Chapter 6
Surface drainage

INTRODUCTION

On flat agricultural lands, with slopes often below 0.5 percent, ponds form where the infiltration into the soil is less than the amount of water accumulated after rainfall, snowmelt, irrigation or runoff from higher adjacent places. In cold climates, a combination of snowmelt and frozen subsoil is particularly troublesome, while in dry regions so is an irrigation followed by unexpected heavy rain. Ponds form on the ground surface, especially where the infiltration rate is below the precipitation intensity. This process also occurs where the groundwater is deep.

Fine-textured soils, especially ones with a weak structure, and soils that form crusts easily are most susceptible to low infiltration and ponding. The cause is usually at or very near to the ground surface, in the form of natural pans or human-induced compacted layers such as plough soles. Deeper layers of low permeability are sometimes the cause of the formation of a perched water table.

Another cause of pond formation is insufficient subsurface drainage (natural or artificial), causing groundwater tables near or even above the surface. In this case, the flow is not restricted by insufficient infiltration into the soil but by the limited discharge of groundwater. The two processes sometimes interfere. A temporary high groundwater level may cause slaking and crust formation, which then causes stagnation of water on the surface, even after slight rains. Such pools tend to become larger during further rains.

In temperate climates with low-intensity rainfall, the precipitation rate is usually lower than the infiltration into the soil. Thus, surface runoff is limited to special cases, i.e. steep and barren slopes, very impermeable soils, land compacted by heavy machinery during the harvest of root crops, and soils that are susceptible to crust formation. In summer, the land is dry enough to absorb even a heavy shower. In such climates, subsurface discharge dominates (Chapter 7). In places where surface runoff occurs, local or temporal solutions are common (usually small ditches).

In climates where rainfall is more torrential, the volumes of surface runoff can be considerable, especially on soils with low infiltration rates and from land that has been conditioned (smoothened, beds, furrows, etc.) to reduce the incidence of ponding in high-value vegetable crops and orchards. Both rainfall intensity and infiltration rate are functions of time, and their combination leads to a critical period when conditions are worst. Such a period usually lasts a few hours. Where the type of agriculture requires its removal, as is usual in flat areas, surface drainage is needed. In addition, part of the infiltrated water must also be removed by subsurface drainage, but this flow comes later.

Surface water stagnation has negative effects on agricultural productivity because oxygen deficiency and excessive carbon dioxide levels in the rootzone hamper germination and nutrient uptake, thereby reducing or eliminating crop yields. In addition, in temperate climates, wet places have a relatively low soil temperature in spring, which delays the start of the growing season and has a negative impact on crop yields. Excess water in the top soil layer also affects its workability.

The length of the critical period of crop inundation must be determined from local experience as it varies according to climate, soils, crop tolerance, crop development stage and cropping conditions. In humid temperate regions, common field crops, such as maize and potato, usually require designs to remove ponded rainfall from the drainage area within 24 hours. Some higher value horticultural crops may require a 6–12-hour removal time during the growing season, while other crops (e.g. sugar cane) can tolerate ponding for a couple of days.

The objective of surface drainage is to improve crop growth conditions by providing timely removal of excess water remaining at or near the ground surface before the crops are damaged. Surface drainage is also needed to guarantee soil workability and trafficability, so preventing delays in soil preparation operations and harvesting, respectively.

In order to do this effectively, the land surface should be made reasonably smooth by eliminating minor differences in elevation. It should preferably have some slope towards collection points, such as open field drains or shallow grassed waterways, from where water is discharged through outlets especially designed to prevent erosion of the ditch banks. Land smoothing is the cheapest surface drainage practice and it can be performed on an annual basis after completion of tillage operations (Ochs and Bishay, 1992).

On sloping and undulating lands, generally with natural slopes of more than 2 percent, ponding is not usually much of a problem, except for a few small depressions. However, the resulting runoff may cause severe erosion during heavy rains. Where this occurs, reshaping of the land surface into graded terraces that generally follow the contours is needed in order to promote the infiltration and the storage of useful moisture in the soil. The necessary earth movement can at the same time be used to fill the small depressions where runoff tends to collect. Earth movement is expensive (at least US\$2/m^3 even in low-income countries) and it requires considerable expertise and further maintenance because of soil subsidence and settling. This chapter does not address land grading and levelling aspects. Instead, reference is made to Sevenhuijsen (1994) for land grading and levelling calculations.

The field surface drains (furrows or shallow ditches) discharge into a network of open ditches or grassed waterways and larger watercourses. The main drainage system (described in Chapter 5) removes excess water to points outside the project area. Care must be taken to protect stretches where surface runoff collects and enters into field surface drains or where these drains enter larger ones. These are the points where gullies can start and where sediments enter into the main drainage system. At these transition points, provisions are needed to control erosion, even in flat lands.

In this chapter, the drainage systems required to remove safely the excess of surface water are described first. Later, methods to estimate surface drainage coefficients, which are required in order to design each component of the drainage system, are considered with technical details added in the annexes. Flat lands and sloping lands are considered separately because of their specific conditions concerning surface runoff and soil erosion control.

SURFACE DRAINAGE SYSTEMS FOR FLAT LANDS

In flat lands, the approaches to cope with excess surface water depend on the circumstances. Where high groundwater is not a problem, surface systems, such as furrows and raised beds, are sufficient. However, a system of shallow ditches, combined with surface drains where necessary, is often used to cope with high groundwater as well as surface water.

Furrow at the downstream end of a field

Where there is a small slope (either natural or by land grading), surface runoff from an individual field may be discharged into a furrow running parallel to the collector ditch at the downstream end of the field. Bank erosion may be prevented by a small dyke along the ditch. The water collected in the furrow is then discharged safely into the open ditch through a short underground pipe (Figure 17).

The same drainage outlet is generally used for removing excess irrigation water, especially in rice fields.

Ridges and furrows

Where crops are grown on ridges with furrows in between, their somewhat higher elevation protects plants from inundation. The furrows also serve as conduits for the flow of excess water, which is collected by an additional furrow at the downstream sides of the field and discharged into the ditch in a similar way as described above.

The ridged fields may have a small slope towards the sides. Where fields are made highest in the middle (e.g. by land grading), this position can also be used for irrigation supply to the furrow (Figure 18).

The length and slope of the furrows depend on the field dimensions and the soil conditions. The length usually ranges from 150 to 250 m. The slope along their length is usually some 0.5–5 per thousand. This guarantees a flow velocity of less than 0.5 m/s, low enough to prevent erosion on most soils.

Convex raised beds and furrows

In flat lands with low infiltration rates, surface runoff is facilitated by shaping the land into raised beds with a convex form between two furrows. Beds run in the direction of the prevailing slope, as shown in Figure 19.

A rather low lateral direction slope of these beds (1–2 percent) is sufficient. In some soils, beds that are too high may become subject to erosion. Raised beds can be made on-farm by repeated directional ploughing or by land grading. The intervening furrows are shallow enough to be passable for agricultural implements and cattle. These furrows should have a

FIGURE 17
Furrow, dyke and pipe to provide surface drainage and to prevent bank erosion in the ditch

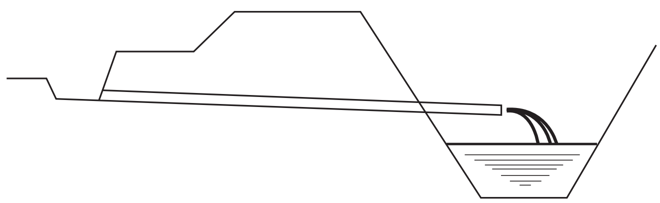


FIGURE 18
Layout of a field for irrigation and surface drainage through furrows

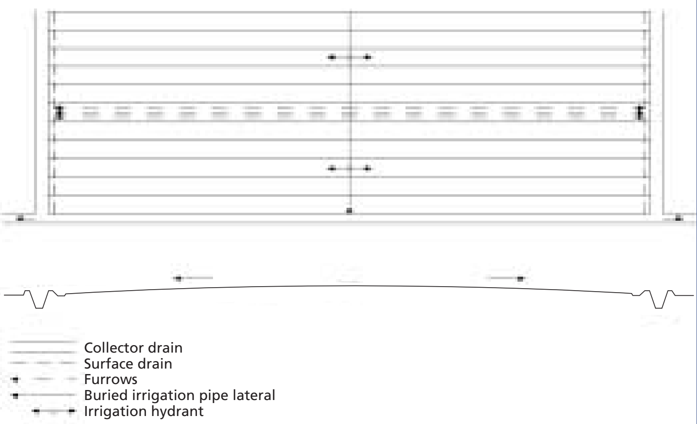
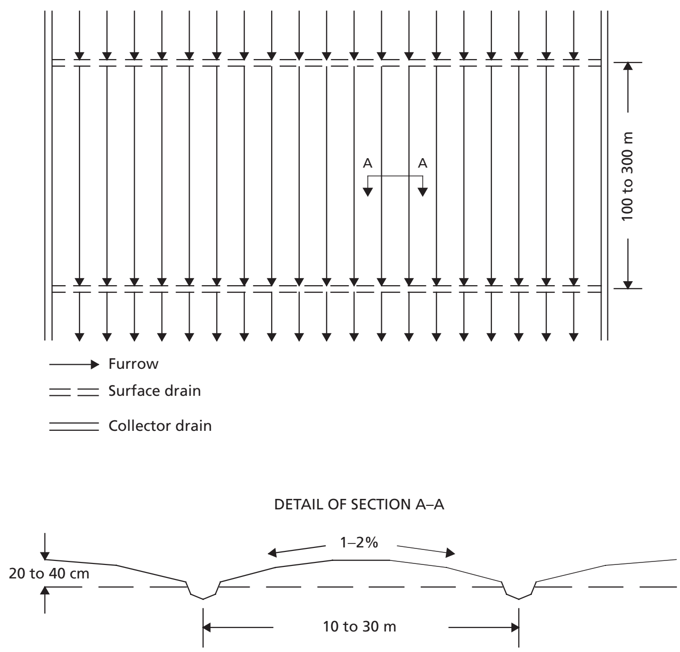


FIGURE 19
Layout of a drainage system with convex beds and furrows



slight longitudinal slope for their discharge, either directly to the collector ditch, as in grassland where the soil is sufficiently protected, or to a system with a downstream furrow acting as a surface drain (described above).

While normal ploughing operations must always be carried out in the same way the beds were ploughed originally, all other farming operations can be carried out in either direction.

The beds have a length of about 100–300 m. The bed widths and their slopes depend on soil permeability, land use and farm equipment. Some recommendations, according to Raadsma and Schulze (1974) and Ochs and Bishay (1992), are:

- 8–12 m for land with very slow internal drainage (K = 0.05 m/d);
- 15–17 m for land with slow internal drainage (K = 0.05–0.10 m/d);
- 20–30 m for land with fair internal drainage (K = 0.1–0.2 m/d).

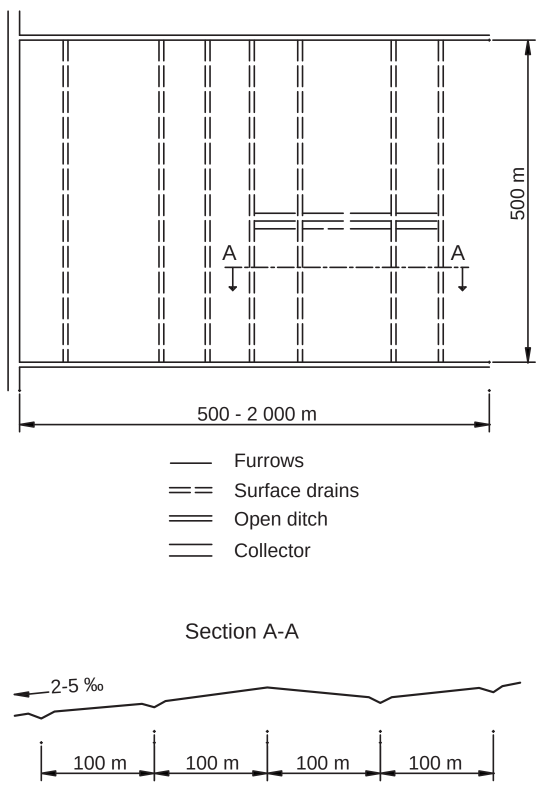
The elevation of the beds, i.e. the distance between the bottom of a furrow and the top of the bed, can range from about 20 cm for cropland up to 40 cm for grassland, where land covering reduces erosion hazard. The furrows between the beds are normally about 25 cm deep with gradients of at least 0.1 percent.

The bedding system does not provide satisfactory surface drainage where crops are grown on ridges, as these prevent overland flow to the furrows. Bedding for drainage is recommended for pasture, hay or any crop that allows the surface of the beds to be smoothed. It is less expensive but not as effective as a parallel furrow drainage system. The system cannot be combined with surface irrigation, although sprinkler and drip irrigation remain possible.

Parallel surface drains at wide spacings

Parallel field drainage systems are the most common and generally the most effective design recommended for surface drainage of flat lands, particularly where field surface gradients are present or constructed. Parallel field drainage systems facilitate mechanized farming operations.

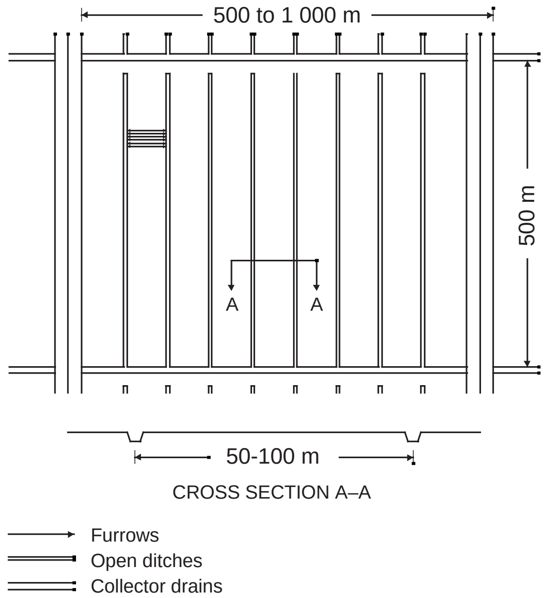
FIGURE 20
Layout of a parallel field drainage system



Shallow field drains are generally parallel but not necessarily equidistant, and spacing can be adjusted to fit farm equipment. The spacing of parallel field drains depends on the crops to be grown, soil texture and permeability, topography and the land slope. Drain spacing generally ranges from 100 to 200 m on relatively flat land, and it depends on whether the land slopes in one direction or in both directions after grading (Ochs and Bishay, 1992). Parallel field drains should usually have side slopes not steeper than 1: 8 (if equipment will be crossing) and longitudinal grades ranging from 0.1 to 0.3 percent (never less than 0.05 percent). Figure 20 shows some of the details for a typical parallel field drainage system.

To enable good surface drainage, crop rows should be planted in a

FIGURE 21
Layout of a drainage system with parallel small ditches



direction that will permit smooth and continuous surface water flow to the field drains. Ploughing is carried out parallel to the drains, and all other operations are perpendicular to the drains. The rows lead directly into the drains, and should have a slope of 0.1–0.2 percent. Where soil erosion is not probable, the row slope may be as high as 0.5 percent.

Under some conditions, deeper field drains are also used to provide subsurface drainage. In several places, especially at the outlets, small filled sections with culverts are often needed to provide access to the fields.

Parallel small ditches

This system employs small ditches 0.6–1.0 m deep. It is used with the dual purpose of removing surface runoff and controlling high water tables. The system is especially useful where the groundwater stagnates on a poorly permeable layer at shallow depth (perched water tables), but also functions to prevent a high rise of the real groundwater during wet periods. In this case, all farming operations are carried out parallel to the drains.

The distance between the small ditches is usually 50–100 m, with a length up to 500 m (Figure 21).

With wider spacings or low-permeability soils, additional shallower ditches can be used instead of the furrows shown in Figure 21. The length of these ditches depends on the spacing of the ditches receiving the discharge. Longitudinal slopes of 2–5 per thousand are recommended in order to secure their discharge and, at the same time, to prevent their erosion. Where surface runoff is a problem, shaping the land will provide either one- or two-sided discharge to these ditches.

Erosion protection for parallel ditches is sometimes needed, especially on arable land. The system in Figure 17, with a small parallel furrow that discharges at its lowest points through pipes into field collector ditches, can be used for this purpose. In pastures, the side slopes of the ditches are usually covered with vegetation, and protection against surface runoff is seldom needed.

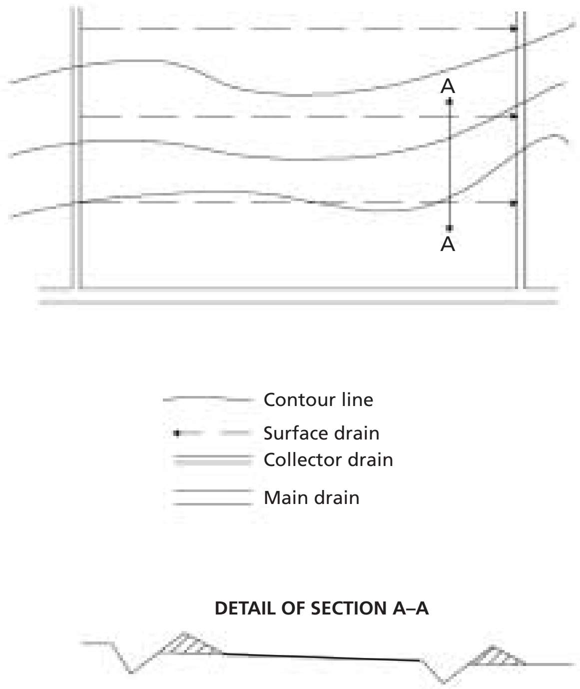
SYSTEMS FOR SLOPING AND UNDULATING LANDS

With undulating and sloping lands, there is ample opportunity for free surface runoff, and often also for natural underground drainage to a deep water table. However, erosion of such lands often causes sedimentation elsewhere, while the runoff leads to inundations in the lowest parts of the area. Groundwater flow may cause seepage in lower places.

Cross-slope drain systems

Where surface runoff threatens agricultural fields in sloping lands, small cross-slope ditches can be made at their lower end, running almost along contours. Ditch spacings depend on factors such as gradient, rainfall, infiltration into the soil, hydraulic

FIGURE 22
Surface drainage with small cross-slope ditches in a sloping field



conductivity, erosion risk and agricultural practices. No general rules can be given. Surface runoff is discharged into open collector ditches running in the direction of the natural slope to discharge into a main waterway (Figure 22).

The open collector ditches should not erode. Therefore, the slope of the land should be not more than a few percent; otherwise, the collector ditches must be provided with weirs or drop structures.

To facilitate agricultural operations, the ditches can be made passable for machinery or (where this is not desired) provided at their ends with a dam and an underground pipe leading to the collector drain. The width of the dam and the length of pipe depend on the type of machines to be used, but a pipe length of about 5 m is sufficient. When constructed, the excavated materials should be used in low areas and on the downhill side of the ditches.

Random field drainage systems

Random drains are applicable where fields have scattered isolated depressions that cannot be easily filled or graded using landforming practices. The system involves connecting one depression to another with field drains, and conveying the collected drainage waters to suitable outlets. Drain depths should be at least 0.25 m, with dimensions depending on the topography of the area and on discharge design, considering the contributing area. This minimum depth is usually applied in the uppermost depression areas. To permit crossing by farm machinery the side slopes should be no steeper than 1:8. The spoil or excavated material from random field drains should be used to fill small depressions or be spread uniformly so that it does not interfere with surface water flows. Smoothing is sometimes required in order to improve the effectiveness of the surface drainage in some of the flatter parts of these fields (Ochs and Bishay, 1992).

Surface drainage in undulating lands

On undulating lands, the layout of an improved drainage system must follow as much as possible the natural topography of the existing watercourses (Figure 23). In narrow valleys, one open drain is usually sufficient, but wider plains may require interceptor or diversion drains, often in addition to contour embankments at the foot of the surrounding hills, to protect areas from flooding caused by surface runoff from higher lying adjacent lands.

A surface drainage system as shown in Figure 23 not only captures runoff from the higher grounds, but it can also intercept groundwater flow. Infiltrated water can reappear in the valley as seepage, causing a more permanent type of waterlogging, and in dry climates severe salinization. This situation is common near the foot of hills bordering flat valleys, and also in low-lying lands that receive tail-end water and/or seepage water from adjacent higher lying irrigated areas.

