3.1 STRAIGHTNESS. Boreholes should be sufficiently straight to permit a straight pipe of length at least 7 m and having an external diameter that is within the relevant range given in Table 15 to be lowered to the full depth of the borehole.

Table 15EXTERNAL DIAMETER OF PIPE

1	2	3
Diameter of borehole, mm	External diameter of pipe, mm	
	min.	max.
100	88,1	89,5
150	138,7	140,6
200	164,1	166,1
250	215	220
300	270	275

- 3.2 VERTICALITY. The centre of the borehole should not, at any depth, deviate from the vertical through the centre of the hole at the top by more than 0,3% of the distance from the top of the borehole to the depth tested, i.e. the maximum permissible deviation is 3 mm per 1 m of depth.

If the recommendations for verticality are met, and the entire depth of the borehole conforms to the recommendations for straightness given in 3.1, the borehole should be regarded as being acceptably vertical and acceptably straight.

The exact determination of the extent to which a borehole deviates from the vertical requires careful measurement and, in order to eliminate as far as possible the factors that may adversely affect the test results, it is recommended that the apparatus and method described in Annexure C should be used to determine the deviation(s). Other apparatus and methods may be used provided that the results obtained are not less accurate than those obtainable with the apparatus and method described.

4. TESTING THE YIELD OF BOREHOLES

4.1 GENERAL

- 4.1.1 In order to establish the conditions of equilibrium described in 2.2 it is necessary for the capacity of the test pump to exceed the maximum yield of the borehole.
- 4.1.2 The intake should be installed as near to the bottom of the borehole as is practicable, but not nearer than 1,5 m from the bottom. The depth of the intake below the top of the borehole should be recorded¹.

- 4.1.3 In order to determine the amount of recovery, the depth of the static water-level below the top of the borehole should be determined both before and after the test pumping, and the time that has elapsed between the completion of the test pumping and the second determination of water-level should be noted (see 4.4(g)).

NOTE: Apparatuses suitable for measuring the depth of the water-level and the quantity delivered by the test pump are described in Annexures A and B.

- 4.1.4 To avoid flow or seepage back into the hole, the delivery of the test pump should be discharged at an adequate distance from the borehole, and the joints in the piping of the test pump should be such as to prevent leakage.

- 4.2 ACCURACY OF MEASUREMENTS. It is recommended that

- a) the depths of the static levels of the water should be measured to an accuracy of within 100 mm;
- b) where indirect measuring methods are used, the quantity discharged by the test pump should be determined with the flow of water at the measuring point steady and to an accuracy of within 5%; and

- c) in order to prevent the recording of misleading results, the arithmetic mean should be calculated from three successive measurements (taken at short intervals) that do not differ from one another by more than 10%.

4.3 RÉSUMÉ OF TEST. The test consists of the following two stages:

- a) The removal of stored ground-water in the immediate vicinity of the borehole; and
- b) the determination of the maximum yield which the borehole can, at the time of the test, maintain continuously for the duration of the test.

4.4 TEST PROCEDURE. With the test pump and the apparatus for measuring water-level installed, proceed as follows:

- a) Measure the depth of the static water-level from the top of the borehole (it can be assumed that static conditions have been reached as soon as the difference in level is less than 100 mm in 3 h).

NOTE: At least 12 h should be allowed to elapse between pumping and measurement of level.

- b) Commence pumping with the test pump delivering at maximum rate, and maintain this rate until the water-level falls below the intake of the pump and air is drawn in. Then adjust the rate of pumping till steady conditions are reached and a small amount of air is drawn in.

NOTE: When a test pump that may be adversely affected when air is drawn in is used (e.g. a centrifugal or a turbine pump), the water-level in the borehole during the test should be maintained at approximately 300 mm above the pump intake.

- c) If any fluctuation in the water-level occurs during pumping, adjust the pumping rate until the water-level remains constant. (The rate of discharge may then steady to a fairly constant value. Alternatively it may fluctuate or decrease steadily during the test period.)
- d) Maintain the conditions described in (c) above and measure and record the discharge at hourly intervals (or more frequently if necessary in order to ascertain the rate of discharge).
- e) Provided that the rate of discharge did not change by more than 5% over the last hour of the test period, consider the average rate obtained during that period as the maximum yield of the borehole.

NOTE: For private boreholes the test period should be at least 6 h and for boreholes to be used by public bodies for domestic supplies it should be at least 72 h.

- f) For the determination of the test period in the event of accidental or unavoidable stoppage during the test consider, with the relevant period before the stoppage, only that part of the resumed test after which the yield and the water-level of the borehole have reached the values achieved immediately before the stoppage.

- g) On completion of the test, measure the static level (from the top of the borehole) to which the water returns and record the time that has elapsed between the completion of the test and the recording of this level. (During this period frequent readings should be taken.)

Consider the water-level to have reached its static level when three successive readings, taken at intervals of 3 h do not differ from one another by more than 100 mm.

NOTE: The static water-level cannot be determined if boreholes in the vicinity are being pumped and this pumping is affecting the water-level of the borehole under test.

5. DETERMINATION OF PUMPING RATE

- 5.1 Because the yield is influenced by a number of factors (e.g. geological formation, rainfall, neighbouring boreholes), the pumping rate should in general not exceed 60% of the yield determined by testing. It is strongly recommended, however, that geological advice be obtained on the percentage to be adopted for each location.

ANNEXURE A. APPARATUS FOR THE MEASURING OF WATER-LEVELS

A-1 ELECTRICAL APPARATUS

A-1.1 Double-Pole Unit. The apparatus consists of the following, assembled as shown in Figure 50:

- a) Double-pole contact. A double-pole contact so weighted as to ensure that the twin electric cable on which it is suspended remains taut (refer to Figure 50)
- b) A battery capable of supplying a voltage of at least 6 V.
- c) A reel with a length of twin electric cable wound on the drum
- d) An electric indicating instrument.
An instrument such as a high-resistance voltmeter with a range, the upper limit of which is at least equal to the voltage of the battery, or a milliam-meter.

Each conductor at one end of the cable is connected (preferably permanently) to a pole of the contact, and the other end of the cable is permanently connected, through one flange of the reel, to the indicating instrument and battery.

As the cable is lowered into the borehole the circuit will be completed when the two electrodes in the contact strike the water, and the indicating instrument will register. The reading on the instrument is of no importance as the instrument only serves to indicate when the contact reaches the water-level. The depth of the water-level is obtained by measuring the length of the twin cable from the electrodes to the point on the cable opposite the top of the borehole.

A-1.2 Single-Pole Unit. This apparatus is similar to the one described in A-1.1 except that the contact housing is fitted with one electrode only, a single-conductor cable is used, and the battery and indicating instrument are connected, in series, to the borehole casing (provided that it extends from the top of the borehole to below the water-level to be measured) or to the rising main.

Upon contact with the water, the electrical circuit is completed through the casing or through the rising main, as relevant.

A-2 AIR PRESSURE LINE APPARATUS. This apparatus consists of

- a) a tube positioned between the rising main and the casing or borehole side, and extending to below the water-level to be measured;

- b) a suitable calibrated pressure gauge graduated either in metres of water or in kPa; and
- c) an air pump and non-return air valve.

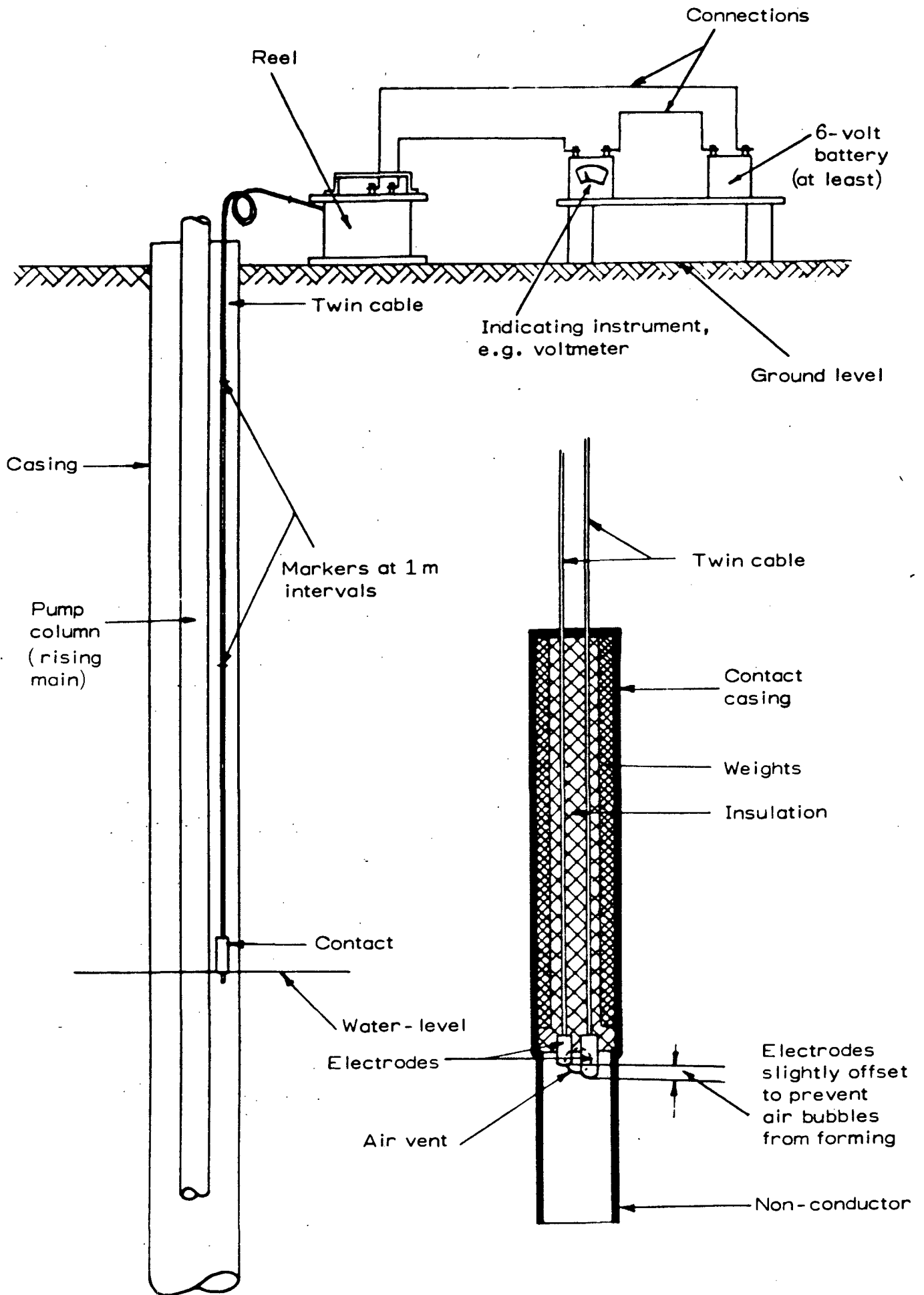
The installation is as shown in Figure 51.

When the water-level, L, is to be determined, air is pumped into the air line until the maximum dial reading is obtained. The entire air line will then be filled with air, and the pressure reading will correspond to the head, A.

If the gauge is graduated in kPa, the reading must be divided by 9,8 to give the submersion in metres.

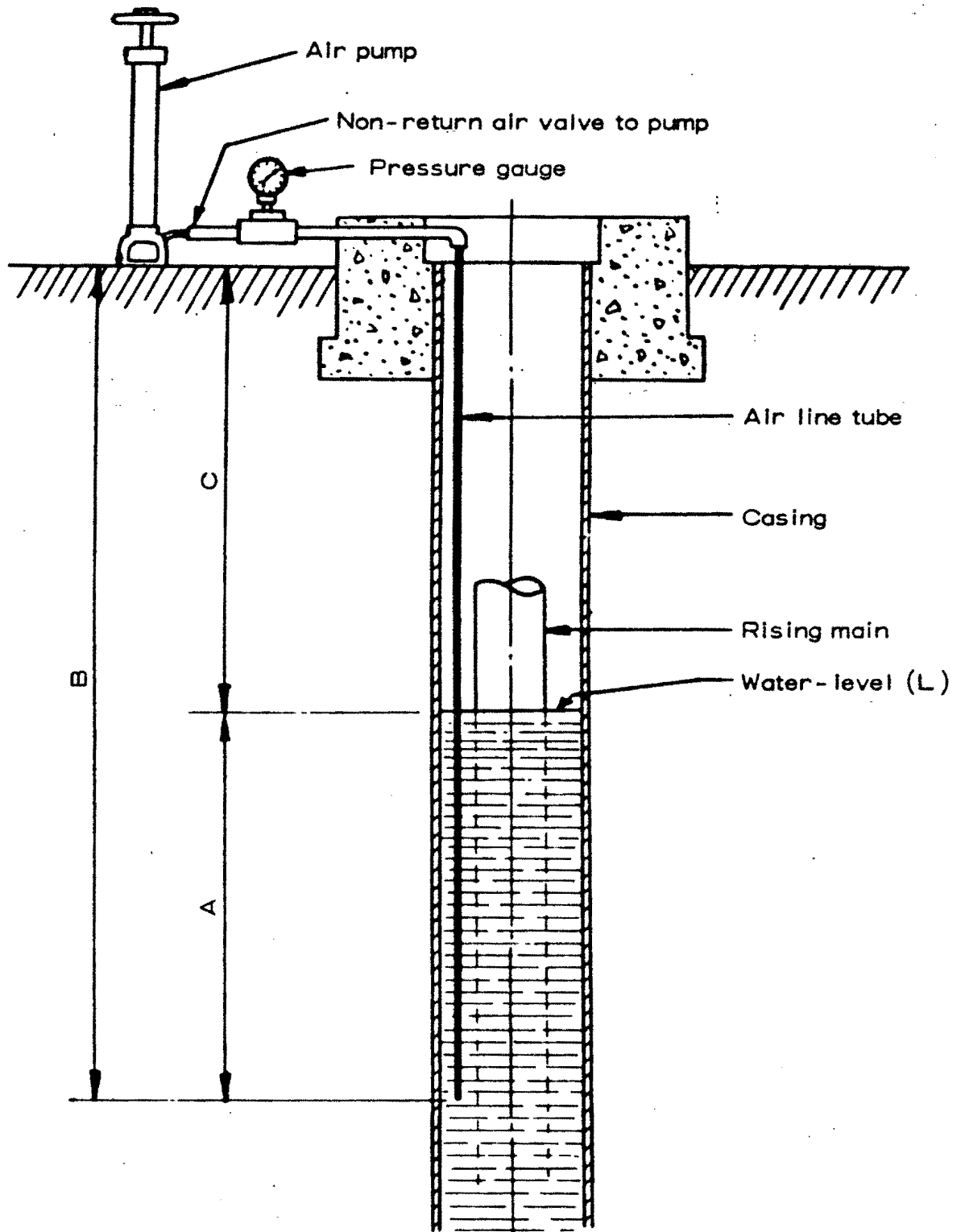
At the time of installation the distance, B, is recorded. The required depth of water-level, C, is therefore B-A.

- A-3 A WEIGHTED STEEL TAPE (preferably with a backing, for a suitable distance at the weighted end, that indicates clearly the extent of submersion of that end)
- A-4 A SUSPENDED FLOAT (suspended on a metal wire)



Enlarged view of contact

Electrical Apparatus with Double-Pole Contact for Determining the Water-Level in a Borehole



Air Pressure Line Apparatus for Determining the Water-Level in a Borehole

ANNEXURE B. APPARATUS FOR THE MEASURING OF DISCHARGE

B-1 GENERAL.

A variety of apparatuses and methods may be used for the measuring of discharge of a test pump. The two types of apparatus described in this annexure have been chosen because they are in general the ones least affected by the frequent handling and transportation associated with the testing of boreholes.

Of the two types of apparatus it has been found that weirs that are calibrated for direct reading, if they are properly constructed (especially as far as upstream conditions are concerned), are more convenient, although they are not necessarily more accurate. The use of weirs is however not recommended for capacities of less than 900 l/h in the case of a 28° Vee notch and 2 000 l/h in the case of a 90° Vee notch.

B-2 WEIRS.

Rectangular or triangular weirs may be used for determining the discharge from the test pump. These weirs should be calibrated (by a recognized testing authority) to give direct measurements of the discharge in litres per hour or litres per minute or litres per second. The certificate of calibration for each weir used should be made available for inspection to the owner of the borehole by the driller or by the person in charge of the test.

The weir should be properly constructed, sharp crested with smooth crest walls and free overfall, and should be set at right angles to the flow. The crest should be formed of metal and should, at the time of test, be smooth and free from rust and grease.

The channel of approach should be straight and of uniform cross-section, and its side walls should be smooth and parallel.

Care should be taken to ensure that, in the channel of approach, the flow is smooth and free from eddies, surface disturbance and, as far as practicable, air bubbles.

A trap or other means of preventing sand and other foreign matter (which may affect the accuracy of the measurement) from entering the channel of approach should be incorporated in the apparatus.

B-3 VOLUMETRIC APPARATUS

B-3.1 Apparatus. The apparatus consists of

- a) Container. A container of known capacity, suitably graduated in litres.
- b) Stop-watch
- c) Means of quickly diverting discharge into and away from the container

B-3.2 Method. The method of determining the discharge is as follows:

The time taken, in seconds, to deliver a quantity of water into the container is measured by means of the stop-watch. The actual quantity of water is then determined from the calibrations on the container. Thus, if q litres of water are delivered into the container in \underline{t} seconds, the rate of discharge of the pump

$$\begin{aligned}
 &= \frac{q}{\underline{t}} \text{ l/s, or} \\
 &= q \times \frac{60}{\underline{t}} \text{ l/min, or} \\
 &= q \times \frac{3\ 600}{\underline{t}} \text{ l/h} \qquad (62)
 \end{aligned}$$

The accuracy with which the discharge is determined depends upon the accuracy of the calibration of the container and of the time and quantity measurements. If the time is short or the quantity is small (or both), relatively small errors in their measurements may give rise to considerable inaccuracies in the calculated discharge rates, and it is therefore recommended that the capacity of the containers used should not be less than that shown in Table 16.

Table 16
CONTAINER CAPACITIES

1	2	3
Discharge		Minimum capacity of container, <i>l</i>
<i>l/s</i>	<i>kℓ/h</i>	
Up to 1	Up to 3,6	100
Over 1 up to 2	Over 3,6 up to 7,2	150
Over 2 up to 5	Over 7,2 up to 18	300
Over 5 up to 10	Over 18 up to 36	600
Above 10	Above 36	3 000

ANNEXURE C. DETERMINATION OF THE DEVIATION OF A BOREHOLE
FROM THE VERTICAL

WARNING :

The determination of deviation from the vertical by means of the plumb-bob as described below is restricted to depths up to which the plumb-bob cable does not touch the side of the borehole or casing at the top or at any other point lower down. When the plumb-bob method becomes ineffective, deviation from the vertical may be determined by means of a recording drift indicator. Several types and makes of such indicators are available.

C-1 APPARATUS. The apparatus consists of

- a) Stand. A rigidly constructed stand.
- b) Pulley. A single-sheave pulley
- c) Screen. A screen or other adequate means of protecting the plumb-line from the wind.
- d) Plumb-bob. Refer to Figure 52.
- e) Tripod. Refer to Figures 53, 54 and 55.
- f) Cable. A length of high tensile steel cable of diameter 3-6 mm.
- g) Compass

The apparatus is assembled as shown in Figure 56. The single-sheave pulley is attached to the stand with its centre at least 4,6 m above the top of plate A.

The tripod, F, is placed in position over the top of the casing and the top plate, A, is levelled by means of the levelling screws, G, on the legs, H, with the assistance of the spirit level, B. The plate, D, (which is graduated in degrees) is oriented with a compass placed at least 6 m from the tripod. The set screws, E, are adjusted until the centre of plate A, as indicated by the scale, C, coincides with the centre of the borehole. Plate A, which has an arrow mark in line with scale C, can then be taken off and put on without further adjustment.

The distance between the centre of the pulley and the top of plate A should be accurately determined.

A plumb-line is suspended over the pulley, and the pulley is adjusted laterally until the plumb-bob hangs immediately above the centre of the borehole. The plumb-line is protected from any wind effect.

C-2 METHOD:

The plumb-bob (Figure 52) is lowered into the borehole and the top plate A is rotated in the tripod until the plumb-line hangs freely through slot J. For each of a series of depths of the plumb-bob the deflection of the plumb-line from the centre of the plate and the bearing of the deviation from a horizontal axis of the borehole, e.g. East-West or North-South axis, are measured.

C-3 CALCULATION:

Total deviation from the vertical at each depth at which the deflection is measured is

$$\frac{A(B+C)}{C} \text{ metres} \quad (63)$$

where A = deflection from centre of plate, m
 B = depth of top of plumb-bob below top of plate A, m
 C = suspension height, i.e. height from pulley centre to top of plate A, m (refer to Figure 56).

The bearing of the deflection is measured directly on the plate for each depth.

Example

Suspension height	5,0 m
Depth of plumb-bob	25,0 m
Deflection from centre of plate	= 0,012 m
Bearing of deflection	= 87°

Therefore

total deviation from the vertical at 25 m depth

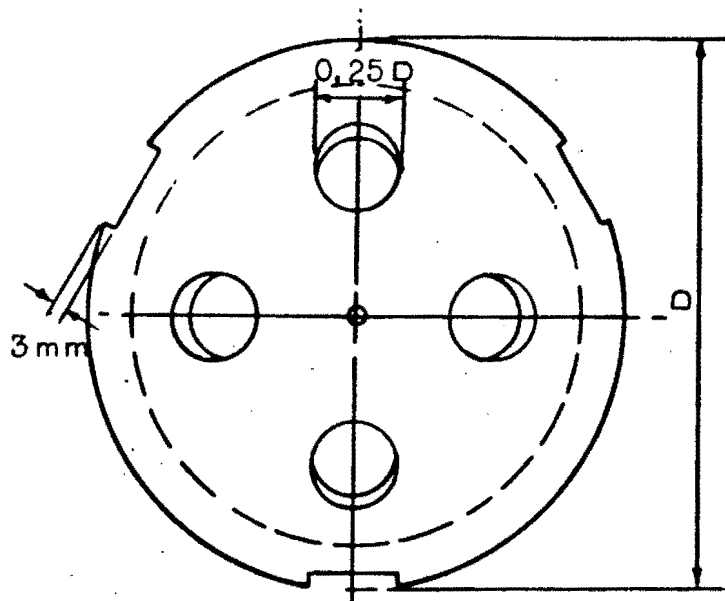
$$= 0,012 \left(\frac{25+5}{5} \right) \text{ m}$$

$$= 0,012 \times 6 \text{ m}$$

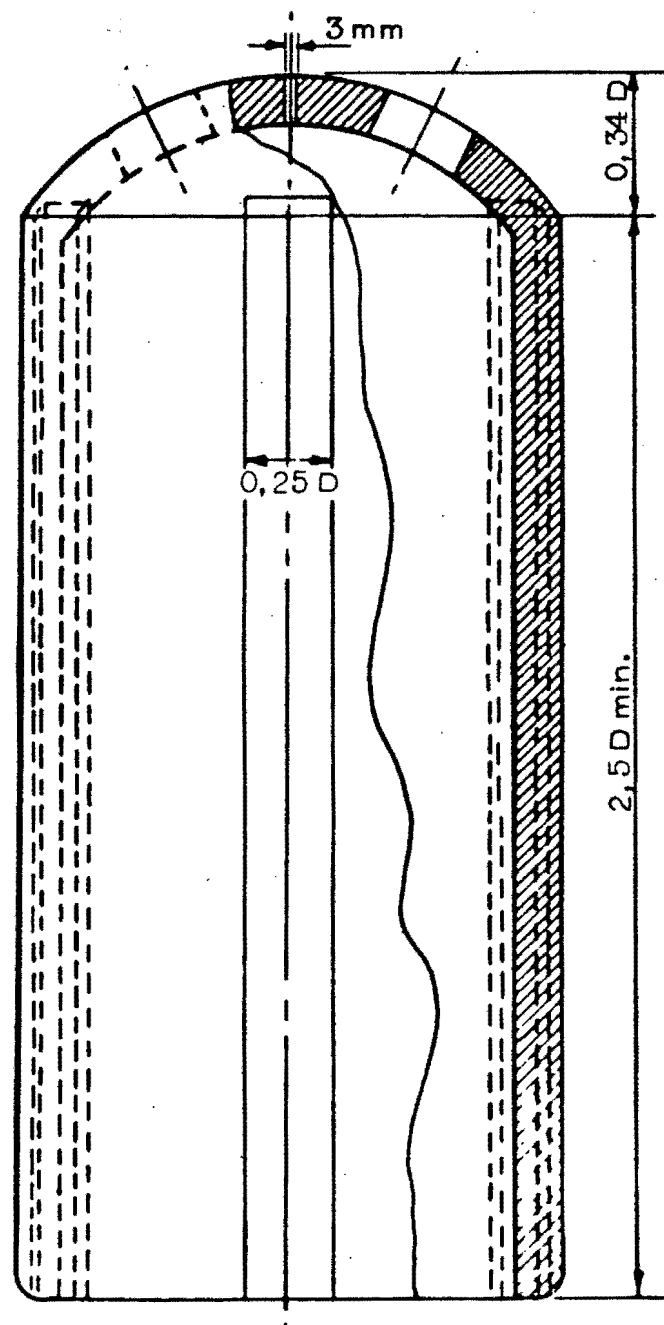
$$= 0,072 \text{ m}$$

$$= 72 \text{ mm}$$

Thus total deviation from the vertical at 25 m depth is 72 mm in a direction 3° North from East.



