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Guidelines and computer programs for the planning and design of land drainage systems

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Drained lands of the Lower Guadalquivir Irrigation Scheme, Spain (left bank) and paddy fields with surface drainage systems (right bank). M.M. Ridao, Spain.

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62

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- PC with Intel Pentium® processor and Microsoft® Windows 95 / 98 / 2000 / Me / NT / XP
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Foreword

Agriculture and, consequently, food production depend, among other factors, on the proper management of water. Land drainage, an integral component of water management, is well known to have ameliorated salinity and waterlogging problems in rainfed and irrigated agriculture. In so doing, it has contributed substantially to sustainable agricultural development through enabling increased crop production, decreased farming costs, and the maintaining of soil quality. In areas where rainfall is excessive, it is necessary to manage land drainage, both surface and subsurface, in order to prevent waterlogging. In areas where rainfall is deficient, drainage management is still important in order to minimize soil salinization.

In the arid and semi-arid regions, soil salinity still limits crop production significantly. Hence, it has a negative effect on food security. This is especially true in irrigated agriculture because of the salts added with the irrigation water and the buildup of saline groundwater where natural drainage is insufficient. Although only approximate figures are available, FAO estimated in 2002 that salinity had damaged about 20–30 million ha of irrigated land worldwide, and that 0.25–0.50 million ha were being lost from production every year as a result of soil salinization.

In the wetter regions, flooding and waterlogging still limit crop production in many parts of the world. In the inland valleys of sub-Saharan Africa with shallow groundwater tables, controlled drainage may help to increase crop production and improve the health of rural populations. In certain lands of the humid tropics, drainage is also needed in order to increase rice production and promote crop diversification. As the global population and the demand for food increase, additional new drainage systems will be installed in a broader range of climate, soil and hydrological conditions, and existing systems will be renovated.

FAO has already addressed waterlogging and salinity control through its normative and field programmes in the past 50 years. However, the context of land drainage has changed considerably in recent decades. This change has come about owing to concerns for the environment and the recognition of the need to integrate system users into the planning, design, operation and maintenance process. In addition, the experience gained and the research of recent years have led to improvements in the technology and methods.

This FAO Irrigation and Drainage Paper is intended to serve as a tool for an integrated drainage approach by providing guidelines for: (i) the appropriate identification of drainage problems; (ii) the planning and design of drainage systems; and (iii) the careful integration of technical, environmental and socio-economic factors.

The main text of this paper provides critical general information about the planning and design of land drainage systems and their relationship with technical, socio-economic and environmental aspects. The annexes provide more detailed information with technical background, appropriate equations, some cross-references for finding appropriate methodologies, and computer programs for applications developed by Professor W.H. Van der Molen, which are included on a CD–ROM.

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List of acronyms

CN	Curve Number
EC	Electrical conductivity
EIA	Environmental impact assessment
$FeSO_4$	Iron sulphate
GIS	Geographical information system
ICID	International Commission on Irrigation and Drainage
ILRI	International Institute for Land Reclamation and Improvement
IMTA	Mexican Institute for Water Technology
IWRM	Integrated water resources management
K	Potassium
KD	Transmissivity of soil layers
LAI	Leaf area index
M&E	Monitoring and evaluation
MSL	Mean sea level
N	Nitrogen
O&M	Operation and maintenance
P	Phosphorus
PE	Polyethylene
PVC	Polyvinyl chloride
R&D	Research and development
RS	Remote sensing
SAR	Sodium adsorption ratio
SCS	Soil Conservation Service
SEBAL	Surface Energy Balance Algorithm for Land
TDS	Total dissolved solids
UNEP	United Nations Environment Programme
USBR	United States Bureau of Reclamation
WHO	World Health Organization