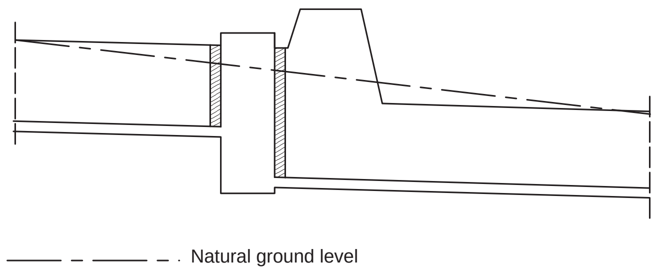
The type of interceptor drains used depends on the relative amounts of runoff and seepage. The former usually dominates, in which case open ditches are needed. Their side slopes, especially the upstream one, must be very flat in order to prevent erosion, and grassed waterways are often useful. A grassed filter strip is also recommended for the upslope side of the interceptor ditch. It catches sediments carried by the water and prevents erosion of the slope.

Where seepage is of importance, deeper ditches are required, and pipe drains can be used if there is little or no surface runoff. Drainage for intercepting subsurface flow is described in Chapter 7 (with more detail in Annex 21).

Some narrow valleys still have a considerable longitudinal slope, the open ditch being liable to erosion. By grading the land, the valley may be divided into compartments separated by small transverse dams. An open drain situated near the centre of the valley collects water from upstream and transports it to the lower end of each compartment. There, a weir or drop structure leads to the next one. In some cases, pipes can be used in combination with inlets of surface water situated at the downhill end of the compartment. Such inlets can be made from large-diameter plastic pipes surrounded with gravel (Figure 24).

FIGURE 23
Random drains in undulating lands

FIGURE 24
Pipe drain, surface water inlet and drop structure in a levelled sloping valley

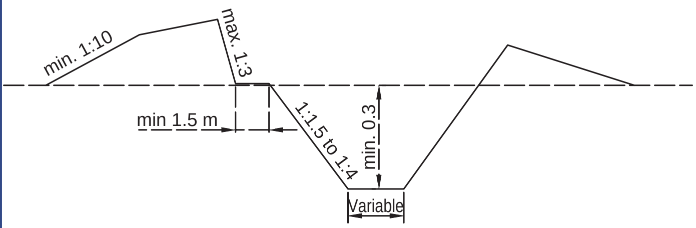


CROSS-SECTIONS OF SURFACE DRAINS

Ditches must have enough capacity to transport the drainage water in wet periods. However, they are sometimes made wider than needed in order to create more storage in the open water system. Such temporary storage is a good way of diminishing the peak outflows from the area, as occurs after heavy rains. Thus, it reduces the required capacity of downstream constructions, such as the larger watercourses, culverts, and pumping stations.

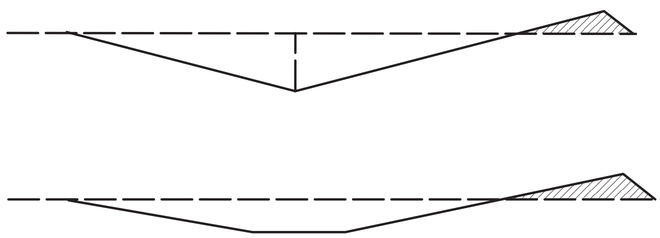
The cross-sections of ditches are usually trapezoidal (Figure 25) although small ones may be V-shaped. Their dimensions vary according to: the expected runoff, the necessity for open water storage, the capacity to be passable for machinery, the risks of bank erosion, and the available means for maintenance.

FIGURE 25
Cross-section of a shallow ditch



Source: Adapted from Raadsma and Schulze, 1974

FIGURE 26
Cross-sections of passable drains



Because ditches tend to hamper agricultural operations, passable drains are often used (Figure 26), designed with respect to agricultural land use rather than on hydraulic properties. Where they tend to erode, they are sown with grasses (grassed waterways). However, grassed waterways are not always a solution because sometimes the grass does not grow or it does not survive the dry season.

As a guide, Table 5 gives some values recommended by the International Institute for Land Reclamation and Improvement (ILRI) (Raadsma and Schulze, 1974; Sevenhuijsen, 1994) and others for small ditches and surface drains.

Ridges and furrows are made by ploughing with ridge-forming agricultural machinery, passable ditches usually by grader, and steeper ones may be constructed by a special plough that shapes the required profile in one pass. Larger ditches are usually made using a backhoe. Details on machinery for construction of surface drains are given in Vázquez Guzmán (1999).

TABLE 5
Recommended dimensions of trenches and open ditches

Type of drain	Depth (m)	Bed width (m)	Side slope (*v*:h)	Maximum side slope (*v*:h)
Furrows	0.20–0.30	-	-	-
Passable drains, V-shaped	0.15–0.30	-	1:10	-
Passable drains, trapezoidal	0.25–0.50	2.0–2.5	1:10	1:8
Ditches, V-shaped	0.30–0.60	-	1:6	1:3
Ditches, V-shaped	> 0.60	-	1:4	1:3
Ditches, trapezoidal	0.30–1.0	As required	1:4	1:2
Ditches, trapezoidal	> 1.0	As required	1:1.5	1:1

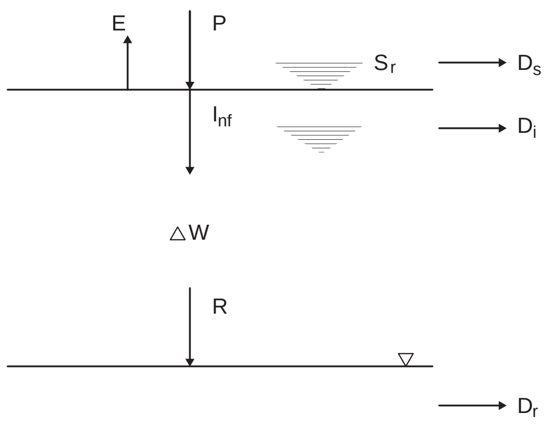
DESIGN DISCHARGES

The discharge of excess surface water to be expected determines not only the dimensions of the structures described in the previous sections, but also those of drainage elements of the main system further downstream (Chapter 5). Peak discharges are caused almost exclusively by rainfall or snowmelt; in rare cases, they stem from irrigation losses. First, the drainage coefficient, defined as the rate of water removal per unit of area, is estimated. Then, the flow rate, which varies with the size of the area, is calculated.

In flat lands, design discharges depend on the amount of excess rainfall to be removed by the surface drainage system during the critical period. The first item can be estimated from the water balance or through empirical formulae.

In sloping land, although surface stagnation is generally not the problem, design discharges are needed to dimension the different components of the main drainage system. Discharges stem from overland runoff processes in the basin considered. There are several methods to obtain the hydrograph of the basin (from this the design discharge can be estimated); some of them are quite sophisticated. Therefore, before describing some of the methods for calculating design discharges in flat and sloping areas, the following section considers some principles on surface runoff.

FIGURE 27
Components of the water balance after a heavy rain



Basic concepts concerning overland runoff

Water balance

The amount of excess rainfall to be drained superficially during a critical period can be estimated from the water balance at the ground surface (Figure 27):

$$S_r = P - E - I_{nf} \quad (1)$$

where:
E = direct evaporation (mm);
I_{nf} = infiltration into the soil (mm);
P = total precipitation (mm);
S_r = excess of water at the soil surface (mm).

The excess of surface water is generally drained freely in sloping lands, but commonly through surface drainage systems in flat lands (D_s). Part of the infiltrated water sometimes interflows through the topsoil (D_i), but most replenishes the unsaturated zone and percolates, recharging the groundwater table (R). Where natural drainage is not sufficient, subsurface drainage (D_r) is required (Figure 27).

The evaporation in a period of a few hours is usually small and negligible compared with the other terms of the water balance.

The amount of rain to be expected with a given frequency in a critical period can be estimated from meteorological data. For extreme values, Gumbel's method may be used to obtain such forecasts (Annex 2 for the method, and Annex 23 for the computer program).

Generally, only rainfall data for 24 hours are available. However, the length of critical periods can be 6–12 hours and, moreover, heavy rainfalls usually occur in this time interval. Nevertheless, estimations for these short periods can be made, for example with the following coefficients (Smedema, Vlotman and Rycroft, 2004):

$$P_6 / P_{24} = 0.5–0.7 \quad (2)$$

$$P_{12} / P_{24} = 0.6–0.8 \quad (3)$$

where:
P_6 = estimation of the amount of rainfall in 6 hours (mm);
P_{12} = estimation of the amount of rainfall in 12 hours (mm);
P_{24} = amount of precipitation in 24 hours (mm).

The distribution of the amount of rainfall accumulated in 6 hours can be estimated with the coefficients shown in Table 6.

Where only rainfall data for one-year return period are available, estimations for 5 and 10 years can also be made with the following coefficients (Smedema, Vlotman and Rycroft, 2004):

$$P_{T5} / P_{T1} = 1.5–2.0 \quad (4)$$

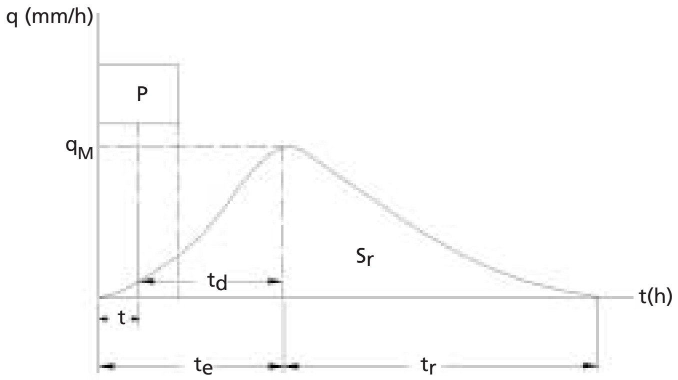
$$P_{T10} / P_{T1} = 1.7–2.5 \quad (5)$$

TABLE 6
Model of distribution of the amount of rainfall accumulated in 6 hours

t (h)	0.0	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0
%	0	2	8	15	22	60	70	78	84	88	92	96	100

Source: WMO, 1974.

FIGURE 28
Hydrograph of surface runoff



where:
P_{T1} = precipitation for 1-year return period (mm);
P_{T5} = precipitation for 5-year return period (mm);
P_{T10} = precipitation for 10-year return period (mm).

For snowmelt combined with a still frozen soil, it should be expected that the total precipitation accumulated as snow during the foregoing frost period (minus some evaporation by sublimation) will become runoff within a few days.

More important to a water balance is the infiltration, which depends greatly on the soil properties. While coarse sands will take almost any rainfall intensity, finer sands (e.g. wind-blown dunes) can show surface runoff during heavy showers. Silt loams have a tendency to form crusts, and some clay soils have a low infiltration rate whereas other well-structured ones may remain very permeable.

However, all soils show an infiltration rate that varies with time. When still dry at the surface, they have a much higher intake rate than after wetting. The main reason is that at the beginning the hydraulic gradient between the wet top and the dry subsoil is very large. Eventually, the intake rate becomes constant because the soil is ultimately saturated and the hydraulic gradient has become unity owing to the effect of gravity only. Another cause of reduced infiltration is that clay swells on wetting. The determination of infiltration forms the main difficulty, but field methods are available (Chapter 4).

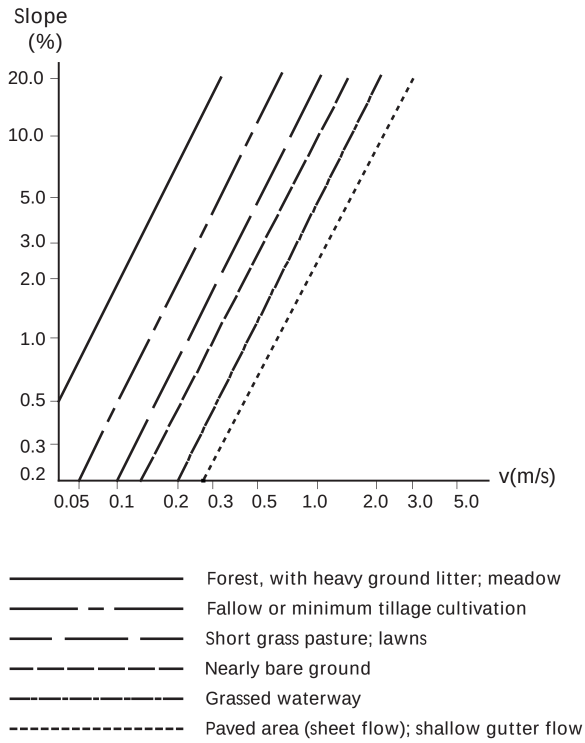
Hydrographs of surface runoff

In an agricultural area, surface runoff depends on some physical characteristics of the basin, such as its form and size, soil conditions, land slope, natural vegetation and land use. The peak flow of drainage water also depends on the characteristics of the main drainage system, such as drain density, cross-sections and gradients of the watercourses, as well as their maintenance conditions (which may restrict their water transport capacity).

After a certain amount of precipitation (P), the specific discharge of surface drainage water at the outlet of the basin (q) increases progressively during the elevation time or time to peak (t_e). Once the maximum value (q_M) is reached, the specific discharge decreases progressively during the recession time (t_r). The time interval between the average time of the storm (t) and the time when maximum discharge occurs is called the lag time (t_d). These concepts are represented by their corresponding symbols in Figure 28, where the total amount of surface runoff (S_r) can also be determined. The hydrograph for total drainage discharge can be obtained by superimposing the groundwater hydrograph on this hydrograph.

FIGURE 29

Flow velocity on the ground surface



Source: Wanielista, 1978.

In a basin, the values of the times described above are constants as they depend on the concentration time (t_c). This is the time interval since the beginning of the storm and the moment when runoff coming from the most distant point from the outlet of the basin contributes to the water flow at the outlet. For basins of less than 1 500 ha, the concentration time can be considered equal to the time to peak (Boonstra, 1994). If the duration of the storm is less than the t_c, only part of the basin contributes to the peak flow at the outlet; if the t_c is higher, the whole area contributes, but generally the rainfall intensity decreases with time. The t_c value depends on the flow velocity and on the length of each section of the main drainage system:

$$t_c = \sum_{i=1}^{n} \frac{l_i}{v_i} \qquad (6)$$

where:

t_c = concentration time (s);

l_i = length of section i of the main drainage system (m);

v_i = average flow velocity in section i (m/s).

Where the drain hydraulic cross-section, the slope and the Manning coefficient are known, the flow velocity in the watercourses can be calculated with the Manning formula. The flow velocity on the ground surface depends on the covering (land use) and slope. Figure 29 shows indicative values.

However, in agricultural areas of less than 50 ha, the concentration time can be estimated with the empirical formula developed by Kirpich:

$$t_c = \frac{K^{0.770}}{3080} \qquad (7)$$

where:

h = difference in elevation between the most distant point in the basin and the outlet (m);

l = maximum distance between the above two extreme points (m);

$s = h/l$ = gradient;

$K \frac{l}{\sqrt{s}}$ = basin constant (m).

Methods to determine design discharges

Different methods have been developed to determine peak water flows and design discharges. The approaches differ from sloping lands, where surface runoff is free, to flat lands. In addition to this distinction, the selection of the appropriate method for a specific project area depends on data availability.

A brief description of each method and the information required to apply it is provided below. Annexes 11–16 provide technical details and application examples. Additional information can be consulted in the literature references, especially in Boonstra (1994) and Smedema, Vlotman and Rycroft (2004).

The batch method for flat lands

For humid flat lands, a simple and approximate method, called the batch method, is based on rainfall, outflow, and storage in different reservoirs, this being:

- soil storage;
- storage in channels and ponds;
- storage by field inundations.

In the batch method, a water balance is set up in order to obtain an approximation of the consequences of different drainage coefficients on crop growth during the critical period. This method can be used to check the effectiveness of existing drainage systems, as shown in an example in Annex 11, or to select the most appropriate specific discharge for designing new drainage systems.

Empirical formulae for flat areas

In flat areas, empirical formulae can also be used. Special formulae are available for specific regions and their use is recommended if they are based on sufficient experience. As an example, the Cypress Creek formula, developed for flat lands in the east of the United States of America, is given in Annex 12. As actual conditions may differ in a project area, this formula can only be used as a first approximation to be verified later.

Statistical analysis of measured flows

The maximum discharge at the outlet of the main drainage system can be determined statistically where a data series of measured flows is available covering a period of at least 15–20 years in an area where the hydrological conditions and the land use have not changed during the historical period considered. Annex 13 shows an example of statistical analysis of measured flows.

Unit hydrograph

In agricultural areas, long data series of measured flows are rarely available to determine statistically the design discharge. However, in basins of 10 000–50 000 ha, where it is possible to assume that 2–6-hour storms are covering the area uniformly, flows have sometimes been measured for different duration rainfalls. Therefore, some hydrographs are available. By using these hydrographs, a precipitation/surface runoff relationship can be obtained. This can be used to predict the surface runoff for other series of rainfall data. The unit hydrograph developed by Sherman is based on this principle. Annex 14 provides details on this method.

Rational formula

In agricultural areas of 100–200 ha, surface runoff is produced just after precipitation where the storage capacity of water in the soil is low. No unit hydrographs are usually available, but there are sometimes some gauge points in the main drainage system. In this case, with water flow data and the characteristics of the section affected by the measurement of the water flow (surface area and hydrological conditions), a relationship can be established between the amount of surface runoff and rainfall. This relationship can be applied to other areas with similar characteristics to the reference section. The rational formula, which is based on the above principle, is described with an example in Annex 15.

Curve Number method

In agricultural areas, the most frequent case is to have rainfall data available but no surface runoff information. In this case, surface runoff can be estimated with the available rainfall data and information on the physical characteristics of the basin concerning the rainfall/runoff relationship, by using a method based on this relationship. A method widely applied is the Curve Number (CN) method. This method was developed by the Soil Conservation Service (SCS) after studies and investigations made in basins with surface area below 800 ha.

To apply the CN method three phases are followed:

1. The amount of surface runoff expected after the design rainfall is estimated, by considering the physical characteristics of the basin.
2. The distribution of the estimated runoff during the storm period is determined by using an undimensional hydrograph.

TABLE 7
Summary guidelines for the selection of method to determine design discharges

Type of lands [1]	Aim	Drainage flow conditions	Drainage basin area (ha)	Available data	Recommended method	Remarks
Flat (slope < 0.2%)	To discharge excess surface water in a critical period	Field and canal storage are relevant; overland flow, interflow and subsurface flow	Up to some thousand	Data series of measured flows (m^3/s) (at least 15–20 years)	Statistical analysis of flows	Most reliable method but information not commonly available; need to check land-use changes.
			Up to some thousand	Rainfall distribution (days or hours) Evaporation (mm/d) Soil storage (mm/d) Storage in channels and ponds (mm/d) Maximum time of ponding (days or hours)	Batch method	Suitable to check performance of existing drainage facilities or to determine the design discharge
			< 5 000	24-hour excess rainfall (mm) Area served by the drain (km^2)	Cypress Creek formula	To be used only as a first approximation as this formula was developed for flat lands in the east of the United States of America
Sloping (slope > 0.5 %)	To discharge peak runoff	Free overland flow	Up to some thousand	Data series of measured flows (m^3/s) (at least 15–20 years)	Statistical analysis of flows	Most reliable method but information not commonly available; need to check land-use changes.
			Up to some thousand	Series of rainfall (mm) Some measured flows for 2–6 hours rainfall Unit hydrographs for 10 mm rainfall	Unit hydrograph	Method based on precipitation/surface runoff relationships not always available
			100–200	Rainfall intensity (mm/h) Area of the basin (ha) Slope (%) Soil infiltrability	Rational formula	To be used only as a first approximation as indicative values developed in the United States of America to determine the surface runoff coefficient are used
			Up to some thousand	24-hour rainfall (mm) In each land mapping unit (ha): natural vegetation and land use; agricultural practices; hydrological soil conditions associated to vegetation density; soil infiltrability; and soil moisture content previous to the design storm	Curve Number	To be used only as a first approximation as the original CN numbers were determined in the United States of America and the specific discharge is based on the SCS unit hydrograph

[1] For lands with slope between 0.2 and 0.5%, other factors (rainfall intensity, soil type, vegetation cover, cultivation methods, etc.) should be considered to classify the land as flat or sloping (Smedema, Vlotman and Rycroft, 2004).

3. The maximum value of the specific discharge is determined in the hydrograph obtained for the total discharge. Then, with the surface area value, the peak flow at the outlet of the main watercourse draining the basin is calculated.

Details of the CN method and an example are included in Annex 16.

This method has a wider scope of application than the rational method as it can be applied in basins with a surface area of several thousand hectares. However, the result obtained can only be considered an estimation of the peak flow. This must be further checked with measured flows in gauge stations in similar locations to the place of application (as original curve numbers were developed in the United States of America).

Table 7 provides summary guidelines for the selection of the appropriate method for determining design discharges of surface drainage according to the data available in one specific project area.

Chapter 7
Subsurface drainage

INTRODUCTION

In flat lands, subsurface drainage systems are installed to control the general groundwater level in order to achieve water table levels and salt balances favourable for crop growth. Subsurface drainage may be achieved by means of a system of parallel drains or by pumping water from wells. The first method is usually known as horizontal subsurface drainage although the drains are generally laid with some slope. The second is called vertical drainage.