D = Diameter of borehole or casing less 15 mm

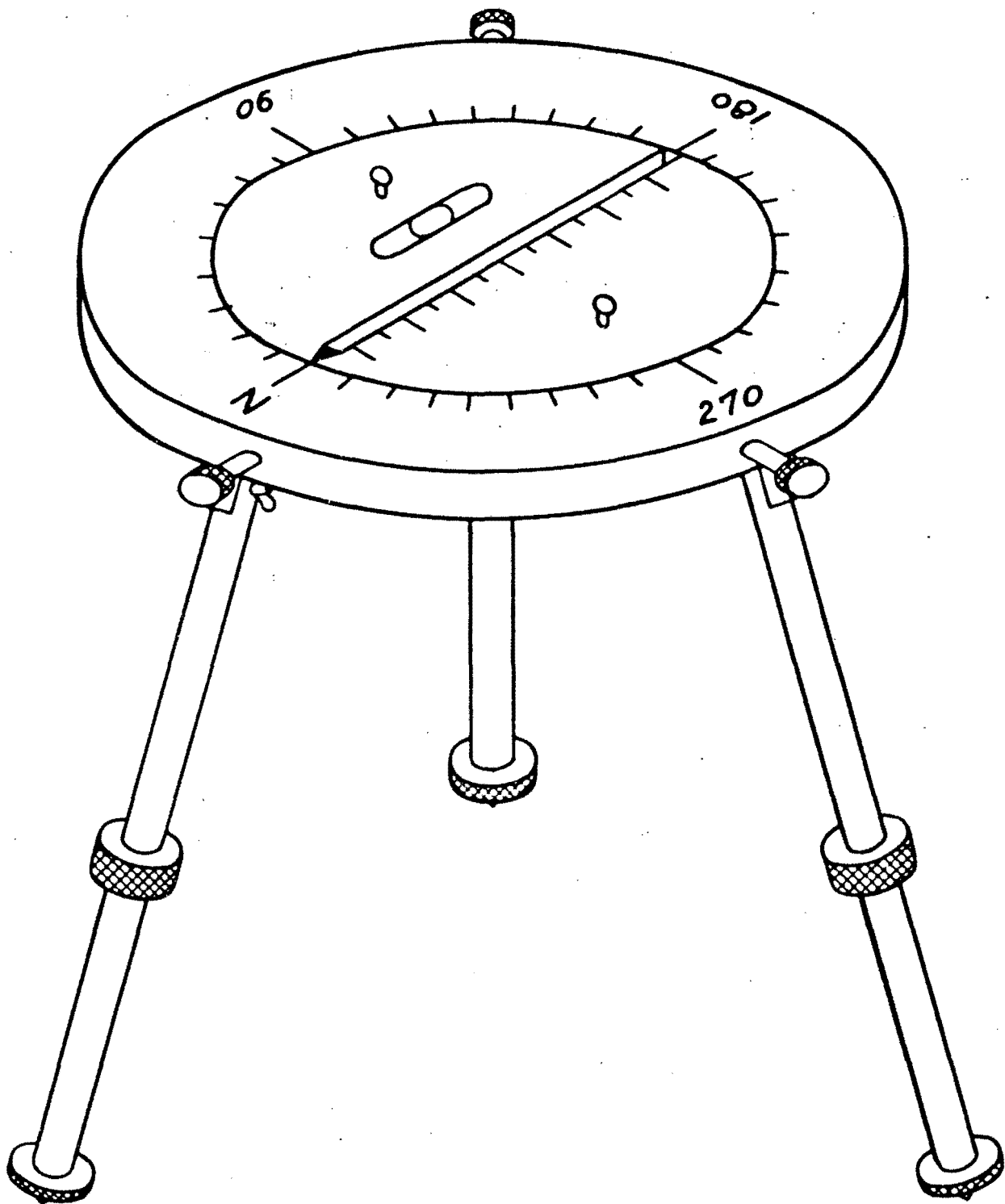


Minimum mass = 12 kg

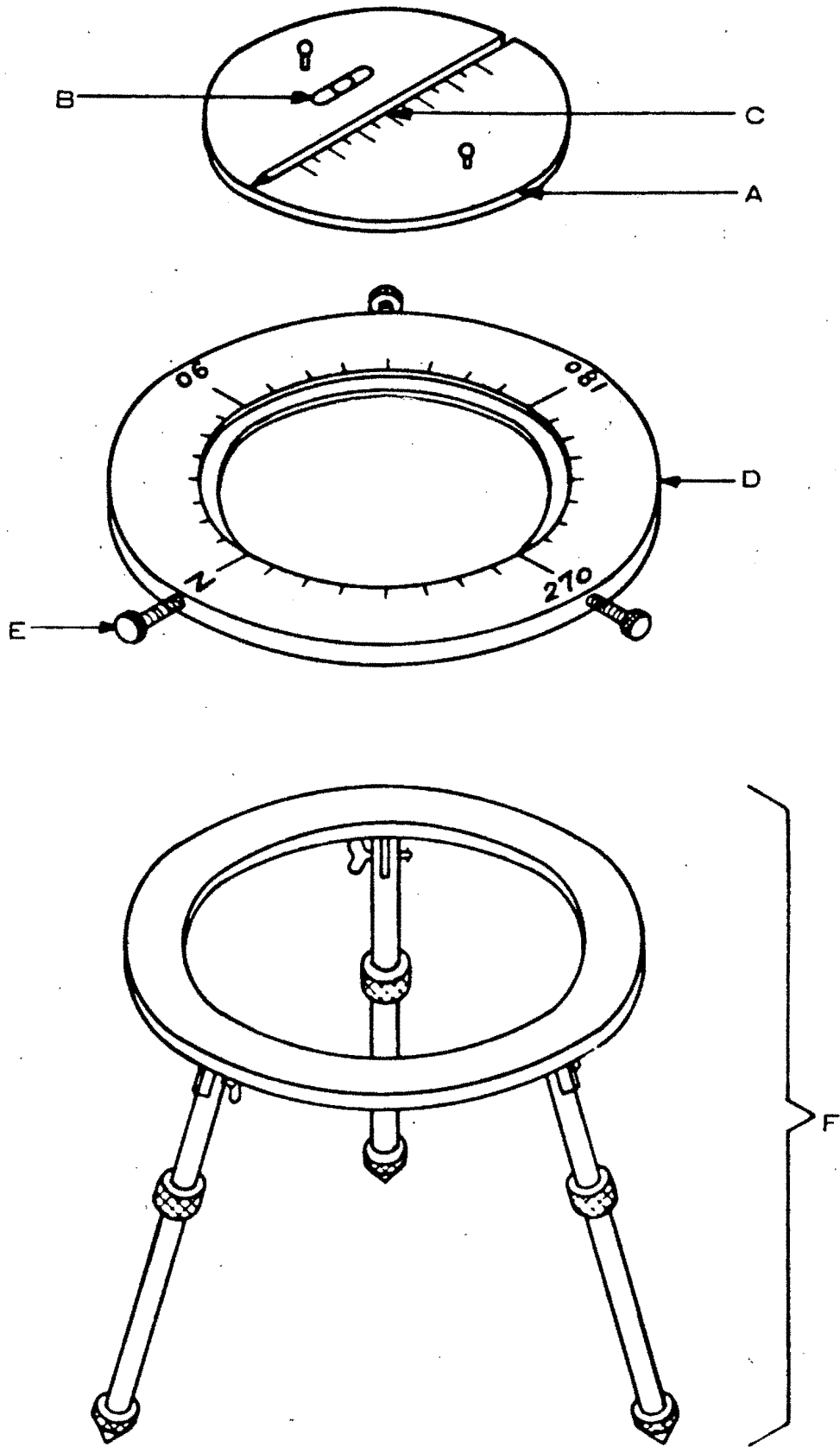
Material: Cast iron

Plumb-Bob

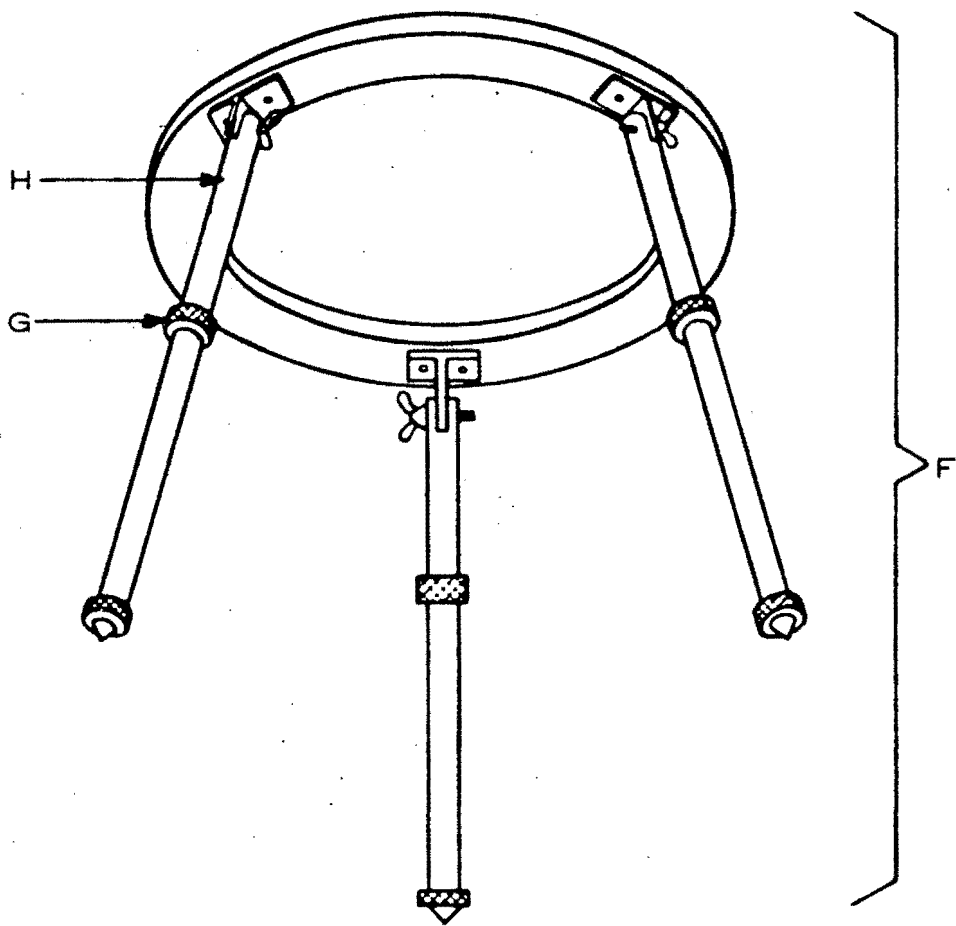
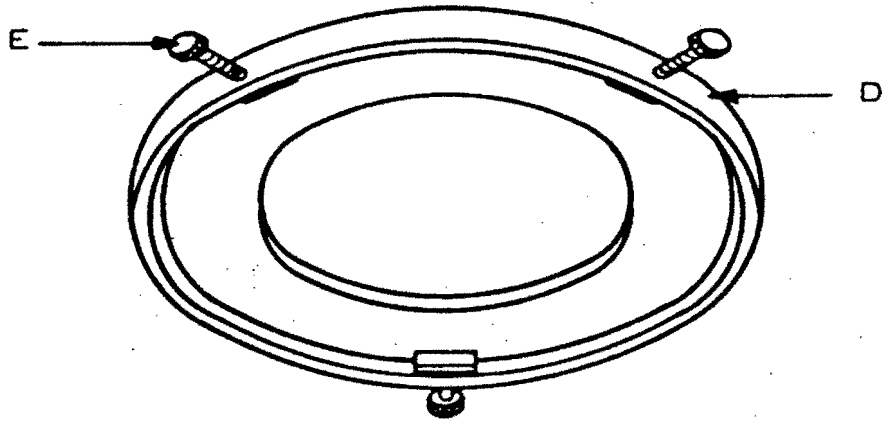
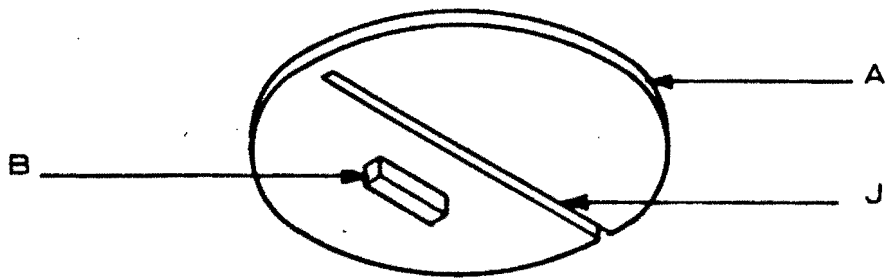
NOTE: In the case of boreholes of varying sections the diameter of the borehole on casing shall be taken as the smallest nominal diameter



Tripod Assembly

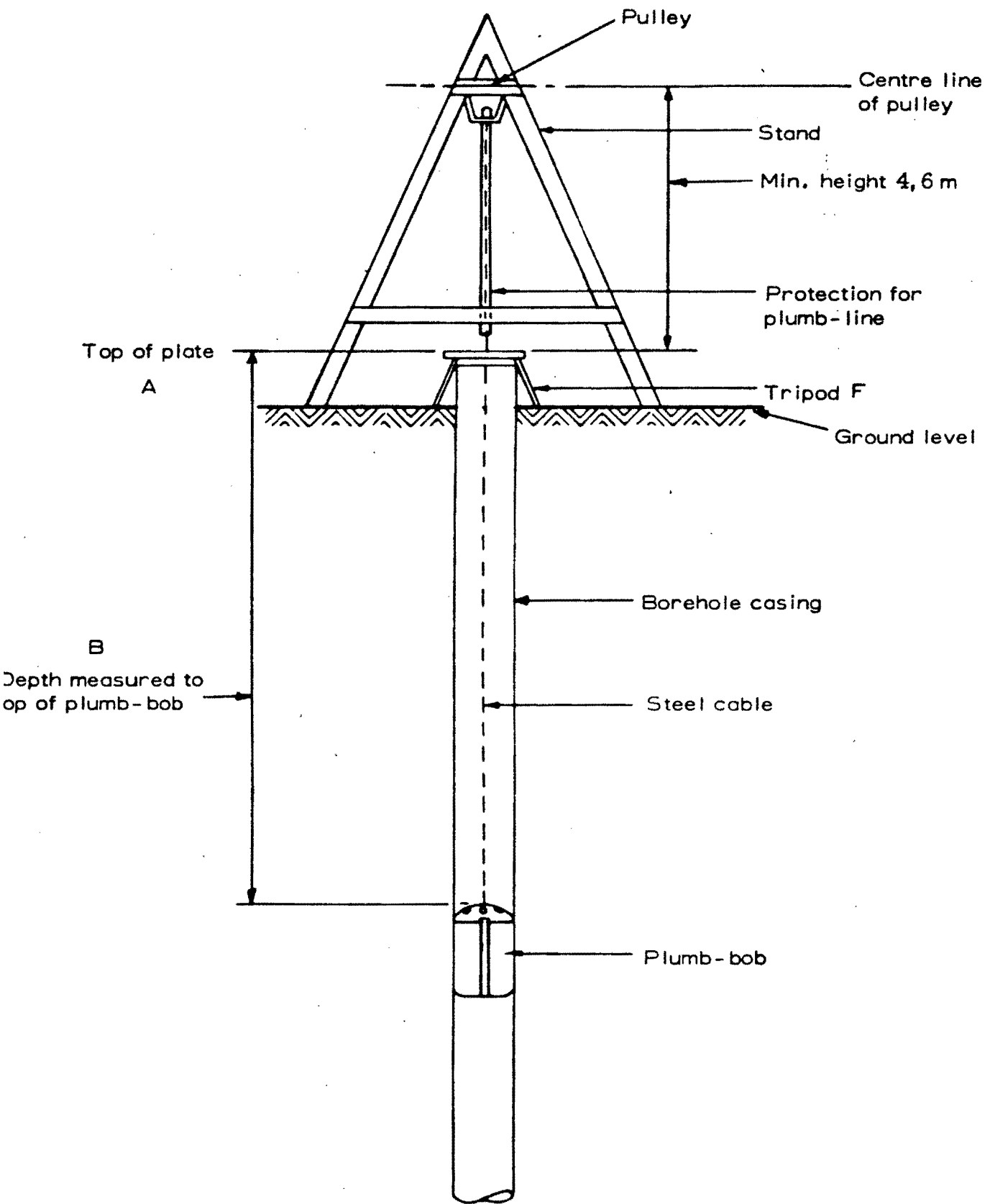


Tripod Components (Viewed from above)



Tripod Components (Viewed from below)

Tripod Components



Apparatus for Determining the Deviation of the Centre of a Borehole from the Vertical

ANNEXURE D. CERTIFICATE FOR BOREHOLE TESTED IN ACCORDANCE WITH CODE OF PRACTICE SABS 045 'TESTING WATER BOREHOLES'

D-1 A copy of a certificate for a borehole tested in accordance with Code of Practice SABS 045 'Testing water boreholes is given below:

CERTIFICATE FOR BOREHOLE TESTED IN ACCORDANCE WITH CODE OF PRACTICE SABS 045 'TESTING WATER BOREHOLES'

To be completed and signed by the Drilling Foreman or by the person conducting the test.

TEST No.(e.g. first, second, etc.)
 SITE
 FARM No. PORTION No.
 DISTRICT PROVINCE
 FULL NAME OF OWNER
 BORING SITE SELECTED BY
 LOCATION OF BOREHOLE
 (with reference to house, beacon, camp, fence, etc.)
 DESIGNATION OR NUMBER OF BOREHOLE
 (e.g. first, second, etc.)
 DATE DRILLING COMMENCED DATE DRILLING CEASED.....
 OWNER OF DRILL FOREMAN

PARTICULARS OF BOREHOLE

Geological formation		
Section		Strata
From	To	
Surface		

DIAMETER

..... mm from surface to m
 mm from m to .. m
 mm from m to .. m

CASING

Plain length m diam. mm
 Perforated between m
 and m diam. mm

DEPTH

Total depth of borehole from surface m
 Depth from surface at which water was struck m
 Depth from surface to which water rises m

TESTING TIME

Observed yield at hourly intervals			Water level
Date	Time	Yield, /s	Depth, m

Commencement of test
 Date hour

Completion of test:
 Date hour

Total testing time Hours

Yield determined by test /s
 /min
 /h

PARTICULARS OF TEST

Depth from surface of water-level at start of test m (see 4.4(a) of code)

Depth to which intake was inserted m

Depth to which water rises after completion of test m
 h after test (see 4.4(g) of code)

DETAILS OF TEST PUMP

Type of test pump
 (e.g. centrifugal, reciprocating, etc.)

Centrifugal: Bore mm

Capacity /s, /min, /h

Maximum number of revolutions per minute

Reciprocating: Inside diameter of pump cylinder mm

Maximum length of stroke mm

Maximum number of strokes per minutes

Other types (specify)

I certify that the above particulars are correct and that the yield test was made in accordance with SABS 045.

Signature

Date

PURPOSE OF BOREHOLE

Apparent quality of water

For what purpose is water to be used (e.g. domestic, stock, irrigation, etc.) ?

DATE

SIGNATURE OF OWNER OR REPRESENTATIVE

Appendix C

DIN 4021 ; Part 3

SUBSOIL EXPLORATION BY TRIAL PITS AND BORINGS AS
WELL AS SAMPLING AND INVESTIGATIONS OF GROUNDWATER

DIN 4021 : Part 3
August 1976

Translated by H. Kreft & B. Rossouw,

Baugrund
**Erkundung durch Schürfe und Bohrungen
sowie Entnahme von Proben**
Aufschluß der Wasserverhältnisse

DIN
4021
Teil 3

Subsoil; exploration by trial pits and borings as well as sampling; investigations of ground water

Zu dieser Norm gehört DIN 4021 Teil 1 „Baugrund; Erkundung durch Schürfe und Bohrungen sowie Entnahme von Proben; Aufschlüsse im Boden“, Ausgabe Juli 1971 und DIN 4021 Teil 2 „Baugrund; Erkundung durch Schürfe und Bohrungen sowie Entnahme von Proben; Aufschlüsse im Fels“, Ausgabe Februar 1976.

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1 Geltungsbereich

Diese Norm gilt für Aufschlüsse in Böden (Lockergestein) nach DIN 4021 Teil 1 und für Aufschlüsse im Fels nach DIN 4021 Teil 2, die für bautechnische Aufgaben ausgeführt werden. Sie gilt nicht für Bohrungen zur Wasserversorgung und des allgemeinen Grundwasserdienstes.

2 Zweck

Diese Norm soll bei dem Aufschluß des Baugrunds für bautechnische Aufgaben nach DIN 4021 Teil 1 und DIN 4021 Teil 2 als Richtlinie für das Messen der Wasserstände, der Wasserdruckverhältnisse und der Wasserbewegung im Untergrund sowie für die Feststellung bautechnisch wichtiger Eigenschaften des Wassers dienen. Hierzu gehören:

- a) die Einmessung der bei dem Niederbringen von Baugrundaufschlußbohrungen angetroffenen Wasserstände;
- b) der Ausbau von Baugrundaufschlußbohrungen für Grundwasserstandsbeobachtungen kurzer Dauer (bis zu etwa 1 Jahr);
- c) das Herstellen von Grundwassermeßstellen für Grundwasserstandsbeobachtungen langer Dauer;
- d) die Entnahme von Wasserproben;
- e) im Fels zusätzlich die Bestimmung der Wasseraufnahmefähigkeit.

3 Zu beachtende Normen, Vorschriften und Merkblätter

- | | |
|-----------------|---|
| DIN 1054 | Baugrund; Zulässige Belastung des Baugrunds |
| DIN 4021 Teil 1 | Baugrund; Erkundung durch Schürfe und Bohrungen sowie Entnahme von Proben, Aufschlüsse im Boden |
| DIN 4021 Teil 2 | Baugrund; Erkundung durch Schürfe und Bohrungen sowie Entnahme von Proben, Aufschlüsse im Fels |
| DIN 4022 Teil 1 | Baugrund und Grundwasser, Benennen und Beschreiben von Bodenarten und Fels, Schichtenverzeichnis für Untersuchungen und Bohrungen ohne durchgehende Gewinnung von gekerneten Proben |
| DIN 4023 | Baugrund- und Wasserbohrungen; Zeichnerische Darstellung der Ergebnisse |
| DIN 4030 | Beurteilung betonangreifender Wässer; Böden und Gase |
| DIN 4049 Teil 1 | Gewässerkunde; Fachausdrücke und Begriffsbestimmungen, Teil I: quantitativ |

Fortsetzung Seite 2 bis 14

Fachnormenausschuß Bauwesen (FNBau) DIN Deutsches Institut für Normung e.V.

- DIN 4049 Teil 2 Gewässerkunde; Fachausdrücke und Begriffserklärungen, Teil II: qualitativ
DIN 4924 Filtersande und Filterkiese für Brunnenfilter

Unfallverhütungsvorschriften der Tiefbauberufsgenossenschaften und der Bergämter.

Grundwasser-Richtlinien für Beobachtung und Auswertung. Herausgegeben vom Deutschen Grundwasserausschuß, 1961.

Vorläufiges Merkblatt über Untersuchung von Wasser und Boden auf betonangreifende Bestandteile. Herausgegeben von der Forschungsgesellschaft für das Straßenwesen, 1967.

4 Grundsätze für den Aufschluß der Wasserverhältnisse in Böden

4.1 Wasser im Untergrund (Grundwasser) kann in verschiedener Weise auftreten (siehe Bild 1 bis 3). Ziel der Untersuchungen ist es, alle Wasservorkommen zu erfassen und vorhandene Grundwasserstockwerke festzustellen. Hierzu dienen in erster Linie Grundwassermeßstellen. In oberflächennahen Zonen können auch Schürfe Aufschlüsse vermitteln, im Fels zusätzlich auch Stollen und Schächte.

Anmerkung zu Bild 1: Die Grundwassermeßstellen zeigen den Grundwasserspiegel jener Schicht an, in der die Filterstrecke des jeweiligen Peilrohrs liegt. In den Rohren B und C taucht die Filterstrecke in das Grundwasser ein, und der Grundwasserspiegel zeigt hier die freie Grundwasseroberfläche an. Bei den Rohren D, F und G ist das Grundwasser gespannt; der Grundwasserspiegel entspricht hier der Grundwasserdruckfläche, die bei Rohr G über das Gelände hinaufreicht (artesisches Grundwasser). Bei Rohr A liegt der Wasserspiegel tiefer als die Grundwasseroberfläche, da die Filterstrecke in der Kiesschicht liegt, die unmittelbar zu einem Vorfluter H entwässert (Dränagewirkung); infolge des Druckunterschieds bewegt sich das Wasser im Sand hier auch abwärts gegen die Kiesschicht. Auch im sandigen Schluff bei Rohr E vermag sich noch eine Grundwasseroberfläche auszubilden, doch dauert es hier viel länger als bei den Rohren B und C, bis sich zwischen dem Wasser im Rohr E und dem Grundwasser der Druckausgleich eingestellt hat.

Anmerkung zu Bild 2: Die Grundwassernichtleiter trennen das Grundwasser in drei Grundwasserstockwerke. Im Bereich von den Rohren C und D und zwischen den Rohren F und G dringt Niederschlags- und Oberflächenwasser als „Sickerwasser“ in den Untergrund ein. Es sammelt sich auf der Schluffschicht zwischen den Rohren F und G und auf der Tonschicht zwischen den Rohren F und G als zeitweise vorhandenes „Stauwasser“ an. Es kann auch bei größerer Ausdehnung – wie über der Tonschicht zwischen den Punkten A und H – als „Schichtwasser“ ein Grundwasserstockwerk bilden. Dieses Grundwasser zwischen den Rohren C und D tritt dann bei Punkt A zutage und speist das zweite Stockwerk bei Punkt H. Bei den Rohren E und E' wird die Lage der Grundwasseroberfläche des ersten Grundwasserstockwerks beobachtet. Das Grundwasser des zweiten Stockwerks fließt dem Vorfluter unterhalb Punkt B zu. Es ist bei Rohr F gespannt und bei Rohr D ungespannt. Das dritte Grundwasserstockwerk ist überall gespannt, seine Druckfläche wird durch den Wasserstand in Rohr G angezeigt; er ist vom Grundwasser des ersten und zweiten Grundwasserstockwerks nicht beeinflusst.

Anmerkung zu Bild 3: Ein Grundwasserleiter (Feinsand beziehungsweise kiesiger Sand) wird durch einen Grundwassernichtleiter streckenweise in zwei Grundwasserstockwerke geteilt. Die Grundwassersole

des gesamten Grundwasserleiters liegt an der Grenze zwischen dem kiesigen Sand und dem Ton. Die untere Begrenzung der oberen Tonschicht bildet die Deckfläche des gespannten Grundwassers im zweiten Grundwasserstockwerk.

Die Grundwasseroberfläche wird durch die Spiegelhöhen in den flachen Rohren A, C, D und E wiedergegeben und die Grundwasserdruckfläche des zweiten Grundwasserstockwerks durch den Wasserspiegel in den tiefreichenden Rohren B und F angezeigt. Da in dem tiefer gelegenen kiesigen Sand die Strömungsverluste geringer sind als in dem Feinsand kommt es im Bereich der hydraulischen Verbindung beider Grundwasserstockwerke bei Rohr B zu aufsteigendem Grundwasser; auf die aufsteigende Bewegung des Grundwassers weist auch die unterschiedliche Lage des Wasserspiegels in den Rohren B und C hin.

In die untere Tonschicht ist eine Sandlinse eingelagert, in der Grundwasser eingeschlossen ist. Wegen des sehr geringen Wassernachschubs aus dem fast undurchlässigen Ton würde die Druckhöhe des eingeschlossenen Grundwassers beim Anbohren der Sandlinse meist bleibend gestört werden.

4.2 Wegen der Mannigfaltigkeit der Bodenarten und der Wasserverhältnisse im Untergrund einerseits und infolge der Vorgänge beim Niederbringen von Bohrungen andererseits ist das Erkennen der tatsächlichen Wasserverhältnisse meist sehr schwierig. So können durch das weiträumig zusammenhängende oder auch durch das örtliche Auftreten wenig durchlässiger bindiger Bodenschichten verschiedene Grundwasserstockwerke oder auch lokal und zeitlich begrenzte Wasservorkommen (z. B. Schichtwasser, Stauwasser) ausgebildet sein.

In Tidegebieten ist zusätzlich zu beachten, daß die Grundwasserstände durch die Gezeitenbewegungen (Ebbe und Flut) beeinflußt werden, was sich in zeitlichen Verzögerungen im Auftreten der Hoch- und Niedrigwasserstände äußert; das Ausmaß der Beeinflussung ist in Abhängigkeit von der Entfernung der Grundwassermeßstelle zum freien Tidewasser sowie den geologischen Verhältnissen von Ort zu Ort verschieden. Diese Erscheinung wirkt sich auch in tieferen Grundwasserstockwerken aus.

Auch die Vorgänge beim Bohren (Durchörtern von Sperrschichten, laufende Wasser- und Bodenentnahme beziehungsweise Wasserzugabe) können zu Fehlbeurteilungen der wirklichen, das heißt der vom Bohrvorgang unbeeinflussten Wasserverhältnisse führen.

Bei der Ausführung von Baugrundaufschlußbohrungen kann deshalb die Einmessung der Wasserstände nur unter bestimmten Voraussetzungen ein zutreffendes Ergebnis liefern, z. B. wenn die Grundwasseroberfläche in einer gut durchlässigen Bodenschicht verläuft. In den meisten Fällen können jedoch die wirklichen Wasserstände bei der Durchführung von Aufschlußbohrungen nicht zutreffend eingemessen werden, weil diese selbst im allgemeinen durch den Bohrvorgang beeinflusst werden:

- Der Bohrfortschritt ist zum Ausspiegeln der wirklichen Wasserstände gewöhnlich zu schnell.
- Die notwendige Verrohrung behindert den Wasserausgleich, vermag aber auch mehrere Grundwasserstockwerke meist nicht völlig voneinander zu trennen.
- Die Bohrlochsole wird durch sedimentierte Feinbestandteile oft zugesetzt.

4.3 Für eine zutreffende Angabe der Wasserstände ist es in der Regel erforderlich, die Baugrundaufschlußbohrungen nachträglich mit Meßpegeln zu versehen (siehe Abschnitt 6) oder von vornherein besondere Grundwassermeßstellen einzurichten (siehe Abschnitt 7). Hierbei ist zu beachten, daß die Grundwasserstände bzw. -Druck-

höhen aller wasserführenden Schichten, die durch Sperrschichten voneinander getrennt sind, unabhängig voneinander festgestellt werden müssen. Auch in relativ wenig durchlässigen Schichten ist ein etwaiger Grundwasserspiegel durch Meßstellen gesondert zu erfassen. Bei Grundwassermeßstellen für langdauernde Beobachtungen kann allerdings im Laufe der Zeit die Beobachtung dadurch gestört werden, daß an Stahlfiltern eine Korrosion bzw. an allen Filtern eine Verockerung auftritt.

4.4 Schließlich besteht die Möglichkeit, aus der Feuchtigkeit, Konsistenz oder aus der Verfärbung des Bodens zusätzliche Schlüsse über die Wasserverhältnisse zu ziehen (siehe Abschnitt 8).

4.5 Anzahl, Art, Tiefe und Anordnung von Grundwassermeßstellen, die Entnahme von Wasserproben, sowie Dauer und Häufigkeit der Beobachtungen sind vom Auftraggeber oder seinem Beauftragten — bei Einschaltung eines Sachverständigen im Einvernehmen mit diesem — rechtzeitig festzulegen.

Dabei sind zu berücksichtigen:

- a) die hydrogeologischen Verhältnisse unter Beachtung örtlicher Erfahrungen und vorhandener Aufschlüsse sowie verfügbarer Unterlagen und dergleichen;
- b) die Möglichkeit, daß kurz- und langzeitige Spiegelschwankungen im natürlichen Wasserhaushalt oder als Folge baulicher oder betrieblicher Maßnahmen eintreten können;
- c) bei Flachgründungen die Tiefenlage der Gründungsohle zum höchsten zu erwartenden Grundwasserstand, bei Tiefgründungen die Einbindetiefe der Gründungselemente in das Grundwasser, bei Erdbauten die Lage der Wasserspiegel in den Böschungen und zur Sohle;
- d) die Grundrißabmessungen des Bauwerks;
- e) die Auswirkung von Grundwasserabsenkung auf Anlagen Dritter;
- f) etwaige Verdachtsmomente hinsichtlich einer Grundwasserverschmutzung.

Hinsichtlich der Lage und Höhe des Ansatzpunktes der Grundwassermeßstellen ist nach DIN 4021 Teil 1, Ausgabe Juli 1971, Abschnitte 4.7 und 4.8 zu verfahren.

5 Messen der Wasserstände im Bohrloch bei der Ausführung von Baugrundaufschlußbohrungen in Böden

5.1 Sobald bei einer Baugrundaufschlußbohrung Wasser im Bohrloch angetroffen wird, ist der Bohrvorgang zu unterbrechen und der Wasserstand nach einer Bohrpause (siehe Abschnitt 5.4) einzumessen. Er ist außerdem täglich bei Beginn und Ende der Arbeitszeit sowie vor und nach längeren Arbeitsunterbrechungen und schließlich vor dem Verfüllen der Bohrung zu messen (siehe Abschnitt 5.5).

Besonderes Augenmerk ist auch auf das etwaige Auftreten von Schichtwasser oberhalb der Grundwasseroberfläche zu richten.

Alle Erscheinungen, die auf das Vorhandensein von Wasser, wie z. B. Neigung zum Fließen, Nachfall oder Sohlaufbruch schließen lassen, sowie deutliche Konsistenzänderungen des Bodens sind sorgfältig zu beachten. Wenn Wasser im Bohrloch merkbar steigt oder sinkt, ist festzustellen, wann, bei welcher Tiefe und bei welchem Wasserstand die Veränderung beginnt und endet.

Die auf diese Weise festgestellten Wasserstände entsprechen allerdings nur unter günstigen Bedingungen den wirklichen Wasserständen (siehe Abschnitt 4.2). Die beim Bohren

ausgeführten Messungen und Beobachtungen sind trotzdem für die überschlägliche Beurteilung der Wasserverhältnisse unentbehrlich.

5.2 Die Messung der Wasserstände nach Abschnitt 5.1 führt in der Regel zu unrichtigen Werten, wenn beim Bohren:

- a) mit Spülung oder Dickspülung,
- b) mit Spülhilfe,
- c) mit Wasserüberdruck,
- d) mit Wasserzusatz,
- e) mit zu großem Bohrfortschritt

gearbeitet wird (siehe DIN 4021 Teil 1, Ausgabe Juli 1971, Abschnitt 5.2.2).

Anmerkung: Bei gut durchlässigen nichtbindigen Bodenarten, bei denen sich ein schneller Ausgleich des Wasserspiegels einstellt, wirken sich jedoch Spülhilfe oder ein Wasserzusatz und ein großer Bohrfortschritt im allgemeinen nicht nachteilig aus. Das Ausspiegeln tritt bei gleichem Boden in Bohrlöchern mit kleinem Durchmesser schneller ein als in Bohrlöchern mit großem Durchmesser, was besonders bei weniger durchlässigen, z. B. sandig-schluffigen Böden, von Bedeutung ist.

Vom Untergrund her liegen für die Messung der Wasserstände günstige Bedingungen dann vor, wenn die wasserführenden Schichten gut durchlässig sind (nichtbindige Böden ausschließlich Feinsand). Je feinkörniger die Bodenart, d. h. je geringer die Durchlässigkeit des Wasserleiters ist, desto länger dauert das Ausspiegeln des Wassers im Bohrloch und um so stärker fallen Einflüsse des Bohrvorgangs, der Bohrwerkzeuge und des Bohrfortschritts störend ins Gewicht.

5.3 Bei geschichtetem Untergrund mit mehreren Grundwasserstockwerken ist DIN 4021 Teil 1, Ausgabe Juli 1971, Abschnitt 5.2.3 zu beachten. Bei Verwendung nur einer Rohrtour ist grundsätzlich nur der Wasserspiegel des höchsten Grundwasserstockwerks unter den in Abschnitt 5.2 gemachten Einschränkungen zuverlässig zu messen (siehe Bild 4a), da sich eine einwandfreie Abdichtung der einzelnen Grundwasserstockwerke längs des Bohrrohrs im allgemeinen nicht erreichen läßt. Dadurch kann der Wasserspiegel des unteren Stockwerks stark verfälscht werden (siehe Bild 4c und 4d).

Soll der Wasserstand eines zweiten Grundwasserstockwerks während der Baugrundaufschlußbohrung gemessen werden, so ist die erste Rohrtour in der Sperrschicht (Grundwassernichtleiter) dicht abzusetzen, und es ist mit einer zweiten Rohrtour weiter zu verrohren (siehe Bild 4b); in dieser kann der Wasserstand des zweiten Stockwerks unter den Einschränkungen des Abschnittes 5.2 erfaßt werden.