List of symbols

Symbol	Description	Dimension
A	Cross-sectional area of drain (m^2)	L^2
	Surface area (m^2, ha)	L^2
	Basin area (ha)	L^2
	Area served per drain (km^2)	L^2
	Area served per well (m^2)	L^2
	Factor of the piezometer method depending on shape (cm)	L
A'	Reduced cross-sectional area of drain because of sedimentation (m^2)	L^2
A_8	Factor of the piezometer method for $H = 8d$ (cm)	L
a	Coefficient	-
	Conversion factor	-
	Blasius coefficient	-
	Distance to more permeable subsoil in interceptor drain (m)	L
a_c	Corrected coefficient	-
an_i	Anisotropy factor of layer i	-
an_1	Anisotropy factor of layer 1	-
an_2	Anisotropy factor of layer 2	-
B	Length of subsurface drain (m)	L
	Length of waterway in wind direction (km)	L
B'	Horizontal distance of subsurface drain (m)	L
B_c	Length of collector drain (m)	L
B_{i-1}	Begin of drain section (m)	L
B_i	End of drain section (m)	L
B_1	Length of first section in drain line with increasing diameter (m)	L
b	Bottom width of drainage channel (m)	L
	Width of drain trench or ditch bottom (m)	L
	Crest width in weir (m)	L
C	Constant	-
	Coefficient for surface runoff	-
	Installation cost of subsurface drainage system (US$/ha)	-
C_u	Cost per unit length of installed drains (US$/m)	-
c	Euler's constant	-
	Hydraulic resistance of semi-confining layer (d)	T
c_b	Entry resistance of drain (d)	T
D	Layer thickness (m)	L
	Thickness of aquifer (m)	L
	Average thickness of flow region (m)	L
	Depth of impermeable layer below bottom of auger-hole (cm)	L
	Distance to impermeable layer from piezometer cavity bottom (cm)	L
D'	Thickness of semi-pervious layer (d)	L
	Maximum initial thickness of flow region for numerical solution (m)	L
D_i	Amount of water interflows through topsoil (mm)	L
	Thickness of layer i	L

Symbol	Description	Dimension
D_r	Amount of subsurface drainage (mm)	L
D_s	Amount of surface drainage (mm)	L
D_0	Downstream thickness of flow in interceptor drain (m)	L
D_1	Average thickness of layer above drain level (m)	L
	Upstream thickness of flow in interceptor drain (m)	L
D_2	Thickness of layer below drain level down to impermeable subsoil (m)	L
	Thickness of first layer below drain level (m)	L
D_3	Thickness of second layer below drain level (m)	L
D_4	Thickness of third layer below drain level (m)	L
d	Thickness of equivalent layer (m)	L
	Inside drain diameter (m)	L
	Diameter of cavity in the piezometer method (cm)	L
d'	Thickness of semi-confining layer (m)	L
d_1	Diameter of first section in drain line with increasing diameter (m)	L
d_2	Diameter of second section in drain line with increasing diameter (m)	L
E	Amount of direct evaporation (mm)	L
	Actual evaporation from groundwater (md^{-1})	LT^{-1}
E_0	Potential evaporation from groundwater (md^{-1})	LT^{-1}
EC	Electrical conductivity (dSm^{-1})	$T^3I^2M^{-1}L^{-3}$
EC_e	Electrical conductivity of soil saturated paste (dSm^{-1})	$T^3I^2M^{-1}L^{-3}$
EC_g	Global electrical conductivity measured by remote sensing (dSm^{-1})	$T^3I^2M^{-1}L^{-3}$
EC_i	Electrical conductivity of irrigation water (dSm^{-1})	$T^3I^2M^{-1}L^{-3}$
EC_1	Initial soil electric conductivity (dSm^{-1})	$T^3I^2M^{-1}L^{-3}$
$EC_{1:2}$	Electrical conductivity in 1 to 2 soil solution (dSm^{-1})	$T^3I^2M^{-1}L^{-3}$
$EC_{1:5}$	Electrical conductivity in 1 to 5 soil solution (dSm^{-1})	$T^3I^2M^{-1}L^{-3}$
ET	Evapotranspiration (mm)	L
ET_c	Consumptive use during irrigation cycle (mm)	L
	Actual crop evapotranspiration (mm)	L
e	Base of natural logarithms	-
e_a	Irrigation application efficiency	-
F	Freeboard in open drain (m)	L
F_c	Calculation coefficient for corrugated pipes	-
F_s	Calculation coefficient for smooth pipes	-
Fr	Froude-Boussinesq number	-
F_1	Calculation coefficient for first section in drain line with increasing diameter	-
F_2	Calculation coefficient for second section in drain line with increasing diameter	-
f	Function	-
	Total correction factor for pipes with sediment	-
f_i	Leaching efficiency coefficient as a fraction of irrigation water applied	-
f_r	Leaching efficiency coefficient as a function of percolation water	-
f_1	Partial correction factor for pipes with sediment	-
f_2	Partial correction factor for pipes with sediment	-
G	Function	-
	Amount of capillary rise (mm)	L
g	Acceleration of gravity (ms^{-2})	LT^{-2}
H	Total allowed hydraulic head in drainpipe at design discharge intensity (m)	L
	Hydraulic head in aquifer (m)	L
	Depth of borehole below groundwater (cm)	L
	Depth of top cavity (piezometer method) below groundwater (cm)	L
H_x	Hydraulic head in drain at distance *x* (m)	L