A system of parallel drains sometimes consists of deep open trenches. However, more often, the field drains are buried perforated pipes and, in some cases, subsurface collector drains for further transport of the drain effluent to open water are also buried pipes. The drainage water is further conveyed through the main drains towards the drainage outlet. Less common are vertical drainage systems consisting of pumped wells that penetrate into an underlying aquifer.

In sloping lands, the aim of subsurface drainage is usually to intercept seepage flows from higher places where this is easier than correcting the excess water problem at the places where waterlogging occurs from shallow seepage.

LAYOUT OF SINGULAR AND COMPOSITE DRAINAGE SYSTEMS

There are several options for the layout of systems of parallel drains:

- singular drainage systems consisting of deep open trenches flowing directly into open outlet drains of the main system;
- singular drainage systems consisting of perforated pipe field drains (laterals) flowing directly into open drains of the main outlet system;
- composite drainage systems in which perforated pipes are used as laterals and closed or sometimes perforated pipes as collector drains. The latter discharge into the main drain outlet system.

As open trenches hamper agricultural operations and take up valuable land, field drainage systems with buried perforated pipes are usually preferred.

Several factors must be considered in order to select the appropriate drainage system (Martínez Beltrán, 1999), such as:

- the need to discharge surface runoff;
- the slope of the land to be drained;
- the depth of the lateral outlets;
- the maintenance requirements and possibilities;
- the design depth of the water table.

Singular subsurface drainage systems, with pipe laterals only, are appropriate:

- where, in addition to the subsurface flow, it is necessary to discharge excess rainfall through a shallow surface drainage system;
- where a certain amount of water must be stored in the open drains in order to reduce the peak flow in the outlet system;
- in very flat lands where the drainage flow is high and the available slope is low.

As an example of a singular subsurface system, Figure 30 shows the layout of the system installed in the Lower Guadalquivir Irrigation Scheme, Spain. Field drains are laid at 10-m spacing and open collector drains at 500-m spacing.

FIGURE 30
Layout of a singular drainage system of parallel drains

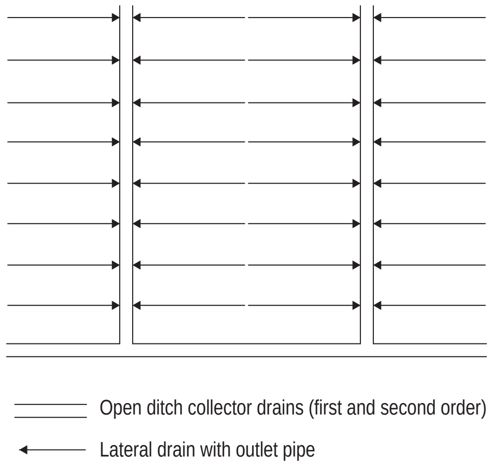
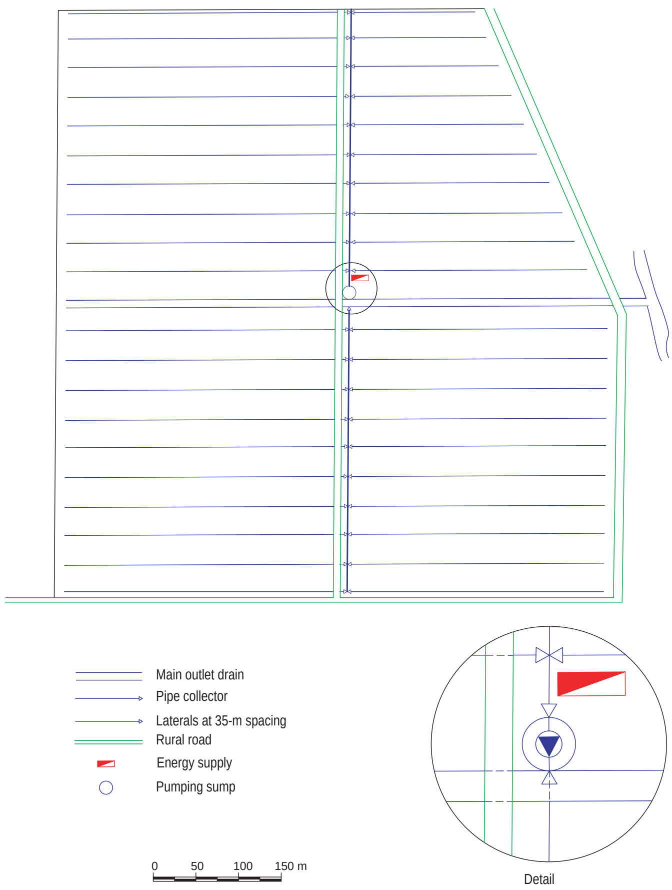


FIGURE 31
Layout of a composite subsurface drainage system with central sump pumping



Composite subsurface drainage systems, with pipe lateral and collector drains, are generally recommended in the irrigated lands of arid regions because:

- The depth of field drains is usually greater than in the temperate zones and, consequently, large excavations would be required if open ditches were used as field or collector drains.
- The excess rainfall is generally negligible; as a consequence, drainage rates are low (although often very salty) and thus the discharge of a considerable number of parallel pipe drains can readily be collected and transported by a subsurface collector system.
- Weed proliferation increases the maintenance costs of open ditches.

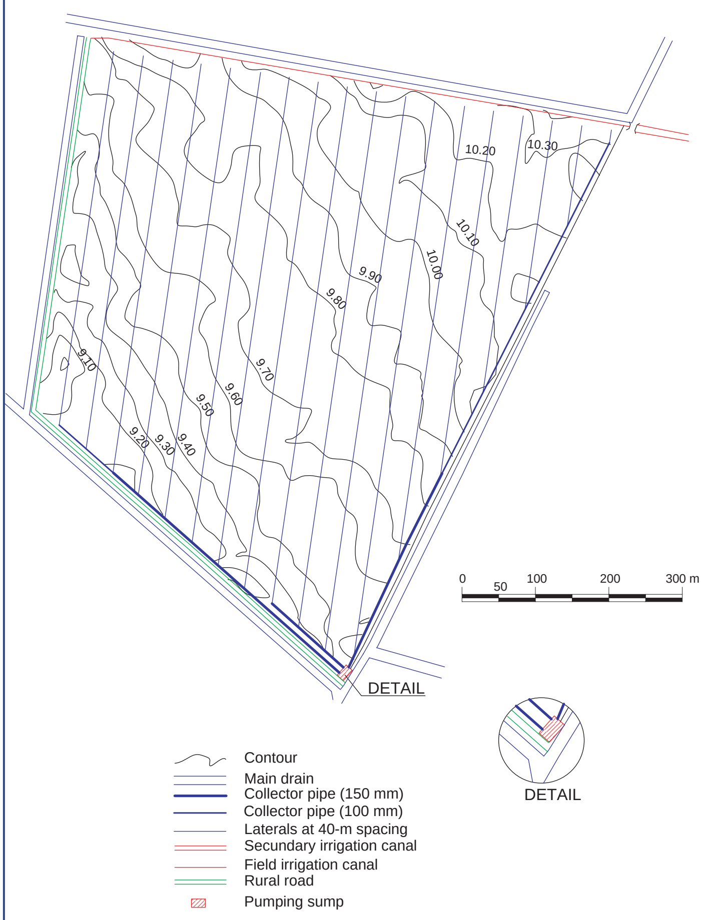
This type of system is common in the Nile Delta, Egypt, where subsurface drainage systems discharge only the necessary leaching to control soil salinity and keep the groundwater level sufficiently deep to prevent salinization caused by capillary rise of saline groundwater.

Composite systems are also recommended in: sloping areas where soil erosion must be controlled and/or drainage problems are mainly manifest in patches or in topographic lows; in areas where the land is very valuable; and in the case of unstable subsoils that cause unstable banks of open drains.

In some areas, especially where the maintenance or availability of deep open drains is difficult, groups of pipe collector drains discharge into tanks (sumps), from where the water is pumped into a shallow main outlet system (where the external water level is above the field groundwater level). This is the case for arable crops and mango orchards in some parts of the Lower Indus Plain, Pakistan, and in some areas of the Ebro Delta, Spain, where horticultural crops are grown. In the latter case, subsurface drainage systems, as in Figure 31, have been installed to control the saline groundwater table.

Controlled drainage is sometimes used to slow drainage during dry periods, and increasingly to control water requirements of rice in rotation with dry-foot crops. Then, the water table is maintained at a higher level by technical means, such as temporary plugs in subsurface drainage systems, raising seasonally the open drain water levels, or rising lateral/collector pipe outlets. Thus, a certain amount of water is saved from flowing away during droughts, or when fields are flooded during a rice crop. In Egypt, during rice cultivation in otherwise dry-foot crop cultivated land, such plugs are used to close the orifice in the bottom part of a specially constructed overflow wall inside inspection maintenance hatches of composite drainage systems. Water tables can also be controlled by subirrigation, where water from outside sources flows into the drain if the outside water level in the whole area is kept high for a considerable period. Apart from these uses, it is effective for preventing clogging with iron compounds, and the outflow of nitrates from the drainage system may be reduced by denitrification. However, great care should be taken with such systems in arid areas subject to salinization.

FIGURE 32
Layout of a composite subsurface drainage system with extended field drains



Although there are no physical restrictions on the length of subsurface field drains, it is usually governed by the size of the agricultural fields and the maintenance requirements of the drain. In composite systems, the same applies to the length of collectors. Where cleaning is required, the maximum length of pipes is usually limited by the maximum length of the cleaning equipment, which is about 300 m.

However, where there is enough slope and no constraints (owing to field dimensions) on designing pipe drains longer than 300 m, extended systems can be designed. However, they require a suitable access construction for cleaning devices at about every 300 m. As longer drains require larger diameter pipes, maintenance hatches should be installed to facilitate the connection between pipes of different diameters, as well as for inspection and cleaning, notably in the case of collector drains. Accessible junction boxes should be placed at the junctions between laterals and collectors.

Figure 32 shows details of an extended composite drainage system of the type installed in the irrigation districts of northwest Mexico. In this example, the pipe diameter changes only in the collector drains, and a second collector drain has been installed on the southern side instead of increasing again the diameter of the first collector drain.

FIGURE 33
Cross-section of two pipes of a subsurface drainage system

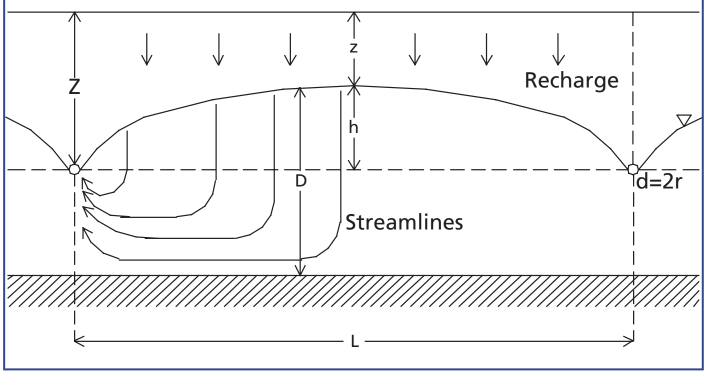
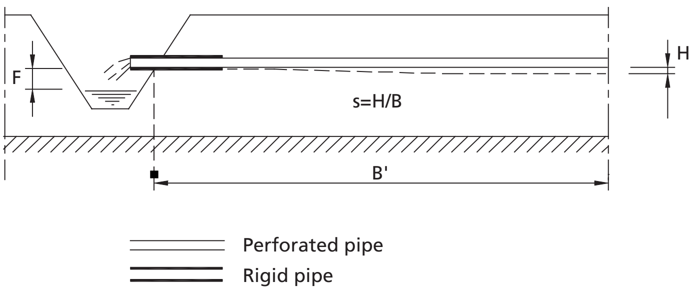


FIGURE 34
Longitudinal section of a lateral drain



DESIGN CONCEPTS AND APPROACHES

In designing horizontal subsurface drainage systems, in addition to the drain length *B* described above, the following dimensions are needed:

- drain depth *Z*;
- drain spacing *L*;
- drain slope *s*, or total allowed head loss in the drain at design discharge intensity *H*;
- drain diameter *d*.

Moreover, for composite systems, the dimensions of the collectors (depths, slopes and diameters) must be determined.

The type of pipes and possible types of protective drain envelopes must be selected, preferably from among the types and sizes that are readily available in the country. In addition, the method of installation (trenchless or in dugout trenches) and the method of maintenance must be chosen.

Figure 33 shows some drainage parameters (the average thickness of the groundwater-bearing layer *D* is also shown).

Figure 34 (longitudinal section) shows other dimensions of a field drain, such as the drain slope *s*, as well as the outlet structure into the open drain and its freeboard *F*.

The drain slope *s* is defined as the difference in elevation between the upstream and downstream ends *H* divided by the horizontal distance *B'*. However, for small *s*, the drain length *B* can be taken instead of *B'*. In practice, $s = H/B$ is usually used. The difference is negligible where $s < 0.01$.

The design dimensions, such as the average drain depth, drain slope and allowed head losses, are usually the same for large areas, often over an entire project. Sometimes, they are prescribed quantities. On the other hand, drain spacings, lengths and pipe diameters may vary considerably from place to place, as spacings depend on crops and soil conditions, lengths on the system layout, and diameters on spacings, lengths and slope.

The lengths and diameters of field and collector drains depend considerably on the dimensions of the plots to be drained, thus on the parcelling of the area. Both are interrelated, as the longer the drains are, so the greater their diameter must be. As the price of pipes increases with diameter, in the case of long drains, where all diameters of pipes are readily available, it can often be profitable to begin upstream with smaller pipes, using increasing diameters further downstream. The switch in diameter has to be done at a logical place (maintenance hatch), otherwise mistakes can be made during installation and/or problems may occur with the cleaning of the drains.

The drain spacing is also related to cost. In singular drainage systems, the costs are almost inversely proportional to the spacing.

The drain spacing and the drain depth are mutually interrelated – the deeper the level of the drains so the wider the drain spacing can be. Thus, increased spacing might lower the amount of the subsurface drainage work, and consequently the costs. However, in some cases, the cost advantages of greater drain depth may be offset by an increase in construction cost per unit length, by larger diameter of field drains, by higher costs of deeper collectors and ditches, and by costlier O&M, especially where deeper drains need a lower outlet level (which might indicate that pumping is necessary or pumping costs are higher). Moreover, deeper drainage is often restricted by other factors, e.g.: by soil conditions, as in heavy clay soils with shallow impervious layers; outside water levels, as happens in lowlands; or, less frequently, by the availability of appropriate machinery.

For example, in Egypt, during often relatively short fallow periods, groundwater must be lowered in order to limit topsoil salinization by capillary rise. Detailed cost calculations resulted in the conclusion that deeper and wider spaced drainpipe installation only entailed modest installation cost savings owing to the extra cost stemming from larger drain diameters (although the total installation cost was still lower compared with drains installed at a shallower depth). For example, a system where the water level between drains is designed at 1.50 m below field level with a hydraulic head of 0.30 m requires a drain depth of 1.80 m and drain spacing of 80 m. During the fallow period in this arid area, the actual water level between drainpipes will be slightly higher than 1.80 m. Where the pipes are installed at 1.60 m depth to fulfil the requirement of a water table at 1.50 m, the pipes have to be spaced at 50 m. This means a depth gain of only 20 cm, for a cost increase of about 60 percent. During the fallow period, the water table depth is then about 1.60 m (instead of 1.80 m). However, in the heavy clay soils of the Nile Delta, capillary rise is very slow, and as irrigated cropping intensity is high, both depths are sufficient to prevent soil re-salinization.

Once a design drain depth has been selected, there are two different approaches to calculating the drain spacings:

- for conditions of steady-state groundwater flow towards the drains, where the flow in wet periods is assumed to be constant in time;
- for non-steady-state flow conditions, where flow is time-dependent.

In the former case, an outflow intensity, which is assumed constant, is used as a criterion; in the latter case, the time to obtain a given drawdown after a critical recharge event is taken as design datum.

The steady-state method can be used where the recharge to the water table is approximately constant during a critical period. Then, it is possible to design the system with a discharge equal to the recharge. If, at a design water table height, the inflow of water to the soil is constant and equal to the drain outflow (so that storage effects can be ignored), the water balance in the saturated zone is in equilibrium and the groundwater level remains at a constant depth.

In practice, steady-state flow is a good approximation:

- in temperate zones with long periods of low-intensity rainfall that are critical for drainage;
- in areas recharged by deep upward seepage from a semi-confined aquifer;
- in areas where there is lateral seepage from outside waterbodies;
- in irrigated lands where water is continuously applied through high-frequency irrigation methods, such as drip irrigation and central-pivot systems.

The steady-state approach is less applicable where high recharges occur in a short period of time only, such as after heavy irrigation or sudden rainfall. In this case, the water balance is not in equilibrium as when the recharge is higher than the discharge, the groundwater level rises; and when the recharge ceases, the system is still draining, and the water level falls. The conditions where soil water storage is important in design are frequent in:

- areas with heavy showers of short duration, common in some Mediterranean areas and in the humid tropics;

- in irrigated lands with intermittent irrigation where applications of 60–120 mm are common.

However, under certain assumptions, non-steady drainage flow conditions can be converted mathematically to steady flow conditions. Therefore, steady flow considerations can be used as a substitute for processes that are essentially non-steady in nature.

Technical details for the steady and non-steady drainage design approaches are given below.

DRAINAGE CRITERIA

In humid temperate areas, agricultural drainage must be able to prevent damage to crops in periods with abundant rainfalls occurring with a frequency of once in 2–5 years. In arid areas, drainage should prevent the accumulation of harmful amounts of salt and provide adequate drainage after a heavy irrigation or after heavy rains as occur in monsoon-type climates.

Artesian conditions (deep aquifers under pressure) often lead to upward seepage flow of water from deeper layers. This flow has a great influence on the design of a drainage system. It often makes a "normal" drainage unable to prevent waterlogging or salinization. Thus, extra measures are necessary in upward seepage areas. Where the seepage water can be reused, vertical drainage may be an option for controlling the water table.

Drainage requirements result in two important factors for drainage design, which are used in the steady-state determination of drain spacing: the specific discharge q; and the hydraulic head midway between two drains h, which should be available for causing the required groundwater flow. This head represents the drain depth Z minus the required groundwater depth z (Figure 33).

For non-steady calculations, an additional input parameter is needed. This is the storage coefficient (μ) (described in Chapter 4).

Therefore, the dimensions of a subsurface system depend on the following drainage criteria:

- the design groundwater depth z or the depth of the water table below the soil surface, midway between drains, during times of design discharge (for crop season, fallow periods, etc);
- the outflow intensity q or the design discharge of the drains per unit area, and usually expressed in millimetres per day;
- in non-steady cases, the time in which the groundwater should regress from the initial high water tables (or complete inundation) to a given water table depth (midway between the drains) is used. This recession time depends on crop and temperature; for horticultural crops, it is usually short, especially under high temperatures.

Fundamental criteria such as design groundwater depth and design outflow are derived from guidelines, local experience, research plots, theoretical considerations and models. For example, the DRAINMOD model (Skaggs, 1999) allows evaluation of criteria or checks on those derived by other means.

The following sections provide some indications for values of these drainage criteria.

Design groundwater depth

Critical to crop growth and soil trafficability is the depth at which the groundwater remains/fluctuates under critical circumstances. At design discharge for field crops, this depth z is usually of the order of 0.9 m, but it varies by crop, soil and climate. For shallow-rooting horticultural crops on pervious soils, depths of 0.5 m may be reasonable. Tree crops require greater depths than vegetables, but the latter can stand water near the surface only for a few hours and, thus, are vulnerable to extreme high water table situations, especially when temperatures are high.

In temperate zones, controlled drainage permits two design groundwater depths: a deep depth to provide aeration and trafficability in periods with excess of water; and a shallower depth to facilitate subirrigation in dry periods. Controlled drainage also permits high water levels for nitrate reduction and preventing iron precipitation in the pipes.

In climates with low-intensity rainfall, the following minimum depths to the steady-state design groundwater depth (midway between drains) during short wet periods are usually recommended:

- 0.3–0.5 m for grassland and field crops for design outflows of about 7–10 mm/day;
- 0.5–0.6 m for vegetables grown on sandy loam soils.

In arid areas, two design depths are frequently required: one during the cropping season to provide aeration to the rootzone (unless rice is grown); and a second one for fallow periods in order to prevent capillary rise and associated salinization (where seepage from irrigation elsewhere would cause too high groundwater levels). As the drainage discharge is also different during the cropping and the fallow seasons, the drain spacing/depth has to be designed for the most critical period (the smallest h/q), bearing in mind the required groundwater depth during the fallow period (smaller h, lower q).