Anmerkung zu Bild 4: Ist unter einer Ton-schicht ein zweites Grundwasserstockwerk zu erwarten, dessen Druckfläche mit der Grundwasseroberfläche des oberen Grundwasserstockwerks nicht übereinstimmt (siehe Bild 4a), so kann die Druckhöhe des unteren Grundwasserstockwerks nur dann richtig erfaßt werden, wenn die beiden Grundwasserstockwerke durch das Bohrloch nicht miteinander verbunden werden. Hierzu ist es notwendig, zu „teleskopieren“. Dabei wird nach dichtem Einbinden der Verrohrung in den Grundwassernichtleiter eine zweite Rohrtour mit geringerem Durchmesser tiefer in das untere Grundwasserstockwerk geführt (siehe Bild 4b).

Wird die Verrohrung nicht abgesetzt, so wird die zu beobachtende Druckhöhe des unteren Grundwasserstockwerks infolge der Umläufigkeit der Verrohrung besonders dann verfälscht, wenn dieser Teil des Grundwasserleiters geringer

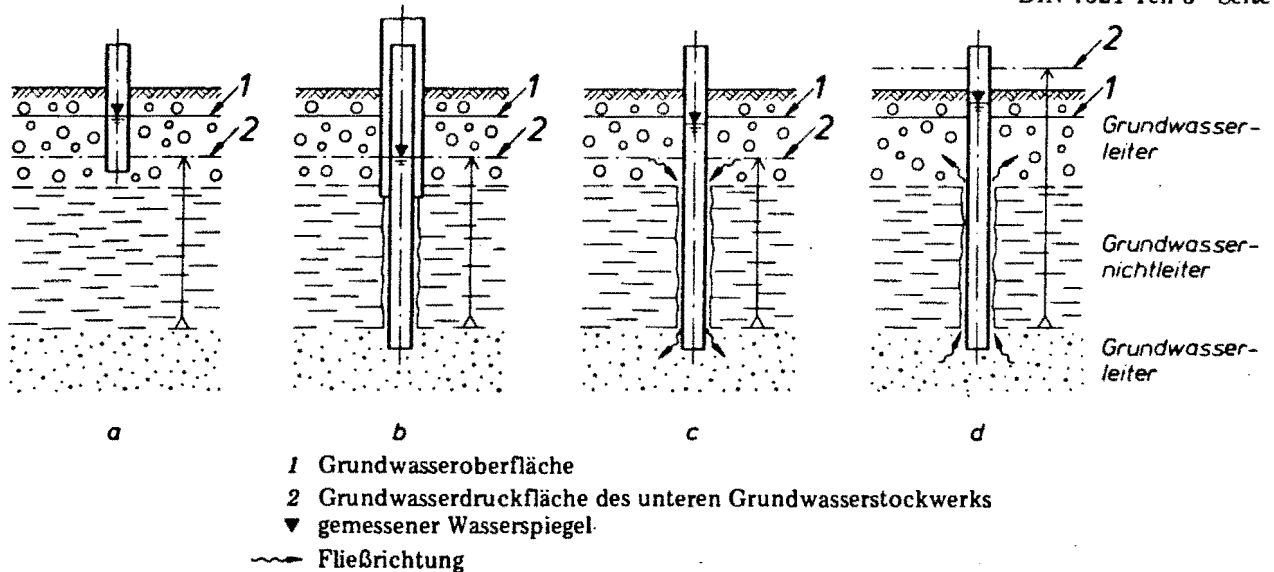


Bild 4. Mögliche Beeinflussung der Spiegelhöhenmessung im Bohrloch beim Durchfahren eines Grundwassernichtleiters
 a) richtige Messung der Grundwasseroberfläche
 b) richtige Messung der Grundwasserdruckfläche des unteren Grundwasserstockwerks
 c) und d) fehlerhafte Messungen der Grundwasserdruckflächen des unteren Grundwasserstockwerks

durchlässig ist als der obere (siehe Bild 4 c und 4 d). Es wird eine zu hohe Lage der Druckfläche vorgetäuscht, wenn die Grundwasserdruckfläche des unteren Stockwerks unter der Grundwasserdruckfläche des oberen Stockwerks liegt (siehe Bild 4 c). Eine zu niedrige Lage der Druckfläche wird gemessen, wenn die Grundwasserdruckfläche des unteren Stockwerks oberhalb der Grundwasserdruckfläche des oberen Stockwerks liegt (siehe Bild 4 d).

5.4 Für die Bohrpause nach Abschnitt 5.1 reicht in gut durchlässigen Böden und bei Beachtung der Einschränkung nach Abschnitt 5.2 zum Wasserspiegelausgleich im Bohrloch eine Zeitspanne von etwa 5 Minuten aus. In weniger durchlässigen Böden kann der Ausgleich des Wasserspiegels im Bohrloch in der Regel mit Rücksicht auf den Bohrfortschritt, d. h. ohne zusätzliche Stillstandszeit, nicht abgewartet werden. In diesen Fällen läßt sich durch Wiederholung der Messung während einer verlängerten Bohrpause allenfalls feststellen, ob das Wasser im Bohrloch steigt oder sinkt.

5.5 Alle Wasserstände sind mit einer Meßgenauigkeit von 1 cm festzustellen und im Schichtenverzeichnis bzw. als Anlage hierzu mit Datum und Uhrzeitangaben zu vermerken. Auch alle hinsichtlich des Wassers sonst gemachten Beobachtungen (siehe Abschnitt 5.1) sind zu protokollieren (siehe DIN 4022 Teil 1, Ausgabe November 1969, Abschnitt 11.11b und Tabelle 1, Spalte 3, Punkt 2). Als Meßgeräte kommen z. B. Brunnenpfeife oder Lichtlot in Frage.

6 Ausbau von Baugrundaufschlußbohrungen zu Grundwassermeßstellen in Böden

Soll eine Baugrundaufschlußbohrung zu einer Grundwassermeßstelle ausgebaut werden, so ist hierbei nach Abschnitt 7 zu verfahren. Hierzu ist in der Regel ein Außendurchmesser der Bohrrohre von mindestens 159 mm erforderlich (siehe DIN 4021 Teil 1, Ausgabe Juli 1971, Abschnitt 5.2.4).

7 Anlage von Grundwassermeßstellen in Böden

7.1 Anordnung

Grundwassermeßstellen dienen allein der Beobachtung von Grundwasserständen; hierzu gehören auch die Wasser-

stände örtlich und zeitlich begrenzter Vorkommen, z. B. von Schichtwasser, Stauwasser (siehe Bild 2) und Kluftwasser. Die Meßstellen bestehen in der Regel aus einzeln angeordneten Beobachtungsrohren (Peilrohre, siehe Bild 5), die auch zu Gruppen zusammengefaßt werden können.

Die Lage der Grundwassermeßstellen im Baugelände wird meist aufgrund vorher durchgeführter Bohrungen gewählt. Wenn Grundwasserbeobachtungen während und auch noch nach der Ausführung von Bauarbeiten erforderlich werden, ist dies bei der Festlegung der Meßstellen zu berücksichtigen. In besonderen Fällen kann auch die Anordnung von Grundwassermeßstellen außerhalb des Baugeländes notwendig sein (siehe Abschnitt 4.5e)). Die unmittelbare Nähe von Lagerplätzen, Baustraßen, Wasserläufen, Geländeeinschnitten und Brunnenanlagen ist zu meiden, um eine Beschädigung bzw. Beeinflussung der Meßstelle auszuschließen. Im Hinblick auf spätere Baumaßnahmen ist auch zu prüfen, ob deren Einfluß auf das Meßergebnis von Bedeutung ist.

7.2 Ausbau

7.2.1 Grundsätze

Die durch die Untersuchungen zu erfassenden Grundwasserstockwerke, soweit sie auf Grund vorhandener Unterlagen von Beginn der Bohrungen festgelegt werden können, sowie die sich daraus ergebenden voraussichtlichen Endtiefen der Bohrungen sind vom Auftraggeber unter Beachtung des Ziels und Zwecks der Grundwassermeßstellen anzugeben. Dies gilt auch für die Anzahl der Peilrohre und die Höhenlage der einzubauenden Sumpf-, Filter- und Aufsatzrohre sowie für die erforderlichen Abdichtungen; die Angaben sind nach Vorlage der Bohrergebnisse zu korrigieren.

Die voraussichtliche Beobachtungsdauer ist rechtzeitig vor Beginn der Arbeiten festzulegen. Dies beeinflußt nicht nur die Wahl der Filter und des Rohrwerkstoffs, sondern unter Umständen auch die Wahl der Bohrdurchmesser und den Ausbau im allgemeinen.

Der Aufwand für den Ausbau von Pegeln kann bei kurzer Beobachtungsdauer (bis zu etwa einem Jahr) und geringer Tiefenlage des Grundwasserstockwerks (bis max. 10 m) vielfach gering gehalten werden (siehe Abschnitte 7.2.2.1 und 7.2.2.2). Bei langer Beobachtungsdauer (mehr als ein Jahr) oder großer Tiefenlage des Grundwasserstock-

werks müssen die Meßstellen dagegen als Kiesschüttungspegel ausgebildet werden (siehe Abschnitte 7.2.2.3 und 7.2.3).

Werden mehrere wasserführende Grundwasserleiter vermutet oder angetroffen, so sind ihre einzelnen Grundwasserstockwerke getrennt zu erfassen. Dabei sollte wegen der Schwierigkeiten beim Abdichten der einzelnen Grundwasserstockwerke gegeneinander nach Möglichkeit für jedes Peilrohr eine besondere Bohrung niedergebracht werden. Der gemeinsame Einbau mehrerer Peilrohre in ein Bohrloch sollte deshalb möglichst vermieden werden und auf Sonderfälle beschränkt bleiben.

Bei nur geringer Mächtigkeit des Grundwasserstockwerks sollte die Filterunterkante der Peilrohre bis zu seiner jeweiligen Sohle reichen. Bei größerer Mächtigkeit des Grundwasserstockwerks ist die Filterunterkante mindestens 3 m unterhalb des tiefsten erwarteten Grundwasserspiegels anzuordnen.

7.2.2 Geringe Tiefen und kurze Beobachtungsdauer

7.2.2.1 Gerammte Peilrohre (Rammpegel)

Bei geringen Tiefen (bis max. 5 m) sind in Grundwasserleitern mit hoher Durchlässigkeit Rammpegel brauchbar. Es müssen Stahlrohre mit Filterteil und Rammspitzen verwendet werden.

7.2.2.2 Eingespülte Peilrohre

Bei spülfähigen Böden und begrenzten Tiefen (bis max. 10 m) können Peilrohre mit eingesetzter oder gesonderter Spüllanze eingespült werden. Es werden Kunststoff- oder Stahlrohre mit geschlitztem Rohrteil verwendet; dieser ist mit einem Filter gegen Sandeintritt zu schützen.

7.2.2.3 Einbau von Peilrohren in verrohrte oder unverrohrte Bohrungen mit Kiesschüttung

Diese Ausbauart ist anzuwenden, wenn die Voraussetzungen für den Ausbau nach den Abschnitten 7.2.2.1 und 7.2.2.2 nicht gegeben sind. Sie ist deshalb nach ähnlichen Grundsätzen durchzuführen, wie sie für Meßstellen mit langer Beobachtungsdauer gelten (siehe Abschnitt 7.2.3).

7.2.3 Längere Beobachtungsdauer

7.2.3.1 Allgemeines

Für diese Ausbauart ist zunächst ein Bohrloch abzuteufen, das in der Regel mit Verrohrung hergestellt sein muß. Auf die Verrohrung darf in Ausnahmefällen mit Einwilligung des Auftraggebers verzichtet werden. Die Peilrohre sind mit Kies- bzw. Sandschüttung einzubauen.

DIN 4021 Teil 1, Ausgabe Juli 1971, Abschnitte 5.2, 6.2, 7 und 8 sowie DIN 4022 Teil 1 und DIN 4023 sind sinngemäß zu beachten.

7.2.3.2 Bohrlochdurchmesser

Der Durchmesser der Verrohrung bzw. die Anfangs- und Enddurchmesser bei gestaffelter Verrohrung richten sich nach dem Außendurchmesser und der Anzahl der vorgesehenen Peilrohre. Folgende Mindestbohrlochdurchmesser werden für Peilrohre bis 50 mm Durchmesser empfohlen:

bei Einbau von 1 Peilrohr 159 mm Außendurchmesser
in Sonderfällen:

bei Einbau von 2 Peilrohren 203 mm Außendurchmesser
bei Einbau von 3 Peilrohren 267 mm Außendurchmesser

Der Einbau von nur einem Peilrohr ist vorzuziehen (siehe Abschnitt 7.2.1).

7.2.3.3 Herrichten des Bohrlochs für den Einbau

Die Bohrlochsohle ist für den Einbau der Peilrohre freizuräumen. Spülbohrlöcher sind vor Einbaubeginn klar zu spülen und so weit abzuteufen, daß unterhalb des untersten Peilrohrs ein genügend langer Bohrlochsumpf zur Aufnahme von Nachfall vorhanden ist.

7.2.3.4 Einbauplan

Vor Beginn des Einbaus sind Art und Anordnung der Peilrohre mit Sumpf-, Filter- und Aufsatzrohren, der Abdichtungen sowie der Verkiesung bzw. Verfüllung des Bohrlochs in einem Einbauplan unter Beachtung von Abschnitt 7.2.1, Absatz 1 festzulegen.

7.2.3.5 Peilrohr und Filter

Für Sumpf-, Filter- und Aufsatzrohre müssen korrosionsbeständige, gegebenenfalls auch gegenüber aggressivem Wasser widerstandsfähige Rohre verwendet werden, z. B.:

- Kunststoffrohre;
- durch Anstrich, Kunststoffüberzug, Hartgummibelag oder Verzinken geschützte Stahlrohre;
- Edelstahlrohre.

Bei Kunststoffrohren ist zu beachten, daß mechanische Beanspruchungen und niedrige Wasser- bzw. Lufttemperaturen während des Einbaus ihre Verwendbarkeit beeinträchtigen können.

Als Innendurchmesser der Peilrohre genügen im allgemeinen 25 bis 50 mm; bei Verwendung von Schreibpegeln sind größere Innendurchmesser (> 100 mm) zu empfehlen.

Unter der Filterstrecke sollte ein mindestens 0,3 bis 1,0 m langes, unten verschlossenes Sumpfrohr angeordnet werden. Bei Verzicht auf ein Sumpfrohr ist das Filterrohr am unteren Ende durch eine Kappe oder ähnliches abzuschließen; dabei ist zusätzlich eine Unterschüttung mit Filtersand entsprechender Höhe einzubringen.

Die Länge des Filterrohrs soll im allgemeinen 1 bis 2 m betragen; bei geringer Mächtigkeit des Grundwasserstockwerks ist sie dieser anzupassen. Die Filterrohre können mit Geweben, Kiesklebefiltern oder ähnlichem ummantelt werden.

Die Korngröße des Filtersandes oder des Filterbelags ist der geringsten Korngröße der zu verfilternden Schicht des Grundwasserleiters den Filterregeln entsprechend anzupassen und sollte deshalb im allgemeinen nicht größer als 1 bis 2 mm sein. Die Körnungen von

0,5 bis 1,0 mm oder

0,7 bis 1,4 mm oder

1 bis 2 mm

nach DIN 4924 sind zu bevorzugen. Bei flachen Grundwassermeßstellen (bis etwa 5 m Wassertiefe) genügt auch vielfach die Verwendung von weiter abgestuften, gewaschenen Körnungen, z. B. 0,1 bis 3 mm.

7.2.3.6 Einbau der Peilrohre

Die Peilrohre werden möglichst zentrisch in das Bohrloch abgelassen. Sie sind dazu mindestens unterhalb der Filterrohre mit Distanzhaltern zu versehen. Für Schreibpegel müssen die Peilrohre lotrecht stehen. Auf einen dichten Anschluß aller Rohrverbindungen ist zu achten. Schädliche Setzungen des Peilrohrs selbst oder des umgebenden Bodens sind zu verhindern.

Die Oberkante des Peilrohrs ist sofort nach dem Einbau so einzumessen, daß ein Mitgehen des Peilrohrs beim Ziehen der Verrohrung rechtzeitig erkannt und verhindert werden kann.

7.2.3.7 Einbringen des Filtersandes und Verfüllen des Bohrlochs

Um eine etwaige Schlammhaut am Filterrohr zu beseitigen, ist unmittelbar vor Einbau des Filtersandes etwa ein Eimer Wasser in das Peilrohr zu schütten. Auch beim Einbringen des Filters ist in das Peilrohr ständig Wasser nachzufüllen, um den Wasserstand im Peilrohr höher als im Bohrrohr zu halten.

Die Filterschüttung muß das Sumpfrohr und das Filterrohr allseitig umschließen und mindestens 1 m über das Filterrohr hinaufreichen. Bei wechselnder Schichtung innerhalb eines Grundwasserleiters kann der Aufbau einer Filterschüttung über eine größere Strecke zweckmäßig sein, unter Umständen bis zur nächsten Abdichtung. Bei dem Einbau von mehreren Peilrohren sollte in den unteren Grundwasserstockwerken der gesamte Bereich zwischen den Abdichtungen (siehe Abschnitt 7.2.3.8) vollständig mit Filtersand ausgefüllt werden.

Der Filtersand ist sorgfältig kontinuierlich in kleinen Mengen so einzubringen, daß keine Verstopfung entsteht. Bei verrohrten Bohrlöchern müssen die Mantelrohre gleichzeitig mit dem Schütten des Filtersandes gezogen werden. Der Rohrschuh soll dabei etwa 0,3 bis 0,5 m unterhalb der jeweiligen Oberfläche der Filterschüttung stehen, was durch Loten zu verfolgen ist.

Über der Filterschüttung ist das Bohrloch mit Bohrgut oder körnungsmäßig ähnlichem Boden zu verfüllen. Unmittelbar oberhalb der Filterschüttung darf jedoch kein Boden eingefüllt werden, der diese verunreinigt.

7.2.3.8 Wiederherstellung der Sperrschichten

Alle Sperrschichten, die Grundwasserstockwerke trennen, sind wiederherzustellen. Dies kann mit Ton, Bentonit-Zement-Gemischen oder Bentonit-Schwerspat-Gemischen geschehen.

Für die Tonabdichtung dürfen nur gering- bis mittelplastischer, lufttrockener Ton in etwa 1 bis 3 cm großen Stücken oder handelsübliche Tonkugeln verwendet werden. Der Ton ist vor dem Einbringen in das Bohrloch gegen Aufnahme von Feuchtigkeit zu schützen. Tonkugeln können eingeschüttet werden; Stückton ist mit Schüttröhren einzubringen. Die Dicke der Tonabdichtung richtet sich nach dem aufzunehmenden Wasserdruck und der Mächtigkeit der natürlichen Sperrschicht; sie sollte mindestens 1 m betragen. Die Abdichtung ist beim Einbau ständig mit einer Wassersäule zu belasten, die im Bohrrohr höher reichen muß als die Druckhöhe des darunterliegenden Grundwasserstockwerks. Nach Einbringen des Tons wird die Verrohrung bis 30 cm unterhalb der Oberfläche der Tonabdichtung gezogen. Die Wartedauer bis zum weiteren Verfüllen des Bohrlochs sollte dann noch mindestens 8 Stunden betragen.

Für die Bentonit-Zement-Abdichtung ist Bentonitmehl mit Wasser und Zement etwa im Verhältnis 25 : 30 : 25 in einem Rührwerk kurz anzumischen und durch ein Verpreßgestänge sofort im Kontraktorverfahren einzupumpen. Hinsichtlich der Dicke der Abdichtung und des Ziehens der Verrohrung gelten die Ausführungen des vorhergehenden Absatzes. Dagegen kann die Wartedauer bis zum weiteren Verfüllen des Bohrlochs bis auf 1 bis 2 Stunden verkürzt werden.

Die Bentonit-Schwerspat-Abdichtung wird bei Auftreten von artesischem Grundwasser benötigt, um ein dem Wasserdruck entsprechendes Gegengewicht herzustellen. Hierzu muß der Schwerspatzusatz von Fall zu Fall berechnet werden, und zwar derart, daß die Abdichtung nicht dicker wird als die Sperrschicht; das Gewichtsverhältnis der Mischung Bentonit : Wasser beträgt etwa 1 : 10. Das aufbereitete Gemisch muß im Kontraktorverfahren eingebracht werden. Hinsichtlich des Ziehens der Verrohrung gelten wieder die Ausführungen im Absatz über die Tonabdichtung. Eine Wartedauer bis zum weiteren Verfüllen des Bohrlochs ist hier nicht erforderlich.

7.2.3.9 Abschluß der Peilrohrköpfe

Das Eindringen von Oberflächenwasser ist durch eine mindestens 50 cm unter Gelände reichende Lehm- oder Tonschicht zu verhindern.

Die Oberkante der Peilrohre kann über Gelände liegen, wenn dies nicht hinderlich ist. Die Rohre sind dann gegen Beschädigung durch geeignete Maßnahmen zu sichern (tief eingeschlagene Pflöcke mit Dreiecksverband oder ähnlichem). Dort, wo herausstehende Peilrohre unerwünscht sind, ist es zweckmäßig, sie unter Gelände abzuschneiden und einen Schutzkasten (Straßenkappe) anzuordnen, dessen Oberkante geländegleich ist, damit er keine Gefährdung für Fahrzeuge darstellt. Die Benutzung durch Unbefugte muß ausgeschlossen sein. Bei Verwendung von Schutzkästen ist darauf zu achten, daß ein Abfluß des in die Schutzkästen eingedrungenen Oberflächenwassers sichergestellt ist. Die das Bohrloch nach oben abschließende Lehm- oder Tonabdichtung sollte in diesem Fall 1 bis 2 m unter Gelände enden.

Die Pegel sind durch verschließbare Kappen zu sichern und mit einem Lüftungsloch zu versehen. Bei artesischem Grundwasser ist das Peilrohr durch ein Druckmeßgerät zu schließen, das die Ablesung des Wasserdrucks ermöglicht. Sind in einer Bohrung mehrere Peilrohre eingebaut, so müssen die Rohroberkanten zur Vermeidung von Verwechslungen eindeutig gekennzeichnet werden. Stehen in Oberflächennähe frostgefährdete Böden an, so kann es notwendig sein, die Peilrohre gegen Frosthebungen bis unter die Frostgrenze durch einen Schlupfmantel (siehe Bild 6) zu sichern.

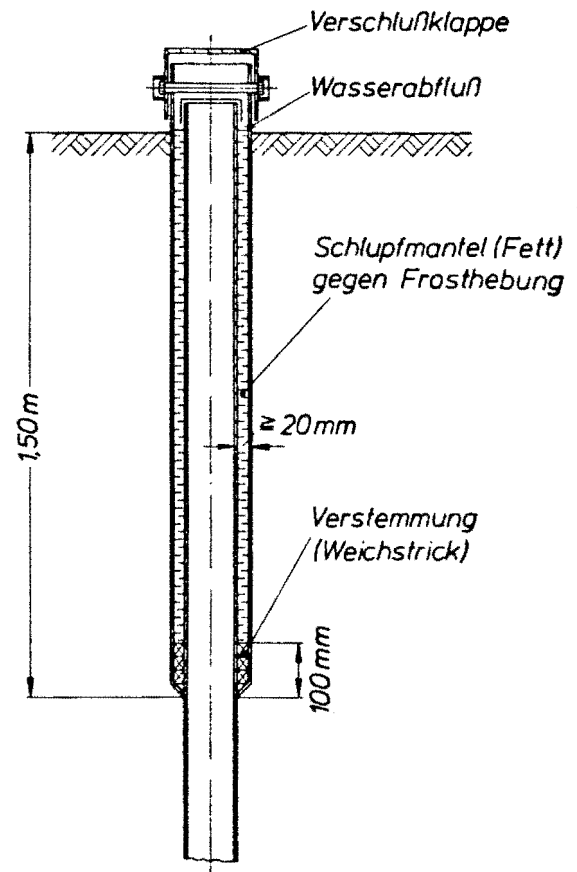


Bild 6. Grundwassermeßstelle mit Schlupfmantel gegen Frosthebung

7.2.4 Maßnahmen nach der Errichtung der Grundwassermeßstellen

Nach Fertigstellung der Grundwassermeßstelle muß ihre Funktionsfähigkeit geprüft werden. Dazu wird der Grundwasserstand im Peilrohr gemessen und anschließend das Rohr mit etwa 5 l Wasser aufgefüllt oder eine entsprechende Wassermenge abgepumpt. Durch Messungen in kurzen

Zeitabständen wird geprüft, ob eine Ausspiegelung in angemessener Zeit eintritt und damit die gewünschte Verbindung zum Grundwasser besteht.

Die Beobachtungsergebnisse sind auf dem Arbeitsbericht in folgender Form festzuhalten:

- a) ausgespiegelter Wasserstand im Peilrohr vor der Prüfung;
- b) Wasserstand nach Auffüllen oder Abpumpen;
- c) Messung des jeweiligen Wasserstands mit genauer Uhrzeitangabe.

Da nach Fertigstellung einer Grundwassermeßstelle oft noch geringe Setzungen stattfinden können, ist die Peilrohroberkante erst nach einigen Tagen auf NN einzumessen. Die Höhe des Peilrohrkopfes über NN ist nach gewissen Zeitabständen zu kontrollieren und die Prüfung der Funktionsfähigkeit der Meßstelle zu wiederholen. Dabei ist durch Lotung auch zu prüfen, ob das Peilrohr durchgehend bis zur Filterunterkante frei ist.

7.3 Messen der Wasserstände

Für bautechnische Zwecke interessieren in erster Linie die wahrscheinlichen Grundwasserstände während der Bauzeit sowie ihre Extremwerte im Hinblick auf das spätere Bauwerk. Während der für Bauaufgaben verfügbaren Beobachtungszeiten können die Extremwerte, wie sie in größeren Zeitspannen auftreten, dagegen nur selten erfaßt werden. Anhaltspunkte für die Abschätzung solcher Werte, die für das Bauwerk Bedeutung haben können, müssen gegebenenfalls in Verbindung mit den langjährigen Messungen des Grundwasserbeobachtungsdienstes abgeleitet werden.

Durch die Messungen sollen die witterungsbedingten und jahreszeitlichen oder durch extreme Wasserstände naher Gewässer beeinflussten Grundwasserschwankungen bestimmt werden. Die zeitliche Folge der Messungen ist so einzurichten, daß nach Möglichkeit die höchsten und niedrigsten Grundwasserstände festgestellt werden können. Alle Wasserstände sind mit einer Meßgenauigkeit von 1 cm abzulesen und zu protokollieren. Als Meßgeräte werden überwiegend Brunnenpfeifen, Lichtlote und Schwimmergeräte eingesetzt. Ist eine durchgehende Erfassung der Grundwasserspiegelschwankungen erforderlich, so ist dies allein durch den Einsatz selbstregistrierender Meßgeräte möglich.

8 Hinweise auf Wasserstände in Böden

Hinweise auf langzeitige Wasserhochstände und Wasser-tiefstände sowie auf den Schwankungsbereich des Grundwassers können unter Umständen an Baugrubenböschungen, in Schürfruben und an Bohrkernen aus der Farbe der Bodenarten, ihrer eventuellen Fleckigkeit und aus gegebenenfalls vorhandenen Konkretionen erhalten werden (siehe Tabelle). Die Hochstände zeichnen sich durch die Verfärbung an der unteren Grenze der Oxydationszone ab. Aus der Lage der oberen Grenze der Reduktionszone lassen sich Schlüsse auf die Tiefstände ziehen. Derartige Merkmale können sich bei mehreren Wasserhorizonten im Profil wiederholen. Örtlich begrenzte Wasservorkommen können am räumlichen Nebeneinander von unterschiedlichen Farben (z. B. braun und blau) erkannt werden.

Anmerkung: Konkretionen sind Mineralsubstanzen in einem Gestein bzw. Lockergestein, die sich in unregelmäßiger, meist linsenförmiger, kugelig, knolliger oder traubenförmiger Form ausgeschieden haben.

Die Oxydationszone ist der Bereich im Boden oberhalb des Wasserspiegels, der ständig Luft enthält. Die Reduktionszone unterhalb des Wasserspiegels ist ständig wasser-gesättigt.

9 Entnahme von Wasserproben in Böden

9.1 Allgemeines

Im Rahmen dieser Norm wird auf die Entnahme von Wasserproben ausschließlich im Hinblick auf bautechnische Folgerungen eingegangen. Im einzelnen handelt es sich dabei um:

- a) Allgemeine Untersuchungen der chemischen Bestandteile im Zusammenhang mit dem mineralogisch-chemischen Aufbau der Bodenschichten;
- b) Untersuchung auf betonschädliche Bestandteile;
- c) Eignungsprüfung als Anmachwasser für Beton;
- d) Untersuchung auf Korrosionsgefahr;
- e) Untersuchung auf eine Gefährdung von Dränagen, Filtern oder Versickerungsanlagen durch Ausfällungen und ähnlichem;

Tabelle Hinweis-Merkmale für Wasserstände

Bereiche	Bodenart	Merkmale		
		Boden		Konkretionen
		Farbe	Flecken und Streifen	Farbe
oberhalb der Hochstände (Oxydationszone)	bindige Böden	braun bis gelb	—	—
	nichtbindige Böden	braun bis hellgelb	—	—
Schwankungsbereich	bindige Böden	braun bis gelb, grau, grünlich	vorhandene Rostflecken	braun, dunkelbraun bis schwarz
	nichtbindige Böden	braun bis gelb, grau	vorhandene Rostflecken und Roststreifen	braun, dunkelbraun bis schwarz
ständig im Wasser (Reduktionszone)	bindige Böden	blau, grün, grau	—	wenn vorhanden, dann olivfarben
	nichtbindige Böden	blau, grün, grau, jedoch mit schwacher Farbintensität	—	wenn vorhanden, dann olivfarben

- f) Untersuchung hinsichtlich Grundwasseränderungen infolge bautechnischer Maßnahmen, z. B. in Wassergewinnungsgebieten;
- g) Untersuchung auf chemische oder biologische Verunreinigung durch Abwässer oder Sickerwasser aus Halden, Industrieanlagen, landwirtschaftliche Betriebe und ähnlichem.

9.2 Grundsätze für die Entnahme

Die Entnahmestellen sollen nach den Erfordernissen der vorliegenden Bauaufgabe unter Berücksichtigung der örtlichen geologischen und hydrologischen Verhältnisse festgelegt werden. Darüber hinaus sind in der näheren Umgebung von Industrieanlagen, von dicht besiedelten Wohngebieten, von Ablagerungen von Abfallprodukten (z. B. chemischen Rückständen, Müll, Schutt) und von Schüttungen von organischen oder auslaugfähigen Böden aus mindestens zwei verschiedenen Stellen Wasserproben zu entnehmen. Bei mehreren Grundwasserstockwerken kann die Entnahme aus jedem Stockwerk notwendig sein. Grundsätzlich sind aus Brunnen oder Grundwassermeßstellen zuverlässigere Proben als aus Bohrlöchern zu erwarten, weil die Beschaffenheit des Grundwassers durch den Bohrvorgang verändert wird.