Symbol	Description	Dimension
h	Horizontal	-
	Difference in basin elevation between most distant point and outlet (m)	L
	Hydraulic head midway between subsurface drains at design discharge (m)	L
	Total hydraulic head (m)	L
	Height of water column in auger hole (cm)	L
	Hydraulic head above crest level in weir (m)	L
	Total hydraulic head in pumping station (m)	L
h_a	Head in artesian aquifer above drain level (m)	L
h_c	Corrected hydraulic head considering anisotropy (m)	L
h_h	Hydraulic head for horizontal flow (m)	L
h_p	Hydraulic head in drainpipe in numerical solution (m)	L
h_s	Lift or static hydraulic head in pumping station (m)	L
h_r	Hydraulic head for radial flow (m)	L
h_t	Hydraulic head midway between subsurface drains at time t (m)	L
h_v	Vertical hydraulic head loss (m)	L
h_{ap}	Hydraulic head for approach flow (radial + entry) to the drain (m)	L
h_{des}	Design head for outflow system in numerical solution (m)	L
h_{init}	Initial hydraulic head in numerical solution (m)	L
h_0	Initial hydraulic head midway subsurface drains (m)	L
	Hydraulic head near drain in numerical solution (m)	L
h_1	Initial water column in auger hole (cm)	L
	Piezometer reading in tube laid midway between drains (m)	L
	Upstream water head in weir (m)	L
	Hydraulic head in first compartment in numerical solution (m)	L
	Hydraulic head in canal above original groundwater level (m)	L
h_2	Final water column in auger-hole (cm)	L
	Piezometer reading in a tube laid at some distance from drain (m)	L
	Downstream water head in weir (m)	L
	Hydraulic head in canal above drain level (m)	L
h_3	Piezometer reading in tube laid close to drain trench (m)	L
h_4	Piezometer reading in tube laid on the drainpipe (m)	L
h_{10}	Hydraulic head midway drains in numerical solution (m)	L
I	Gross irrigation depth applied at field level (mm)	L
	Rainfall intensity during concentration time (mmh^{-1})	LT^{-1}
I_a	Amount of water intercepted and infiltrated before overland flow (mm)	L
I_n	Net amount of irrigation water infiltrated into soil profile (mm)	L
I_{nf}	Accumulated infiltration into soil (mm)	L
	Infiltration rate (mmh^{-1})	LT^{-1}
i	Index for distance step in numerical solution	-
j	Index for time step in numerical solution	-
K	Basin constant (m)	L
	Hydraulic conductivity (md^{-1})	LT^{-1}
	Permeability of aquifer (md^{-1})	LT^{-1}
K'	Permeability of semi-confining layer (md^{-1})	LT^{-1}
K_h	Horizontal hydraulic conductivity of soil (md^{-1})	LT^{-1}
K_M	Reciprocal parameter of Manning's roughness coefficient (m$^{1/3}$ s^{-1})	L$^{1/3}$T^{-1}
K'_M	Corrected K_M coefficient (m$^{1/3}$ s^{-1})	L$^{1/3}$T^{-1}
K_v	Vertical hydraulic conductivity of soil (md^{-1})	LT^{-1}
K_{hi}	Horizontal hydraulic conductivity of layer i (md^{-1})	LT^{-1}
K_{vi}	Vertical hydraulic conductivity of layer i (md^{-1})	LT^{-1}