In irrigated lands, the following design depths for groundwater for steady-state design outflow (in dry climates, e.g. 2 mm/d) can be used as a starting point:

- 0.8–0.9 m for field crops;
- 1.0–1.2 m for fruit trees, depending on soil texture.

In the case of irrigation of rice, controlled drainage permits the elevation of the groundwater level up to the ground surface in order to prevent excessive water losses. Here, there is no danger of salinization owing to the absence of upward flow in the inundated soil.

To control capillary rise and related soil re-salinization processes, groundwater must remain below a certain depth in periods without rain or irrigation. This safe design depth is determined mainly by the capillary properties of the soil and the salinity of the top layers of the groundwater mound. In particular, silts and silt loams require deep drainage.

Where the critical depth to control capillary rise is excessive and higher groundwater levels have to be accepted, then, in order to secure acceptable soil salinity levels, the salts accumulated during the fallow period must be leached by irrigation where there is no excess rainfall.

Design outflow

In humid temperate areas, the design discharge occurring with a frequency of once in 2–5 years is usually taken as the design criterion. Under these circumstances, crops should not suffer from waterlogging.

In arid climates, prevention of salinization is the main purpose of drainage, and for most cases a discharge capacity of 2–4 mm/d is sufficient for leaching. Annex 7 provides details on design discharge for salinity control in irrigated land.

In humid tropical areas (including those with monsoon climates), the rains are often so heavy that the infiltration capacity limits recharge, and surface runoff may occur. In addition, the subsurface drainage system is usually unable to cope with the inflow. The same applies to other climates with intense rains. In such cases, a combination with a surface drainage system is needed. After the rains, when the soil is saturated, the subsurface drainage system then lowers the groundwater to a sufficient depth in a reasonable time (non-steady state, see below), whereas in the dry season it prevents the accumulation of salts.

TABLE 8
Examples of design discharges

Climate	q (mm/d)
Humid temperate climates	7–15
Humid tropical climates	10–15
Irrigated lands in arid climates	1–2

The exact figures for the design discharge q are extremely dependent on the local climate conditions and/or irrigation practices (Annex 6). Therefore, the outflow intensity is usually derived from local experience. Where local criteria are not available, the use of drainage models is recommended. To indicate the order of magnitude, Table 8 gives some examples of design discharges in current use.

Where considerable seepage occurs, the amount of seepage water must be added to the design discharge, and the pipe sizes adjusted accordingly. For example, this is the case where relief wells are used to tap the aquifer – the drains must be able to convey this extra amount of water.

Groundwater lowering

In the non-steady-state design method, both z and h are functions of time. After a heavy rain or irrigation, the groundwater should fall a given depth in a given time so that its depth z increases (e.g. 0–0.30 m in 4 hours for vegetables). Because Z cannot change with time, h also falls by the same amount. Such a requirement can be used to calculate drain spacings. In this non-steady case, the storage coefficient and not the discharge is used as an input parameter. In this case, the drain discharge rate varies with time.

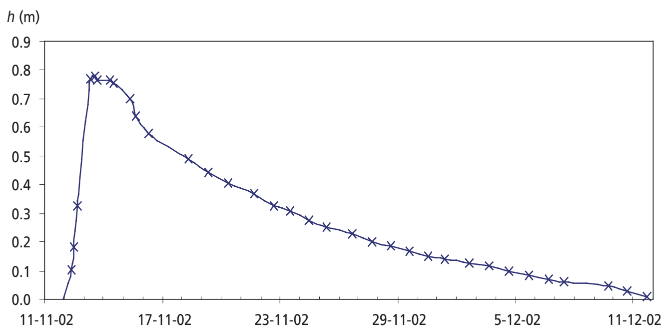
Where heavy rains or irrigation have caused water to stand on the surface, the following criteria for the lowering of the groundwater could be used under non-steady flow:

- for horticulture, a lowering after complete inundation of 0.30 m in 4–6 hours;
- for most crops in hot climates, a lowering or 0.30 m in 1 day;
- in cool climates, a lowering of 0.20 m in 1 day.

In irrigated lands, in addition to these criteria, the soil provides storage for the percolation water, and the drainage system must be able to remove this storage before the next irrigation. Therefore, between two irrigation applications, the drawdown of the water table must be similar to the elevation produced by the irrigation water losses (Figure 35). A low outflow criterion (e.g. 2 mm/d) is usually sufficient for this purpose.

For example (Figure 35), where 40 mm of percolation is stored in a soil with a storage capacity of 5 percent, this gives a rise of 0.8 m. The groundwater level must be low before the following irrigation, for example 30 days, and the stored water must be removed in this period, requiring on average a drainage coefficient of 1.3 mm/d.

FIGURE 35
Example of lowering of the groundwater table between two consecutive monthly irrigations



Under these circumstances, the best approach is to design the system with a steady-state method and a low outflow intensity, and then to simulate its behaviour after complete flooding. If the outcome is not satisfactory, the steady-state discharge must be changed by increasing the steady outflow criterion. This will lead to a narrower spacing, which can be tested again for its non-steady behaviour. The process is repeated until a satisfactory solution is obtained.

TABLE 9
Examples of depths of pipe lateral drains

Region	Drain depth (m)	Remarks
Temperate	1.0–1.5	from 1.0–1.2 m in rainfed areas to 1.0–1.5 m in irrigated lands
Humid tropical	0.8–1.5	
Arid (sandy soils)	1.0–1.5	capillary rise is limited in height
Arid (clay soils)	1.5–2.0	capillary flow is very slow
Arid (silt loam soils)	2.0–3.0	capillary rise and seepage of saline water are major concerns

SYSTEM PARAMETERS

Drain depth

The selection of the drain depth is a crucial and early decision in a drainage project. This is because of the technical aspects involved, and because of the direct influence of the drain depth on the overall cost of the system. As mentioned above, deeper drains allow wider drain spacings with fewer drains per unit area, but other factors, such as construction and O&M costs of field and main drains and outlet structures, play a role in the overall cost.

The depth of the laterals Z is equal to the sum of the depth to the water table z and the hydraulic head h both taken midway between two drains (Figure 33). Under steady-state conditions, the required groundwater depth must be adjusted by the head loss h required to cause groundwater flow towards the drains:

$$Z = z + h \tag{8}$$

or, with a given drain depth, limited by the discharge level, etc.:

$$h = Z - z \tag{9}$$

where:
h = head loss for flow in soil, at design discharge (m);
z = groundwater depth midway, at design discharge (m);
Z = drain depth (m).

As mentioned above, the design value for z depends on climate, crop requirements (crop calendar, rooting depth, crop salt tolerance), and soil and hydrological conditions. Moreover, to select an adequate drain depth Z, the hydraulic conductivity and the soil stability of the layers situated above the impervious barrier should be considered (because drains should not be installed in or below impervious layers). Unstable soils such as quicksand are to be avoided. Although quicksand can be handled, it requires a special installation technique with sometimes modified machines. In addition, the drain depth is often limited in practice by the water level at the outlet of laterals or collectors into the main drainage system.

The minimum depth of open trenches for subsurface drainage is about 0.6 m, and for pipes it is about 0.8 m. Pipes installed at a shallower depth may become clogged if crop or tree roots (orchards; windbreaks) penetrate into the drain through the pipe slots. In addition, shallow pipe drains can be damaged during subsoiling operations, which are common in the management of clay soils with low permeability. In cold climates, pipes must be deep enough to prevent freezing. Table 9 gives some indications of commonly applied depths of installation pipe drains.

Drain spacing

Drain spacing is an important factor because the cost of subsurface drainage is related closely to the installed length of drains per unit area:

$$C = \frac{10000}{L} C_u \tag{10}$$

where:
C = installation cost of the system (in terms of monetary units per hectare);
C_u = cost per unit length of installed drains (in terms of monetary units per metre);
L = drain spacing (m).

Although the field drains form a major component of the cost, collectors and the main drainage system are important items, as are the capitalized costs of O&M. Therefore, if it is decided to install deeper drains to allow wider spacings, the additional costs of the required deeper main system must be compared with the savings on field drains.

There are various methods of calculating drain spacings from the drainage requirements and the soil characteristics. Of these, the soil permeability, the layering and anisotropy are especially important factors (Chapter 4). The calculation methods fall into the two categories mentioned above: steady and non-steady flow. In steady-state calculations, the inputs (apart from the soil data) are the design head loss h or midpoint water table height and the design discharge q. In the non-steady case, the design factors comprise a prescribed increase in groundwater depth z with time in combination with the storage coefficient μ.

Steady-state methods may form a first step in designing drain spacing, but non-steady methods can represent the changing conditions more accurately. Therefore, as a second step, drain systems designed tentatively with steady criteria may be subjected to more realistic, variable inputs in order to evaluate the design. In this way, the design can be tested and adapted as necessary.

Annexes 17 and 19 describe the respective drainage equations for steady and non-steady groundwater flow that are commonly used for drain spacing calculations. Where vertical seepage is relevant, the Bruggeman method (Annex 18) can be applied.

Chapter 8 and Annex 23 provide descriptions of available computer programs for designing subsurface drainage, and some calculation examples.

The distance between two parallel laterals may vary between 50 and 150 m in permeable soils. In pervious clay soils, spacings of 20–50 m are common; in heavy clay soils and certain silt loams, spacings of 10–20 m are frequently required (Martínez Beltrán, 1999). In irrigated lands with an arid climate, the drain spacings are usually much wider than under rainier conditions owing to smaller discharges of the drains.

Drain slope and allowed head loss in the pipes

The cost per unit length of installed field drains C_u (Equation 10) is related closely to the drain diameter. This diameter depends on the expected outflow and on the available hydraulic head difference along the drain. Consequently, the drainpipe might be constructed without any slope. However, for practical reasons (e.g. to reduce the incidence of sunken pipe stretches which silt up easily and may ultimately cause blocked pipes) and cost-saving considerations, slopes are designed as high as possible in order to minimize the drain diameter.

In sloping lands, drains can be laid parallel to the ground slope, especially where the surface has been graded. Thus, the pipe depth is maintained along the drain. The usual criterion for sloping drains is that, at design discharge, no water is standing above the drain at its upstream end. However, interceptor drains, intended to collect and remove seepage water entering the top of the field, should follow the groundwater or soil surface contours.

In flat lands, a shallower drain depth of the upper end of the drain must be chosen in order to maintain a minimum slope. However, very small slopes (even horizontal drains) are possible, if the drains are constructed carefully and are sometimes used if subirrigation is to be practised. In such horizontal drains, water must be allowed to temporarily stand above the drain in wet times, which by itself is not a problem as long as it remains deep enough below the soil surface. The argument that slope is needed to transport sediments out of the lateral is valid only at slopes of more than 1 percent, which are seldom possible in flat lands where drain slopes are usually in the range of 0.1 to 0.3 percent. Such a flat slope is not enough to remove incoming soil by the water flow. Therefore, precautions against clogging are needed, i.e. careful construction of the drains and, in many cases, the use of protective drain envelopes.

However, horizontal drains are not recommended because the installation tolerances are never negligible even where the drainage machine is equipped with a laser device. In practice, minimum slopes of 0.07 percent or in extreme cases 0.05 percent can be considered.

Drain diameter

The design of the drain diameter should take into account the available diameters and the costs thereof. As cost increases with diameter, finances play a role in the choice of diameter.

In designing the drain diameter, the total head loss in the drain during a very wet period H is considered. It is often required that, at design discharge, no water be standing above the upstream end of the drain. Therefore, with a slope of 0.2 percent and a length of 250 m, the available head for pipe flow is 0.50 m. If in flat land the drain outlet is 1.50 m below the surface, the depth of the drain at the upstream end will only be 1.00 m. With an allowed head loss of 0.50 m, there will be no water above the pipe. In this case, drain slope and allowed head loss are the same. However, the same drain with the same outlet depth, but with a slope of 0.10 percent, has an upstream depth of 1.25 m below surface. With an allowed head loss of 0.50 m, there will be 0.25 m of water standing above the drain at design discharge, but the depth of this water will also be 1.00 m. The same reasoning applies to any slope below 0.2 percent and even for a horizontal drain. This example shows that there is no direct relation between drain slope and allowed head loss. Therefore, the allowed head loss in the pipes will be taken as an input for calculations of the required drain diameters. This head loss determines the groundwater depth near the drain during critical times at the least favourable places.

The diameter of lateral and collector drains can be calculated using various formulae, which are based on the laws for pipe flow. These calculations are different for smooth and corrugated pipes, because of pipe roughness. The available head loss at design discharge and the amounts to be drained under that condition form the base for calculations concerning pipe diameters. Annex 20 describes formulae commonly used for drain diameter calculations. Description of available computer programs is also given in Chapter 8 and, in more detail, in Annex 23.

Pipes with an outer diameter of 80–100 mm are common for wide drain spacings; 65–80-mm pipes are frequently used in systems in the temperate regions; and 50–65-mm pipes are used in drainage systems for clay soils.

DRAINAGE MATERIALS

FAO Irrigation and Drainage Paper No. 60 (FAO, 2005) provides full details about materials for subsurface drainage and their use. Therefore, only limited reference is made here.

Pipes

Corrugated plastic pipes with adequate perforations are most frequently used as field drains because of their flexibility, low weight and their suitability for mechanical installation, even for a drain depth of 2.5 m and more. Polyvinyl chloride (PVC) is commonly used in Europe, and polyethylene (PE) pipes are commonly used in North America, but both are technically suitable. Although PE material is less resistant to soil loading than PVC and is sensitive to deformation at high temperatures, it is more resistant to ultraviolet radiation during storage and handling, and is less brittle at temperatures below 3 °C. However, the choice is usually based on availability and price considerations.

Water enters into the drainpipe through perforations. These openings are uniformly distributed in at least four rows. The perforated area varies from 1 to 3 percent of the total pipe surface area. Where the perforations are circular, diameters range from 0.6 to 2 mm. Elongated openings have a length of about 5 mm. In Europe, the perforation area should be at least 1 200 mm^2 per metre of pipe (FAO, 2005).

Baked clay or concrete pipes about 30 cm long are still sometimes used, the former for field drains, and the latter mostly for large collector drains, especially where the required diameter is more than 200 mm. These pipes may be considered as “technically smooth”. Clay tiles have a circular cross-section with an inside diameter of 50–200 mm. For collectors, the inside diameters of concrete pipes range from 100 mm upwards. Where the diameter is more than 300 mm, reinforced concrete should be used. Where the sulphate content of the groundwater is high, it is necessary to use high-density cement resistant to gypsum. Additional details for clay and concrete pipes can be consulted in FAO (2005).

Drainage pipes should fulfil technical specifications that are verified in laboratories before installation. For plastic pipes, these specifications include impact resistance, weight, flexibility, coilability, opening characteristics and hydraulic characteristics (and with concrete pipes, resistance to sulphates). The draft European standard on corrugated PVC pipes has been published by FAO (2005).

Pipe accessories and protection structures

At the upstream end of the drain, caps are used to prevent the entry of soil particles. Snap-on couplers are used to connect plastic pipes of the same diameter, and plastic reducers are used where the pipes are of different sizes. Where couplers and end caps are not available, the drainpipes can be manipulated in the field to fulfil the same functions.

Rigid pipes, of sufficient length to prevent the penetration of roots of perennial plants growing on the ditch bank, are used as outlets. These pipes are also used where a drain crosses unstable soil, or a row of trees that may cause root intrusion.

Details on pipe accessories and protection structures are described in FAO (2005).

Envelopes

To prevent soil intrusion in unstable silt and sandy soils, drainage pipes should be surrounded by envelope material. Envelope material can be made of: fine well-graded gravel; pre-wrapped organic materials, such as peat, or natural fibres, such as coconut fibres; or woven and non-woven synthetic materials, such as granular polystyrene and fibrous polypropylene. In soils consisting of stable clays at drain depth, such envelopes may often be omitted, which reduces drainage costs.

Envelopes prevent the entrance of soil particles, but they also promote the flow of water into the drain. A good envelope conveys water to the perforations, thus considerably reducing the entrance resistance. Moreover, voluminous envelopes increase the effective radius of the drain, from the pipe radius to that of the pipe plus envelope thickness. This further promotes water flow and improves the hydraulic efficiency of the drain.

In addition to the entrance resistance restriction by soil clogging, drainage pipes have to face other problems, such as clogging of the pipe openings by penetration of roots into the pipe, by biochemical processes such as ochre formation, and by precipitation of less-soluble salts, such as gypsum and carbonates, which are difficult to prevent.

It is not easy to predict the need for an envelope but tentative prediction criteria are available. These criteria are based on clay content, soil particle size distribution, and salt and sodium content of the soil solution.

Fine, well-graded gravel forms an excellent envelope, but the high cost of transport and installation constrains its use in practice. Organic fibres may decompose with time. Therefore, synthetic envelopes, such as pre-wrapped loose materials and geotextiles with appropriate opening sizes, are in widespread use.

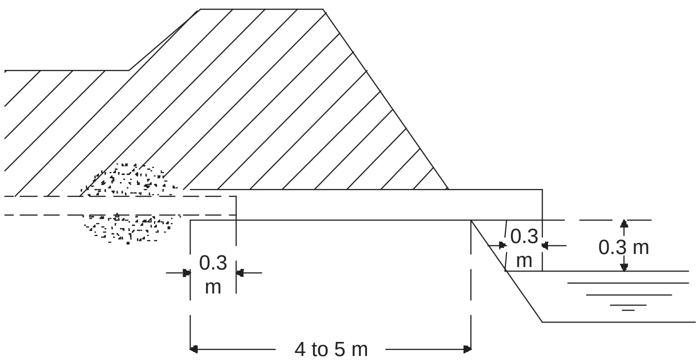
Envelopes should also fulfil technical specifications, such as: thickness, mass per unit area, characteristic opening size and retention criteria, hydraulic conductivity and water repellence, and some mechanical properties.

Guidelines for predicting the need for envelopes and for selection of the appropriate material are available (FAO, 2005; Vlotman, Willardson and Dierickx, 2000), but the selected material must be field-tested for local conditions. Requirements for envelopes used for wrapped pipes are also included in the draft European standard (FAO, 2005).

Auxiliary structures

Where singular subsurface drainage systems are used, a rigid outlet pipe (Figure 36) is necessary. The rigid pipe should be long enough for water to flow directly into the outlet drain ditch water in order to prevent erosion of the ditch bank and to impede clogging by roots of bank vegetation. As these pipes hamper mechanical ditch cleaning, the bank may also be protected by concrete or plastic chutes.

FIGURE 36
Outlet of a lateral drain into a collector drain



In composite subsurface drainage systems, cross-connectors, T-pieces and elbows are used to join buried laterals and collectors. Junction boxes or fittings are used to connect pipes where the diameter or the slope of the pipe changes. Where inspection and cleaning are required, maintenance hatches replace junction boxes.

Blind and surface inlets can be used to evacuate surface water through subsurface drainage systems. However, provision should be made to prevent entry of trash and eroded soil by using appropriate envelope material.

Details on connection structures, outlets and special structures on pipe drains for controlled drainage are described in FAO (2005).

INTERCEPTION DRAINAGE

Inflow from higher places (Figure 37) or from leaky irrigation canals can sometimes be captured by interceptor drains, especially where it passes through relatively shallow aquifers. The effect of interception drainage is only significant if the

FIGURE 37
Interception drainage on a hill slope

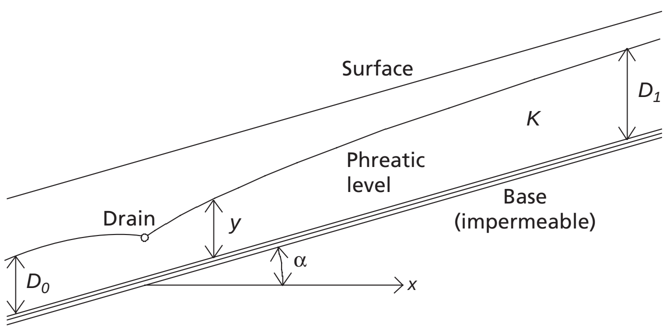
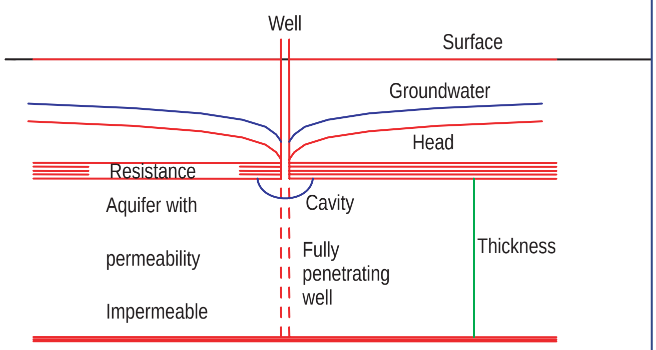


FIGURE 38
Vertical drainage



impermeable layer is about at the drain depth. Otherwise, the effect is roughly only proportional to the percentage that the interceptor drain depth is of the thickness between the phreatic level and the impermeable layer.