9.3 Entnahme

9.3.1 Geräte und Chemikalien

Für die Entnahme einer Wasserprobe sind im Regelfall (Feststellung der Betonaggressivität, siehe DIN 4030) folgende Geräte und Chemikalien erforderlich:

- a) Saubere Probenflaschen (aus Glas, möglichst der hydrolytischen Klasse 1) mit luftdichtem Verschluß; zwei Flaschen mit je 1 l Inhalt, zwei Flaschen mit je 0,5 l Inhalt;
- b) Pumpe mit Abflußschlauch (kein Gummischlauch);
- c) Sonderentnahmegesetz mit Boden- oder Seiteneinlauf;
- d) Marmorpulver (chemisch reines Kalziumkarbonat), 5 g, bereits in eine 0,5-l-Flasche eingefüllt;
- e) Zinkacetat, 3 g, bereits in eine 0,5-l-Flasche eingefüllt;
- f) Bleiacetatpapier;
- g) Reagenzpapier einschließlich Farbskala;
- h) Thermometer, Meßbereich 0 bis 30 °C, Skalenwert 0,2 °C;
- i) Transportkiste.

Für alle anderen im Abschnitt 9.1 genannten Untersuchungen hängen Geräte und Chemikalien vom Umfang und Zweck der Untersuchung ab.

9.3.2 Durchführung

Die Wasserprobe muß aus frisch angesammeltem Wasser entnommen werden. Abgestandenes oder verschmutztes Wasser ist vorher abzupumpen. Dies ist in einwandfreier Weise nur bei der Entnahme aus Brunnen und Grundwassermeßstellen möglich; bei Bohrlöchern ist der erforderliche Wasserzufluß nur in einem grobsandig kiesigen Grundwasserleiter ohne Feinanteile gegeben. Für eine einwandfreie Entnahme aus Bohrlöchern müssen darüber hinaus folgende Bedingungen erfüllt sein:

- a) kein Zufügen von Fremdwasser zum Druckausgleich;
- b) kein Zufluß von Wasser von der Oberfläche oder aus anderen Grundwasserstockwerken (bei nicht dicht abgesetzten Rohrtouren in einzelnen Sperrschichten; siehe Abschnitt 5.3);
- c) kein Zutritt von Luft durch Einfahren von Bohrwerkzeugen.

Die Entnahme kann auf zwei Arten durchgeführt werden:
Entnahme mittels Schöpfgerät
Das Schöpfgerät muß möglichst ohne Wirbelbildung abgesenkt werden. In der vorgesehenen Tiefe muß das Wasser durch einen Boden- oder Seiteneinlauf ohne Wirbelbildung in das Gerät einfließen können. Während des Entnahme- und Füllvorgangs ist darauf zu achten, daß sich das Wasser nicht mit Luft vermischt und möglichst wenig mit Luft in Berührung kommt.

Entnahme mittels einer Pumpe

Bei Wasserständen bis zu 6 m unter Rohroberkante können handbetriebene Flügelumpen eingesetzt werden. Besser sind vakuumfreie Tauchpumpen mit eigenem Kraftantrieb. Bei Peilrohren setzt dies einen Innendurchmesser ≥ 50 mm voraus.

Zur Entnahme wird der Schlauch am Entnahmehahn oder Ablaufrohr der Pumpe befestigt. Das andere Schlauchende muß bis auf den Grund der Flasche geführt werden. Wenn Wasserproben in Verbindung mit größeren Förderleistungen (Pumpversuch, Wasserabsenkung) entnommen werden, so ist die Zapfstelle unmittelbar am Brunnen anzuordnen.

Die Flaschen, die kein Marmorpulver oder Zinkacetat enthalten, müssen unmittelbar vor dem Füllen gründlich mit dem zu untersuchenden Wasser ausgespült werden, wobei das Spülwasser nicht wieder in die Entnahmestelle zurückgeschüttet werden darf.

Die Flaschen sind so zu füllen, daß nach dem Verschließen eine kleine Luftblase zurückbleibt.

Die mit Marmorpulver und die mit Zinkacetat versehenen Flaschen sind nach dem luftdichten Verschluß intensiv zu schütteln.

9.4 Ergänzende Feststellungen an Ort und Stelle

Die Temperatur des Wassers ist möglichst im Pumpenstrom oder im Sondergerät auf 0,1 °C zu messen.

Zum qualitativen Nachweis eines etwaigen Sulfid- oder Schwefelwasserstoffgehalts kann Bleiacetatpapier verwendet werden. Das Papier ist mit dem Wasser zu benetzen. Sulfide oder Schwefelwasserstoff führen zu einer hell- bis dunkelbraunen Färbung.

Zur überschläglichen Bestimmung des pH-Wertes wird Reagenzpapier mit dem Wasser benetzt und die nach etwa 30 s auftretende Färbung mit der Farbskala verglichen, auf der die pH-Werte abgelesen werden können.

9.5 Kennzeichnung und Versand

Die Probenflaschen sind an Ort und Stelle mit Aufkleber und Anhänger in Anlehnung an DIN 4021 Teil 1, Ausgabe Juli 1971, Abschnitt 6.2.3 zu kennzeichnen; zusätzlich ist anzugeben, ob und welche Zusätze die Flaschen enthalten.

Für jede Wasserprobe ist ein Entnahmeprotokoll anzufertigen, das Angaben über die zur Wasserprobe gehörige Anzahl und Art der Flaschen, die Temperatur des Wassers, die Reaktion am Bleiacetatpapier sowie den bei der Entnahme ermittelten pH-Wert enthält.

Die Proben sind möglichst unverzüglich nach der Entnahme an die Untersuchungsstelle zu senden. Sie müssen gegen Wärme, Frost, Licht und Bruch geschützt werden. Proben, die später als eine Woche nach der Entnahme bei der Untersuchungsstelle eingehen, sind für eine Untersuchung unbrauchbar.

10 Messung der Grundwasserbewegung

10.1 Notwendigkeit

Verschiedene Verfahren des Tiefbaus setzen nicht nur die Kenntnis der Grundwasserstände, sondern auch der Fließrichtung und der Strömungsgeschwindigkeit des Grundwassers voraus. So kann eine zu große Strömungsgeschwindigkeit die Anwendbarkeit von Injektionsmaßnahmen behindern oder die Durchführbarkeit von Gefriergründungen in Frage stellen. Auch die Aggressivität des Grundwassers gegen Bauteile ist in bewegtem Grundwasser größer als in stehendem. Das Messen der Grundwasserbewegung kann aber auch zur Überprüfung der Wirksamkeit von Abdichtungsmaßnahmen notwendig werden.

10.2 Fließrichtung

Die Fließrichtung des Grundwasserstroms entspricht der Richtung des Gefälles der Grundwasseroberfläche beziehungsweise der Grundwasserdruckfläche. Sie ergibt sich aus den Grundwasserständen in mindestens drei Meßstellen, die im Dreiecksverband angeordnet sind. Bei schnell veränderlichen Grundwasserständen müssen die Messungen in einer möglichst kurzen Zeitspanne durchgeführt werden.

Anmerkung: Bei einer großen Anzahl von Meßstellen in flächenhafter Anordnung lassen sich die Grundwasserhöhenkurven (Grundwassergleichen: Kurven gleicher Grundwasserstände, bezogen auf NN zu einem bestimmten Zeitpunkt) zeichnen. Die Fließrichtung verläuft rechtwinklig zu den Grundwassergleichen.

Nach der „Einbohrloch-Methode“ mit Hilfe von Isotopen-, Temperaturmessungen oder ähnlichem, läßt sich die Fließrichtung nur für einen eng begrenzten Raum oder eine örtliche Abweichung von der allgemeinen Fließrichtung ermitteln.

10.3 Strömungsgeschwindigkeit

Zur Feststellung der Strömungsgeschwindigkeit wird das Grundwasser in einem Bohrloch (Geberloch) mit einem Markierungsstoff geimpft und sein Erscheinen in weiteren Bohrlöchern (Empfängerlöcher) beobachtet. Die Empfängerlöcher sollen in Fließrichtung des Grundwassers angeordnet werden. Wegen der Inhomogenität des Bodens stimmen vielfach die örtlichen Fließrichtungen nicht mit der allgemeinen Fließrichtung überein. Deshalb werden neben einem in der allgemeinen Fließrichtung gesetzten Empfängerloch beidseitig weitere Empfängerlöcher angeordnet.

Der Abstand zwischen Geberloch und Empfängerlöchern ist so zu wählen, daß das Erscheinen des Markierungsstoffs in wenigen Stunden bis Tagen erwartet werden kann (je nach Bodenart 3 bis 30 m). Alle Löcher sind in den interessierenden Horizonten mit Filtern auszubauen.

Als Markierungsstoff kommen im wesentlichen Farbstoffe, Salze oder radioaktive Isotope in Frage. Er wird entweder über die Höhe des Grundwassers beziehungsweise Filters gleichmäßig verteilt oder in einer durch Einfach- oder Doppelpacker begrenzten Strecke eingegeben. Hierbei muß sichergestellt sein, daß der Wasserstand im Geberloch nicht verändert wird. Der Versuch ist auch nur auswertbar, wenn während der Versuchsdauer annähernd stationäre Strömungsverhältnisse herrschen.

Das Eintreffen des Markierungsstoffs in den Empfängerlöchern wird bei Farbstoffen durch Probenentnahme, bei Salzen durch Messen der Leitfähigkeit des Wassers mit ergänzender Probenentnahme und bei radioaktiven Isotopen mit Hilfe von Zählrohren beobachtet. Zweckmäßigerweise wird in jedem Empfängerloch bei Untersuchungen mit Salzen oder Isotopen zur gleichzeitigen Beobachtung des jeweiligen Filterbereichs eine Kette von Meßwertgebern eingehängt.

In allen Fällen sollte der Gehalt des Grundwassers an Markierungsstoffen vor Beginn der Impfungen ermittelt werden.

Festgestellt werden der Zeitpunkt des ersten Erscheinens des Markierungsstoffs und der Zeitpunkt des Auftretens seiner größten Konzentration. Das Meßergebnis kann dadurch fehlerbehaftet sein, daß der Markierungsstoff von den Bodenteilchen teilweise absorbiert wird oder sich unabhängig von der Wasserbewegung auch durch Diffusion ausbreitet. Zur Korrektur können im Bedarfsfall, vor allem in feinkörnigen Böden und bei sehr geringen Strömungsgeschwindigkeiten, die Abweichungen aufgrund von Eichversuchen bestimmt werden.

Die größte Abstandsgeschwindigkeit v_a wird aus der Zeitspanne Δt zwischen Einbringen des Markierungsstoffs und dem ersten Erscheinen im Empfängerloch sowie aus dem Abstand Δl zwischen Geber- und Empfängerloch berechnet. Sie ist:

$$\max v_a = \frac{\Delta l}{\Delta t}$$

Wegen der Inhomogenität der Böden können die Meßwerte $\max v_a$ zwischen einem Geberloch und mehreren Empfängerlöchern sehr unterschiedlich sein. Auch ist oft in ein und demselben Meßprofil die größte Abstandsgeschwindigkeit um ein Vielfaches größer als die mittlere Abstandsgeschwindigkeit, die sich aus dem Auftreten der größten Konzentration ergibt.

Anmerkung: Die Abstandsgeschwindigkeit v_a ist nicht zu verwechseln mit der Bahngeschwindigkeit eines Grundwasserteilchens oder der Filtergeschwindigkeit eines Grundwasserkörpers (siehe DIN 4049 Teil 1). Sie kann nicht zur Bestimmung des Durchlässigkeitskoeffizienten benutzt werden.

Zur Ermittlung der Abstandsgeschwindigkeit in örtlich eng begrenzten Bereichen kann nach der „Einbohrloch-Methode“ verfahren werden. Bei ihr wird das Grundwasser in einem Bohrloch zwischen Doppelpackern auf begrenzter Strecke mit einem Markierungsstoff geimpft. Gemessen wird die zeitliche Abnahme der Farbintensität, der Salzkonzentration oder der Radioaktivität des Markierungsstoffs in dem durch den Doppelpacker und die Bohrlochwandung begrenzten Wasservolumen. Aus diesen Meßwerten kann auf die Abstandsgeschwindigkeit des Grundwassers in der Umgebung des Bohrlochs und in dem betrachteten Horizont geschlossen werden.

11 Aufschluß der Wasserverhältnisse im Fels

11.1 Grundsätze

11.1.1 Allgemeines

Für viele Bauvorhaben sind Kenntnisse über die Wasserverhältnisse im Fels notwendig. Da die Wasserverhältnisse im Fels entscheidend durch geologische Vorgänge bedingt sind, müssen bei ihrer Erkundung die geologischen Gegebenheiten im besonderen berücksichtigt werden.

Anmerkung: Sowohl bei Wasserentzug als auch bei Wasserzufuhr können im Fels große hydrostatische Druckhöhen ab- oder aufgebaut und dadurch Verformungen oder Bewegungen (Rutschungen) ausgelöst werden. Zur Beurteilung wichtiger Probleme, wie zum Beispiel der Standsicherheit oder Durchlässigkeit, sollten Wasserdruck, Fließrichtung und Wassermenge bekannt sein.

Fels ist gewöhnlich ausgeprägt inhomogen und anisotrop (siehe DIN 4021 Teil 2, Ausgabe Februar 1976, Anmerkung zu Abschnitt 5.1). Er besteht aus Klüftkörpern, die von Trennflächen unterschiedlicher Entstehung, Richtung, Häufigkeit, Öffnungsweite und Rauigkeit begrenzt werden. Störungen oder Verwerfungen, an denen die Schichten versetzt sind, stellen im allgemeinen besonders wirksame Trennflächen dar.

Bei löslichen Gesteinen (Kalkstein, Steinsalz, Gips beziehungsweise Anhydrit und anderen) entstehen durch chemische Lösung und mechanische Erosion häufig größere Hohlräume (Karst), die durch röhrenartige Verbindungskanäle eine stärkere Wasserwegsamkeit hervorgerufen. Bei weniger verfestigtem Sandstein, Mergelstein, Tonstein und so weiter sind ähnliche Erscheinungen möglich.

11.1.2 Durchlässigkeitseigenschaften (Wasserwegsamkeit)

Im Fels ist grundsätzlich zwischen der Durchlässigkeit des Gesteins und der des Gebirges (siehe DIN 4021 Teil 2, Ausgabe Februar 1976, Tabelle) zu unterscheiden. Für die Wasserwegsamkeit im Gestein sind die Porenverbindungen und für die im Gebirge zusätzlich die Trennflächen maßgebend.

Hinsichtlich der Durchlässigkeit von Fels kann man von drei Modellvorstellungen ausgehen:

- Das Gestein ist undurchlässig, das Gebirge aber durchlässig; das Wasser bewegt sich auf den vorhandenen Trennflächen (Klüfte, Spalten und so weiter).
- Auch das Gestein ist durchlässig; das Wasser bewegt sich sowohl in den Poren, als auch in den viel durchlässigeren Trennflächen.
- Das Gestein ist durchlässig oder undurchlässig; das Gebirge enthält aber außer den Trennflächen kanalartige Hohlräume, zum Beispiel bei Verkarstung, in denen die Wasserbewegung bevorzugt stattfindet.

Entsprechend der Erstreckung, Öffnungsweite und Ausbildung der Trennflächen ist der Fels meist in mehreren Richtungen unterschiedlich durchlässig.

Da Klüfte, Spalten oder sonstige Trennflächen und Hohlräume nicht immer miteinander in Verbindung stehen, kann die Druckhöhe in benachbarten Trennflächen oder Hohlräumen unterschiedlich sein.

Durch Belastung, Felsausbruch oder Änderung des Wasserdrucks und damit des Spannungszustands wird die Durchlässigkeit des Gebirges oft entscheidend verändert. Ebenso können Verwitterung und Ausspülung die Durchlässigkeit beeinflussen.

11.2 Aufgaben der Erkundung

Die Erkundung der Wasserverhältnisse im Fels soll Aufschluß bringen über:

- Vorkommen und Verbreitung vorhandener Grundwassersysteme;
- Druckhöhen in verschiedenen Grundwassersystemen;
- Höhenlage einer freien Grundwasseroberfläche und ihrer Schwankungen;
- Wasseraufnahmefähigkeit;
- Richtung und Geschwindigkeit der Wasserbewegung, Wassermengen;
- Chemische Zusammensetzung des Grundwassers;
- Hydraulische Einflüsse auf das Gebirge bei Änderung der natürlichen Grundwasserverhältnisse in Verbindung mit einer Baumaßnahme.

11.3 Bestimmung der Wasserverhältnisse

11.3.1 Allgemeines

Erkundungsstollen, Schächte, Brunnen und Bohrungen werden in ihrer Richtung unter Berücksichtigung der geologischen Verhältnisse und der Geometrie der Trennflächen so angelegt, daß sie möglichst viele der für die Durchlässigkeit maßgebenden Trennflächen erfassen.

11.3.2 Messen der Wasserstände

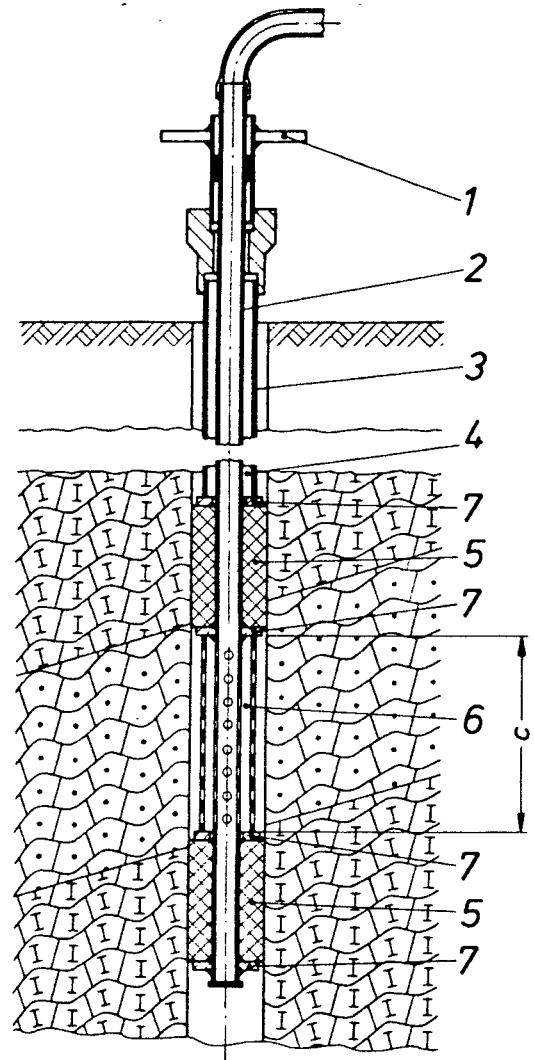
Da bei Felsbohrungen in der Regel mit Spülung gearbeitet wird, bleibt das Antreffen von Wasser beim Bohren meist

unbemerkt. Diesem Mangel kann unter Umständen durch Bohren mit Luftspülung abgeholfen werden.

Eine Messung des Wasserstands, der sich im Bohrloch ausspiegelt, ergibt oft kein ausreichendes Bild über die Wasserverhältnisse im Fels, doch sollte auf solche Messungen trotzdem nicht verzichtet werden. Hierzu ist das Bohrloch zu einer Grundwassermeßstelle auszubauen, wobei Abschnitt 7.2 sinngemäß zu beachten ist. Auch in standfestem Gebirge ist der Ausbau erforderlich, sofern Wasserstandsbeobachtungen über eine längere Zeitspanne durchgeführt werden sollen.

11.3.3 Messen der Druckhöhen

Zur Messung der Druckhöhe des Wassers in bestimmten Trennflächensystemen sind diese jeweils getrennt zu erfassen. Dies geschieht mit Hilfe von unten geschlossenen Peilrohren, deren gelochte Filterstrecke beidseitig mit Packern (sogenannte Doppelpacker) gegen die Bohrlochwandung dicht abgeschlossen wird (siehe Bild 7).



- | | |
|---------------------|---|
| 1 Spreizvorrichtung | 6 Geschlitzter Abschnitt des Innen- und Außenrohrs innerhalb der Meßstrecke |
| 2 Packerinnenrohr | 7 Klemmbacken |
| 3 Packeraußenrohr | c Meßstrecke |
| 4 Ringraum | |
| 5 Gummimanschette | |

Bild 7. Doppelpacker

Anmerkung zu Bild 7: Die beiden Gummimanschetten des Doppelpackers werden durch Zusammenpressen infolge Verkürzung des Klemmbackenabstands mit Hilfe der Spreizvorrichtung zwischen Packerinnen-

und -außenrohr dicht gegen die Bohrlochwand gepreßt. Dadurch wird der zwischen beiden Gummimanschetten befindliche Bohrlochabschnitt nach oben und unten abgeschlossen (Meßstrecke c).

Aus Trennflächen, die von der Bohrlochwand angeschnitten werden, dringt Grundwasser in die Meßstrecke c ein und kann durch das geschlitzte Innen- und Außenrohr entsprechend seiner Druckhöhe im Innenrohr ansteigen. Nach Eintreten des Beharrungszustands kann der Wasserspiegel im Packerinnenrohr mittels Brunnenpfeife oder Lichtlot gemessen werden.

Die Packer sollen an solchen Stellen angeordnet sein, an denen der Fels in Längsrichtung des Bohrlochs nicht geklüftet ist, das heißt, die Packer dürfen nicht umläufig sein. Ist diese Forderung nicht erfüllt, so muß der Bohrlochabschnitt außerhalb der Packer für die Messung mit Zement oder ähnlichem verschlossen werden.

Die Gummimanschetten müssen zur Verhinderung einer Umläufigkeit mindestens 0,4 m lang sein.

Für Messung ist der Beharrungszustand des Wasserspiegels im Peilrohr abzuwarten. Bezüglich der Messung siehe Abschnitt 7.3. Ist das Peilrohr oberhalb der Meßstrecke abgedichtet, so kann der Wasserdruck, der sich innerhalb der Meßstrecke aufbaut, mit einem Wasserdruckgeber (Luftdruckwaage oder Wasserdruckgeber mit elektrischer Fernübertragung) gemessen werden (siehe Bild 8).

Anmerkung zu Bild 8: Nach dem Anpressen der Gummimanschette an die Bohrlochwand ist die Meßstrecke c nach oben abgedichtet, durch das offene Packerinnenrohr führt das Meßröhrchen in den entsprechenden Bohrlochabschnitt. Der Grundwasserspiegel steht oberhalb der Meßstrecke c im Packerinnenrohr 2 sowie im Meßröhrchen 6. Der Druck der Wassersäule an der Unterkante des Meßröhrchens kann bestimmt werden, indem das Wasser im Meßröhrchen mittels Druckluft bis an die Unterkante des Meßröhrchens heruntergedrückt wird. Dies geschieht durch stufenweises Steigern des Luftdrucks im Beobachtungsglas 8. Bei jeder Steigerung treten im Beobachtungsglas Luftbläschen aus. Solange bei gleichbleibendem Druck das Aufsteigen von Luftbläschen mit der Zeit aufhört, ist dieser Druck kleiner als derjenige der Wassersäule. Übersteigt der Luftdruck den Druck des Grundwassers an der Meßröhrchenunterkante, tritt dort ständig Luft aus, was sich durch ein gleichbleibendes Perlen im Beobachtungsglas bemerkbar macht. Der Luftdruck in diesem Zustand wird auf dem Diagramm des Druckschreibers abgelesen.

Sollen die Druckhöhen in verschiedenen Trennflächensystemen in einem Bohrloch gleichzeitig gemessen werden, so ist die Verwendung von Mehrfachpackern zweckmäßig. Druckhöhen über Bohrlochoberkante können in Standrohren oder mit Manometern gemessen werden, während bei Druckhöhen unter Bohrlochoberkante der Einsatz einer Luftdruckwaage oder von Wasserdruckgebern mit elektrischer Fernübertragung notwendig wird.

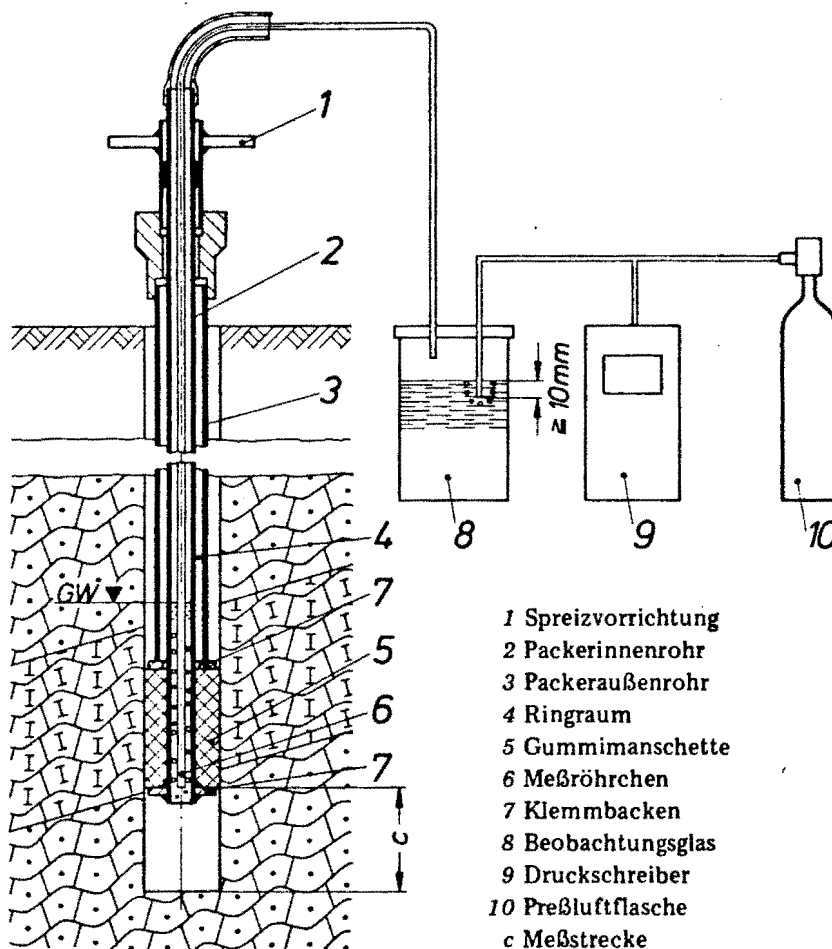


Bild 8. Wasserdruckmessung durch Luftdruckwaage

11.3.4 Messen der Wasseraufnahmefähigkeit

Bei Wasserabpreßversuchen zur Bestimmung der Wasseraufnahmefähigkeit soll in der Regel eine Prüfung von 5 m langen Bohrstrecken mit Einfachpackern vorgenommen werden (siehe Bild 9). Vielfach wird von der jeweiligen Bohrlochsohle ausgehend der Packer auch stufenweise um 1 m zurückgezogen und Wasser mit vorher festgelegten Drücken eingepreßt; dabei ist der vorhandene Grundwasserstand zu berücksichtigen. Der festgestellte Unterschied der eingepreßten Wassermengen (jeweils nach Eintritt des Beharrungszustands über eine Dauer von 10 Minuten gemessen) gibt ein Maß für die Wasseraufnahme der einzelnen Meterstufen.

Durch Wiederholung der Wasserabpreßversuche mit verschiedenen Druckstufen kann ein Aufschluß über mögliche Veränderungen der hydraulischen Verhältnisse im Gebirge in Verbindung mit Baumaßnahmen gewonnen werden.

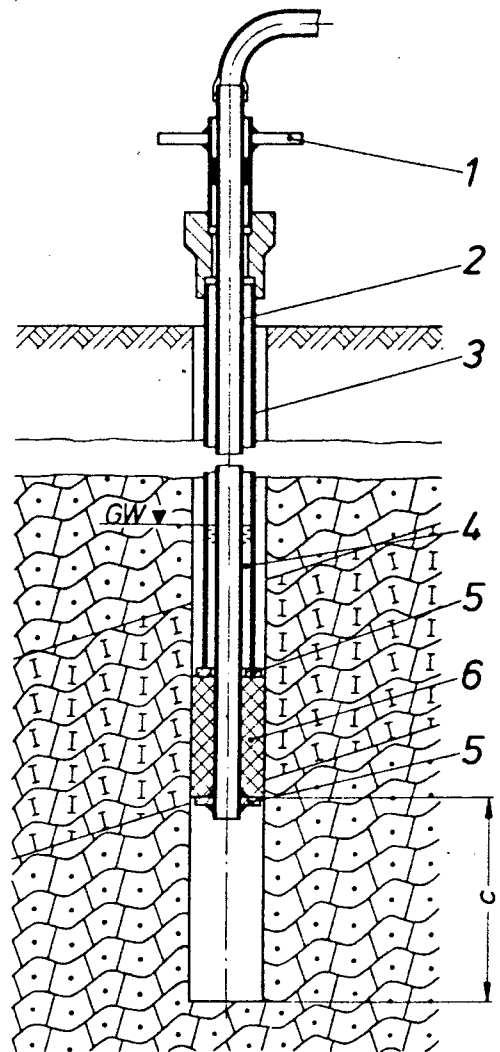
Anmerkung zu Bild 9: Nach Einführen des Packers in das Bohrloch wird durch Verkürzung des Klemmbackenabstands die Gummimanschette dicht gegen das Bohrloch gepreßt. Dadurch wird das gesamte Bohrloch nach oben abgedichtet. Der Einpreßdruck wirkt auf die Meßstrecke c.

11.3.5 Ergänzende Untersuchungen

Zur Bestimmung von Fließrichtung und Strömungsgeschwindigkeit wird sinngemäß nach den Abschnitten 10.2 und 10.3 verfahren.

In Verbindung mit der Untersuchung von Bohrkernen, durch Beobachtung der Bohrlochwand mit einer optischen Bohrlochsonde oder andere Bohrlochmessungen können zusätzliche Angaben über die Wasserführung in den Trennflächensystemen und ihrer Geometrie gewonnen werden (siehe DIN 4021 Teil 2, Ausgabe Februar 1976, Tabelle, Spalte 15).

Bei der Entnahme von Wasserproben zur Untersuchung der chemischen Zusammensetzung des Grundwassers ist sinngemäß nach Abschnitt 9 zu verfahren. Dabei ist zu beachten, daß verschiedene Trennflächensysteme Grundwasser unterschiedlicher chemischer Zusammensetzung enthalten können. Bei der Entnahme sind die Systeme deshalb durch Packer zu trennen.



- | | |
|---------------------|-------------------|
| 1 Spreizvorrichtung | 4 Ringraum |
| 2 Packerinnenrohr | 5 Klemmbacken |
| 3 Packeraußenrohr | 6 Gummimanschette |
| | c Meßstrecke |

Bild 9. Einfachpacker

4. BASIC LAWS OF GROUNDWATER BEHAVIOUR

4.1 Groundwater can appear in different ways. The goal of this paper is to investigate the different groundwater conditions, at varying depths. The investigation of groundwater can be conducted in holes whether dug for shallow water or in wells and mine shafts used for deeper water.

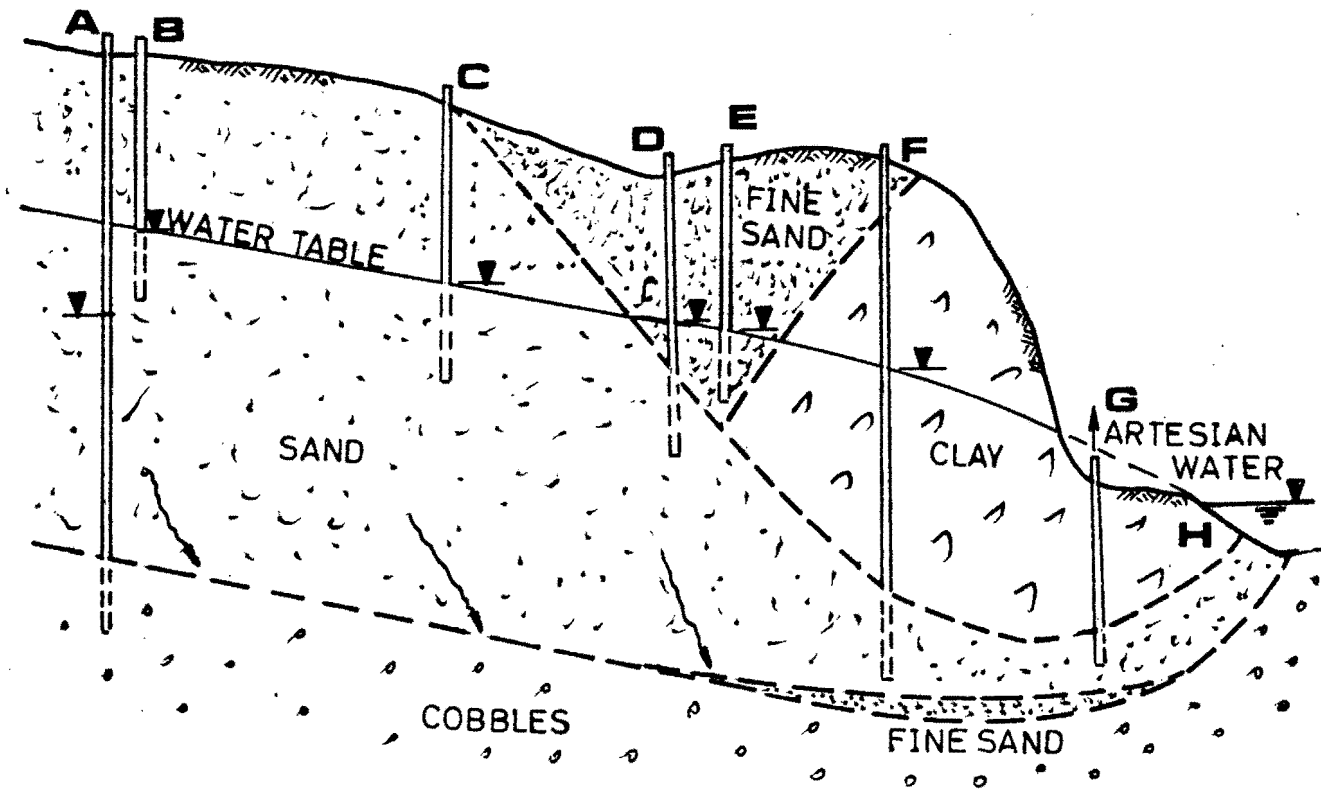
Comments on Figure 57 : The groundwater metering points show the groundwater level of each layer in which the filter depth of the different boreholes are located. In the pipes B and C the ends transverse below the water table, which is a freely formed surface. In pipes D, F and G the groundwater is under pressure, the groundwater will have the same pressure head as if it had been free forming, hence could exhibit artesian conditions. In pipe A the water level is lower than the water table because the end of the pipe lies in a cobble layer which underlies the saturated upper sand layer; because of the pressure difference the water will drop into the lower cobble layer. In the sandy silt at pipe E the water level will be established at the water table, but at a slower rate than in pipes B and C, when the pressure difference has equilised.

Comments on Figure 58 : Impermeable material can separate the groundwater into three different zones. Near the pipes C and D and between the pipes F and G the surface water percolates underground. This water then accumulates on the silt layer between the pipes F and G and also on the clay layer between pipes F and G.

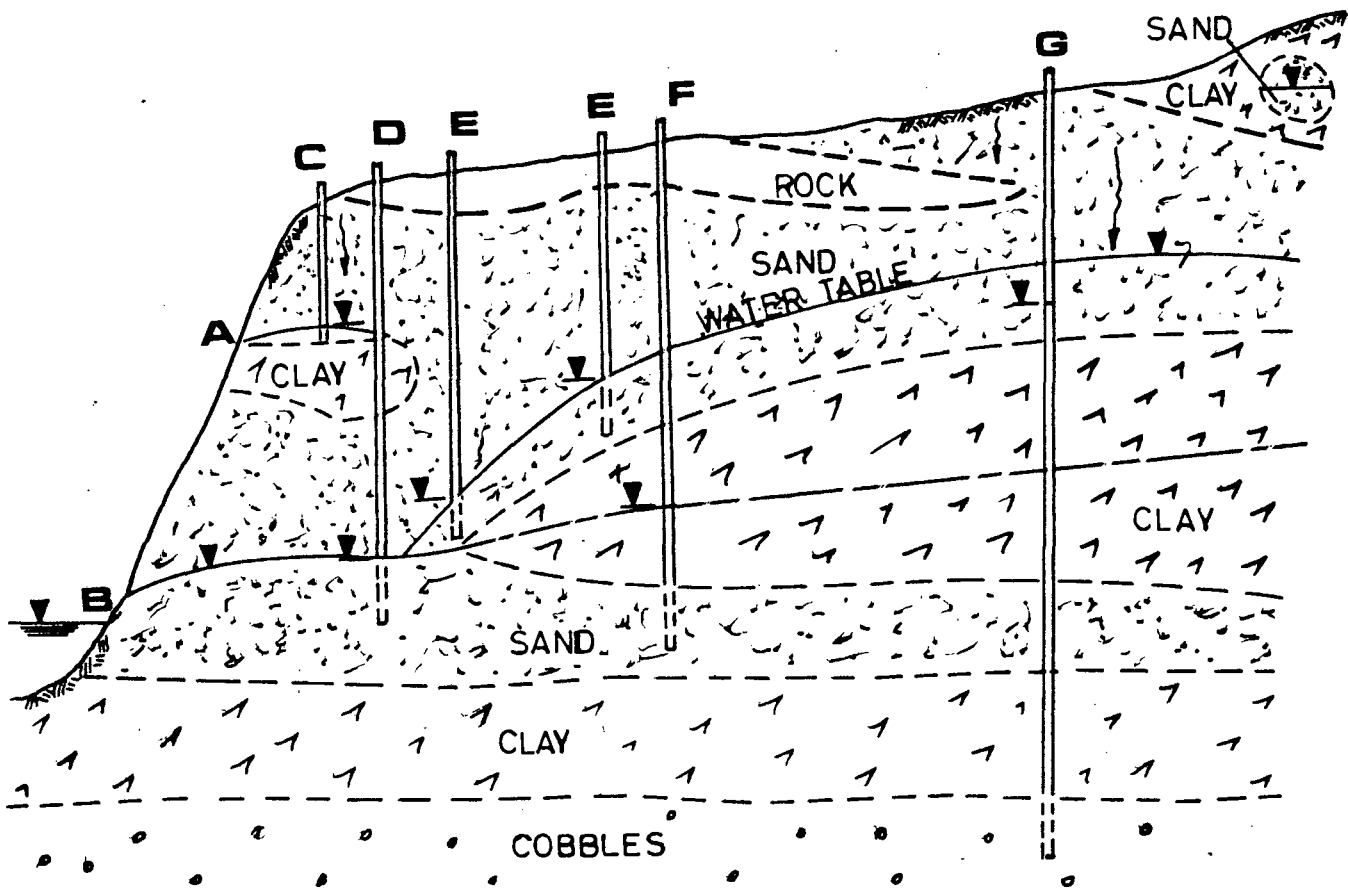
If large enough deposits of clay occur, as between points A and H, a perched water table may form. The groundwater between pipes C and D will rise to the surface at point A and will feed the adjacent water zone back to point H. At pipes E and E¹ note the water table of the first groundwater zone. The groundwater of the second zone flows towards a stream at point B. The groundwater at pipe F is under pressure whilst pipe D is not under pressure. The third water zone is always under pressure, the pressure head being shown by the water level in pipe G, and is not influenced by the groundwaters of the first and second zones.

Comments on Figure 59 : The permeable water layer (fine sand and cobbles) is divided partly by impermeable material, into two groundwater zones. The groundwater level of the total permeable material is located at the boundary between the cobbles and the clay. The lower boundary of the upper clay layer forms a ceiling of the groundwater under pressure in the lower zone.

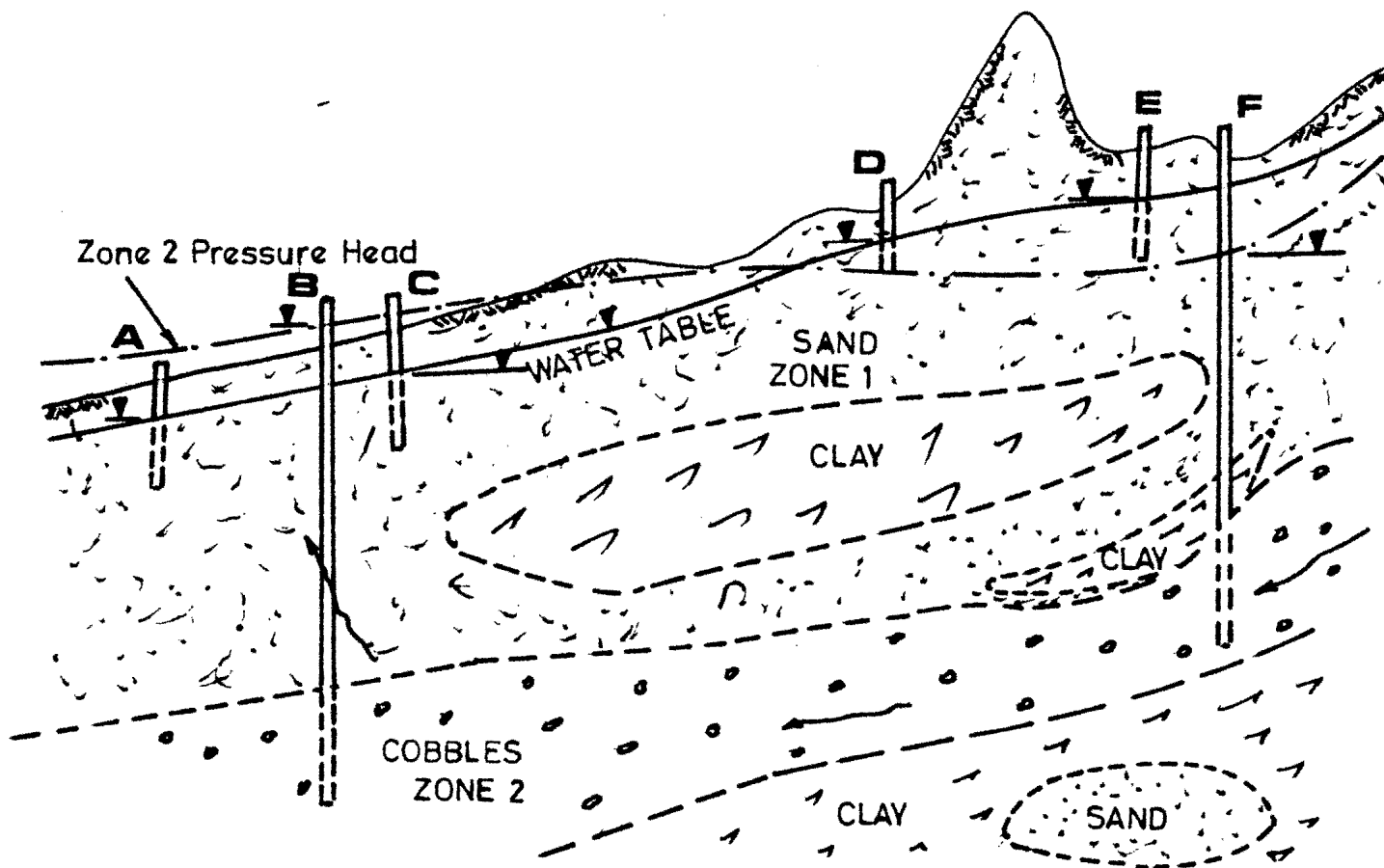
The water table is shown by the levels in the short pipes A, C, D and E and the groundwater pressure plane of the lower zone is shown by the water level in the deeper pipes B and E. Because of the lower layer of the cobbles the flow resistance is lesser than in the fine sand, causing a difference in hydraulic heads between the groundwater zones at pipe B which could result in an artesian condition. The upwards movement of the groundwater can be seen in pipes B and C.



SUB SURFACE PROFILE TYPE A



SUB SURFACE PROFILE TYPE B



SUB SURFACE PROFILE TYPE C

In the lower clay layer a sand pocket is isolated and the groundwater therein contained. Because of the small amount of additional water from the impermeable clay layer the pressure head of the enclosed water will be permanently damaged if drilled into.

- 4.2 As a result of different types of soils and the underground water conditions on the one hand and because of the preparation of the drilling on the other it becomes very difficult to establish the water pattern. Because of a local occurrence of impermeable material underground water may be contained within an area. In lagoons, where the water level fluctuates appreciably, the water table will be affected accordingly but will experience a time lag.

The way the water table is being influenced is dependent on the distance from the groundwater recording point to the lagoon as also the geological conditions occurring. This phenomena also affects the deeper groundwater zones.

The nature of the drilling may result in false information due to the presence of free water, clay, etc. While drilling for building sites the measurement of the water table should be conducted under predetermined conditions, especially where the water flows through a permeable layer. In most cases the water

level may not be determined because the drilling may be influenced by the following.

- a) the drilling operation is too rapid for accurate determinations.
- b) the lining of the borehole hinders the groundwater equilibrium, or it may not separate the groundwater zones by inter-connecting the different zones.
- c) the borehole may be clogged by fine particles of sand.

4.3 To obtain a realistic picture of the water pattern a metering device will have to be installed after the completion of the drilling, refer to Section 6, or observation boreholes should be located, refer to Section 7. It should be noted that the water pressure heads of all the water carrying zones, which are permanently separated, must be located individually. Also in relatively impermeable layers the groundwater layers should be monitored by individual boreholes. In groundwater observation boreholes, for long term monitoring, the results may prove to be unreliable because of possible corrosion of the borehole linings.

4.4 There is another way of obtaining the water content and this is done by observing the type, texture and colouring of the soils.

4.5 Before engaging an experienced driller it is important to establish the pattern of boreholes, the depth required, the type of boreholes and the number and frequency of the observations. The following should be kept in mind:

- a) the hydro-geological conditions of the observation area, of local expertise and existing data, etc.
- b) the possibility of short or long term water fluctuations, in the local water system, can occur because of variations of the supply source
- c) if open excavation is undertaken the nature of the water bearing strata.
- d) the foundation size of the structure.
- e) the lowering of the groundwater may affect adjacent structures.
- f) the contamination of the groundwater.

5. MEASUREMENT OF THE GROUNDWATER LEVEL IN BOREHOLES WHILE DRILLING FOR FOUNDATIONS

5.1 As soon as water is found in a borehole, whilst core drilling, the drilling process should be stopped and the water level recorded after a reasonable time lag. Refer to paragraph 5.4. The water level should be recorded daily at the beginning and end of each drilling shift, as well as before and after any lengthy discontinuity of drilling and finally after the end of the core drilling. Refer to paragraph 5.5

Care should be taken to record isolated water pockets above the actual water table.

Any indication which may show the presence of water, such as may cause a flow which could result in the collapse of the surrounding material, should be closely monitored. If the water in the borehole fluctuates appreciably then these fluctuations should be recorded, as also the depths and the extent of the variations. In this manner the established water level will, under certain conditions, be the correct water level. Refer to paragraph 4.2 The results and the observations recorded while drilling are still necessary to give a realistic picture of the water conditions.

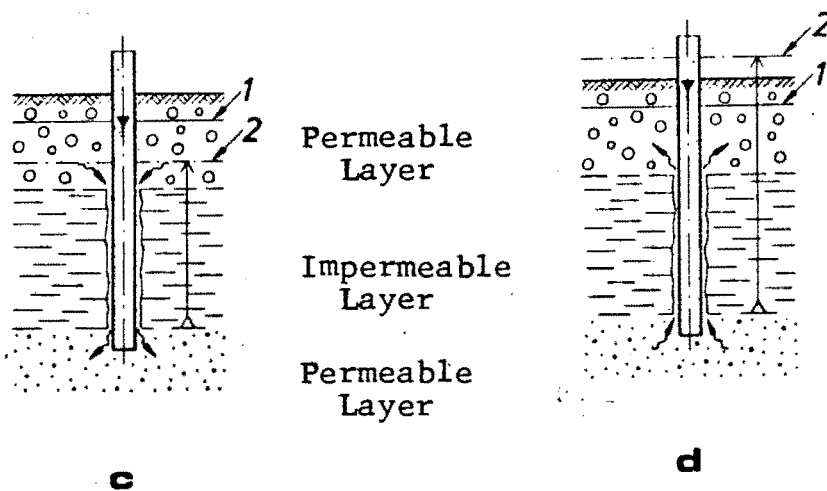
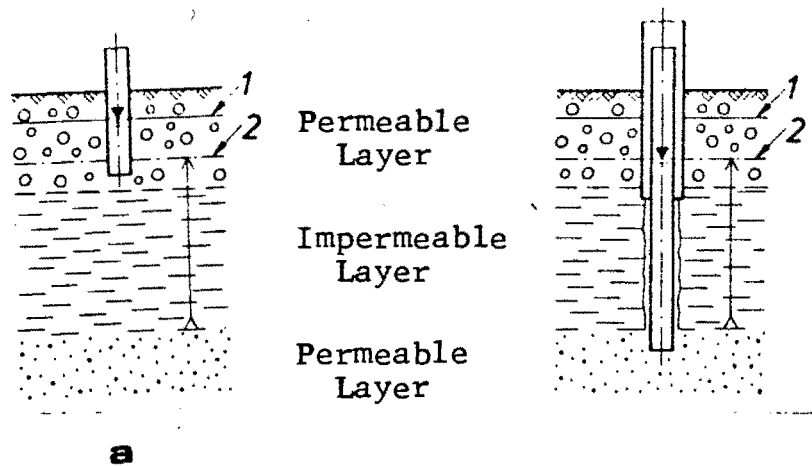
5.2 The recording of the water level, as set out in paragraph 5.1, gives an incorrect result, if while drilling the following is done:

- a) using lubricants in the drilling
- b) using water in the drilling
- c) using water under pressure
- d) using water additives
- e) when drilling depth is very rapid

Remark: With very permeable cohesionless materials, where the groundwater level is rapidly re-established, the lubricants and rapid drilling processes will not have a

detrimental affect on the results. The water level will be re-established better in a small borehole than in a large borehole, in similar material, which will be of some importance when drilling in a less permeable material. The best conditions for recording the water level is in material of good permeability (a cohesionless material, excluding fine sands or silts). The finer the particle grain of the soil the less the permeability the longer it will take to re-establish the water level in the borehole and the more pronounced the influence of the drilling process and type of equipment.

- 5.3 With respect to different soil layers containing different groundwater zones refer back to DIN 4021 Part 1 published in July 1971, Section 5.2.3. If a single sleeve pipe is used the water level of the highest groundwater zone should only be recorded, refer to Figure 60(a), because the different groundwater zones cannot be sealed in the sleeve. The water level of the lower water zones can be incorrectly obtained. Refer to Figure 60(c). If the water level of the second zone is required then the upper portion of the sleeve should be sealed in the impermeable material and a second sleeve should be introduced, refer to Figure 60(b), in which the second water level can be measured within certain parameters, as discussed in Section 5.2.



- 1 Water Table
- 2 Lower Zone Water Head
- ▼ Water Level In Pipe
- ↔ Water Flow

INTERCEPTION OF GROUNDWATER LAYERS

Remarks to Figure 60 : If under a clay layer a second groundwater zone is anticipated, but the pressure plane of the groundwater surface of the upper groundwater zone differs from the lower zone, then the pressure head of the lower groundwater zone can be correctly recorded if the zones are not connected directly through the borehole. Refer to Figure 60(a). Therefore it is necessary to use telescopic pipes with the second pipe below the sealed upper pipe and in this manner advance into the lower groundwater zone. Refer to Figure 60(b).

If the sleeve is not sealed then an incorrect recording of the pressure head, of the lower zone, will be obtained. Especially if this part of the permeable material is less permeable than the upper material, refer to Figure 60(c) and (d). Too high a pressure head is recorded if the groundwater phreatic surface of the lower zone lies below that of the upper zone. Refer to Figure 60(c). Too low a water pressure head is recorded if the groundwater phreatic surface of the lower zone lies above that of the upper zone. Refer to Figure 60(d).

- 5.4 Where a pause in drilling is necessary to enable the water table to re-establish, it will normally take about 5 minutes in permeable material. In less permeable material the re-establishment of the water level in the borehole cannot be accurately postulated because of the drilling process, and it is therefore not worthwhile interrupting the drilling because of the delay to obtain an accurate reading. In this case only through

repeated recordings during a longer interruption in drilling, may it be established by the movement of the water level, whether the water table is rising or falling.

- 5.5 All the water levels should be accurately measured within one centimetre, all the information should be dated, the time noted and all other observations made should be recorded. (Refer to paragraph 5.1 and also DIN 4022 Part 1, November 1969, paragraph 11.11(b) and Table 1, column 3, Point (2). As an example the recording devices which can be used are a light-weight plumb rod or a sonic sounder.

6. EXTENSION OF THE FOUNDATION DRILLING TO MONITOR THE GROUNDWATER LEVELS IN THE VICINITY

If the foundation drilling is to be used as a monitoring station for groundwater, then it should be controlled as set out in Section 7. For this the outside diameter of the drilling tube should not as a rule be less than 159 mm. (Refer to DIN 4021, Part 1, July 1971, Section 5.2.4).

7. THE LAYOUT OF THE GROUNDWATER MONITORING STATION IN THE SUBSTRATA

7.1 Layout

Groundwater monitoring points used to obtain the groundwater levels must include the local water levels and the temporary water occurrence, for example, trapped water, storage water and fissured water. Refer to Figure 58. The monitoring stations generally exist as single

observation boreholes, refer to Figure 61, which also could be combined to form a group.

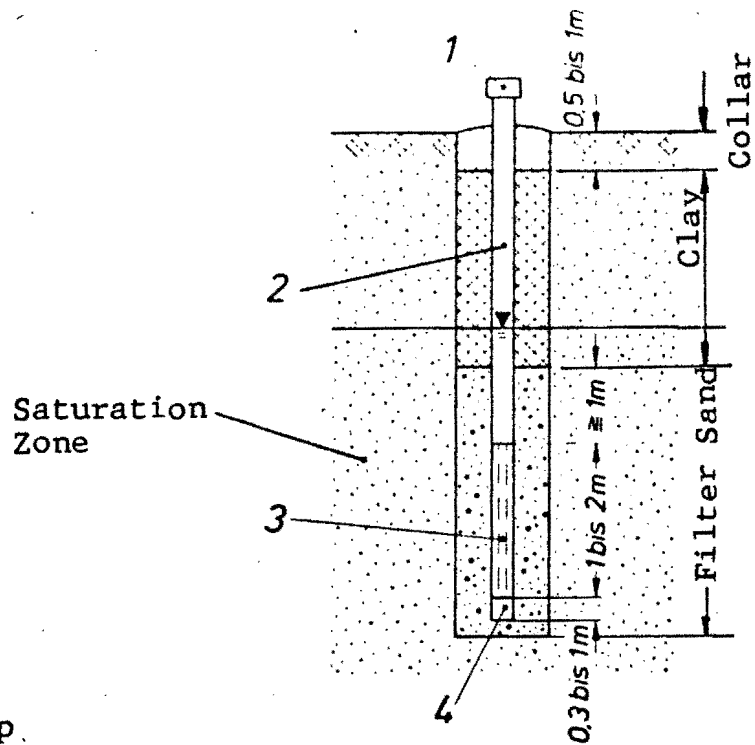
The location of the monitoring station on a building site will normally be chosen from the results of previous boreholes. If it becomes necessary to monitor the groundwater during, or subsequent to, the building operations then the location of the borehole should take this into cognizance. In certain cases it may be necessary to locate the monitoring stations away from the building area, Refer to Section 4.5(e). Try to avoid storage areas, haul roads, drainage paths and existing wells so that no damage is caused to them and also that the monitoring station is not influenced by them.

7.2 Enlargement

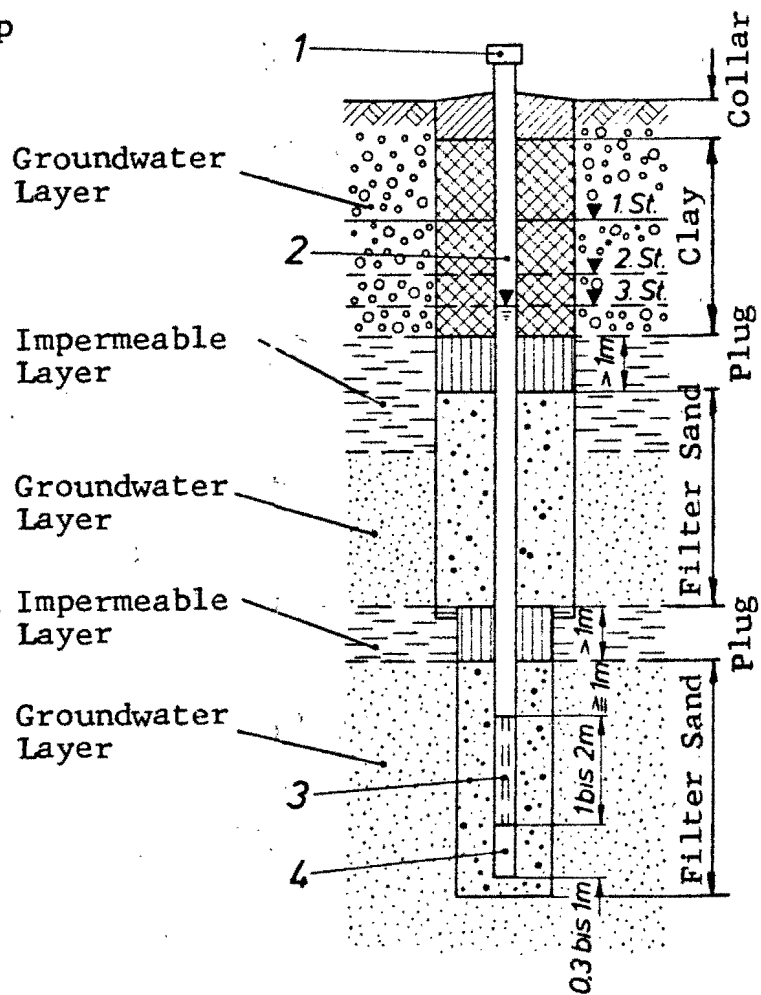
7.2.1 Basic Rules

The client should specify what must be achieved from the monitoring stations as well as the depths to be drilled. It should also be specified how many boreholes are required and at what level the sumps and filters are necessary.

The time of the observation should also be fixed before the drilling starts. This influences not only the choice of the filter and the sleeves but also, under certain circumstances, the choice of the borehole diameter and the general completion.



- 1 Cap
- 2 Sleeve
- 3 Filter
- 4 Sump



GROUNDWATER RECORDING STATION

The erection expenditure of a water gauge for a short observation period (up to one year) and a shallow depth of the groundwater level (maximum ten metres) is generally kept to a minimum, (Refer to Section 7.2.2.1 and 7.2.2.2). With a long term observation zone the monitoring station should be fitted with a good quality water gauge, (refer to Sections 7.2.2.3 and 7.2.3).

If numerous permeable layers are expected, or are found, then every groundwater zone should be separately monitored. Because of the difficulties to seal the different groundwater zones from each other different boreholes should be used for each layer. It is not advisable to install different borehole sleeves in a single borehole and should be resorted to only in special cases.

With a small quantity of groundwater in a zone the lower portion of the filter should reach down to the lower level. With a larger quantity of the groundwater in the zone the bottom of the filter should be a minimum of three metres below the expected lowest groundwater level.

7.2.2 Shallow Depths and Short Observation Time.

7.2.2.1 Piled Sleeves

With shallow depths of permeable material, down to five metres, piled sleeves can be used. Steel pipes with a filter pipe and a piling shoe should be used.

7.2.2.2 Water Bored Pipes

In soft ground with a depth of not more than ten metres, sleeves can be used which are provided with a head with water jets to sink the sleeves. Synthetic or steel slotted pipes can be used which must be protected by a filter from intrusion by sand and silt.

7.2.2.3 Hydraulic Boring Sleeves

In ground which can be hydraulically washed and has a maximum depth of 10 metres hydraulic sleeves can be used with built-in or additional hydraulic jets. Either synthetic or stainless steel pipes with slotted sections are used which should be protected by a filter against sand infiltration.

7.2.2.4 Built-in sleeves can be used in boreholes with or without linings and with cobble backfill.

This type of development should only be used if the conditions for the criteria stated in 7.2.2.1 and 7.2.2.2 do not exist. They should be handled in the same way as the recording stations used for long term observations. Refer to section 7.2.3.

7.2.3 Long Term Observations

7.2.3.1 General

For this type of development the borehole should be constructed in accordance with the lining used. The lining can under certain conditions, with the client's

permission, be omitted. The sleeves should be installed with a backfill of sand or cobbles. DIN 4021 Part 1 published July 1971, Sections 5.2, 6.2, 7 and 8 as well DIN 4022 Part 1, DIN 4023 should be referred to.

7.2.3.2 Borehole diameter

The diameter of the lining, with reference to the starting and end diameters of the interlocking pipes, are dependent on the outside diameter and the number of the sleeve sections used. The following minimum borehole diameters for sleeve pipes up to 50 mm should be used :

- a) for the use of a single sleeve 150 mm outside diameter.
- b) for the use of two sleeves 203 mm
- c) for the use of three sleeves 267 mm

It is preferable to use a single sleeve. Refer to section 7.2.1.

7.2.3.3 Preparation of the borehole for installation.

The bottom of the borehole should be cleaned before the installation of the sleeves. Boreholes that are hydraulically sunk should be washed before installation and should be prepared such that below the lowest sleeve a gap be left to allow for collapse or quicksand conditions.

7.2.3.4 Planning of the borehole installation.

Before commencing the installation, the type, connection details of the sleeves and the filter head should be determined as well as the backfilling with either cobbles or sand as mentioned in paragraph 7.2.1..

7.2.3.5 Sleeves and Filters

For the sump, filter and interlocking sleeves non-corrodable material should be used, in addition if aggressive water is encountered resistant materials should be used.

For example

- a) synthetic pipes
- b) painting or using a protective cover, (hard rubber sleeve), or galvanised steel pipes
- c) stainless steel pipes

With synthetic pipes the following should be remembered. The mechanical usability, low water and temperatures during the installation could be of paramount importance.

As an internal diameter of the sleeve of 25 to 50 mm is normally acceptable it must be remembered that if recording instruments are to be installed a minimum diameter of 100 mm should be used. Below the filter a sump pipe of 0,3 to one metre long should be used against the bottom of the hole. If a sump pipe is not used then the filter pipe should be capped at the end and in addition sand backfill introduced. The length

of the filter pipe should generally be one to two metres long with precautions being taken against a fluctuation in water level. The filter pipe can be of a fabric or a no-fines concrete surround. The grain size of the filter material should be determined by the smallest size of the surrounding soil and should generally not exceed one to two millimetres of the grain sizes.