Interceptor drains can take the form of pipes or open ditches. However, with the latter, the stability of the side slopes is often problematic where large volumes are to be captured. Better solutions are gravel-filled trenches provided with a suitable pipe of sufficient capacity to carry the discharge.

Annex 21 provides further details and calculation methods, and Chapter 8 describes a computer program (more detail in Annex 23).

Vertical drainage

Vertical drainage is achieved by an array of properly spaced pumped wells that lower the head in an underlying aquifer (Figure 38) and lower the water table.

Vertical drainage can be used successfully under special physical circumstances:

- the presence of a good aquifer underneath (unconfined or semi-confined), so that wells give a good yield;
- a fair connection between the soil to be drained and the aquifer, so that the lowered head in the aquifer results in a lower groundwater table. The layers between the aquifer and soil to be drained must be permeable enough to convey the recharge of the groundwater by rainfall and irrigation losses to the aquifer. In other words, the resistance between groundwater and aquifer must not be too high;
- the system should be sustainable.

The aquifer should not be pumped dry. Where the water is to be used for irrigation or for municipal supply, a suitable quality is required that must not deteriorate rapidly with time. This sometimes occurs because vertical drainage may attract salt from deeper layers (where the deeper parts of the aquifer are brackish or saline, in which case, vertical drainage can only be a temporary solution). Chapter 2 has already addressed the other water quality aspects, e.g. the presence of toxic substances.

As constant pumping is needed, the O&M costs are rather high. This leads to the following economic restrictions:

- The method is only economically viable where the pumped water is fresh and can be used for the intended purposes. However, mixing with better quality waters can sometimes be a solution where undiluted use is not allowed.
- Where the water is too salty, it causes disposal problems in the project area that need special provisions. These add to the costs, making vertical drainage still more uneconomic in these cases.
- The O&M costs and complexities of relatively dense well-fields limit the application of vertical drainage.

Despite these constraints, the method is applied widely in some areas where the soil and aquifer conditions are favourable and where the pumped water can be used. In such areas, it has often led to a depletion of the aquifer and sometimes to extraction of salts from deeper layers.

Vertical drainage may also be an option in locations with severe seepage problems. Here, pumping is not always needed, because of overpressure in the aquifer. Where technically feasible, vertical extensions of a horizontal drainage system may be a cheap substitute.

Relief wells consist of vertical wells that reach slightly into the aquifer. In a drain trench, vertical boreholes are made into the aquifer and provided with blind-ended perforated pipes as well casings. They are usually made of corrugated plastic and are the same as the drain itself. These pipes are connected with the horizontal laterals by T-junctions. The method has been successful in several cases. However, the extra discharge of water may be a burden for the outlet system, and its salinity may harm downstream users.

Annex 22 provides details on the design and calculation of vertical drainage systems. A computer program for drainage by vertical wells is described in Chapter 8 (more detail in Annex 23).

Chapter 8
Calculation programs for drainage design

INTRODUCTION

Since the advent of the electronic computer, models have found wide application. For drainage, various models are used in research and engineering (Table 10). Universities and research institutes have developed sophisticated models, and governmental institutes, engineering companies and individual consultants use various calculation methods for design. Information on applications of GIS for planning and design of land drainage systems can be consulted in Chieng (1999). Computer programs for drawings, such as topographic and layout maps, and detailed design of open drains and ancillary structures of the main drainage system are widely used by engineering firms. Additional information on computer applications related to land drainage is given in Smedema, Vlotman and Rycroft (2004).

TABLE 10
Some models involving drainage

Model	Reference	Remarks
DRAINMOD	Skaggs, 1999	extensive model for drainage
DUFLOW	STOWA, 2000	non-steady one-dimensional canal flow
ESPADREN	Villón, personal communication, 2000	calculates drain spacings using several formulae, in Spanish
SAHYSMOD	ILRI, 2005	influence of aquifer on seepage, drainage and salinity
SWAP	Van Dam *et al.*, 1997	extensive model for saturated/ unsaturated soil including drainage

The CD–ROM version of this FAO Irrigation and Drainage Paper includes several programs for drainage design, largely based on formulae given earlier in this publication. The aim is not to clarify the underlying fundamentals or provide great sophistication, but rather to facilitate their direct application to drainage design under practical circumstances. In addition, some related problems are addressed that have influenced the design itself, such as backwater effects and seepage (as described in earlier chapters).

The programs are in FORTRAN and run under both Microsoft Windows and DOS. Inputs are in the form of questions and answers. Choices between various possibilities have to be made by typing certain numbers, and input data have to be provided in the same way. The units are metric, in accordance with FAO standards.

GENERAL STRUCTURE

The programs have a common basic structure, allowing easy retrieval. For this purpose, certain rules have to be followed regarding notation of decimals, the abbreviated name of the project and the location.

The following items are considered:

- The program mentions its name and purpose in order to check that it is appropriate. If not, it can be terminated easily.
- A point must be used as decimal separator. A question is raised about national usage; if a comma is the norm, a warning is given.
- A "project" name of a maximum of four characters is required (letters or numbers in single quotes). This shortness is because of the restricted length of filenames under DOS.
- Within this project, several locations can be used, each of which characterized by a name of a maximum of ten characters in single quotes (letters or numbers).

TABLE 11
Programs and file listing

Program	Background	Description	Purpose
Extreme values			
GUMBEL	Annex 2	Annex 23	Extreme values (rainfall, discharges)
Calculation of permeability			
AUGHOLE	Annex 3	Annex 23	Permeability from auger-hole data
PIEZOM	Annex 3	Annex 23	Permeability from piezometer data
Spacing of drainpipes and wells			
SPACING	Annex 17	Annex 23	Steady-state flow
ARTES	Annex 18	Annex 23	Drainage under artesian pressure
NSABOVE	Annex 19	Annex 23	Non-steady flow, above drains only
NSDEPTH	Annex 19	Annex 23	Non-steady flow, also below drains
NSHEAD	Annex 19	Annex 23	Non-steady flow, heads given
WELLS	Annex 22	Annex 23	Vertical drainage by well network
Drain diameters			
DRSINGLE	Annex 20	Annex 23	Single drains, one diameter
DRMULTI	Annex 20	Annex 23	Multiple drains, various diameters
Main drainage system			
BACKWAT	Annex 10	Annex 23	Backwater effects on main system
Interceptor drains			
INCEP	Annex 21	Annex 23	Homogeneous profile
INCEP2	Annex 21	Annex 23	Drain or ditch in less permeable topsoil

- At the end of the session, the project receives a unique name for the output file, showing the results for the various locations.
- For easy retrieval, all filenames are listed in a file LIST**, where ** indicates the kind of program used (e.g. SP for drain spacings).

Annex 23 provides further details. Table 11 lists the different programs.

SPECIFIC PROGRAMS

Extreme values

GUMBEL

Extreme values are the largest and smallest elements of a group. In many cases, they obey Gumbel's probability distribution. Applications are: the highest precipitation in a certain month and the highest discharge of a river in a year.

The program GUMBEL allows an easy method for interpolation and extrapolation. For a given return time, it calculates the value to be expected. A graph is shown to enable visual inspection of the fit of the data and a possible trend. A poor fit indicates uncertainty in the basic data; a distinct upward (or downward) trend that the data do not obey the GUMBEL distribution and that the extrapolated values are far too low (or too high). In this case, other methods must be used.

By extrapolation, a prediction can be given for return periods of 100 or 1 000 years. However, the uncertainty becomes considerable at such long times. Nevertheless, such extrapolation is valuable for engineering purposes, such as for the height of river embankments needed to withstand a "100-year" flood. The flood will almost certainly not take place after 100 years, but it has a probability of 1 percent of occurring next year (and maybe tomorrow) and has a good chance of occurring in a lifetime. Last, it must be borne in mind that natural and human-induced changes may influence the events in question. Examples are: the increase in impermeable surfaces (roads and cities) and deforestation will increase drainage flows; and climate changes (whether natural or human-induced) will have either positive or negative effects.

For drainage design, return periods of 2–10 years are often taken (2–5 years for agricultural field systems, 5–10 years for the main system), but these must be far higher if human safety is involved. For example, in the Netherlands, return periods up to 10 000 years are used for sea dykes in critical areas.

The theory can be found in Chapter 4 (with more detail in Annex 2). Annex 23 provides details about the use of the programs and examples.

Calculation of permeability

AUGHOLE

The auger-hole method is widely used for measuring soil permeability. The water level in an auger hole is measured before pumping, and afterwards its rise is determined. In dry soils, the fall of the water level after filling can be observed, but this "inverse" method is less reliable. Moreover, some soils swell slowly and have a much lower permeability in the wet season than when measured dry.

The program AUGHOLE can process the data obtained for both the normal and inverse methods. The results within the same auger hole are usually quite consistent. Where more than one observation is made in the same hole, the program takes the average and gives its standard error. When large variations are encountered, a message appears: "Not reliable".

Between different holes, even nearby ones, differences may be considerable owing to local soil variations. However, in predicting drain spacings, these errors are diminished because the resulting spacings are proportional to the square root of *K*.

The resulting *K* values can be used as input for programs such as SPACING and the NS series.

The principles and the basic equations are given in Annex 3. Annex 23 provides details about the use of this program and an example.

PIEZOM

In an open auger hole, a kind of average permeability is measured for the layers between the groundwater level and the bottom of the hole. Where data are required for a specific layer, Kirkham's piezometer method can be used. The auger hole is covered by a tightly fitting pipe, and, with a narrower auger, a short open cavity is made below its open bottom. Alternatively, an auger hole is covered partially by the open pipe and the remainder forms the cavity below. In the former case, the diameters of pipe and cavity are different; in the latter, they are almost equal. As with the auger-hole method, water levels are measured at different times. The permeability is measured of the layer in which the cavity is located.

The underlying theory is explained in Annex 3. The program PIEZOM can find the permeability from the collected data. Annex 23 provides details about the use of this program and an example.

Spacing of drainpipes and wells

SPACING

This program includes an earlier program for the Töksös–Kirkham equations (J.H. Boumans, personal communication, 1999).

The program allows the calculation of spacing of pipe drains under steady-state conditions in cases where upward or downward seepage towards deeper layers is insignificant. If such seepage is considerable, ARTES must be used instead. If non-steady situations have to be considered, a preliminary steady-state solution by SPACING can be checked with programs from the NS series.

In SPACING, up to five soil layers can be considered, and anisotropy may be accounted for. However, in practical cases, sufficient data are seldom available and estimations are usually needed. Nonetheless, the effect of additional layers and anisotropy can be investigated by entering trial values.

The theory is given in Annex 17. Annex 23 provides details about the use of this program and an example.

NSABOVE, NSDEPTH and NSHEAD

These programs analyse the non-steady behaviour of a proposed or existing drainage system after complete or nearly complete saturation of the soil after heavy rainfall, snowmelt or irrigation.

NSABOVE can be used if the drains are at the impermeable base, so that the flow is above drain level only. The program gives the expected lowering of the groundwater table from zero to a given depth within a given time. These data can be based on agricultural requirements that depend on the tolerance of the crop or on soil tillage and trafficability needs.

NSDEPTH is used if also deeper layers take part in the drainage process. As in NSABOVE, the criterion is the lowering of the groundwater. It uses numerical calculations, and allows inclusion of the radial and entrance resistances near the drainpipe and the limited outflow capacity of the drainpipe and the main drainage system.

NSHEAD is similar to NSDEPTH but mentions the head above drain level instead of the water depth.

The related principles and equations are given in Annex 19. Annex 23 provides details about the use of these programs and examples.

ARTES

Artesian conditions may cause upward seepage where a deeper lying aquifer is under pressure, or natural drainage (downward seepage) where the pressure is lower than the pressure of the shallow groundwater. These conditions can exert a large influence on the layout of a subsurface drainage system. Strong upward seepage can lead to failure, whereas natural drainage can diminish the required intensity and even make subsurface drainage unnecessary.

In principle, geological information and a model such as SAHYSMOD are needed. However, for a first estimate, ARTES can be used to see whether serious effects are to be expected. At this stage, good data about the aquifer and the top layer are seldom available, but estimates can provide some insight about the effects to be expected. The program gives two solutions – one for a wet and one for a dry season. The latter is usually critical because of capillary rise and salinization hazards.

The principles and the basic Bruggeman equations are given in Annex 18. Annex 23 provides details about the use of this program and an example.

WELLS

Instead of drainage by a network of pipes or open channels, a network of wells may be used (vertical drainage). However, this method can only be used under specific circumstances:

- A good aquifer must be present.
- This aquifer must have sufficient contact with the overlying soil, so that pumping can influence the groundwater levels.
- There must be no danger of attracting brackish or saltwater from elsewhere.
- Overpumping must be avoided, although it may be allowed temporarily.

Under favourable circumstances, such a network may be useful. The program provides a simple approach for steady-state conditions. However, a more sophisticated method, based on geohydrological studies, is recommended for estimating the effects such as overpumping and salinization.

The principles and equations are given in Annex 22. Annex 23 provides details about the use of this program and an example.

Drain diameters

DRSINGLE and DRMULTI

For long drains and wide spacings, and especially for collectors, it is often more economical to start with a small diameter and change to a larger size further on. Moreover, different materials may be used in the same drain. The program DRMULTI calculates such "multi" drains. Which of the two programs should be chosen depends on the local availability of pipes and on local prices.

The theory of drainpipe flow is given in Chapter 7 (more detail in Annex 20). Annex 23 provides details about the use of these programs and examples

Main drainage system

BACKWAT

Where the main system discharges into a river or the sea, or indeed any waterbody that shows fluctuations in water level, backwater effects occur. Especially during high outside levels, they interfere with the discharge from above. Open outlets may even allow a rapid flooding of the area.

The program gives an initial steady-state approach to such backwater effects. It gives the steady backwater curves, positive at high outside levels, negative at low ones.

The theory is given in Chapter 5 (more detail in Annex 10). Annex 23 provides details about the use of these programs and examples.

Interceptor drains

INCEP and INCEP2

In undulating terrain, waterlogging and salinization often occur at the foot of slopes or below higher irrigated or rainfed lands. Stagnation of groundwater also occurs in places where the thickness of an aquifer or its permeability diminishes suddenly. This may be caused by the presence of a rock sill. A related problem is the interception of water leaking from irrigation canals (although then an improvement of the irrigation system is a better solution).

The programs calculate the width of a drain trench or ditch sufficient to cope with the intercepted flow. INCEP is valid for a homogeneous profile, INCEP2 for a drain or ditch located in less permeable topsoil. The size of the drains needed to discharge the flow must be found from the program DRMULTI, using the inflow per metre given by the programs INCEP.

In homogeneous soil, a normal drain trench is wide enough in many cases. However, drains in a less permeable top layer require much wider trenches or broad ditches. A practical solution is to put more than one drain in such locations. As the hydrological circumstances are often complicated and little known, the programs can only give global guidelines. In practice, the problem is usually solved by trial and error – if a single drain is insufficient, more are added.

The theory can be found in Chapter 7 (more detail in Annex 21). Annex 23 provides details about the use of the programs and examples.

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Annex 1

Estimating soil hydrological characteristics from soil texture and structure

It is possible to derive rough estimates of the hydraulic conductivity (K) and the drainable pore space (μ) from observations of the soil profile. This is because these soil hydraulic qualities depend on soil texture and structure. Table A1.1 average presents μ values, compiled by FAO (1980) and based on data from the USBR (1984), together with K values estimated from the μ/K relationship. For soils with distinct horizontal layers, the vertical K may be taken as being at least 10 and on average 16 times lower than the horizontal one.

As these estimates may be imprecise, more realistic K values are obtained through field measurements, as described in Annex 3.

However, interpreting the soil structures mentioned in Table A1.1 may not be easy. It should be done through observations of soil profiles, but shallow groundwater levels often prevent excavation of soil pits. Moreover, soil texture and structure should be evaluated when the soil is moist throughout.

However, in special cases, it is possible to estimate drain spacings directly from the visual aspects of the soil profile, as was done by people with detailed local experience in the Zuiderzee polders, the Netherlands, where it was the only possible method – drain spacings of 8, 12, 16, 24, 36 and 48 m were distinguished and the choice between possibilities was possible.

For pure sands (almost without clay and silt), an estimate is:

$$K = \frac{m_{50}^2}{2000}$$

where:

K = permeability (m/d).

m_{50} = median size of grains above 50 µm. Half of the weight is above this size, half below.

TABLE A1.1
***K* and μ values according to the soil texture and structure**

Texture (USDA)[1]	Structure	μ	K (m/d)
C, heavy CL	Massive, very fine or fine columnar	0.01–0.02	0.01–0.05
	With permanent wide cracks	0.10–0.20	> 10
C, CL, SC, sCL	Very fine or fine prismatic, angular blocky or platy	0.01–0.03	0.01–0.1
C, SC, sC, CL, sCL, SL, S, sCL	Fine and medium prismatic, angular blocky and platy	0.03–0.08	0.1–0.4
Light CL, S, SL, very fine sL, L	Medium prismatic and subangular blocky	0.06–0.12	0.3–1.0
Fine sandy loam, sandy loam	Coarse subangular block and granular, fine crumb	0.12–0.18	1.0–3.0
Loamy sand	Medium crumb	0.15–0.22	1.6–6.0
Fine sand	Single grain	0.15–0.22	1.6–6.0
Medium sand	Single grain	0.22–0.26	> 6
Coarse sand and gravel	Single grain	0.26–0.35	> 6

[1] C: clay; L: loam; S: silt; s: sand.
Source: Adapted from FAO, 1980, with further elaboration.

The presence of silt (< 50 µm) and especially clay (< 2 µm) will lower this value considerably. Therefore, this formula should not be used for such soils.

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Annex 2
Statistical analysis of extremes

GUMBEL'S METHOD

The Gumbel distribution can be used for extrapolating from a limited number of extreme values (Gumbel, 1954 and 1958). The basic data appear in groups, such as the daily rainfall in August (31 days per year), or the water levels in a river per year (365/366 days). The highest value in such a group is the extreme. The groups should contain at least ten elements, and the minimum number of extremes (often years) is at least ten.

The method assumes that the underlying process remains constant. This supposition is doubtful because of recent climate changes, which also influence data such as river flows. These changes are especially noticeable in the extreme values. Therefore, the method should be used with care.

Extreme values are obtained as follows:

- Select the highest (sometimes lowest) value in a group, e.g. the highest autumn rainfall or the highest river discharge in a year. Each group should contain at least ten values.
- These extremes are sorted according to their magnitude in order to prepare for further analysis.

The probability that a certain value x does not exceed a limit x_0 is:

$$P(x \le x_0) = \Phi(y) = \exp[-\exp(-y)] \quad \text{with} \quad y = \alpha(x_0 - u) \tag{1}$$

where:

P = probability;
n = number of extremes;
u = constant (shift);
x = values of the extremes. The average is $\bar{x}$ the standard deviation is s_x;
x_0 = limiting value;
y = reduced Gumbel variable, with average c and standard deviation s_y. For y and for a very large number of observations, c = 0.57722 = Euler's constant;
α = constant (slope).

The probability that x exceeds x_0 is:

$$P(x > x_0) = 1 - \Phi(y) \tag{2}$$

The return period T is the number of groups in which the limit x_0 is exceeded. If there is one group per year, T is in years (as in the above examples). T is defined as:

$$T = \frac{1}{1 - \Phi(y)} \tag{3}$$

For the x values, the procedure is:

$$\bar{x} = \frac{\sum x}{n}$$

$$s_x = \sqrt{\frac{\sum x^2 - \frac{(\sum x)^2}{n}}{n-1}}$$

$$s_y = \frac{\pi}{\sqrt{6}} = \text{ standard deviation of } y.$$

Table A2.1 shows the values derived by Kendall for a smaller number of observations.

The line $y = \alpha(x - u)$ has two parameters: the slope α, and the shift u. They can be found by plotting on Gumbel probability paper, usually with the return period T on the horizontal axis, the value of the extremes on the vertical. The line may be drawn visually through the points to allow extrapolation. In this way, the once-per-century rainfall or the river discharge can be estimated. This is even possible for much longer return periods.

The program GUMBEL calculates the parameters automatically and provides estimates for the extremes to be expected with a certain return period.

For agricultural drainage design, a return period of 2–10 years is often taken, 2–5 years for field drainage and even 10 years for crop systems with high planting costs, and 5–10 years for the main system where it does not affect inhabited places.