0,5 mm to 1 mm

0,7 mm to 1,4 mm

1,0 mm to 2,0 mm

is recommended in DIN 4924. At very shallow recording points (less than 5 metres in depth) it is adequate to use graded sand, for example, between 0, to 3 mm in size.

7.2.3.6 Installation of Sleeves

The sleeves should be central down the borehole. For this purpose the sleeves below the filter pipe should have guide spacers. For the recorder the sleeves must be truly vertical. The sleeve connections should be water tight. Incorrect installation of the sleeves should be avoided.

The top of the sleeves should be levelled immediately after the installation so that the slipping of the sleeves during the installation of the borehole lining can be established in the initial stages and can be corrected.

7.2.3.7 Placing of Filter Sand in the Borehole.

To remove a possible mud film from the filter pipe, before installation of the filter sand, a bucket of water should be poured down the sleeve during the filling process of the sand, to retain the water level at a higher level in the pipe than that in the surrounding material.

The sand filter should surround the filter and end pipe (shoe) and should be a minimum of one metre above the filter pipe. The filter sand must be determined such that it will retain the in-situ soil in the water zone. During the installation of more than one sleeve the entire hole should be filled with filter sand, and different zones separated with impervious material. Refer to Section 7.3.2.8. The filter sand should be carefully inserted in small portions to avoid blockage down the pipe. With lined boreholes the lining should be installed at the same time as the filter sand. The end pipe should be between 0,3 to 0,5 metres below the filter sand during installation. By using a plumb this can be monitored.

Above the filter sand the remaining space should be backfilled with the material from the excavation or a material consisting of a similar grain size. The backfill over the filter material must however be of such quality as not to contaminate the filter sand.

7.2.3.8 Reconstruction of the Impermeable Layers

All impermeable layers which separate the different groundwater zones must be reconstructed with clay or with a bentonite cement mixture.

For the clay waterproofing a clay with a low to medium plasticity, air-dried and with a maximum modular size of one to three centimetres, should be used. The clay should be protected against moisture intrusions before placing in the borehole. The clay should be introduced using a tremi. The thickness of the clay layer depends on the water pressure above it and also the thickness of the impermeable layer; it should be a minimum of one metre. Whilst compacting the clay collar a water head should be maintained over it which must be greater than the water pressure in the lower water zone. After placing the clay the sleeve pipe should be lifted to 30 centimetres above the top of the clay layer and a time lag of at least eight hours must be observed before the next sequence is commenced.

For the bentonite cement mixture the bentonite flour with water and cement, the mix proportions being 25:30:25 respectively, should be used. This mixture should be mixed for a short time in a mixer and then pumped under pressure directly down the sleeve. The thickness of the mixture and the lifting of the

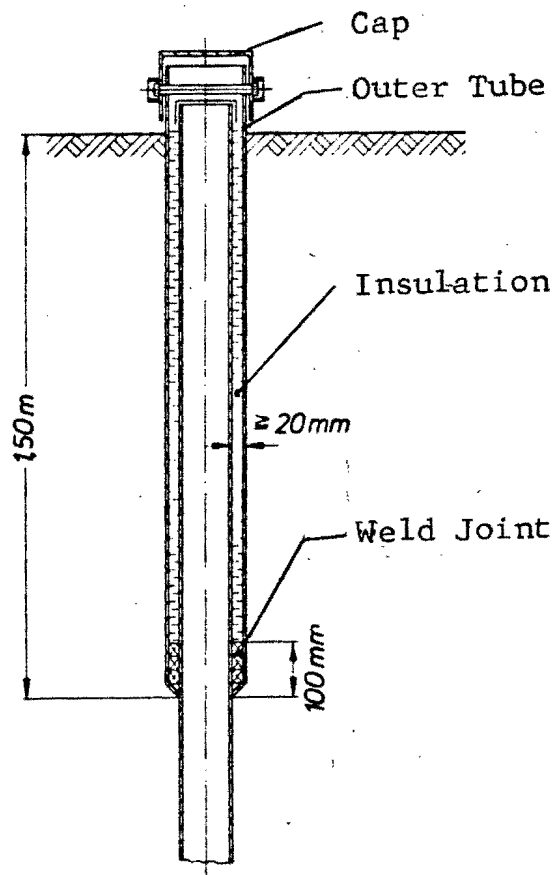
pipe will be as discussed above. The time lag using this method can be shortened to one to two hours.

7.2.3.9 Sealing of Sleeves

To prevent water from the surface getting into the borehole the top 50 cm should also be sealed with clay.

The top portion of the sleeve can protrude above the surface if it does not cause an obstruction. If the sleeve cannot protrude a cap should be used to protect it and prevent material from falling down the borehole. The borehole should also be protected against unauthorised persons. If protection chambers are built then surface water must not collect in it and should be drained away. The top sealing layer should be one to two metres thick to accommodate a chamber. The top cap should have a breather hole. Where artesian water is encountered a pressure gauge should be mounted on the cap to record the resultant upward thrust.

If more than one sleeve is inserted in a borehole the caps should be marked to prevent confusion of results. If the top layer is subject to freezing then insulation must be provided in the affected zone. Refer to Figure 62.



CAP PROTECTION AND FROST INSULATION

7.2.4 Procedure after Installation of Recording Station

After the completion of the installation of the recording station the functioning of the station must be proven. For this the groundwater level in the sleeve should be measured and then either filled with five litres of water or a certain quantity pumped out. By measuring, in short time intervals, the re-establishment of the groundwater level can be monitored. The observations should be recorded as set out below:

- a) the original groundwater level
- b) the level after adding or pumping out the water.
- c) the recording of the re-establishing levels over predetermined time intervals.

After the completion of the installation of the recording station small settlements may occur and this should be allowed to take place before determining the exact level (MSL) on the cap. The height of the cap above MSL should be determined over a period of time to establish if any further settlements take place, and at the same time determine the reliability of the recording device. By plumbing it should also be ascertained if the sleeve is still clear down to below the filter section.

7.3 Measuring of Water Level

For construction purposes the establishment of the water table must be determined as also the maximum oscillation of these levels. During the construction period it is unlikely that the maximum variations will occur as will be the case over a longer period of time. Due to the short period under observation results can be compared with other stations which have longer records hence enabling a more accurate forecast of the fluctuations which will occur.

The fluctuations which are recorded can be associated with weather changes, seasonal variations or the influence of nearby water sources. The periodical sequence of the recordings should be established in such a way that the high and low groundwater levels can be determined. All water levels should be recorded within an accuracy of one centimetre. The measuring devices normally used are either a light plumb, sonic sounder or a float system. If the total fluctuation of the water level is required then a self-measuring device should be used.

8. INDICATION OF GROUNDWATER LEVELS

Indications of long period high and low water levels as well as fluctuation limits can be established under certain conditions in building foundations, in borrowpits or in drilling cores by observing the colour of the soil, the variation of soil colour and possible concretions of soluble salts. Refer to the table below.

Table 17
WATER LEVEL CHARACTERISTICS IN SOILS

LIMITS	SOIL TYPE	CHARACTERISTICS		
		SOIL		CONCRETION
		Colour	Marks	Colour
Above the high water level (oxidation zone)	Plastic material	Brown - Yellow	-	-
	Non-plastic material	Brown - light yellow	-	-
Fluctuation Limits	Plastic material	Brown to yellow, grey, greenish	Rust spots present	Brown, dark brown to black
	Non-plastic material	Brown to yellow, grey	Rust spots and Rust strips	Brown, dark brown to black
Always under water (Reduction zone)	Plastic material	Blue, green, grey	-	If present olive
	Non-plastic material	Light blue, green, grey	-	If present olive

The high water level shows a colour sedimentation which corresponds with the lower boundary of the oxidation zone. From the position of the upper level of the reduction boundary the lower water level can be established. These characteristics can re-occur, in profile, in different water horizons. In areas of different water occurrence these can be established by the colour variation of the water. (e.g. brown and blue).

9. SAMPLING OF UNDERGROUND WATER

9.1 General

This sampling is undertaken for building purposes only. The following points should be considered:

- a) General investigation of chemical constituents in the water together with the mineral and chemical composition of the surrounding soil.
- b) Investigation for possible concrete corrosion.
- c) Testing for the acceptability of the water for concrete.
- d) Investigation for the corrosion danger to pipes.
- e) Investigation whether the water will damage the filter drains, the drainage systems etc.
- f) Investigation of the possible change in the groundwater in adjacent areas.
- g) Investigation of the chemical or biological impurities of the water resulting from nearby industrial areas.

9.2 Basic Rules for Sampling of Water

The sampling point should be located with consideration for the local hydrological and geological features. In addition, in the local industrial zones, densely populated areas, dumps for waste (chemical residue or industrial waste) and dumps of organic residue a minimum of two sampling stations should be located. In the case of more than one groundwater zone a sample should be taken from every water zone. As a rule the samples from the groundwater recording stations or from wells will give a more reliable result than from a borehole because the water changes during the drilling operation.

9.3 Sampling

9.3.1 Apparatus and Chemicals

For the sampling of the water the following apparatus and chemicals are necessary:

- a) Clean glass sampling bottle with an airtight stopper; two bottles of one litre capacity and two bottles of half a litre capacity.
- b) Pump with an outlet pipe (not made of rubber).
- c) Sampling container with a special inflow pipe.
- d) Five grams calcium carbonate, placed in a half a litre flask.
- e) Three grams zinc acetate placed in a half a litre flask.

- f) Lead acetate paper.
- g) Litmus paper with a colour card.
- h) A thermometer reading every $0,2^{\circ}\text{C}$,
over a range of $0^{\circ} - 30^{\circ}\text{C}$.
- i) Transportation containers.

For all other investigations mentioned in Section 9.1 the apparatus and chemicals will be dependent on the size and necessity of the testing programme.

9.3.2 Procedure

The water sample should be taken from freshly gathered water. Stagnant or dirty water must be removed first. This can only be done by sampling from wells or underground recording stations; in boreholes the inflow water can only be measured if the surrounding material is a highly permeable gravel with no fines. For perfect sampling from boreholes the following procedure must be adhered to:

- a) No additional water should be added for pressure equilibrium.
- b) No water should be allowed to enter from the surface or other groundwater zones.
- c) No air intrusion should be allowed during sampling operations.

The sampling can be conducted in two different ways:

Sampling with bailing equipment:

The bailing equipment should not cause turbulence during sampling. At a certain depth the water should enter through the special inlet which prevents a whirlpool turbulence. While filling the container air should not mix with the sample and the sample should have as little air contact as possible.

Sampling with a pump:

With water levels less than six metres below the ground level a hand-operated pump can be used. Submersible pumps should preferably be used. The sleeves should be greater than 50 millimetres for this purpose.

If the sampling bottles do not contain calcium carbonate or zinc acetate then the bottles should be thoroughly washed beforehand with the water to be sampled. The washing water must not be thrown back into the sampling area.

The bottle should be filled in such a manner as to leave as little air under the stopper as possible.

The bottles prepared with calcium carbonate or zinc acetate should be shaken vigorously after the sample has been added and sealed.

9.4 Additional Data at Sampling Point

The temperature of the water should be recorded in the pump stream or in the sampling container to an accuracy of $0,1^{\circ}\text{C}$.

For the qualitative testing for hydrogen sulphide or a sulphur content the lead acetate paper can be used. Sulphides or hydrogen sulphide will discolour the paper from a light to a dark brown colour.

To obtain the pH value of the water the litmus paper should be used in conjunction with a colour chart.

9.5 Marking and Despatching of Samples

The sample bottles should be labelled and a note made of the additional chemicals contained. For every water sample there should be a form indicating temperature, pH value, the reaction of the lead acetate paper and the total number of samples taken.

The samples should be despatched as soon as possible to the laboratory. The samples should be protected against heat, light, frost and breakage. Samples which arrive later than one week should not be analysed.

10. MEASURING OF GROUNDWATER MOVEMENT

10.1. Requirements

For sub-surface engineering design the engineer needs, in addition to the groundwater levels, the permeability of the soil and the rate of flow of the sub-surface water. If the flow is too high it may become impossible to inject stabilising chemicals into it or to freeze the ground. Moving groundwater is more aggressive to foundations than stationary water. The recording of the movement of the groundwater will enable adequate waterproofing to be provided for foundations.

10.2 Flow directions

The flow direction is related to the pressure head in the water zone. To determine the flow of the groundwater - three recording stations are necessary, located in a form of a triangle. Where rapidly changing groundwater levels are encountered readings must be taken almost continuously. If a large number of recording stations are located in an area it is possible to compile pressure head contours. The direction of flow will therefore be perpendicular to the pressure contours.

10.3 Flow Velocity

To establish the flow velocity the water is discoloured by injection and the reappearance in other boreholes is noted. The observation boreholes should be downstream of the injected borehole. Because of the non-homogeneous

nature of the soil local flow patterns may not correspond with the overall flow direction. As a result of this more than one borehole must be located downstream of the injected borehole.

The distance between the injected and observation boreholes may be a factor as to the time of travel of the injected dye. This may range between a few hours and a few days. The distance between these boreholes should be approximately 30 metres. All boreholes should have built in filters at the levels under the water at which the study is to be conducted.

For a monitoring agent coloration dyes, salts, or radio-active isotopes can be used. This agent must be spread above the groundwater table and above the filter in a single or double pack. It must be ensured that when this agent is included the water level does not alter. The results of the experiment are only valid if the flow is constant during the time of the observations.

To determine the arrival of the dye in the observation borehole frequent visual tests must be conducted. Where salts have been used a conductivity meter must be used. If radio active isotopes have been inserted then a monitoring apparatus to measure the radio-activity of the water will be necessary. If salts or isotopes are used then the water should be tested

beforehand in case these elements already exist in a concentrated form.

During observations the time interval between the arrival of the first diluted traces of the agent and the point at which maximum concentration is observed, must be recorded. The observation results could be suspect because the agent could be absorbed by the soil or when the agent starts to disperse rapidly. This can only be corrected in fine sandy soil with low flows. The greatest velocity, max V, can be derived from the time interval dt of the injection of the marking agent and the receiving time in the observation borehole as well as the distance dl, which is

$$\text{max } V = \frac{dl}{dt} \quad (64)$$

Because of the non-homogeneous nature of the soil the recorded max V will differ between the injection borehole and different observation boreholes. In the same profile a higher velocity may be calculated than the average velocity due to the concentration differences in the marking agent. Care should be taken not to misinterpret the different results obtained.

11 DETERMINATION OF WATER PROPERTIES IN ROCK

11.1 Basic Principles

11.1.1 General

For building purposes the properties of the water in the rock is required. The water content in the rock is dependent

on the geological formation of the rock. If meaningful results are to be obtained then the geological formations must be determined.

Depending on the fluctuation of the water in the rock the pressures in the rock will vary. These may cause instability in the rock structure. To define the problem the water content, flow pattern and water pressure should be determined.

A rock is normally a non-homogenous, non-isotropic material with fracture planes where the number of fractures and passages may vary considerably. In soluble rocks, such as kaolin, calcium, gypsum etc, where the rock has been eroded by solution cavities may occur. These cavities cause stronger water flows which in turn increase the water passage size. In a sandstone, or mudstone similar phenomena may occur.

11.1.2 Permeability Properties

Differentiation must be made in the properties of soluble and insoluble rocks. The permeability in soluble rock is dependent on the cavities and pores, whilst the permeability in insoluble rock is determined by the degree of fracture planes within the rock formation. The permeability can occur in three different ways, namely:

- a) Although the rock may be impermeable water will still flow through the formation via fracture planes.
- b) The rock may be permeable in addition to flow occurring through the fracture planes.

- c) The rock may be permeable or impermeable but water can flow through passages or cavities.

As a result of the type of fracture planes the degree of permeability of the rock formation will vary.

There may not be any connection between the cavities and the fracture planes and hence different water pressures may exist in the same formation. Where rock movements occur, because of internal or external stresses, the internal structure and fracture planes may change hence altering the permeability of the formation.

11.2 Purpose of Investigation

The investigation of the water properties in rock should yield the following results :

- a) The existence and extent of the existing groundwater systems.
- b) Pressure heads in different groundwater systems.
- c) The level of the phreatic surfaces and their fluctuations.
- d) The water absorption ability.
- e) The hydraulic influence on the formation as a result of the erection of a structure.

11.3 Determination of Water Content

11.3.1 General

In order to obtain the maximum information of the permeability in the various strata exploration shafts, boreholes and wells should be located, and inclined if possible, to accommodate the local geological formations and fracture planes.

11.3.2 Measurement of Water Levels

Normally, during rock drilling, a water based coolant is used and therefore the groundwater level cannot be easily determined. To avoid this problem air cooling may be necessary. The measuring of the water level in a borehole does not always give a conclusive result but measurements should still be conducted. With this in mind the borehole should be set up as a recording station, as set out in Section 7.2. In rock formations the recording station must be of a permanent nature to obtain more reliable long term results.

11.3.3 Measurement of the Pressure Head

The measurement of the pressure heads in various fracture planes should be individually measured. This is done with a closed sleeve pipe where the filter section is located between two packers. Refer to Figure 63.

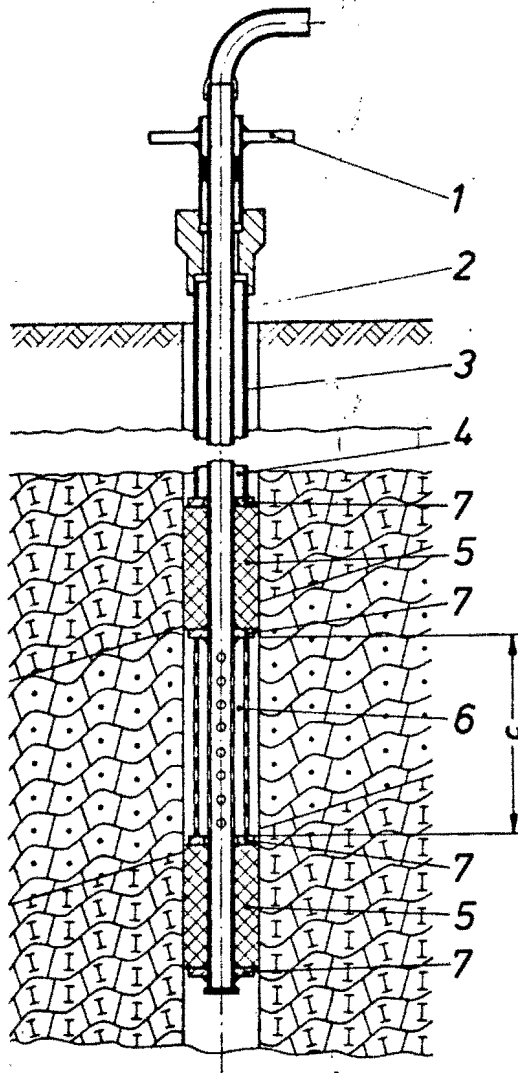
The packers should be arranged such that they are not located on a vertical fracture line. It must be ensured that water cannot bypass the packers. If this cannot be achieved concrete plugs may be necessary. The packers must be a minimum of 0,4 metres long to prevent bypassing by the water.

Before measuring the pressure head in the groundwater the water level must be allowed to re-establish and remain constant. For the procedure for attaining this equilibrium refer to Section 7.3. If the sleeve is above the strata, where measurement is required, then the water pressure, which builds up in the lower strata, can be determined by a pressure gauge. Refer to Figure 64.

If different heads are required in different rock strata multiple packers can be used. Pressure heads above the borehole ground level can be measured in vertical pipes or a manometer. The pressure heads below ground level must be measured using the apparatus shown in Figure 58.

11.3.4 Measurement of Water Absorption

In order to determine the water absorption water is forced into the borehole under pressure. The borehole test section should be a minimum of five metres and single packers should be used as shown in Figure 65. Generally the tests should be started at the bottom of the borehole and lifted in increments of one metre. Water is pumped down the



1. Screw Handle
2. Inner Tube
3. Outer Screw Tube
4. Ring Sleeve
5. Packer
6. Filter
7. Brackets

DOUBLE PACKER

borehole at pre-set pressures and a constant monitoring of the groundwater level should be maintained. The observed difference in the water quantities before and after injection, once equilibrium has been achieved, must be recorded (after equilibrium readings should be made during the following ten minutes). This will give the water absorption properties in single metre steps.

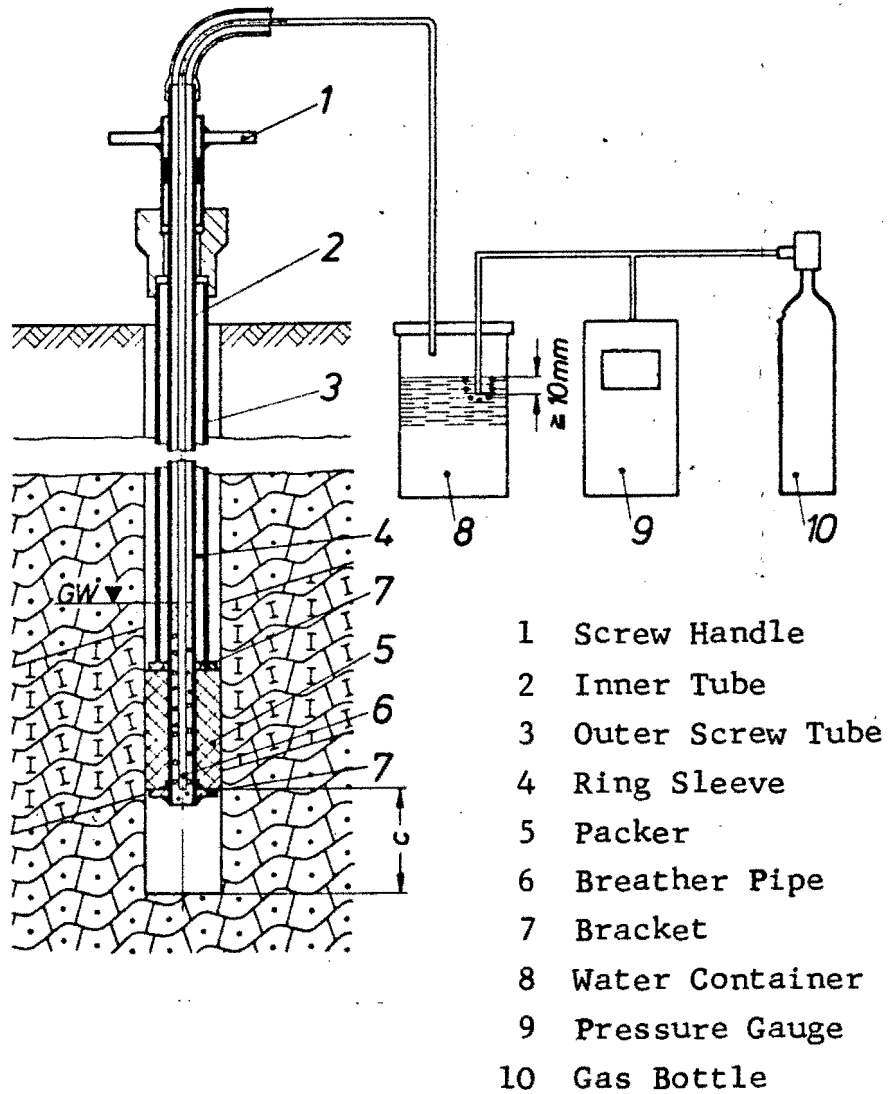
By varying the pressure on the injected water a clearer idea of the hydraulic properties of the formation can be achieved.

11.3.5 Supplementary Testing

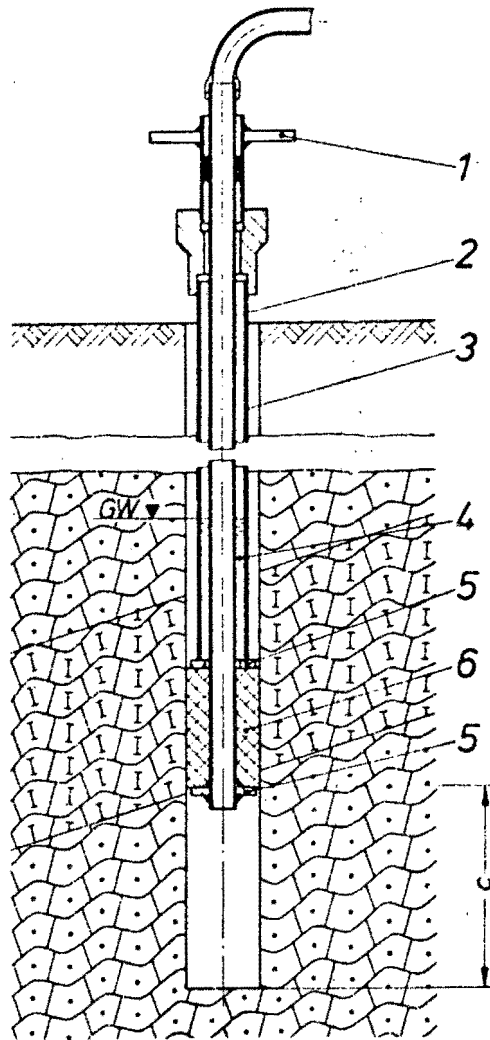
To determine the flow direction and velocity refer to Section 10.2 and 10.3.

Whilst viewing the borehole cores or borehole sides, with the use of periscopes, more information can be gleaned about the different strata, permeability and structural properties (Refer to DIN 3021 Part 2 published February 1976, Table, column 15).

When sampling groundwater for a chemical analysis Section 9 should be adhered to. It should be kept in mind that water from different strata may have different chemical properties. While sampling in each strata packers must be used to isolate them.



WATER PRESSURE APPARATUS



- 1 Screw Handle
- 2 Inner Tube
- 3 Outer Screw Tube
- 4 Filter
- 5 Brackets
- 6 Packer

SINGLE PACKER

Appendix D

Postgraduate Courses

COURSES COMPLETED IN PARTIAL FULFILMENT
OF THE M.Sc (ENG) DEGREE AT THE UNIVERSITY OF CAPE TOWN

Course	Course Number	Year Completed	Credit Value
Coastal Hydraulics	CE 525	1977	5
Coastal Engineering Practice	CE 526	1977	5
Potential Theory And Groundwater Flow	CE 529	1978	5
Urban Transportation, Planning And Modelling	CE 531	1977	5
Geotechnical Site Investigation	CE 534	1977	5
Engineering Economy	CE 535	1977	3
Transportation Theory	U. & R. Planning	1977	3
Total Course Credits			32

Quarter Thesis : 10

Course Credits : 32

Total : 42

UNIVERSITY OF CAPE TOWN

DEPARTMENT OF CIVIL ENGINEERING

M.Sc. IN CIVIL ENGINEERING

UNIVERSITY EXAMINATION : JULY 1977

CE 525 : Coastal Hydraulics

All Questions may be attempted

Time : 3 hours

Constants

Sea water density = 1025 kg/m^3

Sea water weight = 10 kN/m^3

1. A beach site has an average underwater slope of 1 in 50, and the beach material is a coarse quartz sand of relative density 2,65 and average size 1,35 mm, the shoreline being essentially straight.

Two conditions of wave attack are being considered :-

- (A) swell of 10 second period with a deep water wave height of 1,6 m approaching the beach with wave crests parallel to the shore line.
- (B) as in (A) above, but with a deep water wave incidence of 35° , (angle between wave crest and contour)

For case (A) make the following calculations :-

- (a) the wave length and wave celerity in deep water.
- (b) the water depth at which the wave begins to be affected by the presence of the sea bed.
- (c) the wavelength, celerity and height for water depths at 10 m intervals between $d=80 \text{ m}$ and $d=10 \text{ m}$, and at 1 m intervals between $d=10 \text{ m}$ and $d=1 \text{ m}$.
- (d) the water depth in which the wave breaks, the breaker type, and the wave height at breaking. Ignore the effect of wave set up or down.
- (e) the deep water energy flow.
- (f) the wave height and energy flow in a water depth of 1 m.
- (g) the water depth in which the sand is on the point of moving.
- (h) the water depth in the which the sand is in motion but has no net drift.

For case (B) make the following calculations :-

- (i) the water depths in which the angle of incidence becomes 30° , 20° , 10° and 5° , and the wave heights at these depths.
- (j) the water depth and wave height under breaking conditions. (assume the depth at breaking is 80 per cent of the value obtained for parallel waves)
- (k) the thrust on the mass of water in the surf zone, per metre length along the shore.(N)
- (l) an estimate of the bulk sand volume flow rate in m^3/s in the alongshore direction.

2. A cylindrical pipe is laid on the sea bed across a harbour entrance in 10 m of water, the pipe diameter being 0,3 m, and the axis of the pipe is parallel to the local wave crests. If the local wave length is 50 m, estimate the wave period, and find the peak magnitudes of the velocity and acceleration force components per metre length of pipe. Estimate the peak resultant force in the inshore direction, and the timing of this in relation to the passage of the wave crest. The wave height is 2 m, take $C_D = 1,2$ and $C_M = 2,16$

3. (a) A steady wind of speed 15 m/s blows over a fetch for a period of 8 hours, producing a significant wave height of 1,8 m at the downwind end of the fetch. Estimate the fetch length in km and the wave period. Check whether the wind duration is sufficient for this condition to be stable and also check whether this is the fully arisen sea for this wind speed.
- (b) In a zero damage design calculation for the armour protection of a rubble mound breakwater, 3 tonne and 5 tonne dolosse are specified for the trunk and head respectively, the slope of the breakwater face being $\cot \theta = 2$. Estimate the block masses and block heights if tetrapods had been used in the same design. If the design wave height was 3 m, and a storm causes damage of the order 20-30 per cent to the tetrapod scheme, estimate the storm wave height. (concrete density = 2245 kg/m^3)
- (c) An incoming swell has crests parallel to a straight beach with a deep water wave height of 2 m. Estimate the horizontal force (per metre along the beach) acting on the beach inside the refraction zone, due to the dynamic action of the waves.
- (d) In an area where the sea bed is horizontal, and the water depth is 3 m, a wave has a period of 7 s, a wavelength of 38 m, and a wave height of 1,5 m. Estimate the drift velocity at bed level, and indicate the direction. Compare this velocity with the maximum orbital velocity at the same level, and indicate the influence on bed drift of a strong onshore wind.
- (e) A storm at sea generates waves with a period range of 8 to 16 seconds. The resulting swell travels towards a harbour 500 km away. Estimate the time interval between the arrival of the shortest and longest waves, assuming deep water throughout.

UNIVERSITY OF CAPE TOWN

DEPARTMENT OF CIVIL ENGINEERING

COASTAL ENGINEERING PRACTICE - Postgraduate Course CE 526 - 1977

Lecturer: Gerald Rosenthal

Outline of Course1. Dates and Time

The course consists of 14 two-hour lectures held each Monday evening starting on 18 July, 1977 and ending on 31 October. Lectures will not be held on 5 September (Settler's Day) and 10 October (Kruger Day).

Lectures will start at 5.30 pm and will be held in LT2 unless notices to the contrary are posted on the door of this lecture theatre.

An examination will be held at 9 am on SATURDAY 12 November 1977.

2. Assessment

The work of students will be assessed in three parts:

- a. Work submitted by students during the course of lectures
- b. The examination
- c. A report to be submitted not later than 23 December 1977.