By extrapolation, a prediction can be given over much longer periods of time in order to obtain estimates for values to be expected once in 100 years (the once-per-century value) and even for much longer times. However, the uncertainty of the estimates becomes very large for such longer return periods. Moreover, for such periods (and even for a century), the basic data series cannot be considered as constant, owing to human and geological influences.

Nevertheless, such a prediction is valuable for engineering purposes, e.g. the height of a river embankment able to withstand a "100-year flood". This will almost certainly not occur 100 years later, but it has a chance of 1 percent of occurring next year.

The influence of climate changes can be analysed by comparing data from the last 10–20 years with earlier ones (where available), and it is wise to employ the worst prediction. Where not different, the basic data include recent changes already.

TABLE A2.1
Values of c and s_y as a function of n

n	c	s_y
10	0.495	0.950
15	0.513	1.021
20	0.524	1.063
25	0.531	1.092
30	0.536	1.112
40	0.544	1.141
50	0.548	1.161

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Annex 3

Field methods for measuring hydraulic conductivity

INTRODUCTION

The *K* value can be measured directly in the soil layers situated below the groundwater level using the methods described below. Less reliable methods are used to estimate the saturated hydraulic conductivity above this level. For well-moistened granular soils, the soil permeability for saturated flow can be estimated from the capillary hydraulic conductivity of the unsaturated zone. However, this is not the case in well-structured soils where this permeability is caused by cracks, holes or other macropores. Infiltrometer or inverse auger-hole methods are often used as a compromise. They measure conductivity under "almost saturated" conditions.

The field methods for determining *K* are based on a basic principle: water flows through a volume of soil, whose boundary conditions are known, and the discharge is measured; the *K* value is calculated by applying an equation derived from Darcy's Law applied to the specific geometry of the soil volume.

The following paragraphs review the suitability of the field methods most commonly used to measure the soil hydraulic conductivity (auger-hole, piezometer, and inverse auger-hole). The methods are different according to the groundwater depth at the time of measurement. Details on these methods can be found in the bibliographic references (Van Hoorn, 1979; USBR, 1984; Oosterbaan and Nijland, 1994; Amoozegar and Wilson, 1999).

AUGER-HOLE METHOD FOR DETERMINING SOIL PERMEABILITY

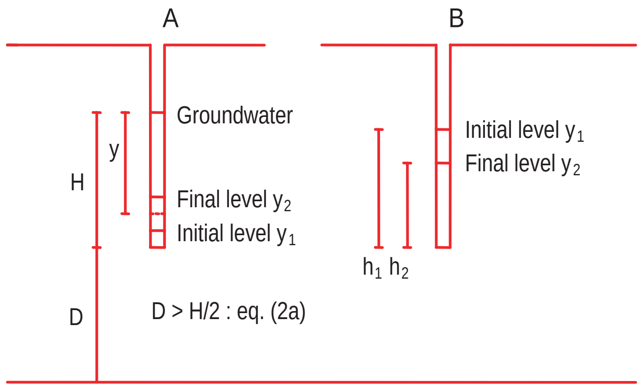
The auger-hole method (Van Beers, 1983) is the most suitable way of measuring the *K* value of saturated homogeneous soils down to a depth of about 3 m. It is based on the relationship between the *K* value of the soil surrounding a hole and the rate at which the water level rises after pumping. The method measures the saturated permeability in a rather large volume, which is an advantage in view of the large variability in natural soils.

Method

This method for determining the soil hydraulic conductivity (Figure A3.1) consists of the following steps:

1. Make a hole of known depth with a soil auger of known diameter to a depth of at least 50 cm below the water table. In unstable soils (e.g. sand), a perforated filter may be needed to support the walls.
2. Find or estimate the depth of any impermeable soil layer. If more than 100 cm below the bottom of the hole, assume an infinite depth.

FIGURE A3.1
Definition sketch: A: auger-hole method, B: inverse method



3. Pump water out (e.g. with a bailer) several times and let that water flow back into the hole.
4. Let the groundwater (where present) fill the hole until equilibrium. For impermeable soils, return the next day; for permeable soils, a few hours are sufficient (sometimes even a few minutes).
5. Measure the groundwater depth below soil surface.
6. Pump water out.
7. Measure the rise of the water level over time. Time intervals should be short initially.

Example

The following data can be considered:

- Depth of 8-cm diameter hole: 150 cm;
- Groundwater at equilibrium: 50 cm;
- Water level, first measurement: 85–83 cm, Δt = 20 s;
- Water level, second measurement: 80–78 cm, Δt = 24 s;
- Water level, third measurement: 70–68 cm, Δt = 31s;
- Impermeable base: deep (300 cm).

From these data (all distances below soil surface), the average permeability K follows. This value is the mean value (mainly horizontal) between the groundwater table and a few centimetres below the bottom of the hole.

It should be noted that:

- The permeability of different layers can be found from measurements in holes of different depths, but this is not very reliable; the piezometer method is better.
- The first measurement may deviate because water is still running off the wall; in this case, it should be discarded.
- Measurements soon after lowering by pumping the water out are preferred.

The above methods cannot be used without an existing groundwater table at the time of measurement. The following methods can be used in such cases. However, they are less reliable.

The inverse method, also known as the Porchet method, may be also applied to determine the saturated hydraulic conductivity above the groundwater level. In this case, water is poured into an augered hole and the rate of lowering of the water level inside the hole is measured (Figure A3.1). The measurements are taken after water has been infiltrating for a long time until the surrounding soil is sufficiently saturated (in order to diminish the effect of unsaturated soil on the rate of drawdown). The equation used to calculate the K value has been derived from the balance between the water flowing through the side walls and bottom of the hole, and the rate of lowering of the water level in the hole. The basic assumption is that the flow gradients are unity. Although less reliable than the measurements using an existing water table, it is often necessary where measurements must be made outside a wet period in dry soils. However, many dry soils swell so slowly that their permeability can only be reliably measured by the auger-hole method during the wet season.

Van Hoorn (1979) made a comparison between normal and inverse methods and found reasonably corresponding values for K, thus confirming the assumption about the gradient.

Theory

According to Ernst and Westerhof (1950), Van Beers (1983) and Oosterbaan and Nijland (1994), for the auger-hole method, the saturated soil permeability is calculated using:

$$K = C\frac{dy}{dt} \tag{1}$$

in which:

$$C = \frac{4000\,\frac{r}{\bar{y}}}{\left(\frac{H}{r} + 20\right)\left(2 - \frac{\bar{y}}{H}\right)} \tag{2a}$$

where the bottom of the hole is far above the impermeable base ($D > H/2$), or:

$$C = \frac{3600\,\frac{r}{\bar{y}}}{\left(\frac{H}{r} + 10\right)\left(2 - \frac{\bar{y}}{H}\right)} \tag{2b}$$

where the bottom of the hole reaches the impermeable base ($D = 0$). In these formulae:

C	=	constant, depending on hole geometry;
dy/dt	=	rate of rise in water level (cm/s);
D	=	depth of impermeable layer below bottom (cm);
$h = H - y$	=	height of water column (cm);
h_1, h_2	=	initial and final water column in hole (cm);
H	=	depth of borehole below groundwater (cm);
K	=	average soil permeability (m/d);
r	=	radius of borehole (cm);
t	=	time (s);
y	=	depth of water level below groundwater (cm);
$\bar{y}$	=	average value of y in the interval where $y > 3/4y_0$ (cm);

$$\frac{dy}{dt} \approx \frac{y_1 - y_2}{t_2 - t_1} \qquad y_1 > y_2;\ t_2 > t_1 \tag{3}$$

Where the impermeable base is close to the bottom of the hole, an interpolation between Equations 2a and 2b is used.

For the inverse method, Oosterbaan and Nijland (1994) recommend:

$$K = \frac{r}{2(t_2 - t_1)} \ln \frac{h_1 + \frac{r}{2}}{h_2 + \frac{r}{2}} \qquad h_1 > h_2;\ t_2 > t_1 \tag{4}$$

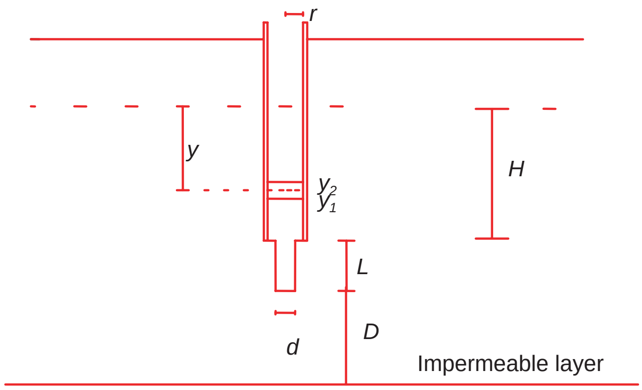
which was derived analytically by integration of the following differential equation:

$$\frac{dh}{h + \frac{r}{2}} = -\frac{2K}{r}dt \tag{5}$$

In Equation 4, the value of K is expressed in centimetres per second. To convert K from centimetres per second to metres per day, it should be multiplied by the factor 864.

The results within the same auger hole are usually quite consistent, but between different holes, even nearby ones, differences may be considerable owing to local soil variations. However, in predicting drain spacings, these differences become less

FIGURE A3.2
Definition sketch: piezometer method



important because the calculated spacings are proportional to the square root of *K*.

The program AUGHOLE makes the necessary calculations according to the above formulae.

The resulting *K* values can be used as input in programs for calculating drain spacings.

PIEZOMETER METHOD FOR DETERMINING SOIL PERMEABILITY

The piezometer method is more convenient than the auger-hole method for measurements of the *K* value in stratified soils and in layers deeper than 3 m. In these cases, water is pumped out of a piezometer, of which only the lowest part is open, while the upper part of the hole is protected by a pipe. The rate of rise in the water level inside the tube is measured immediately after pumping. Therefore, the *K* value of the small layer of soil near the open part is determined.

Method

The piezometer method (Luthin and Kirkham, 1949) differs from the auger-hole method in that the upper part of the hole is covered by a non-perforated pipe (Figure A3.2). The lower part of the borehole is open and collects the water from a specific layer. In this way, the permeability of separate layers can be found easily.

The procedure is as follows:

1. Make an auger hole and cover the upper end with a tightly fitting pipe, while the remaining open part acts as the water-collecting cavity, or cover the entire hole and make a narrower cavity below the pipe with a smaller auger.
2. Measure the groundwater depth at equilibrium.
3. Pump some water out and measure the rise in water level at different times.

It is most convenient to take all measurements with reference to the top of the protecting pipe. The computer program PIEZOM is based on Kirkham's formula. It calculates the permeability *K* (in metres per day) from these observations and the geometric factors.

Theory

The basic formula is:

$$K = \frac{864 \pi r^2}{A \Delta t} \ln \frac{y_1}{y_2} \qquad (6)$$

where *A* is a factor depending on the geometry of the piezometer and the hole below the end of the piezometer and 864 a constant for converting centimetres per second (for *K*) to metres per day. Various authors (Luthin and Kirkham, 1949; Smiles and Youngs, 1965; Al-Dhahir and Morgenstern, 1969; Youngs, 1968) have provided graphs or tables for *A*. Except for very small distances between the top of the piezometer and groundwater (and within certain limits), the tables for A/d given by Youngs (1968) (with the necessary corrections for diameter rather than radius) may be approximated by empirical formulae for the two limiting cases and for the "standard" value $H = 8d$:

$$\frac{A_8}{d} = 4.40\left(\frac{L}{d}\right)^{0.661} + 2.6 \tag{7a}$$

where the bottom of the cavity hole is at the impermeable base, and:

$$\frac{A}{d} = 4.40\left(\frac{L}{d}\right)^{0.661} + 0.2 - 0.06\left(\frac{L}{d} - 1\right) \tag{7b}$$

where the bottom of the cavity hole is far above the impermeable base (more than four times the cavity diameter). For H/d less than eight, rather complicated corrections are made to obtain A/d.

For H/d greater than ten, no values are tabulated. As an approximation, it is supposed that for $H/d > 8$ the cylindrical cavity may be represented by a sphere and that the remaining flow is radial. For this part of the flow, the inner radius is $r_8 = 8d + L/2$, whereas the outer radius is taken as the depth of the cavity centre below the groundwater level, $H + L/2$. These approximations are used in the program PIEZOM; the corrections are small because most of the resistance to flow occurs immediately around the cavity. They are:

$$\frac{A}{d} = \frac{A_8(1/r_o - 1/r_8)}{d(1/r_o - 1/r^*)} \tag{8}$$

where:

$$r_o = \frac{1}{4\pi / A_8 + 1/r_8} \tag{9a}$$

$$r_8 = 8d + L/2 \tag{9b}$$

$$r^* \approx H + L/2 \qquad \text{for } H>8d \tag{9c}$$

In these formulae (see Figure A3.2):

A = factor depending on shape (cm);
A_8 = same, for $H = 8d$;
d = diameter of cavity (cm);
H = depth of top cavity below groundwater (cm);
K = permeability (m/d);
L = length of cavity (cm);
r = radius of protecting pipe (cm);
r_o = radius of sphere equivalent to cavity (for $H > 8D$) (cm);
r_8 = radius $8d$ beyond which flow is supposed to be radial (cm);
r^* = distance centre of cavity to surface, to be used if $H/D > 8$ (cm);
D = distance to impermeable layer from cavity bottom (cm);
t = time (s);
y = water level below groundwater (cm);
y_1, y_2 = initial and final value of y (cm);
π = 3.14…

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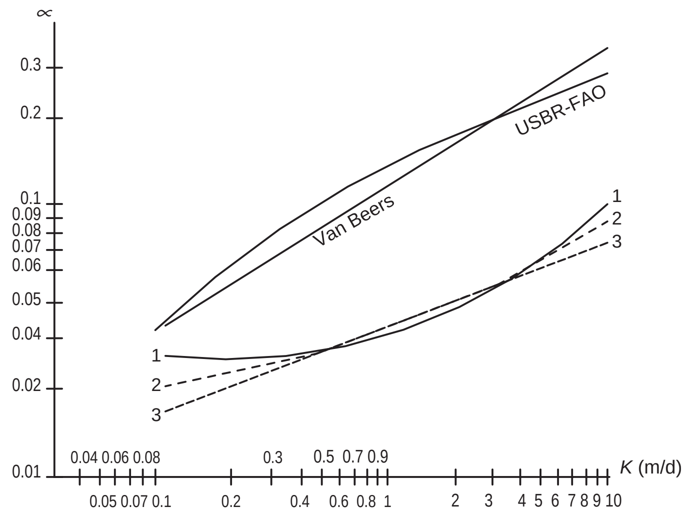
Annex 4
Determining drainable soil porosity

ESTIMATIONS FROM A *PF* CURVE

One option is to estimate the μ value on a *pF* curve as the difference in the water content by volume at saturation and at field capacity. This procedure has an important drawback because of the differences between a small undisturbed soil sample and the actual field conditions. However, an estimated average value of μ can be obtained where several laboratory measurements are taken for the same soil layer.

Figure A4.1
Relationships between the values of *K* and μ



Note: 1. all clay content; 2. less than 15% clay; 3. 15 < clay < 30%.
Source: Adapted from Chossat and Saugnac, 1985.

ESTIMATIONS FROM PERMEABILITY

Another option is to estimate the μ value from empirical relationships between the macroporosity and the hydraulic conductivity. Figure A4.1 shows the relationships developed by Van Beers (ILRI, 1972) and the USBR (1984) and those obtained by Chossat and Saugnac (1985) for soils with different clay contents.

However, as there are large variations, the field methods described below may be preferable.

OBSERVATIONS OF GROUNDWATER-LEVEL VARIATIONS

A better method is to measure the rise in groundwater level at short intervals, for example, before and soon after a heavy rain of short duration. The rainfall is divided by the observed rise, both expressed in the same units. If a sudden rain of 20 mm and no runoff causes a rise of 40 cm = 400 mm, μ = 20/400 = 0.05 (5 percent).

In drained lands, the fall in a rainless period can also be used, in combination with drain outflow measurements, as described in Annex 8.

LARGE CYLINDER

A more laborious method uses a large cylinder of undisturbed soil, carefully dug out. An oil drum (without its bottom) pushed tightly over the remaining column of soil is suitable for the purpose. After taking out, a new bottom is made by sealing the container to a plastic plate or welding it to a steel one. Water is added, and the water table rise inside is measured.

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Annex 5
Determining other soil hydrological characteristics

DEPTH TO IMPERVIOUS BARRIER

The position of an impermeable base (bedrock or tight clay) can be found from borings or soundings, or by geophysical methods. The existence of an impervious or slowly permeable soil layer can be commonly identified by observations in an auger hole where the barrier occurs within the depth of the hole, for example, when a net change in the soil texture or a sharp increase in the soil compactness is observed and, specifically, where a relatively dry material is found below a layer saturated with water. However, it is not always easy to distinguish an impervious layer. In this case, a layer can be considered as such if its hydraulic conductivity (K) is less than one-tenth of the permeability of the overlying layer.

Where the impervious layer is not within the depth range of the auger hole, deep borings must be carried out. Although cumbersome, hand augerings to 8–10 m are possible in moist soils. Where this is not possible or does not give a result, the depth can be estimated from soil maps or geological maps. Existing deep-water wells, or logs from drilled wells, may provide indications of the depth. Other solutions can be found in rough estimates of the aquifer transmissivity as described below.

THICKNESS OF THE FLOW REGION

In very deep homogeneous soils or aquifers, the lateral flow of groundwater tends to be concentrated in the upper part, to a depth about one-third of the distance between source and sink. In anisotropic aquifers ($K_v < K_h$), the active flow depth is even less. Thus, the flow in a drained field with 20-m drain spacing, would be concentrated in the upper 7 m, whereas flow from a hill to a valley, over a distance of 1 km mostly takes place in the upper 300 m (although aquifers are seldom so thick). Such figures form the upper limit of the “equivalent layer” (Hooghoudt, 1940).

The presence of an impermeable soil layer at a greater depth will not have a significant effect on the flow. On the other hand, at shallower depth, the influence becomes noticeable. The difference between real thickness and equivalent thickness is large at first for wide drain spacings, but it becomes less as the aquifer becomes thinner, until finally both become almost equal.

However, in drained fields, aquifers may be much thicker than one-third of the distance between drains. Here, the equivalent thickness (d) is taken. This adjustment is necessary because of the change from an almost horizontal flow through the aquifer to a radial flow near the drain. Consequently, the streamlines are concentrated there, leading to extra “radial resistance” and, thus, a smaller “equivalent” layer thickness, with one-third of the spacing as a maximum. Deeper parts of the aquifer hardly contribute to the flow entering the drain.

However, in thin aquifers, the water flow above the drain level is also relevant and it cannot be ignored. Then, $D = D_1 + d$, D_1 being the average thickness of the flow region above drain level. In some cases, as in many flat deltaic areas at or slightly above sea level with unripened clay subsoils (e.g. the Guadalquivir Marshes in Spain, the lower part of the Nile Delta in Egypt, and the Zuiderzee polders in the Netherlands), drains are laid on the impervious layer and, consequently, water flows only above drain level.

AQUIFER TRANSMISSIVITY

The transmissivity of an aquifer is the product of permeability and thickness (*KD*). In regional groundwater flow, the distances are so large (mostly several kilometres) that the entire thickness of the aquifer can be taken. In almost all cases, it will be thin in comparison with one-third of this distance, so that the real thickness can be taken for *D*.

Estimations of the average value of *KD* may be made by means of a regional approach, by applying Darcy's Law to the flow area:

$$KD = \frac{Q}{Ls} \tag{1}$$

The hydraulic gradient, *s* (dimensionless), is determined on the isohypses map. The discharge Q (cubic metres per day) over a length *L* (perpendicular to the flow) is measured or derived from a water balance.

Therefore, if *Q* is 2 m^3/d over a length of 50 m, and *s* = 2/1 000, *KD* = 20 (square metres per day). If the layer has a thickness of 5 m, *K* = 4 (metres per day).

For drained fields, the *KD* values can be determined by field observations if the impervious layer is not deeper than 3–5 m from the rise in water level in between existing open drains and the water level in the drains and the estimate of outflow to the drainage system at the moment of measuring. Additional details on measurement of *KD* can be consulted in Annex 8. From the *KD* value and the measured *K*, it is possible to derive the *D* value. Where the thickness of the aquifer is greater, pumping tests in drilled wells are required, or regional methods can be applied (described above).

VERTICAL RESISTANCE

Another parameter, useful for estimating regional flow, is the vertical resistance (*c*). Many aquifers are covered by a less permeable (but not impermeable) layer. They are "semi-confined". In many river valleys, there is a clay layer on top of a thick sandy aquifer, the top layer formed in the Holocene, the lower one in the Pleistocene. Groundwater has to pass through the top layer twice: first, as downwards leakage; at the end, as upward seepage.