The final mark will be made up according to the following formula:

$(30\% \text{ of } a) + (40\% \text{ of } b) + (30\% \text{ of } c)$

Students are required to score a minimum of 50% for each of a, b, and c.

3. Field Work

No formal field work will be conducted but students will be required to visit a number of harbours and coastal works in the vicinity of the Cape Town area with the purpose of undertaking evaluation studies, in their own time.

4. Text Book

Substantial reference will be made to the Shore Protection Manual compiled by U.S. Army Coastal Engineering Research Center. It is recommended that students should purchase this book. (A limited number of copies have been ordered for students).

UNIVERSITY OF CAPE TOWN
DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY EXAMINATION NOVEMBER 1977

CE 526 COASTAL ENGINEERING PRACTICE

Time allowed: 2 hours

Answer ALL questions

OPEN BOOK

There is a potential of 142 marks
120 marks will be regarded as 100%

Section 1 is to be handed in at the
end of the first hour

1. Answer all questions on the attached sheets, in the space provided. If additional space is required the answer is to be completed in an Examination Answer Book where the answer must be clearly numbered.

[64]

2. The attached plan shows the bathymetry of False Bay to M.S.L. Using this plan and annotating it if necessary, answer the following questions, stating all assumptions and sources of information.

If the wind were to blow from the North at an average speed of 100 km/hr estimate for a point in the vicinity of Whittle Rock:

- i) The time taken to develop a fully arisen sea (2)
- ii) The significant wave height: H_s (2)
- iii) The significant wave period: T_s (2)
- iv) The depth at which this wave would break (2)

[8]

3. If a wave recorder of the 'Wave Rider' type were to generate a record with the following characteristics:

Record length = 340 seconds

Number of 'zero-upcrossings' = 43

Number of crests = 104

- i) Calculate the zero crossing period (1)
- ii) Calculate the mean crest period (1)
- iii) Calculate the spectral width parameter (1)
- iv) What type of waves are these (i.e. swell, sea, mixed etc) Give your reason (1)
- v) If the height of the highest crest in the record is 2,1 m* and the depth of the lowest trough is 1,9 m* calculate the value of H_s (significant wave height) (2)
and $H_{s_{max}}$ (the 6 hour maximum) (2)
Use the method proposed by L. Draper 1967 in his paper 'The Analysis and Presentation of Wave Data - A Plea for Uniformity'.

* dimensioned from the zero crossing line

[8]

4. A lake of area $4 \times 10^6 \text{ m}^2$ is to be joined to the sea by a navigation channel with sides formed by vertical sheet piles driven into the sand bed. The tidal range is 1,8 m.
- What dimension would you recommend for the width of the channel assuming a bed depth of 2 m below M.S.L. ? (4)
 - Estimate the average outflow velocity (2)
 - If the size and grading of the sand is typical of the Cape Flats at what velocity would you anticipate scour would commence ? (1)

[7]

5. Two vertical aerial photographs of the coast taken at 12 seconds apart are mounted in a viewer. Two adjacent wave crests (A and B) approaching a shallow shoreline are examined. In the first photograph the distance between A and B is 127 m apart. In the second photograph the distance between A and B is 117 m apart. The second position of A is 84 m ahead of its position in the first photograph.
- Estimate the average wave celerity of crest A and of crest B during the twelve second interval (2)
 - Assuming the water is effectively shallow, estimate the average water depth under each crest, and check that the assumption is valid. (3)
 - Calculate the wave period for each crest (2)
 - To what do you attribute the difference in period (2)
 - Have these waves been generated locally, or at a considerable distance (1)

[10]

6. a) Explain the term 'spectral window' as applied to electromagnetic radiation in the region 0,2 to 20 micrometres (3)
- b) Explain why colour-false infrared film is particularly suitable for demarcating the tide line in an estuary (3)
- c) Explain what is meant by the term 'spectral signature' of a ground material such as sand or grass; and hence explain how a 'classification' of a set of multi-spectral images of a ground scene can be achieved (7)

[13]

7. The attached plan shows contours of the sea bed at the Strand, near Gordons Bay. It will be seen that rocks outcrop in many places and provide a relatively calm area which is considered to have some potential for a small craft harbour and in particular a boat ramp.
- a) Outline briefly the investigations and work you would recommend to establish the feasibility of constructing a small craft harbour in this location (7)
 - b) Identify the personnel and equipment required to undertake each of the investigations outlined above in (a). Estimate the time, rates and hence the cost of undertaking this work. (7)
 - c) Draw on the plan provided the main features of a proposal to provide a small craft harbour at this location (10)
 - d) Identify the number of boats at moorings and in dryboat storage that can be accommodated (3)
 - e) Give a rough estimate of quantities for any harbour protection works (e.g. breakwaters) proposed. State any assumptions. (5)

QUESTION 1

Name

1.1 The optimum orientation for the mooring of sailing craft is
----- (1)

1.2 The economic advantage in providing locks in a tidal harbour is in
respect of -----
----- (1)

1.3 The optimum location for waterside fuelling facilities is

because ----- (2)

1.4 Minimum dredged depths in a small craft harbour are the sum of
individual depths allowed for the following
----- typically ----- m
----- m
----- m
----- m
----- m
----- m
----- m
----- m
----- m
----- m (3)

USE ADDITIONAL LINES IF REQUIRED

1.5 Detail in plan and dimension typical floating berthing to provide
double occupancy for boats of length 8 m. Show the system of
mooring proposed.

(3)

2.

Assuming an average overall cost of R100/m² of floating berth deck area, estimate the cost of providing this berthing per boat

----- m² @ R100/m = -----

(2)

1.6 Give a local example of a leeshore anchorage -----

(1)

1.7 Why is a leeshore disadvantageous to a harbour -----

(1)

1.8 Explain the significance of providing a turning basin in the harbour on the Buffalo River at East London -----

(2)

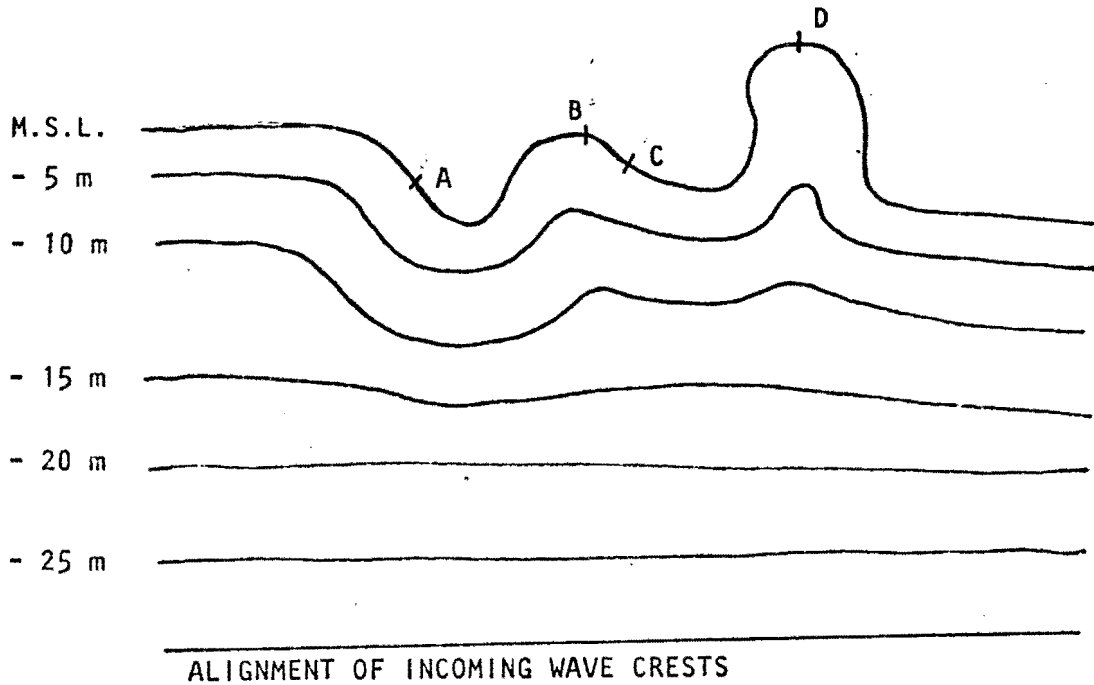
1.9 Explain briefly how the position of a dredger may be ascertained by using a sextant

(3)

1.10 Explain what is meant by the term 'controlling depth' of a harbour

(2)

1.14 Draw the approximate form of the wave orthogonals to reach A, B, C and D as they approach the coastline drawn in plan below. (Assume refraction without diffraction).

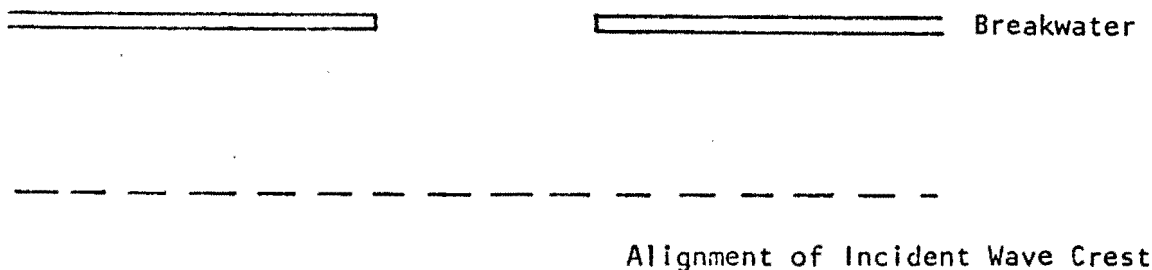


(4)

1.15 For the harbour entrance detailed below sketch

- a) the approximate form of 4 wave orthogonals as they enter the harbour (2)
- b) 3 wave crests (1)
- c) If the gap width is equal to 1 wave length draw in a dotted line the location of diffracted wave heights of one half the incident wave height. (Refer to Fig. 2-44 in CERC Shore Protection Manual) (2)

HARBOUR



5.

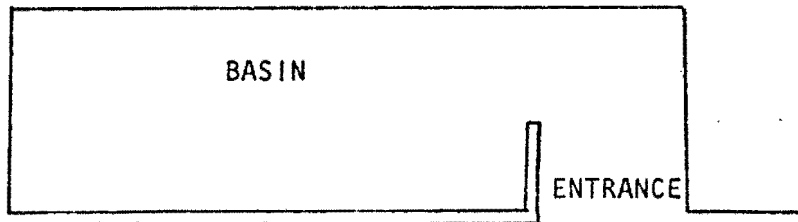
1.16 a) What is the significance of waves entering a harbour with a period equal to the fundamental period of oscillation of one of the basins ?

----- (1)

b) Name two design features that may be incorporated in a harbour design to reduce reflection ? (1)

----- (2)

c) Sketch where you would site the two features described above in 16b) in the basin shown below



(2)

d) Sketch sectional elevations of the two features described above

(2)

1.17 Give an example of a situation in which it would be appropriate to commission:

a) A 3-dimensional hydraulic model -----

b) A 2-dimensional hydraulic model -----

c) A mathematical model -----

(3)

- 1.18 Explain briefly the function of coastal sanddunes in maintaining the stability of a sandy coastline _____

_____ (3)
- 1.19 Identify by means of annotated sketches the procedures involved in implementing the following stages which might occur in the construction of a jetty on a rock bed covered in a thin layer of sand.
- a) Temporary staging (2)
- b) Airlift (2)
- c) Placement of precast bases (2)
- d) Placement of bearer piles (2)
- e) Placement of concrete underwater (2)

UNIVERSITY OF CAPE TOWN
DEPARTMENT OF CIVIL ENGINEERING
INTRODUCTION TO COURSE
COURSE CE 529 - POTENTIAL THEORY AND GROUNDWATER FLOW

A.D.W. SPARKS - 1977

Field theory deals with subjects such as seepage and consolidation in soils, stress distribution in materials, water velocity distributions in channels, deflections of slabs, heat flow, gravity fields and electromagnetic fields.

One of the equations which can be used to describe many fields is the Laplace differential equation. This equation is especially important in considering steady water seepage problems in saturated soils, and velocity distributions in certain 'ideal fluids'. In many cases the velocity distributions in water can be very similar to these distributions for 'ideal fluids'. Certain stress parameters in elastic media can also be described by the Laplace equation. The ability to understand and solve problems based on the Laplace differential equation is essential to a detailed study of theories applicable to Civil Engineering.

Other differential equations of interest to the Civil Engineer are the Poisson equation which is closely allied to the Laplace differential equation, and the Biharmonic equation which can also be considered as a form of combination of the Laplace and Poisson equations.

The Poisson equation is used in torsional stress analysis; while the Biharmonic equation applies to the deflection of slabs, and to stress distributions in plane strain and plane stress problems.

The Diffusion equation is of interest in time-dependent problems such as the consolidation of clays.

Course Sequence :-

It has been decided to start the course with the syllabus pertaining to groundwater (seepage, consolidation etc.) and then to move on to general aspects of field theory, conformal mapping, stress functions and fluid flow. Numerical methods will be introduced when they arise (e.g. in the groundwater section). Portions of the lectures may be used for working on problems and for laboratory work. A project will be set for December, January and February. The subject of the project will be chosen to suit both the lecturer and the candidate.

(At the beginning of April 1978 a short open book examination will be held in which the candidates will take the examination paper home for the week-end.) The project will count for at least 60 per cent of the marks for the course, and will be presented at a seminar session in March 1978.

Copyright of notes:- Because some of the work pertains to a postgraduate thesis which is still in progress, these notes will be issued to members of the class for their private use and for the examination in CE 529, and may not be published or copied without written permission from the lecturer.

A.D.W. Sparks.
29 September 1977.

THE OBJECTIVES AND THE APPROACH OF THE COURSE:

These will be discussed.

TEXTBOOKS FOR THE COURSE:-

Approximately forty texts will be used for the preparation of the lectures, but the following are typical examples :-

For Groundwater Flow:-

- 1) Raudkivi A.J. and Callander R.A. (Softcover available).
"Analysis of Groundwater Flow", Arnold, 1976
The softcover edition is prescribed for this course.
- 2) Todd D.K. (Softcover available)
"Ground Water Hydrology", Toppan, 1959.
This is a simpler, and more practical book.

For the Mathematical solution of Laplace's Differential Equation:-

- 3) Bland D.R. (Softcover)
"Solutions of Laplace's Equation", Routledge and Keegan Paul, 1961
A useful little book. Entirely Mathematical. Not based on vectors or tensors.
- 4) Sneddon I. (Softcover available)
"Elements of Partial Differential Equations", McGraw-Hill, 1957
Good, readable, on introduction and solution of differential equations.

For Conformal Transformations and Fluid Mechanics:-

- 5) Vallentine H.R.
"Applied Hydrodynamics", Butterworths, S.I. Edition, 1969
Excellent, especially on conformal transformations.

For Stress Analysis:-

- 6) Dally J.W. and Riley W.F. (Softcover available).
"Experimental Stress Analysis", McGraw-Hill, 1965.
Excellent, Readable, General Book.

For General Field Analysis:-

- 7) Baden-Fuller A.J. (Softcover)
"Engineering Field Theory", Pergamon, 1973.
Good, Readable. Unfortunately it deals mainly with electrical fields.
However, it could be regarded as a second prescribed book. (After ref 1.)
- 8) Rutherford D.E. (Small book)
"Vector Methods", Oliver and Boyd, 1954.
This introduces the reader to vector notation for differential equations.
- 9) Vitkovitch D. (Hardcover)
"Field Analysis", van Nostrand, 1966.
A good general book. More expensive than the softcover books.

For Numerical, Finite Difference, Finite Element or Matrix Methods:

Several sources will be used, when required.

UNIVERSITY OF CAPE TOWN

DEPARTMENT OF CIVIL ENGINEERING

COURSE CE 531 - URBAN TRANSPORTATION PLANNING & MODELLING

1. Introduction

This course will be taught by Eric Pas on Wednesday evenings from 5.30 p.m. until approximately 7.00 p.m. The course begins on the 6th April and is scheduled to run for 16 weeks. However, some people enrolled for the course have expressed a desire to finish the lectures by the end of June. It is likely that arrangements will be made to accommodate these requests, without reducing the number of contact hours.

2. Course Objectives

- (i) To provide a brief critical review of past and current practice in urban transportation planning.
- (ii) To provide a critical review of past and current practice in urban transportation modelling.
- (iii) To introduce the current developments in both urban transportation planning and modelling.
- (iv) To develop a critical perspective on current issues in urban transportation planning.

3. Method of Presentation of Material

- (i) Lecture/discussion sessions.
- (ii) Printed notes, reproductions from books, journals etc, (a nominal charge to cover the costs of printing will be made).
- (iii) For each lecture (or set of lectures) a reading list will be provided. The readings will be divided into 2 categories, viz. prescribed and additional. The prescribed readings will be handed out to all class participants, who will be expected to familiarize themselves with the material before the relevant lecture. This is an essential requirement in order for the course to proceed at the necessary pace.
- (iv) In discussing the conventional planning process it will be assumed that class participants have some familiarity with the procedures and techniques. Those not having a suitable background will be expected to acquire this by studying the

relevant sections of the undergraduate Transportation Engineering course notes.

4. Method of Assessment

This has not yet been finalized, however it is intended to discuss this aspect at the first class meeting on April 6th. It is likely that a brief term paper/project will be assigned. In addition, an examination might be held at the end of the course.

5. Disclaimer

This course will be covering a very wide range of subject matter. Therefore class participants are forewarned that they will not leave the course with an indepth knowledge of any particular aspect. However, the intention is that you should gain a general understanding of issues and concepts, and develop a critical appreciation.

UNIVERSITY OF CAPE TOWN
DEPARTMENT OF CIVIL ENGINEERING

CE 531 - URBAN TRANSPORTATION PLANNING AND MODELLING

PROVISIONAL LECTURE SCHEDULE

<u>LECTURE NO.</u>	<u>TOPIC</u>
1	Introduction, Course Objectives, Brief Overview, Outline of Issues
2	Historial Overview of Developments in Transportation Planning and Travel Forecasting
3	Introduction to the Conventional Urban Transportation Planning Process
4) 5)	Critical Review of the Conventional Urban Transportation Planning Process
6	Brief Review of Early 'Non-conventional' Forecasting Models (Direct Demand, Entropy etc)
7	Introduction to Individual Choice Models (Theoretical Development)
8	The Structure of Travel and Travel-Related Decisions
9	The Logit Model; formulation, use, estimation, assumptions, statistical tests etc.
10	The "Aggregation Problem" - what it is and how to approach it
11	Attitudes, Perceptions etc. The need for Measurement. Introduction to Measurement Techniques.

PROVISIONAL LECTURE SCHEDULE

CE ⁵³¹~~513~~

12	Data Collection, with special reference to individual choice modelling.
13	Transportation Planning for Special Groups - issues, problems, approaches.
14	Brief review of Evaluation Techniques - Past Failures and Future Possibilities.
15	Current alternatives to the Individual Choice Models
16	Brief Review of Current Policy Issues

Eric I. Pas

April 1977

UNIVERSITY OF CAPE TOWN

DEPARTMENT OF CIVIL ENGINEERING

COURSE CE 531 - URBAN TRANSPORTATION PLANNING AND MODELLING

A S S I G N M E N T NO. 1

AS AN INDEPENDENT CONSULTANT, YOU ARE REQUIRED TO CRITICALLY REVIEW THE STUDY DESIGN FOR THE CAPE METROPOLITAN TRANSPORTATION STUDY, A COPY OF WHICH YOU RECEIVED LAST WEEK.

1. INTRODUCTION

The study design, dated May 1976, states in the introduction "A transportation study of the Greater Cape Town Metropolitan Area is to be conducted and, towards this end, the design of the study is outlined in this report".

Other possibly relevant information, contained in the section which has been deleted from your copy for reasons of confidentiality, is as follows :

- (i) A Metropolitan Transport Advisory Council was duly constituted and a Steering Committee was formed to direct and supervise the execution of the study. The Steering Committee consists of representatives of local and provincial government, the South African Railways and the Cape Metropolitan Planning Committee.
- (ii) The consultants appointed to carry out the study recognized that the complex problems involved required expertise from several specialist disciplines, and a multi-disciplinary team has been assembled to assist the principal consultants.

2. TASK DESCRIPTION

You have been appointed as an independent consultant to review the study design document and to report on your findings to the Steering Committee. As noted above, a critical review is required. In other words, tell the Committee what you do and do not like about the study design. What would you do differently, better?

In carrying out your assignment, keep in mind that the study design should inform the Committee as to what the consultants propose to do, and how the various tasks will be carried out. You should therefore also report on the extent to which you think the study design serves this function.

3. SPECIFICATIONS

The following specifications should be strictly adhered to in your submission :

- * Length must not exceed 15 pages - this excludes the cover sheet, graphics and bibliography (if these are used). This, of course, does not mean that your report cannot be shorter than 15 pages.
- * All papers are to be typed, double spaced, with reasonable margins (i.e. approximately 2,5 cm).
- * The way the report is written (e.g. style and grammar) will be considered in the assessment.
- * Your paper should be linked as closely as possible to the lectures and associated readings. If you wish to take an opposing viewpoint at any stage, feel free to do so. However, your arguments must be clearly presented, and substantiated wherever possible.

NOTE : The mark for this assignment will contribute approximately 40% the final mark for the course.

April 27th, 1977.

UNIVERSITY OF CAPE TOWN

DEPARTMENT OF CIVIL ENGINEERING

COURSE CE 531 - URBAN TRANSPORTATION PLANNING & MODELLING

ASSIGNMENT NO. 2

A household level least-squares trip generation model has been calibrated for a hypothetical city called LAPABURG. The hypothetical model is as follows :-

$$Y = 0,20 + 1,75 X_2 + 2,50 \sqrt{X_6} \quad \dots\dots(1)$$

where Y = number of home-based trips per household per day,

X_2 = number of people per household,

X_6 = number of cars per household.

You are required to use this model to estimate the future home-based daily trip generation for zone number 105 of LAPABURG. The forecast characteristics of this zone are given in Table 1 below. (In carrying out this calculation you should apply the model given by equation (1) only once).

Note :

- (i) The Due Date for this assignment is Wednesday 25th May 1977.
- (ii) This assignment will count approximately 5% to your final course marks, and is to be completed on an individual basis.

Number of Households (i.e. Frequency)	Number of People (X_2)	Number of Cars (X_6)
50	1	0
100	2	0
100	3	0
50	4	0
25	5	0
25	6	0
50	1	1
150	2	1
200	3	1
150	4	1
50	5	1
25	6	1
-	1	2
25	2	2
50	3	2
100	4	2
50	5	2
50	6	2

NOTE: No household have more than six people or more than 2 cars.

TABLE 1 : Forecast Household Characteristics for Zone 105 of LAPABURG.

UNIVERSITY OF CAPE TOWN

DEPARTMENT OF CIVIL ENGINEERING

COURSE CE531 - URBAN TRANSPORTATION PLANNING & MODELLING

ASSIGNMENT NO. 3

Consider the hypothetical city of INDIANIS shown in Figure 1. The city has been divided into five hypothetical zones (1 through 5). You are required to estimate the future trip distribution and modal split of work trips for INDIANIS, using a number of different approaches.

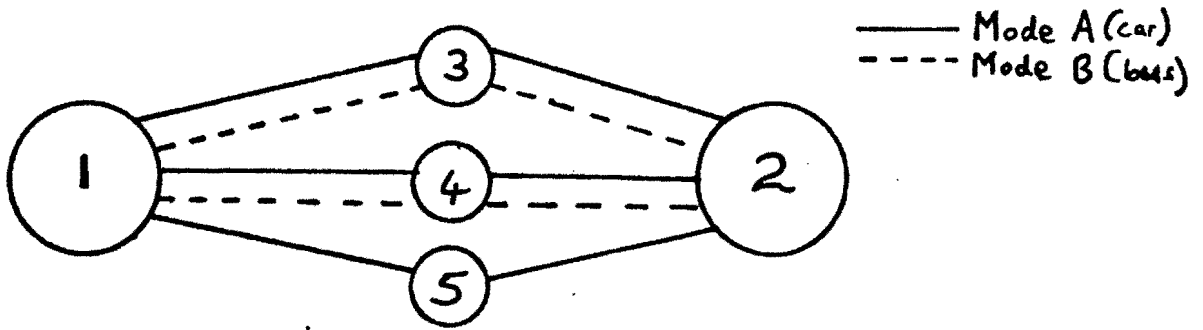


Figure 1

Zones 1 and 2 are residential areas and zones 3, 4 and 5 are employment locations. The following is the estimated future travel-cost matrix.

	3	4	5
1	10 / 15	10 / 8	15 / N/A
2	10 / 10	5 / 10	5 / N/A

travel-cost, mode A (car)

travel-cost, mode B (bus)

N/A = not available

Zones 1 and 2 are expected to have 1 000 and 1 500 households respectively in the design year. It is anticipated that 15% and 20% of the households in zones 1 and 2 respectively will not have a car in the design year. On the average, it is expected that each household will have one worker in the design year.

Zones 3,4 and 5 are expected to have the following number of jobs available in the design year :-

Zone	No. of Jobs
3	750
4	1000
5	750

- A. Use a hypothetical trip-end modal split model which assumes that all those workers living in a household in which at least one car is available utilize a car for the work trip, and that all those workers living in car-less households use the bus. Distribute the work trips by each mode using a gravity model, with a travel-cost factor function of the form $f(C_{ij}) = C_{ij}^{-2}$ for each mode. State any (reasonable) assumptions you make.
- B. Use a gravity model with a travel-cost factor function of the form $f(C_{ij}) = C_{ij}^{-2}$ to distribute all the work trips over the car network. Then apply a hypothetical trip-interchange modal split model of the form :-

$$\frac{T_{ij}^B}{T_{ij}^A} = \frac{C_{ij}^A}{2C_{ij}^B} \quad \begin{array}{l} \text{mode A = car} \\ \text{mode B = bus} \end{array}$$

where T_{ij}^m = trips from i to j by mode m

and C_{ij}^m = travel-cost, i to j, by mode m.

- C. Use the hypothetical trip-interchange model described in B above, but first distribute the trips using a 'weighted uni-modal network' and a gravity model with the same travel-cost factor function as above. (Use the hypothetical trip-end model described in A above to determine the weighting factors).
- D. Comment briefly on your results for A, B and C above.

- E. Demonstrate how one might use an iterative process to improve on the results obtained in C above.

NOTE :-

- (i) The due date for this assignment is Wednesday 1st June 1977.
- (ii) This assignment will contribute approximately 7 1/2 per cent to your final course mark, and is to be completed on an individual basis.

EIP/1977

Assignment No. 4

Consider the following brief extract from the book by Stopher and Meyburg^[1].

Elasticity Properties of Multiple-logit Models

A general logit model may be written as equation 16.32.

$$P_k^j = \frac{\exp[G_j(X_k)]}{\sum_j \exp[G_j(X_k)]} \quad (16.32)$$

It is assumed that $G_j(X_k)$ is a linear function of the vector of characteristics of the alternative j . Since it has been established that the characteristics of the individual, S_i , cannot be linearly additive in the function, it will be assumed that these characteristics enter the utility function through the vector of coefficients of the X_j s. The function $G_j(X_k)$ may therefore be written as equation 16.33.

$$G_j(X_k) = a_0^j + \sum_{i=1}^T a_i^j X_{ki} \quad (16.33)$$

where T = the number of characteristics in the common utility of alternative j

and a_i^j = the coefficients of the characteristics X_{ki}

The direct-elasticity of demand for alternative k with respect to characteristics X_{ks} of that alternative is given by equation 16.34.

$$\varepsilon_{kks}^j = a_s^j X_{ks} (1 - P_k^j) \quad (16.34)$$

In words, equation 16.34 states that the direct-elasticity (ε_{kks}^j) of demand for alternative k with respect to attribute s is proportional to the amount of the attribute possessed by alternative k (X_{ks}), to the weight or importance of that attribute in the utility function (a_s^j), and to the share of the market that alternative k has not yet obtained ($1 - P_k^j$). The first two dependencies of the direct elasticity appear to be intuitively reasonable, that is, that the elasticity is a function of both the importance and amount of an attribute. The final dependency modifies the strength of the direct elasticity by the market share of the alternative. The larger the market share, the smaller will be the direct-elasticity of demand, all other things being equal. Again, this is an intuitively reasonable statement of elasticity and is conformal with concepts of consumer behavior in economics.

You are required to derive the expression for the direct elasticity shown in equation (16.34). (Hint :- First use equation (16.32) to find an expression for $P_k^i (1 - P_k^i) a_s^i$. Then find an expression for the relevant partial derivative).

Note

- (i) Due date for this assignment is Wednesday 8th June 1977.
- (ii) This assignment will contribute approximately 5% to your final course mark, and should be completed on an individual basis.
- [1] Stopher, P.R. and Meyburg, A.H., Urban Transportation Modelling and Planning, Lexington Books, 1975.

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UNIVERSITY OF CAPE TOWN

DEPARTMENT OF CIVIL ENGINEERING

COURSE CE 531 - URBAN TRANSPORTATION PLANNING & MODELLING

ASSIGNMENT NO 5.

1. Assume that, for a particular individual, the utility of the car (for a given trip) is U_A , and the utility of the bus (for the same trip) is U_B . Assume also that this individual is indifferent to the colour of his car (i.e. a change in colour does not change his utility).

Now consider the introduction of a "new" mode, which is a blue car, where previously the individual under consideration had a red car. Except for colour, the two cars have identical "observed attributes".

- (a) For each of the values of U_B and U_A given in table 1 below, use the logit model to compute the probability that the individual chooses the bus for the given trip :-
- (i) assuming that the IIA axiom holds,
 (ii) assuming that the IIA axiom does not hold.
- (b) Compute the difference in the results (as a percentage), for each combination of values for U_A and U_B .
- (c) Comment very briefly on your observations.

Importance of trends and why.

TABLE 1 :

U_A	U_B
6	1
4	1
3	1,5
3	2,5
3	3
2,5	3
1,5	3
1	4
1	6

as % of (II)

$$P_{ii} = \left(\frac{P_{ii} - P_i}{P_{ii}} \right) \times 100$$

by adding an irrelevant alternative what would happen

2. Consider a binary mode-choice logit model of the form :

$$P_k^i = \frac{e^{G(X_k)}}{\sum_{j=1}^2 e^{G(X_j)}}$$

Now consider the following hypothetical utility functions :-

$$G(X_1) = 0,5 - 0,190 T - 0,060 C$$

$$G(X_2) = -0,190 T - 0,060 C$$

where T = travel time(in minutes), C = travel-cost (in cents).

NOTE : - All the variables in these utility functions were found to be statistically significant.

- (i) Have travel-time and travel cost been included as generic or alternative-specific variables in this model? Explain briefly.
- (ii) Assume that this model was built (i.e. estimated) for a hypothetical set of travellers having the following values of the attributes of the 2 alternatives facing them :-

Number of Travellers (in Homogeneous Group)	Attributes of Alternatives			
	T		C	
	Mode 1	Mode 2	Mode 1	Mode 2
25	10	10	20	25
50	15	10	25	30
25	20	10	30	35
100	25	10	40	45
25	10	15	20	15
75	15	15	25	25
125	20	15	30	35
25	25	15	40	45
100	10	20	20	15
50	15	20	25	30
75	20	20	30	35
125	25	20	40	45
100	10	25	30	20
100	15	25	25	20
50	20	25	30	40
50	25	25	40	40

What percentage share of the travel market do each of the two modes have for this set of people? Explain why you could carry out the computation in this case

- (iii) The logit model assumes that travellers make trade-offs between the levels of the attributes of the alternatives available to them. Consider the above data set, and comment on the applicability of this set of data (or parts thereof) for the estimation of a logit model.
- (iv) Based on the coefficients of the above logit model, what is the "value of time" for this set of travellers (Express your answer in R/hr).
- (v) "There is a modal bias towards mode 1 for this set of travellers". Explain this statement, and comment briefly on the desirability of such a "modal bias" in a model of mode choice.

NOTE :

- (i) Due date for this assignment is Wednesday 29th June 1977.
- (ii) This assignment will contribute approximately 10% to your final course mark, and should be completed on an individual basis.

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UNIVERSITY OF CAPE TOWN
DEPARTMENT OF CIVIL ENGINEERING

GEOTECHNICAL SITE INVESTIGATION - Postgraduate Course CE 534 - 1978

Lecturer: Gerald Rosenthal

Outline of Course

1. Dates and Time

The course consists of 10 two-hour lectures held each Wednesday evening starting on 1st March, 1978 and ending on 24th May, 1978. There will be no lecture on 22nd March, 19th April and 17th May. Lectures will start at 5.30 p.m. and will be held in LT2 unless notices to the contrary are posted on the doors of this lecture theatre.

An examination will be held at 9 a.m. on SATURDAY 3rd June 1978.

2. Assessment

The work of students will be assessed in three parts:

- (a) Work submitted by students as assignments during the course of lectures
- (b) The examination
- (c) A report to be submitted not later than 16th June 1978.

The final mark will be made up according to the following formula:
(30% of a) + (40% of b) + (30% of c)

Students are required to score a minimum of 50% for each of a, b and c.

3. Field Work

Students will be required to visit a number of sites in the vicinity of the Cape Town area, with the purpose of undertaking evaluation studies. Some of these studies are to be undertaken in the students' own time, others will be undertaken as formal site visits at times to be arranged soon after the commencement of the course.

4. Text Book

There is no text book for this course but students will be required to make substantial reference to journals and other reading matter available in local libraries.

A number of papers and other notes will be reprinted for the use of students in their research for this course.

5. Seminars

Some of the material covered in this course will be discussed in Seminar. Students will be required to contribute to these seminars by giving short prepared talks. (One week's notice will be given).

UNIVERSITY OF CAPE TOWN
DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY EXAMINATION JUNE 1978

CE 534 GEOTECHNICAL SITE INVESTIGATION

Time allowed : 3 hours

There is a potential of 199 marks
180 marks will be regarded as 100%

Section 1 is to be handed in at the
end of 1½ hours. After this time the
examination will be OPEN BOOK.

PART A : CLOSED BOOK

1. Answer all questions on the attached sheets, in the space provided. If additional space is required the answer is to be completed in an Examination Answer Book where the answer must be clearly numbered.

[106]

PART B : OPEN BOOK

2. Your firm has been commissioned by the Cape Provincial Administration to submit a memorandum detailing and justifying the geotechnical site investigations and specific geotechnical tests that should be undertaken preparatory to the design and construction of a large new hospital to be sited some 5 km north of Muizenberg.

The proposed structures include an eight-storey hospital block covering a ground area of 20 000 m² (for which two underground basement levels are to be provided), a 30 m water tower, parking areas, nurses and doctors quarters and two playing fields.

At the existing site poorly vegetated sand dune remnants can be seen. Although the land is not directly affected by flooding from any river, pools of water are observed to lie over many portions of the site throughout the winter.

Tabulate (Use both sides of the Examination Answer Book pages to use the full width).

- (a) The investigations and tests you propose. (Specify the number of tests and proposed depth, where appropriate).
- (b) A brief description of what the work entails

...../2(c)

2. (c) The purpose of investigation or test.
- (d) The implications of not having the correct information.
- (e) The approximate cost of each investigation or test (State times and assumed rates where appropriate).

e.g. Civil Engineer	10 hrs @ R 15-00 per hr	R 150-00
Technician	20 hrs @ R 10-00 per hr	R 200-00
Labourers	40 hrs @ R 2-00 per hr	R 80-00
Drilling rig (including operators)	40 hrs @ R 20-00 per hr	R 800-00

Note: Some tests can be quoted on time taken or depth - either will be acceptable.

In the absence of known rates reasonable assumptions must be made and stated.

[40]

3. (a) Show by means of annotated sketches 3 situations in which the determination of the shear strength of the soil mass is essential for adequate design. In each case show the assumed failure surface. (3)
- (b) Give 3 field methods that can be used to determine quantitative values for in-situ shear strength (3)
- (c) Briefly explain the physical basis of each of the three tests described above in (b) and how the results are interpreted. (6)
- (d) Explain what is meant by "effective stress" as applied to partially saturated soils. Give a Mohr circle representation of your explanation. (4)

[16]

4. (a) A stereo-pair of black and white photographs is available for for an area composed of two different soil types (the one being TMS (Table Mountain Series) and the other being Cretaceous (as found in the Swartkops River Valley near Port Elizabeth).
What clues would you look for in the photographs to delineate the soil boundaries. Where possible be specific.

(10)

- (b) Tabulate the steps you would take in the preparation of an engineering soils map of a remote area of the northern Cape. Assume that the map you will produce will be used in road route selection.

(10)

[20]

5. (a) Explain what is meant by a spectral signature and hence explain how multispectral scan data may be used in the preparation of maps delineating specific soil and vegetation types.

(10)

(b) Explain why thermal imaging of ground scenes at ambient temperatures is generally done in the 8 to 14 μm range, whereas fire detection is done at the shorter wavelengths of 3 to 5 μm .

(2)

(c) Explain briefly what the following terms mean :

pixel

density slicing

spatial rectification of imagery

(3)

(d) What portion of the electromagnetic spectrum is specifically useful for imaging the vigour of vegetation?

(1)

What natural phenomenon makes this possible.

(1)

[17]

PART A: CLOSED BOOK

QUESTION 1

NAME:

Answer all questions on these sheets in the space provided. If additional space is required use the examination answer book where the answer must be clearly numbered.

- 1.1 Name 3 pedogenic materials found in Southern Africa.
..... (1)
..... (1)
..... (1)

- 1.2 What characterises these materials to be termed pedogenic?
.....
..... (2)

- 1.3 One pedogenic material found in the Cape Peninsula has an economic use. What type of material? (1)
For what purpose is it used? (1)

- 1.4 What does a drift map show?
..... (1)

- 1.5 What is the nature of the weathered soil deriving from granite?
..... (1)

- 1.6 What is Loess?
..... (1)

- 1.7 What special precaution has to be taken in the design of structures bearing on Loess?
..... (1)

- 1.8 Why?
..... (1)

- 1.9 Give two points of engineering significance of the pebble marker in a typical Southern African soil profile.
(i)
..... (1)
(ii)
..... (1)

1.10 By what means can a soils' susceptibility to frost be determined?
.....
..... (1)

1.11 Why is it necessary to specify the temperature at which clays should
be oven dried eg 105 - 110°C?
..... (1)

1.12 For what purpose is sodium hexametaphosphate used in a soils testing
laboratory?
..... (1)
Why?
..... (1)

1.13 Name an index that is used in the specification of the material
properties of the clay core of an earth dam (1)
Give the algebraic expression for its value (1)
Explain briefly the physical basis of how it is determined.

(3)

Give typical values of this index that would be :
acceptable (1)
unacceptable (1)
in the clay core of the earth.

1.14 What is the liquidity index a measure of? (1)
Give an algebraic expression for its value (1)
Give an example of a situation in which it would be relevant to
record the liquidity index in a soil profile (1)

1.14 What is the essential difference between a thin walled and thick walled sampler? Give a quantitative answer and explain what each term implies physically through the use of an annotated sketch.

(5)

1.20 Why are twin core barrel drilling systems used in preference to single core barrel systems for soil sampling? Give annotated sketches to illustrate your answer.

(3)

1.21 Sketch an arrangement by which samplers can be "winched" into the ground.

(2)

1.22 Why are piston samplers preferable to open drive samplers? Give sketches to illustrate your answer.

(4)

1.23 Explain what is meant by the "Activity" of a clay.

(4)

1.24 Explain how you would determine the sensitivity of a clay in the field.

(4)

Of what engineering significance is its value?

.....(1)

1.25 Explain by means of a sketch what is meant by an atmospheric spectral window. (Quantitative values need not be given).

(2)

What is the significance of spectral windows in remote sensing?

.....
.....
.....

(1)

1.26 In South Africa from what source is overall aerial photographic coverage obtainable?

(1)

1.27 Sketch grading curves of gap graded and uniformly graded materials.

(2)

1.28 Annotate the axes to the grading curves given above to show the numerical limits of clay, silt and sand.

(2)

1.29 Give an aspect of engineering significance in the predicted behaviour of :

- a. Gap graded soils
.....(1)
- b. Uniformly graded soils
.....(1)

1.30 (Answers to be tabulated on the form given overleaf, or in Examination Answer Book)

- a. State the names of 4 types of penetrometer test commonly used in South Africa.
- b. Briefly describe the main physical features of the apparatus
- c. Briefly describe the method of execution
- d. Explain briefly how and for what purpose the results are used.

(16)

1.31 Derive an algebraic expression for the time taken for a P-wave to arrive at a geophone in a "2-layer" seismic refraction situation where the seismic wave is generated at the surface by a hammer blow. (Give your answer on the back of this sheet).

(10)

1.32 Why can the seismic refraction method not be used if the seismic velocity of the lower layer is lower than that of the upper layer?

.....
.....
.....

(2)

ANSWER SHEET FOR QUESTION 1.30.

a. Type of Test	b. Physical Features	c. Method of Execution	d. How and what results used
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UNIVERSITY OF CAPE TOWN
DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY EXAMINATION JULY 1977.

CE 535 ENGINEERING ECONOMY

Time Allowed : 3 Hours

ANSWER ALL QUESTIONS

O P E N B O O K

Calculators may be used.

1. Outline and discuss the rationale behind four basic methods of making economy studies.

Indicate the relative advantages (if any) of each method.

[15 marks]

2. Interest and annuity relationships are used to establish the equivalence between sums of money when time and interest rates are taken into consideration.

Solve the following with interest at 6% compounded annually.

- (a) If R3 500 is deposited now, what uniform amount could be withdrawn at the end of each year for 15 years and have nothing left at the end of the 15th year?
- (b) What present investment is necessary to secure a perpetual income of R2 000 a year?
- (c) How much will be accumulated in a fund at the end of 21 years if R2 500 is invested now?
- (d) What annual saving for 20 years must be expected to justify a present expenditure of R7 000?

[12 marks]

3. Suppose a company can save R4 000 a year by replacing a manual process with a machine costing R12 000, but rejects this opportunity. Describe the resulting loss in money if the economic life of the machine is 10 years with R3 000 salvage and if the minimum required rate of return is 15%. On the other hand, describe the loss, if any, if the company approves the installation and the economic life proves to be 4 years with R3 000 salvage value. Neglect tax.

[12 marks]

4. You have the opportunity of making one of two investments. The first will cost R1 000 now and R100 per year at the end of this and the next four years. The second will cost R700 now and R200 per year at the end of this and the next four years. All other factors are equal.

- At what rate of interest are the two equal?
- Over what range of interest rates will you favour the first?
- Discuss your result.

[7 marks]

5. You have purchased a machine for R40 000 and negotiated a life of four years, with the Receiver. What is the present value of tax allowances if :

- (a) The machine was a truck used for transporting personnel and equipment.
- (b) The machine was a truck used for transporting material for road construction.

Motivate your answers. Use $i = 15\%$

6. The Plant Director, in working up the hourly charge out rate for a model D digger (which cost R_x to buy) includes the amount of $R_y \div n \times L$ as a provision for replacement where

L = expected life of the present digger in years.

n = number of hours worked in a year.

R_y = expected cost of the new generation Model D digger in L years time.

Set out an argument to convince him that his formula for calculating the provision is not entirely correct. Propose an alternative formula and argue for its adoption.

[7 marks]

7. A R39 000 investment in machinery directly used in the process of manufacture is proposed. It is anticipated that this investment will cause a reduction in net annual operating disbursements of R10 500 a year for 12 years. The investment will be depreciated for income tax purposes by the straight-line method assuming a 12 year life and zero salvage value. The forecast of zero salvage value is also to be used in the economy study. The effective tax rate is 55%. What is the prospective rates of return after income taxes?

[12 marks]

8. Depreciation has been defined as that amount which must be invested annually for N years at $i\%$ p.a. in order to create a fund in N years, which equals the first cost of the asset. A minimum required profit of $i \times$ first cost is also defined in the annual worth method.

Show that the sum of depreciation and minimal required profit as defined above equals the capital recovery factor ($A/P, i\% N.$)

[10 marks]

9. A new machine can be acquired for R25 800. Its operating disbursements for the first year of operation are expected to be R16 500; thereafter they are expected to increase R692 a year as a result of deterioration. The analyst predicts that the proposed machine will be replaced in the future by "like" machines having the same first cost and operating disbursements as the proposed new machine. Replacements will be necessary because of increasing operating disbursements. The salvage values of all the new machines in any year t are expected to conform to the formula,

$$L = \left[\frac{(15 - t)(16 - t)}{15 \times 16} \right] (25\ 800)$$

The present machine has a net salvage value today of R3 000, and this is expected to decrease R500 a year for every year that the machine is kept by the company. Its operating disbursements will be R22 728 for the coming year, and these are expected to increase R650 a year thereafter. The minimum required rate of return is 20%. Should the present machine be replaced? Neglect tax.

[20 marks]

UNIVERSITY OF CAPE TOWN
DEPARTMENT OF URBAN AND REGIONAL PLANNING

YEAR: 1977
TERM: SECOND
SUBJECT: TRANSPORTATION : THEORY 1
LECTURER: FABIO TODESCHINI
HANDOUT: 23.0

TRANSPORTATION EVALUATION : A CASE STUDY

1. An overview of the Washington Metropolitan Area Rapid Transit system.
2. Setting - up an Environmental Impact study.
3. The case of the Greenbelt Route.
 - Draft Impact study for the whole system (ARS)
 - 1st Contract
 - 2nd Contract
 - 3rd Contract
4. Conclusions.

UNIVERSITY OF CAPE TOWN

DEPARTMENT OF URBAN AND REGIONAL PLANNING

YEAR 1977

TERM SECOND

SUBJECT THEORY 1

CO-ORDINATOR R.A. CHAPMAN

HANDOUT 18.0

COURSE OUTLINE IN TRANSPORTATION PLANNING

COURSE DESCRIPTION

The course is an overview of the nature and role of transportation in relation to the urban environment and of the concerns, processes and tools of planning - with particular reference to problems in the South African context. Within this framework it deals specifically with the tools and techniques of planning and evaluation, and the implementational parameters of transport.

APPROACH AND OBJECTIVES

Transportation is approached as an integrative phenomenon and medium involving activity, spatial, model and physical environmental relationships and social, economic, administrative and physical aspects of consideration, policy and design. As such, transportation is viewed as an important tool of development.

From this, a series of specific course objectives follow:

To understand the nature and role of urban transportation especially its social, economic and physical purpose.

To become aware of transportation as a metropolitan problem and to gain insights concerning the issues involved including the politics of transportation.

To learn about the transportation system (with component sub-systems) as a determinant of urban structure and form and, in this, to appreciate the complex interrelationships between transportation and land use including physical environmental factors.

To appreciate transportation as a multi-model system.

To learn about the legal, financial and planning-administrative framework of urban transportation in South African cities.

To become basically acquainted with the principles, methods and techniques of analysis, design and evaluation used in transportation planning; this includes quantitative analysis.

To become familiar with the notion of "impact" including evaluative methods and criteria pertaining to community and environmental factors.

To assess the role of the planner in relation to transportation and, in this, to evaluate the transportation planning process in terms of the South African urban context.

COURSE REQUIREMENT

The project in Planning Practice, which runs for the duration of the Second Term, is a Strategy to Guide Development in Woodstock - Salt River. You are asked to do a Primer on Transportation Planning which can assist in examining an urban area of this nature in developing transportation policies, systems and programmes. The primer should, essentially, set out

- a. the planning principles to observe
- b. the constraints to observe
- c. the criteria that should apply at all times over the study area.

The primer is to be done on an individual basis and must be submitted no later than 7th October 1977. It should be concisely written, contain diagrammes where relevant, and be carefully referenced. The document submitted must be typewritten on A4 size paper.

Grades in Transportation Planning will be based on this primer.

Over and above the primer the term studio project will necessarily involve a consideration of transportation in an integrative sense. The degree of emphasis which different project teams place on transportation itself is optional and will be determined by the nature of the problems and opportunities and the notion of development, that emerge. The project deals with a total condition so that transportation, at the outset, is no more or less a consideration than, say, housing.

As far as transportation specifically is concerned, data currently being utilized in the metropolitan transportation study for Cape Town will be made available upon request. Conversely any relevant outcome of your project will be made available for potential utilization in the metropolitan transportat study.

Bernal Floor and I will be available to assist you with the individual primers and transportation aspects of the team projects. Bernal Floor will be teaching the major part of the transportation planning course. He is Director of the Department of Transport Economics at the University of Stellenbosch, and is consulting on the metropolitan transportation study for Cape Town. It is suggested that one (or two) studio periods be set aside at a time when you require "feedback".

TIME-TABLE

July 21	INTRODUCTION Raeburn Chapman	Introduction to the course: Approach, programme and requirements. Nature of transportation as a phenomenon and system. Transportation and urban structure and form. The critical issues, particularly at the metropolitan level. The state of the art of transportation planning. Transportation as a developmental process.
July 28	ENVIRONMENTAL IMPACT F. Todeschini R. Chapman	Environmental impact analysis including quantifiable and non-quantifiable measure of evaluation. Case study of environmental impact assessment for the Washington D.C. metropolitan transit system.
Aug. 4	METROPOLITAN BUS SYSTEM Adv. Johan Barnard	Transportation policy in South Africa governing public transport including the role of the metropolitan operating authority. Advantages of buses as a mode of travel. Characteristics of the metropolitan bus system in Cape Town including political, social, economic and geographical factors which affect service efficiency.
Aug. 11	CHARACTERISTICS OF TRAVEL AND SYSTEMS OF MOVEMENT Bernal Floor	The nature of travel and characteristics of traffic. Transportation system characteristics and standards for different modes. Modal split and peak period commuting. Planning principles involved.

Aug. 18 MODELLING
 25 Bernel Floor
 Sept. 1
 8

The transportation planning process incorporating the following four aspects of mathematical modelling. The approach adopted in the Cape Town metropolitan transport study.

Trip Generation

Land use forecasting and methods of trip estimation: Use of demand simulation and projections in which future zone-by-zone distribution of population and employment (activity) areas is forecast for the purpose of traffic analysis.

Modal Split : Achieving Intermodal Balance

Methods of estimating the proportion of trips from or to a given area that select a particular mode of travel. Peak hour splits for trip productions and attractions by analysis zone.

Trip Distribution

Methods of estimating the manner in which trips generated in one zone distribute themselves to all other zones, including gravity and other models.

Trip Assignment

Methods of estimating the manner in which trips from one zone traverse the transportation network to reach another zone, and the subsequent assigning of internal volumes to each link, thereby testing a network hypothesis numerically.

Sept. 15 ECONOMICS OF
 29 URBAN TRANSPORTATION
 Bernel Floor

Economic considerations, principles and methods of analysis and evaluation. Economic role and demand for urban transportation in relation to city size. Externalities and different counteracting measures. Considerations of funding, revenues and expenditures. Effect of transportation policies and investment on property values.

Oct. 6 LEGISLATION AND
 ADMINISTRATION
 Adv. Johan Barnard

Critical overview of the statutes, authorities, and administrative procedures and mechanisms (eg. subsidies) relevant to planning, financing and operating urban transport in South Africa. Important aspects of the Driessen Commission findings.

READINGS

The following is a rationalized list of basic readings and recommended text purchases. These readings are considered necessary and plausible to complete in order to fulfil the course requirement.

TEXTS RECOMMENDED FOR PURCHASEGENERAL TEXTS

Bruton, Michael J. INTRODUCTION TO TRANSPORTATION PLANNING. London: Jutchinson, 1975 DA 711.7 BRUT

AND/OR

Lane, Robert, Powell, T.J., Prestwood-Smith, Paul. ANALYTICAL TRANSPORT PLANNING. London : Duckworth, 1971. DA 711.7 LANE

SOUTH AFRICAN CONTEXT : LEGAL, SOCIAL, ECONOMIC AND PHYSICAL ASPECTS

REPORT OF THE COMMITTEE OF ENQUIRY INTO URBAN TRANSPORT FACILITIES IN THE REPUBLIC. (Driessen Report), 1975.

MOVEMENT AND CITY STRUCTURE

Bourne, L.S. (ed) INTERNAL STRUCTURE OF THE CITY. Oxford University Press, 1971. DA 301.36 BOU

Chapter IV : Bourne, L.S. Transportation, Communication and Linkages, pp. 216 - 221.

Deutch, K.W. On Social Communications and the Metropolis. pp. 222 - 230

Blumenfeld, H. Transportation in the Modern Metropolis pp. 231 - 239.

Wolforth, J. The Journey to Work pp. 240 - 247.

Meyer, Kain, Wohl. THE URBAN TRANSPORTATION PROBLEM. (A Rand Corporation Study) DA 711.7 MEY

Ch. 3 : Recent trends in Urban Locations

Ch. 5 : Trip Pattern and Demand

Ch. 6 : Interrelationships of Housing and Urban Transportation

Mitchell, Robert B., Rapkin, Chester. URBAN TRAFFIC : A FUNCTION OF LAND USE. New York : Columbia University Press, 1954.

Ch. 3 : The Structure of Movement : Spatial and Temporal Organization

URBAN PROBLEMS

Dyckman, J.W. TRANSPORTATION IN CITIES. Sc. American, Sept. 1969. (Besides the Scientific American edition two additional xerox copies of the article are on library reserve)

Greer, Scott. TRAFFIC, TRANSPORTATION AND PROBLEMS OF THE METROPOLIS. In Orleans, P. (ed.) Social structure and social process. Boston : Allyn and Bacon, Inc., 1969 (Optional)

PLANNING PROCESS AND APPROACH

Creighton, Roger. URBAN TRANSPORTATION PLANNING. Urbana: Univ. of Illinois Press, 1970.

Ch. 1: Problems and Policy Issues

Ch. 5: Transportation Networks

Ch. 6: The Urban Transportation Planning Process

Ch. 9: Goals

Gakenheimer, Ralph. TRANSPORTATION PLANNING AS RESPONSE TO CONTROVERSY : THE BOSTON CASE. Cambridge, Mass. : MIT Press, 1976 DA 711.7 GAKE (Skim)

Mannheim, Marvin L. A DESIGN PROCESS MODEL : THEORY AND APPLICATION TO TRANSPORTATION PLANNING. In Moore, Gary T. (ed) : Emerging Methods in Environmental Design and Planning. Cambridge, Mass. : MIT Press, 1968

Perloff, J.S. SUBSIDIARY TRANSPORTATION : ITS ROLE IN REGIONAL PLANNING. JAIP Vol. 41, No. 3, May 1975, pp 170-183. (optional)

Ministry of Transport. LONDON TRAFFIC SURVEY. London : London County Council, 1964. (Vols. 1 and 2). DA 711.7 LON (Peruse this comprehensive study)

MODES

Richards, Brian. NEW MOVEMENT IN CITIES. New York : Reinhold Publishing Corporation, 1966. DA 711.7 RIC.

EVALUATION

Oglesby, Clarkson H., et al. A METHOD FOR DECISIONS AMONG FREEWAY URBAN LOCATION ALTERNATIVES BASED ON USER AND COMMUNITY CONSEQUENCES. Highway Research Council (Publication No. 305)

Owen, W. TRANSPORT, ENERGY, AND COMMUNITY DESIGN. In Future, Vol. 8, No. 2, April 1976. DA 309.205 FUT. (Skim)

Hoel, L.A. EVALUATING ALTERNATIVE STRATEGIES FOR CENTRAL-CITY DISTRIBUTION Highway Research Council (Publication No. 293) (Skim)

Wachs, Martin. CONSUMER ATTITUDES TOWARD TRANSIT SERVICE : AN INTERPRETATIVE REVIEW. JAIP Vol. 42, No. 1, January 1976. (Optional)

URBAN DESIGN AND PHYSICAL ENVIRONMENT

Ritter, Paul. PLANNING FOR MAN AND MOTOR. London : Pergamon Press, 1964. (Also London : HMSO, 1965) (Study the diagrammes)

Buchanan, Colin D. TRAFFIC IN TOWNS: A STUDY OF THE LONG-TERM PROBLEMS OF TRAFFIC IN URBAN AREAS. London : HMSO, 1963. DA 711.7 GRE (Skim, and note diagrammes)

*Optional - Richard Miles - communication theory of urban growth.
Melvin Webber - urban place and non-place realm.
John Friedman - urban field.*

Gruen, Victor. THE HEART OF OUR CITIES. New York : Simon and Shuster, 1967.
DA 711.52

- Ch. 16 : The Taming of the Motorcar
- Ch. 17 : Pedestrianism and other future modes of transportation
- Ch. 18 : The Emerging New Urban Pattern
- Ch. 19 : The Rebirth of the Heart

Pushkarev, Boris, Zupan, Jeffrey M. URBAN SPACE FOR PEDESTRIANS. (A Report of the Regional Planning Council). Cambridge, Mass. : MIT Press, 1975.
DA 711.74

Lynch, Kevin. THE VIEW FROM THE ROAD. Cambridge, Mass. : MIT Press, 1964.
DA 711.73 APP (Optional)

TRANSPORTATION ECONOMICS (Skim)

Heggie, Ian G. TRANSPORTATION ENGINEERING ECONOMICS. London : McGraw Hill, 1972.

- Ch. 1 : National Planning Objectives
- Ch. 2 : The Use of Surplus Criteria
- Ch. 3 : The Preparation of a Social Accounting Framework
- Ch. 4 : Pricing Policy and the Finance of Transport Improvements

Golberg, M.A. et al A COST-BENEFIT EVALUATION OF TRANSPORTATION CORRIDORS.
Highway Research Council (Publication No. 305)

Angel, S., Hayman, G.M. URBAN TRANSPORT EXPENDITURES. Centre for Environmental Studies. (Working Paper 70) DA 711 CENT.

REFERENCE

MODES AND STANDARDS

Berry, D.S. et al THE TECHNOLOGY OF URBAN TRANSPORTATION. Northwestern Univ. Press, 1967.

Brierley, J. PARKING OF MOTOR VEHICLES. London : Applied Science Publishers, Ltd., 1972. DA 711.73 BRIE

Highway Research Board (Special Report 87) HIGHWAY CAPACITY MANUAL.
Washington, D.C. Nat. Academy of Sciences, 1965. DA 711.7 TRA

LEGISLATION

THE MOTOR CARRIER TRANSPORTATION ACT NO. 39 OF 1930, as amended.

REPORT OF CAPE PENINSULA AND DISTRICTS URBAN TRANSPORT COMMITTEE. (Viljoen Report), October 1947.

REPORT OF THE COMMISSION OF INQUIRY INTO THE CO-ORDINATION OF TRANSPORT IN SOUTH AFRICA. (Marais Report)

REPORT OF THE COMMISSION OF INQUIRY INTO ROAD MOTOR TRANSPORTATION.
(Page Report), 1945.

CAPE PROVINCIAL ROAD TRAFFIC ORDINANCE NO. 21 of 1966, as amended.

References

References

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2. Kuichling, Emil The Relation between the Rainfall
and the Discharge of Sewers in
Populous Districts (1889)
3. Mulvaney, T.J. On the use of Self-registering Rain
and Flood Gauges in making observations
of the Relations of Rainfall and Flood
Discharge in a given Catchment (1851)
4. Kirpich, P. Time of Concentration of Small
Agricultural Watersheds (1940)
5. Kringold, D.B. On the Hydrology of Culverts.
Proc. 26th Ann. Mtg. Highway Res.
Vol. 26 (1946)
6. Yarnell, D.L. Miscellaneous Publication No. 204,
U.S. Dept. of Agriculture.
7. War Department Engineering Manual of the War
Department, Part 13, Chapter 1 (1945)
8. Horner & Fluyt Relation between Rainfall and Run-off
from Small Urban Areas: Trans. ASCE,
Vol. 101, 1936.
9. Voster, J.A. The Rational Method for Determining
Maximum Run-off: Farming in South
Africa. Vol. 15 August 1940.
10. Hydrological Design Flood Determination in South
Research Unit Africa. Report No. 1/72
11. Metcalf & Eddy American Sewerage Practice:
McGraw-Hill (1928)

12. Seelye, E.E. Data Book for Civil Engineers:
Volume 1: 3rd Edition (1960)
13. Reich, B.M. S.A.I.C.E. Transactions (May 1962)
14. Holtzhausen, D. Drawing No. 977 kept by the SWA
Administration (January 1954)
15. Department of South West African Administration (1962)
Roads
16. Roberts, D.F. The empirical determination of
flood peak probabilities.
Division of Hydrology Technical
Report No. 33 (1965)
17. Alexander, W.J.R. Course in Hydrological Analysis:
Pretoria (1978)
18. C.O. Wisier and Hydrology; 2nd Edition (1959)
E.F. Brater
19. C.V. Davis and Handbook of Applied Hydraulics (1969)
K.E. Sorensen
20. ICE Manual of Applied Geology for
Engineers
21. Muskat, M Flow of Homogeneous Fluids
through Porous Media (1937)
22. L.K. Wenzel and Methods of Determining Permeability
V.C. Fiskel of Water Bearing Materials (1942)
23. Casagrande, A. Seepage through Dams (1937)
24. Wilkinson, W.B. Constant Head In-Situ Permeability
Tests in Clay Strata : Geotechnique 18
(1968)

25. Kirkham, D. Potential Flow in Circumferential
Openings in Drain Tubes : Journal
of Applied Physics

26. K. Terzaghi and Soil Mechanics in Engineering
R.B. Peck Practice (1948)

27. R.K. Linsley and Elements of Hydraulic Engineering
J.B. Franzini (1955)

28. E. Hoek and Rock Slope Engineer : Institute of
J.W. Bray Mining and Metallurgy (1974)

29. Johnson, E.E. Groundwater and Wells (1966)

30. Burmister, D.M. Soil Mechanics (1948)

31. V.S. Aronovici Soil Permeability as a Criterion
and W.W. Donnan for Drainage Design (1946)

32. Kirkham, D. Seepage into Drain Tubes in Stratfield
Soils (1951)

33. J.N. Luthin and Numerical Solutions for the Tile
R.E. Gaskell Drainage of Layered Soils (1950)

34. Bertram, G.E. An Experimental Investigation of
Protective Filters (1940)

35. H.F. Winterkorn Foundation Engineering Handbook
and H. Fang (1976)

36. A.J. Randkivi and Analysis of Groundwater Flow (1976)
R.A. Callendo

37. DIN 4021 Siel 3: Erkundung durch Schürfe und Bohrungen
DK624.131.33/.34: sowie Entnahme von Proben (1976)
550.822

38. SABS 045-1974 South African Bureau of Standards:
Testing of Boreholes (1974)