Such resistive layers are characterized by their thickness (*D'*) and their vertical permeability (K_v), and *c* is their proportionality quotient for vertical flow contribution:

$$c = \frac{D'}{K_v} \tag{2}$$

For a clay with K_v = 0.001 m/d and *D'* = 2 m, the vertical resistance is *c* = 2 000 days. This value is expressed in days, as electrical resistance is in Ohms. A head difference of 1 m between bottom and top will cause upward seepage of 1/2 000 m/d or about 180 mm/year. If this groundwater contains diluted seawater, with 11 kg/m^3 of salts, the annual salt load will be about 20 tonnes/ha. Even if the water seeping upward through the clay cap is less salty, it will cause heavy topsoil salinization in the long run, especially in arid and semi-arid regions.

CHARACTERISTIC LENGTH

The combination of transmissivity and resistance determines the properties of the system. Thus, the characteristic length (λ) is a measure for the extent of seepage zones and is roughly equal to their width. It is found from:

$$\lambda = \sqrt{Kdc} \tag{3}$$

where:

c = vertical resistance of covering layer (d);

d = "equivalent" thickness of aquifer (m);

K = permeability of the aquifer (m/d);

λ = characteristic length (m).

Values for c are found from pumping tests, estimated directly from experience or derived form the thickness D' and the (measured or estimated) vertical permeability K_v of the upper layer. Pumping tests are the most reliable method (and supply values for KD at the same time). Methods for pumping tests are described in the bibliographic references (Boonstra and De Ridder, 1994; Kruseman and De Ridder, 1994).

Models for such regional flow, such as SAHYSMOD (ILRI, 2005), are also available.

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Annex 6
Estimating recharge due to irrigation

DETERMINING DEEP PERCOLATION IN IRRIGATED FIELDS

Where drainage projects are planned and designed for irrigated lands, actual figures of deep percolation can be estimated from the water balance on the soil surface and in the rootzone. In dry periods when precipitation is negligible, the amount of deep percolation produced by an irrigation application is:

$$R = I_n - \Delta W = \left(I - E - S_r\right) - \Delta W \qquad (1)$$

where:
E = evaporation losses (mm);
I = gross irrigation depth applied at the field level (mm);
I_n = amount of irrigation water infiltrated into the soil profile (mm);
S_r = amount of surface runoff (mm);
R = recharge (mm);
ΔW = change (increase [+] and decrease [-] of the moisture content of the rootzone (mm).

In Equation 1, the gross amount of water applied to a field, whose size is known, can be calculated if the flow is measured with a flume and the time of watering is determined with a watch. In a similar way, the amount of surface runoff can be measured. The value of ΔW can be estimated by determining the water content of soil samples taken before and after the irrigation application. The calculated value should be checked with the amount of water consumed by the crop (ET_c) in the previous period, which can be estimated by several methods (FAO, 1977 and 1998). Where relevant, precipitation should also be considered (FAO, 1974).

However, soil sampling is a tedious procedure that can be avoided by taking the period equal to an irrigation cycle. Just before irrigation, the soil has dried out; whereas just after irrigation, it is at field capacity. Thus, a period from before the first to before the second watering, or one from after the first until after the second, will have $\Delta W \approx 0$, and Equation 1 reads:

$$R = I_n - ET_c = \left(I - E - S_r\right) - ET_c \qquad (2)$$

where:
ET_c = consumptive use during the irrigation cycle (mm).

Once ET_c in that period has been estimated and irrigation and runoff losses have been measured, R can be determined.

Example

Data from irrigation evaluations made in an pilot area of an irrigation scheme, situated in northeast Spain, show that on average 90 mm of water is applied by basin irrigation in the peak period, with an interval between two consecutive waterings of 12 days. Surface runoff is negligible (levelled field with small bunds) and direct evaporation losses during the irrigation application are about 3 mm. The consumptive use in the peak period is about 66 mm ($ET_c \approx 5.5$ mm/d). Therefore, deep percolation is about 21 mm and the average value in the period considered is 1.75 mm/d.

PREDICTING DEEP PERCOLATION IN NEW IRRIGATION PROJECTS

Where the irrigation and drainage systems are designed jointly in new developments, the amount of expected percolation can be determined during the calculation of irrigation requirements from water retention data:

$$R = I(1 - e_a) - (E + S_r) \tag{3}$$

being:

$$I = \frac{\Delta W}{e_a} = \frac{1000 Z_r (\theta_{fc} - \theta_i)}{e_a} \tag{4}$$

where:

$e_a = ET_c/I$ = application efficiency (0.00–1.00), which represents the ratio between the amount of water consumed by crops and the gross application depth;

Z_r = average thickness of the rootzone (m);

θ_{fc} = soil water retained at field capacity (m^3/m^3);

θ_i = minimum soil water fraction that allows for non-stress of the crop (m^3/m^3).

Where the θ_i value is unknown, the amount of water readily available to the crops can be estimated as approximately half the interval between field capacity and the permanent wilting point:

$$\theta_{fc} - \theta_i = \frac{1}{2}(\theta_{fc} - \theta_{wp}) \tag{5}$$

where:

θ_{wp} = soil water retained at wilting point (m^3/m^3).

For this calculation, an average value of e_a must be assumed (see below).

ESTIMATIONS WHERE NO FIELD DATA ARE AVAILABLE

In the planning phase, field data for the project area are usually scarce or non-existent. In these cases, tentative values for e_a and R can be used from literature.

In 1980, FAO provided information on water management from irrigated lands of arid zones (FAO, 1980). These guidelines considered only readily obtainable data, such as soil texture and irrigation method and some qualitative information on water management at the field level (Table A6.1).

TABLE A6.1
FAO guidelines to estimate the values of e_a and R

Irrigation method	Application practices	Soil texture			
		Fine	Coarse	Fine	Coarse
		e_a (%)		R (%I)	
Sprinkler	Daytime application; moderately strong wind	60	60	30	30
	Night application	70	70	25	25
Trickle		80	80	15	15
Basin	Poorly levelled and shaped	60	45	30	40
	Well levelled and shaped	75	60	20	30
Furrow & border	Poorly graded and sized	55	40	30	40
	Well graded and sized	65	50	25	35

Source: Adapted from FAO, 1980.

TABLE A6.2
Estimated values for deep percolation

Application method	Distribution uniformity	Water application efficiency		Estimated deep percolation
		Tanji & Hanson, 1990	SJVDIP, 1999	
	(%)			
Sprinkler				
Periodic move	70–80	65–80	70–80	15–25
Continuous move	70–90	75–85	80–90	10–15
Solid set	90–95	85–90	70–80	5–10
Drip/trickle	80–90	75–90	80–90	5–20
Surface irrigation				
Furrow	80–90	60–90	70–85	5–25
Border	70–85	65–80	70–85	10–20
Basin	90–95	75–90		5–20

Note: Estimates for deep percolation were made on the basis of the following assumptions: no surface runoff under drip and sprinkler irrigation; daytime evaporation losses can be up to 10 percent sprinkling and 5 percent during night irrigation; tailwater in furrow and border irrigation can be up to 10 percent and evaporation losses up to 5 percent; no runoff is expected in basin irrigation and evaporation losses up to 5 percent (FAO, 2002).
Sources: Tanji and Hanson, 1990; SJVDIP, 1999.

In the past 20 years, considerable efforts have been made to improve irrigation application efficiencies in order to save water. Table A6.2 shows data from well-designed and well-managed irrigation systems in California, the United States of America, and potential maximum values for application efficiencies determined in irrigation evaluations in the San Joaquín Valley Drainage Implementation Program as mentioned in FAO (2002).

Tables A6.1 and A6.2 contain data from different types of systems and management. According to the expectations of a specific project area, the order of magnitude for a first approach to deep percolation can be estimated with the help of these tables. However, sensitivity analyses with various values should be performed in order to see the consequences in case the estimates are not correct. In addition, after the first parts of the irrigation system have been constructed, a direct verification in the field is recommended.

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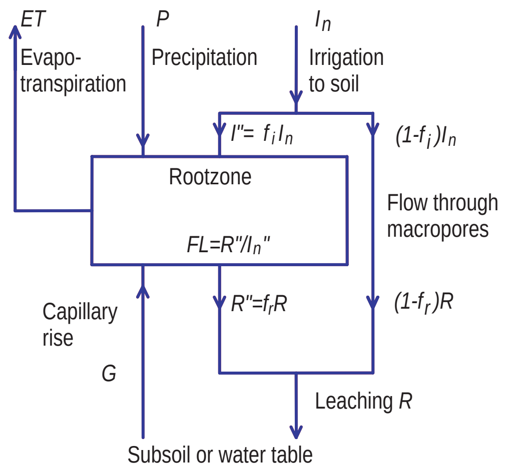
Annex 7
Leaching for salinity control

THE WATER AND SALT BALANCES

FIGURE A7.1
Water balance of an irrigated soil with macropores



During rainfall, snowmelt or irrigation, part of the water is lost by runoff and evaporation, but a considerable part enters the soil and is stored there. This storage is partly taken up by plant roots, while any excess drains below the rootzone. On the other hand, in dry periods, the rootzone may receive water from deeper layers by capillary rise, especially where the water table is shallow and drainage poor. Monthly water balances are generally sufficiently revealing for water table control, while annual soil salinity balances usually provide enough information for soil salinity control.

Coupled to this water balance, a balance can be made for soluble salts. They enter in tiny amounts through rain or snow, and in much larger quantities in irrigation water, even where this is considered as being of good quality. In the soil, these salts are concentrated by drying out, whereas plant roots take up water, but exclude the entry of salts. This increase in concentration should not be allowed to reach harmful levels for crop growth. This requires:

- adequate leaching: the inflow of water during a year must generate enough leaching to keep the salinity levels down;
- adequate natural or artificial drainage to allow removal of the leacheate, and a safe depth of the water table to prevent harmful capillary rise of saline water;
- irrigation water of good quality, or, where poor, an extra amount to provide an increased leaching.

Therefore, a first estimate can be made by estimating the annual balances.

However, a complication is that not all water entering or leaving the soil is effective in leaching. Especially in many clay soils under surface irrigation (basin, furrow or border), part of the water passes downward through cracks and other macropores without contributing much to the removal of salts.

LEACHING FRACTION OF AN IRRIGATED FIELD

This is expressed by a leaching efficiency: the part of the water that is effective. There are two such coefficients: for the surface (fraction of the entering water, f_i); and at the bottom of the rootzone (fraction of the percolating water, f_r).

For irrigated lands, where water conservation and salinity control are required, it is necessary to compare the actual amounts of deep percolation produced by irrigation with the leaching required to ensure soil salinity control. The first step is to determine the actual value of the leaching fraction, which can be taken as a first approximation as:

$$LF = \frac{R}{I_n} \tag{1}$$

However, to allow for flow through macropores it is better defined as:

$$LF = \frac{f_r R}{f_i I_n} \tag{2}$$

This flow usually goes directly to the subsoil. In this case (Figure A7.1):

$$(1 - f_i)I_n = (1 - f_r)R \quad \text{or} \quad f_r = 1 - \frac{I_n}{R}(1 - f_i) \tag{3}$$

Therefore, one of the two coefficients is sufficient.

In these equations:

f_i = leaching efficiency coefficient as a fraction of the irrigation water applied;
f_r = leaching efficiency coefficient as a function of the percolation water;
I_n = net amount of irrigation water (amount infiltrating into soil) (mm);
LF = required leaching fraction;
R = amount of percolation water (mm).

As I is usually much larger than R, so f_i is considerably larger than f_r. The leaching efficiency coefficient f_r was defined by Boumans in Iraq (Dieleman, 1963), and later f_i was introduced by Van Hoorn in Tunisia (Van Hoorn and Van Alphen, 1994). In the literature, both values are used. The f_i coefficient is commonly used. This coefficient depends on soil texture and structure as well as on the irrigation method. It is higher (0.95–1.0) in well-structured loamy soils than in heavy clay cracking soils (< 0.85). It is also higher with sprinkler irrigation than with surface irrigation, and close to 1 under drip irrigation. Where needed, f_r can be found from Equation 3.

Therefore, the actual value of the LF depends on soil characteristics, the irrigation method and the specific water management practised by farmers.

Example

The data in the example in Annex 6 show that farmers apply a net irrigation of about 87 mm during the peak irrigation season, and that about 21 mm of this amount percolates below the rootzone. It was also determined that about 6 percent of the infiltrated water flows directly through cracks without mixing with the soil solution ($f_i \approx 0.94$ and $f_r \approx 0.75$). This means that during this irrigation cycle farmers are irrigating with an LF of about 0.2. Following a similar approach, the average LF during the irrigation season can be obtained where the total values of I_n and R are available.

LEACHING REQUIREMENTS IN TERMS OF A MINIMUM *LF*

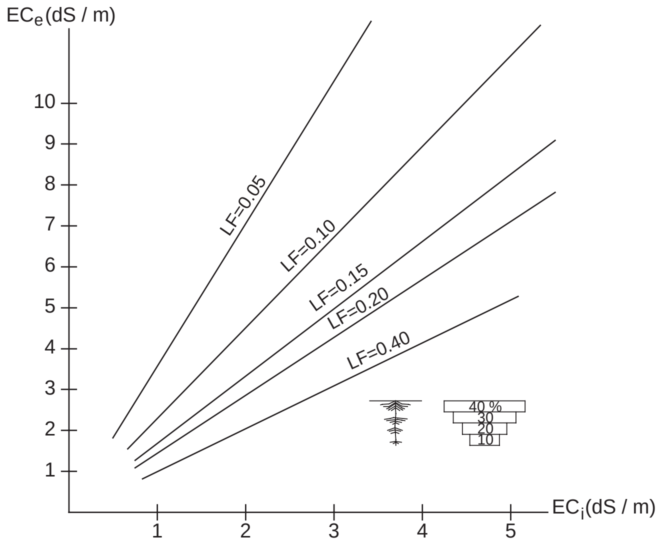
In order to control soil salinity in irrigated lands, a minimum LF is required. This can be calculated where the value of the electrical conductivity of the irrigation water (EC_i) and the salt tolerance of the crop are known. One option is to apply the approach developed by Van Hoorn and Van Alphen (1994) based on the water and salt balances in equilibrium status. In this approach, it was considered that water extraction by crops decreases within the rootzone from 40 percent of the total in the top quarter to 10 percent in the deepest quarter (FAO, 1985). Following this approach, a relationship between the EC_i and the average soil salinity in the rootzone (expressed in terms of the electrical conductivity of the saturated paste [EC_e]) can be obtained for several values of the LF (Figure A7.2). Similar graphs can be obtained from water and salt balances derived considering other water extraction models adapted to specific local conditions, as crop root distribution is affected severely by soil properties and by irrigation water management.

By means of Figure A7.2, the minimum LF to control soil salinization (caused by the salts applied with irrigation water with certain EC_i) can be determined once the threshold value of EC_e that must not be exceeded in the rootzone has been established from crop salt tolerance data. Data provided by Maas and Grattan (1999) about crop salt tolerance can be used (FAO, 2002).

Example

Following the example of the previous section, it is possible to calculate the minimum *LF* required to control the salt buildup caused by the salts applied with the irrigation water, whose salinity content in terms of EC_i is 0.6 dS/m. If maize is the most salt-sensitive crop of the cropping pattern, and its tolerance threshold in terms of EC_e is 1.7 dS/m, then a minimum *LF* of 0.05 is required to control soil salinity (Figure A7.2).

FIGURE A7.2
Relationship between EC_e and EC_i for different leaching fractions



Assuming that the average *LF* during the irrigation season is 0.2 and the minimum *LF* is 0.05, it can be concluded that no salt buildup should be expected in the rootzone, and even the irrigation application efficiency might be increased while keeping soil salinity under control.

In irrigated lands, it is possible to check whether the actual value of the *LF* satisfies the minimum *LF* necessary to control soil salinity. Therefore, if the amount of percolation water is enough to cover the leaching requirements, water might be saved by improving the application efficiency. If not, the leaching requirements must be calculated.

LEACHING REQUIREMENTS

Once the minimum *LF* is known, the long-term leaching requirements, for example, during the irrigation season, can be calculated by means of the salt equilibrium equation developed by Dieleman (1963) and later modified by Van Hoorn and Van Alphen (1994):

$$R^* = (ET_c - P_e)\frac{1 - f_i(1 - LF)}{f_i(1 - LF)} \tag{4}$$

where:
ET_c = actual crop evapotranspiration (mm);
P_e = effective precipitation (mm);
R^* = long-term leaching requirement (mm).
Therefore, the net irrigation requirement (I) is:

$$I = (ET_c - P_e) + R^* \tag{5}$$

Example

This example uses the case of the irrigated lands mentioned in the previous example (in which f_i = 0.94) and assumes that farmers need to irrigate with groundwater with an EC_i of 1.5 dS/m. If they still wish to grow maize in the soil of the previous example, they will need to irrigate with an *LF* of 0.3 (Figure A7.2). If the net irrigation requirement (ET_c - P_e) during the irrigation season is about 560 mm, at least 290 mm will be required to leach the salts accumulated in the rootzone. The net irrigation requirement will be

850 mm. If the actual *LF* is 0.2, about 185 mm of leaching can be obtained during the irrigation season (Equation 4). Therefore, the leaching deficit will be about 105 mm (290 - 185).

Where slightly soluble salts (e.g. gypsum, and magnesium and calcium carbonates) are present in the irrigation water, the leaching requirement is calculated first for the soluble salts. Then, the small contribution of the slightly soluble salts to the total soil salinity is added (Van der Molen, 1973). For average salt contents, the total solubility of gypsum and carbonates is about 40 meq/litre, which is equivalent to an *EC* of 3.3 dS/m. Where bicarbonates predominate in the irrigation water, it is advisable to decrease the sodium adsorption ratio (SAR) by increasing the calcium content of the soil solution by applying gypsum (5–20 tonnes/ha).

Once long-term soil salinity increases are no longer expected, a check should be made on the short term in order to be certain that the salt content of the soil solution does not exceed the threshold value of the crop salt tolerance. For this purpose, the salt storage equation derived for predicting the buildup of soil salinity on a weekly or monthly basis can be used (Van Hoorn and Van Alphen, 1994). The variation of salinity in the short term (Δz) can be calculated thus:

$$\Delta z = z_2 - z_1 = \frac{f_i \, I_n \, EC_i - f_r \, REC_1}{1 + \dfrac{f_r \, R}{2W_{fc}}} \tag{6}$$

where:

$EC_1 = \dfrac{z_1}{W_{fc}}$ = initial soil electric conductivity (deciSiemens per metre);

W_{fc} = moisture content at field capacity (mm);

z_1 = salt content in the rootzone at the start of the period (mm.dS/m);

z_2 = salt content in the rootzone at the end of the period (mm.dS/m).

OPTIONS TO COVER THE LEACHING REQUIREMENTS

Where the actual value of the *LF* does not satisfy the minimum *LF*, options should be considered to cover the leaching deficit.

In monsoon and temperate regions, the salt content in the rootzone may increase during the irrigation season. However, excess rainfall after the irrigation period will supply enough percolation water to leach out the salts accumulated in the rootzone. In this way, the salt content at the beginning of the next irrigation season will be sufficiently low to prevent secondary salinization.

Example

In the case described in the previous example, 100 mm of excess rainfall in winter might provide the percolation required to cover the leaching deficit. Therefore, even when irrigating with water with an EC_i of 1.5 dS/m, the soil salinity might be controlled on an annual basis under actual irrigation management.

However, where no effective precipitation is available for leaching, as is usually the case in arid and semi-arid zones, the leaching deficit must be covered by increasing the annual allocation of irrigation water. To cover uniformity deficiencies in water distribution over the irrigated field, the amount of percolation water should exceed the leaching requirements:

$$I = (ET_c - P_e) + aR^* \tag{7}$$

The *a* coefficient may vary from 1.15 to 1.20 if irrigation uniformity is fairly appropriate.

If, under the current irrigation management, the leaching requirements are not satisfied ($R \leq aR^*$), there are two options: grow crops that are more tolerant of salinity and in this way reduce the minimum *LF*; or find out how to cover the leaching deficit. In the latter case there are two possibilities: remove the accumulated salts before sowing the next crop by applying irrigation water; or split up the leaching requirement during the irrigation period by increasing each irrigation application.

EFFECTS OF LEACHING FOR SALINITY CONTROL ON SUBSURFACE DRAINAGE DESIGN

Where the leaching requirements are covered by the actual irrigation management or after the cropping season by rainfall or out-of season leaching irrigation, salinity control does not affect the drainage coefficient used for subsurface drainage design. However, if more water has to be added with each application in order to increase the *LF*, salinity control affects subsurface drainage design because the drainage coefficient must also be increased.

The option of increasing the irrigation allocation depends on the availability of water resources during or at the end of the growing season. It also depends on the internal drainage capacity of the soils. Coarse-textured soils permit leaching fractions of 0.15–0.25, while in fine-textured soils with low permeability the *LF* should be lower than 0.10 because of their limited internal drainage (unless rice is grown). In addition, the environmental effect of increasing the volume of drainage water on drainage disposal should be considered. Thus, growing more salt-tolerant crops is frequently a better option than using more water and increasing field and disposal drainage needs.

Controlling soil salinity caused by capillary rise generally does not increase the drainage coefficient. This is because it is dependent on adopting a suitable depth of the groundwater table and maintaining a downward flow of water during the irrigation season. Where leaching is required in order to remove the accumulated salts in the rootzone, water is generally applied before the start of the cropping season.

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Annex 8

Procedures for determining soil hydrological characteristics in drained lands

PROCEDURE FOR DETERMINING HYDRAULIC CONDUCTIVITY

Steady-state flow

Where water flows toward the drains under steady-state conditions, an average value of the hydraulic conductivity can be obtained from:

$$K = \frac{qL^2}{8h_h D} \quad (1)$$

where:

B = drain length (m);
D = average thickness of the horizontal flow region (m);
h_h = hydraulic head for horizontal flow (m);
K = hydraulic conductivity (m/d);
L = drain spacing (m);
Q = outflow (m^3/d);

$q = \frac{Q}{LB}$ = specific discharge (m/d).

In Equation 1, L is a design parameter that is known; q is calculated from the value of Q measured at the drain outlet; h_h is measured by difference in piezometer readings in tubes laid midway between two drains (h_1) and at some distance from the drain (h_2), outside the zone where radial flow is important, as shown in Figure A8.1. The radial flow in the vicinity of the drain has been excluded from the measurements.

For shallow aquifers ($D < L/4$), D approaches the real thickness of the permeable layer. However, for deeper ones, the maximum value for D is $L/3$. Where the D value has been determined by augering, an average value of K can be calculated with Equation 1.

Table A8.1 shows an example of the calculation of KD values from groundwater-level observations in piezometers laid midway between two drains (z_{25}) and in the vicinity of the drain ($z_{6.5}$), for drains laid at 50-m spacings and 1.8 m deep in a pilot field of peat soils with a sandy substratum severely recharged by seepage.

FIGURE A8.1
Piezometer line for determining the components of the total hydraulic head in a drained soil

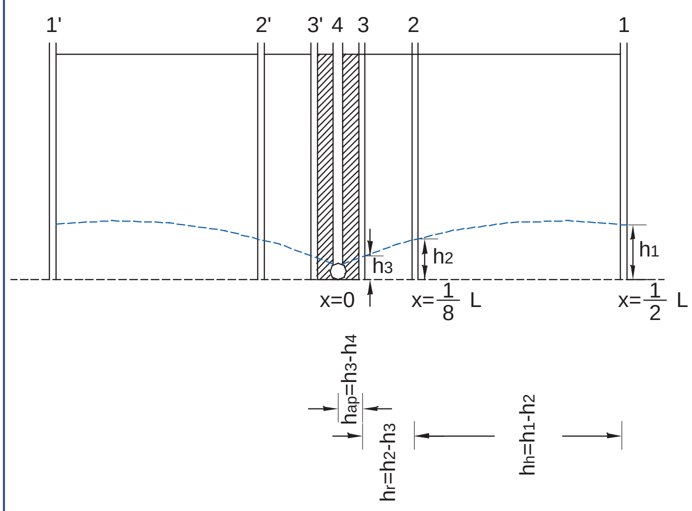
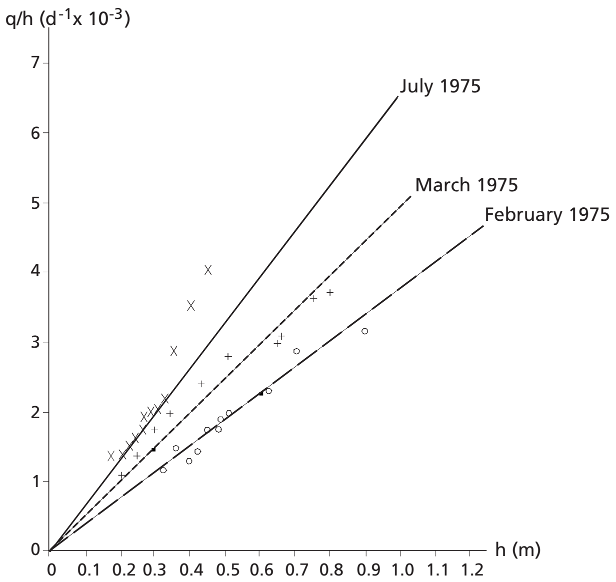


TABLE A8.1
Determination of *KD* values from groundwater-level observations in a drained soil with a sandy substratum

Drain no.	Period of observations (1984)	z_{25} (m)	$z_{12.5}$ (m)	$z_{6.5}$ (m)	$h_1 = 1.8\text{-}z_{25}$ (m)	$h_2 = 1.8\text{-}z_{6.5}$ (m)	h_h (m)	q (mm/d)	KD (m²/d)
13	January–March	0.95	0.97	1.07	0.85	0.73	0.12	22.3	58.1
	April–June	1.03	1.04	1.14	0.77	0.66	0.11	19.5	55.4
	July–October	1.08	1.09	1.17	0.72	0.63	0.09	17.0	59.0
14	January– March	0.86	0.89	0.97	0.94	0.83	0.11	22.6	64.2
	April–June	0.95	0.97	1.05	0.85	0.75	0.10	18.0	56.3
16	January– March	0.52	0.56	0.62	1.28	1.18	0.10	21.1	65.9
	April–May	0.57	0.60	0.66	1.23	1.14	0.09	18.0	62.5

FIGURE A8.2
Example of q_t/h_t relationships during the drawdown of the water table in non-steady flow towards drains laid on an impervious layer in a silt-clay soil



Source: Adapted from Martínez Beltrán, 1978.

The average *KD* value calculated from observations made in three drains over ten months was 60 m²/d. If the sandy layer in which the drains are laid has an average thickness of about 8 m, the average value for the hydraulic conductivity of the sandy layer is 7.5 m/d.

Non-steady-state flow

In drained lands where laterals are laid on the impervious layer, water flow is generally non-steady, especially after an irrigation application or heavy rainfall. However, the average value of the hydraulic conductivity of the permeable layer can be calculated from observations of the drawdown of the water table, where the phreatic level has an elliptic shape. Under these conditions, the Boussinesq equation for the specific discharge reads:

$$q_t = \frac{3.46K}{L^2} h_t^{\,2} \qquad (2)$$

where:
q_t = specific discharge at time t (m/d);
h_t = hydraulic head midway between drains at time t (m).

Therefore, if the function $q_t/h_t = f(h_t)$ is represented graphically, with data from observations made during several drainage periods, straight lines can be obtained, as those represented as an example in Figure A8.2.

The slope of the $q_t/h_t = f(h_t)$ function is equal to:

$$tg\gamma = \frac{3.46K}{L^2} \qquad (3)$$

From Equation 3, *K* values can be obtained, as shown in Table A8.2.

TABLE A8.2
Calculation of hydraulic conductivity with the Boussinesq equation

Period of observations	Drawdown of the groundwater level (m)	Correlation coefficient $q_t/h_t = f(h_t)$	tgγ 10^{-3}	K (m/d)
February 1976	0.30–1.10	0.96	4.05	0.47
July–August 1976	0.10–1.10	0.91	8.67	1.00
January–February 1977	0.60–1.10	0.97	3.81	0.44
June–July 1977	0.50–1.00	0.94	4.80	0.55

Source: Martínez Beltrán, 1978.

Results from Table A8.2 show K values of about 0.5 m/d where the groundwater level is below the top layer (0–30 cm). A higher value of 1 m/d was obtained when the water level was close to the ground surface. However, in this case, the correlation coefficient was lower than in the previous cases (probably because of an almost flat shape of the water table and because of the high hydraulic conductivity of the top layer).

DETERMINING RADIAL RESISTANCE

Resistance to steady-state radial flow towards drains installed above the impervious layer can also be determined from observations in drained lands:

$$W_r = \frac{h_r}{qL} \quad (4)$$

where:
h_r = hydraulic head for radial flow (m);
W_r = radial resistance (d/m).

In Equation 4, h_r is measured by the difference in piezometer readings in tubes laid at some distance from the drain (h_2) and close to the drain trench (h_3), as shown in Figure A8.1.

Table A8.3 shows an example of calculation of W_r values from water-level observations in piezometers laid in the vicinity of the drain ($z_{6,5}$) and close to the drain (z_0), for drains laid at 50-m spacings and 1.8 m deep in a sand layer.

Results from three drains observed during different periods show an average radial resistance of 0.24 d/m.

PROCEDURE FOR DETERMINING THE DRAINABLE PORE SPACE

For drained lands, the μ value of the layer above drain level can be measured from the drawdown of the water table (determined by piezometer recording) and the amount of water drained in the period considered (calculated from measurements of the drain discharge). The restrictions are that evaporation and seepage to or from deeper layers must be low and can be ignored relative to the drain discharge.

TABLE A8.3
Determination of W_r from observations in a drained soil with a sandy substratum

Drain no.	Period of observations (1984)	$z_{6,5}$ (m)	z_0 (m)	$h_2 = 1.8 - z_{6,5}$ (m)	$h_3 = 1.8 - z_0$ (m)	h_r (m)	q (mm/d)	W_r (d/m)
13	January–March	1.07	1.38	0.73	0.42	0.31	22.3	0.28
	April–June	1.14	1.38	0.66	0.42	0.24	19.5	0.25
	July–October	1.17	1.33	0.63	0.47	0.16	17.0	0.19
14	January– March	0.97	1.26	0.83	0.54	0.29	22.6	0.26
	April–June	1.05	1.26	0.75	0.54	0.21	18.0	0.23
16	January– March	0.62	0.87	1.18	0.93	0.25	21.1	0.24
	April–May	0.66	0.87	1.14	0.93	0.21	18.0	0.23

TABLE A8.4
Calculation of the μ value from the water balance in drained lands

Period of observations	Drawdown of the water level	D_r	Δh	μ	$\bar{\mu}$
	(m)	(mm)	(mm)	(%)	(%)
January 1975	0.55–0.80	11.2	219	5.1	4.3
	0.80–0.95	5.3	156	3.4	
	0.95–1.10	4.7	125	3.8	
February 1976	0.95–1.10	4.8	97	4.9	4.7
	1.10–1.20	2.1	46	4.6	
January 1977	0.75–1.10	7.1	169	4.2	3.9
	0.85–1.20	10.2	288	3.5	

Source: Martínez Beltrán, 1978.

Therefore, if the recharge to the water table and natural drainage are negligible and there is no depletion of the water table from plant roots in the time interval selected, the drainable pore space can be found from:

$$\mu = \frac{D_r}{\Delta h} \quad (5)$$

where:
D_r = amount of drainage water converted to an equivalent surface depth (mm);
μ = drainable pore space;
Δh = average drawdown of the water table in the time considered (mm).
D_r and Δh must be expressed in the same units.

To determine the average μ value, it is only necessary to measure, during the interval of time selected, the average drawdown of the water table from piezometer readings and the amount of water drained in the same period. The drainable pore space is a dimensionless fraction, often expressed as a percentage, as in Table A8.4. Table A8.4 shows an example calculation of the average μ value of a silty-clay soil, with data from observations made during three consecutive winters.

The results of this table show the tendency of μ to decrease with soil depth. For example, the 1975 observations show a value of 5.1 percent for a soil layer with a prismatic structure and about 3.9 for the deeper, less-structured soil layer. However, for drain spacing calculations an average value of 4.3 percent can be considered. The average value calculated with the results of the following years was of the same magnitude.

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Annex 9

Procedure for deriving drainage design criteria from drained lands

SUBSURFACE DRAINAGE COEFFICIENTS

From observations of the groundwater level and measurements of drain discharge, hydrographs such as those in Figure A9.1 can be drawn.

This example (from a flat coastal area in eastern Spain) shows that during dry periods (from mid-June to late September), in the absence of irrigation, the subsurface drainage flow towards the observed drain was steady, with a drain discharge of about 17 mm/d, due to seepage. However, in winter and spring, the drainage system was also recharged by percolation of rainfall, and then the water flow was non-steady.

With this information, sound drainage criteria can be formulated for steady-state flow drainage design. If in addition to seepage, during the irrigation season, there is a recharge of about 1 mm/d from irrigation losses, a drainage coefficient of 18 mm/d will be required in order to control the water table during the dry period. However, if after heavy rainfall, high water tables are affecting winter crops or hampering soil trafficability, the drain spacing calculated for steady flow should be checked for non-steady conditions.

In irrigated lands without such high seepage, water flow towards drains is generally non-steady, as Figure A9.2 shows. Information from drainage periods such as those shown in Figure A9.2 is useful for determining the magnitude of the rise of the water table after irrigation and further drawdown during the interval between two consecutive irrigation applications.

FIGURE A9.1
Water depth and drain discharge hydrographs determined by observations in a drainage experimental field (drained peat soil under considerable seepage)

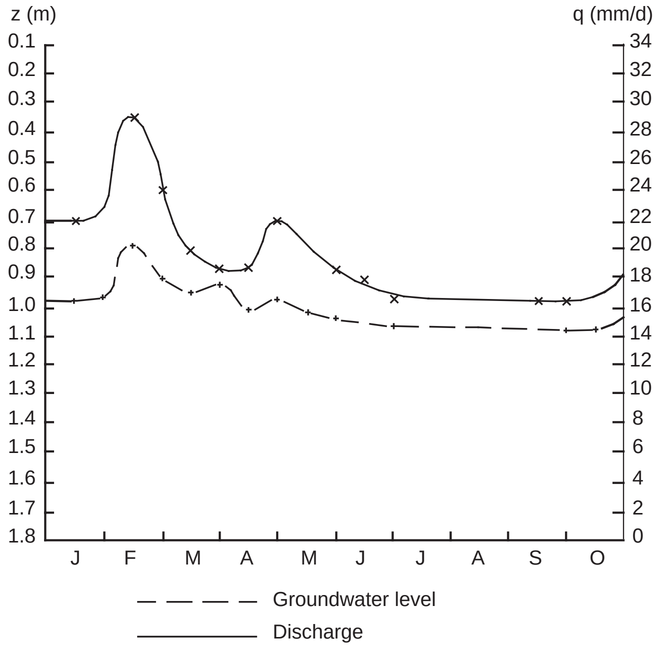
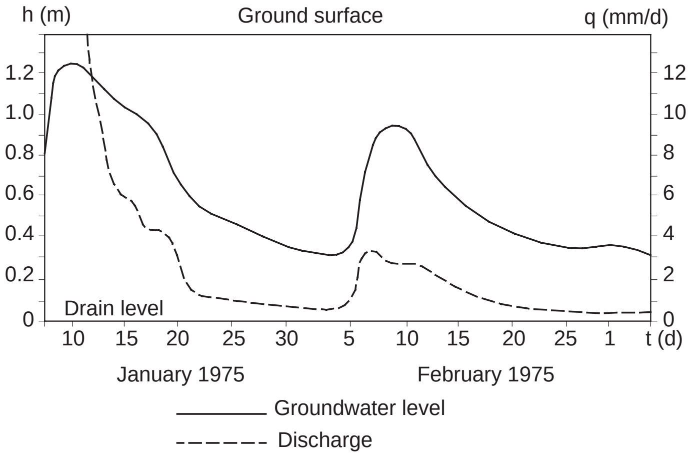
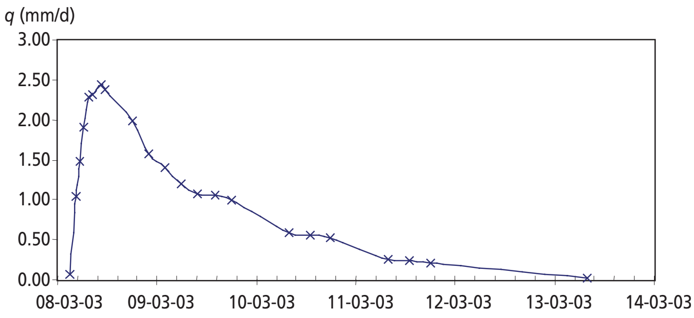


FIGURE A9.2
Drawdown of a water table after irrigation to reclaim saline soils



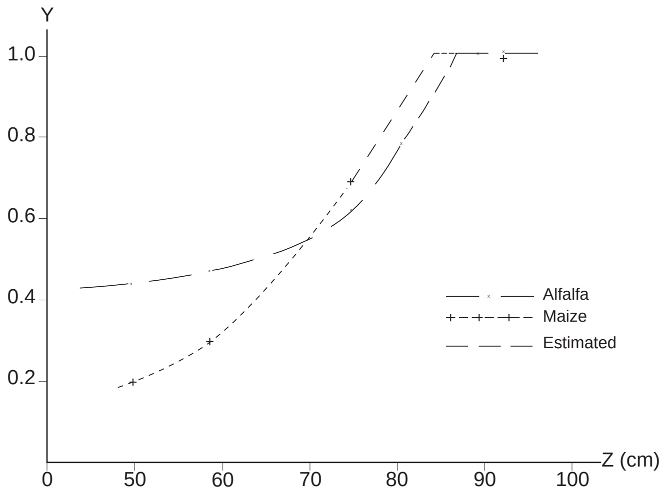
Source: Adapted from Martínez Beltrán, 1978.

FIGURE A9.3
Example of drainage discharge after irrigation



Source: FAO/IMTA, 2004.

FIGURE A9.4
Relationship between the average depth of the water table and maize and alfalfa yields



However, for irrigated lands, the actual non-steady drainage criteria can be translated into more or less equivalent steady-state drainage criteria. For example, the hydrograph in Figure A9.3 shows that after an irrigation application, discharge decreases from a maximum value of about 2.5 mm/d to zero (just before the next irrigation). However, the average discharge during the drainage period was about 1 mm/d. Therefore, this latter discharge can be used as the drainage coefficient for drain spacing calculations using steady-state equations.

DESIGN DEPTH TO THE HIGHEST WATER TABLE

The relationship between the average depth to the water table and crop yields and trafficability or the duration and intensity with which groundwater levels exceed a crop-specific critical depth during the growing season can also be estimated from observations in drained lands.

Table A9.1 shows groundwater depth data from four plots with different drainage conditions and their impact on yields of irrigated maize and alfalfa.

Table A9.1 also includes the *SDW* value, as used in the Dutch polders. It is the sum of days with waterlogging during the period considered (Sieben, 1964). In this case, the SDW_{50} (sum of days with less than 50 cm depth) is also a good measure for crop damage. In the Dutch polders, SDW_{30} (less than 30 cm depth) is usually taken for field crops.

TABLE A9.1
Maize and alfalfa yields compared with data of the groundwater table

Period (1977)	Consecutive days in which the groundwater level was above the depth indicated (cm)															
	25	50	75	100	25	50	75	100	25	50	75	100	25	50	75	100
June	4	5	6	20	5	6	10	30	5	9	22	30	5	20	30	30
July	2	3	4	10	2	3	10	31	1	10	25	31	1	19	31	31
August	2	4	5	16	3	6	10	31	2	14	28	31	3	24	30	31
September	2	4	5	7	3	4	8	23	3	8	17	30	3	8	14	30
SDW_{50}	16				19				41				71			
Alfalfa yield (kg/ha) and relative yield	12 195 1.00				7 600 0.62				5 780 0.47				5 415 0.44			
Maize yield (kg/ha) and relative yield	5 800 1.00				4 000 0.69				1 730 0.30				1 180 0.20			

Source: Adapted from Martínez Beltrán, 1978.

Although under irrigation the water level varies with time, the average depth of the water table is a good indicator concerning crop yields. Figure A9.4 shows the relationship between the relative crop yield (*Y*) and the average depth of the water table ($\bar{Z}$) during the irrigation season, as per the data in Table A9.1.

Although data from only one irrigation season are not sufficient to obtain a statistically sound relationship, these results are useful for providing practical guidance to be confirmed later with further information. It seems that an average depth of 85 cm is critical for maize and alfalfa, which were the most relevant irrigated crops in the study area. In this case, the groundwater depth criterion is dominant because no long dry fallow periods or periods with frequent shortages of irrigation water occur. Where this is not the case, especially where the groundwater is rather salty, deeper groundwater levels during such extended dry periods are required in order to avert soil salinization by capillary rise.

The data in Table A9.1 also show that short periods of high water tables are not harmful for the above-mentioned crops.

In the Dutch polders, with a humid climate, no appreciable damage to crops was found where during heavy rains in winter the groundwater did not rise above 0.30 m depth below the surface, provided that it receded within a few days. Higher groundwater levels led to slaking of the ploughed layer, causing more permanent anaerobic conditions and damage to field crops. These silty-clay soils needed a drainage depth of 1.20 m in order to keep the average levels low enough.

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