Symbol	Description	Dimension
K_1	Permeability above drain level (md^{-1})	LT^{-1}
	Permeability of topsoil (md^{-1})	LT^{-1}
K_2	Permeability below drain level (md^{-1})	LT^{-1}
	Permeability of first layer below drain level (md^{-1})	LT^{-1}
	Permeability of subsoil (md^{-1})	LT^{-1}
K_3	Permeability of second layer below drain level (md^{-1})	LT^{-1}
K_4	Permeability of third layer below drain level (md^{-1})	LT^{-1}
K_{2h}	Horizontal permeability below drains (md^{-1})	LT^{-1}
K_{2v}	Vertical permeability below drains (md^{-1})	LT^{-1}
L	Length (m)	L
	Drain spacing (m)	L
	Length of cavity (piezometer method) (cm)	L
	Well spacing distance (m)	L
L_c	Spacing of collector drain (m)	L
LF	Leaching fraction	-
l	Maximum distance in a basin between most distant point and outlet (m)	L
l_i	Length of section *i* of main drainage system (m)	L
m	Order number in data series	-
	Exponent	-
m_{50}	Median size of soil grains above 50 μm	L
N	Number of total data available in data series	-
n	Number of order	-
	Exponent	-
	Number of extremes in statistical analysis	-
	Coefficient in hill slope (v:h)	-
	Manning's roughness coefficient ($sm^{-1/3}$)	$T L^{-1/3}$
P	Amount of precipitation (mm)	L
	Probability	-
	Power (kW)	ML^2T^{-3}
P'	Amount of precipitation of equal duration in unit hydrograph (mm)	L
P_e	Effective precipitation (mm)	L
P_6	Amount of rainfall in 6 hours (mm)	L
P_{12}	Amount of rainfall in 12 hours (mm)	L
P_{24}	Amount of rainfall in 24 hours (mm)	L
P_{T1}	Precipitation for 1-year return period (mm)	L
P_{T5}	Precipitation for 5-year return period (mm)	L
P_{T10}	Precipitation for 10-year return period (mm)	L
pF	Decimal logarithm of soil suction expressed in cm	-
pH	Minus decimal logarithm of H^+ concentration	-
Q	Water flow, discharge (m^3s^{-1}, m^3d^{-1})	L^3T^{-1}
	Flow towards well (m^3d^{-1})	L^3T^{-1}
$\|Q\|$	Absolute value for Q (m^3s^{-1})	L^3T^{-1}
Q_M	Maximum discharge for return period equivalent to design rainfall (m^3s^{-1})	L^3T^{-1}
Q_w	Discharge of well, absolute value (m^3d^{-1})	L^3T^{-1}
Q_{in}	Flux entering an element per m^1 length (m^2d^{-1})	L^2T^{-1}
Q_{out}	Flux leaving an element per m^1 length (m^2d^{-1})	L^2T^{-1}
Q_2	Flux below drain level per m^1 drain (m^2d^{-1})	L^2T^{-1}
q	Specific discharge (mmd^{-1}, md^{-1}, $ls^{-1}ha^{-1}$)	LT^{-1}
	Flux density to drain (md^{-1})	LT^{-1}
	Surface drainage coefficient ($m^3s^{-1}km^{-2}$)	LT^{-1}
	Upward flow towards interceptor drain (m^2d^{-1})	L^2T^{-1}

Symbol	Description	Dimension
q_M	Maximum specific discharge (mmd^{-1}, md^{-1}, $ls^{-1}ha^{-1}$)	LT^{-1}
q_t	Specific discharge at time t (md^{-1})	LT^{-1}
q_0	Downstream flow per m length of interceptor drain (m^2d^{-1})	L^2T^{-1}
	Original outflow from canal (m^2d^{-1})	L^2T^{-1}
$\|q_0\|$	Flux to drain (absolute value) one-sided (m^2d^{-1})	L^2T^{-1}
q_1	Flux density above drain level (md^{-1})	LT^{-1}
	Upstream flow per m length of interceptor drain (m^2d^{-1})	L^2T^{-1}
	Outflow from canal after interceptor drainage (m^2d^{-1})	L^2T^{-1}
q_2	Flux density below drain level (md^{-1})	LT^{-1}
R	Amount of percolation water (mm)	L
	Amount of recharge (mm)	L
	Recharge by precipitation or irrigation excess (md^{-1})	LT^{-1}
	Hydraulic radius (m)	L
	Radius sphere of influence of well (m)	L
R^*	Long-term leaching requirement (mm)	L
Re	Reynolds' number	-
r	Effective drain radius (m)	L
	Radius of borehole (cm)	L
	Radius of piezometer protecting pipe (cm)	L
	Distance from well centre (m)	L
r^*	Distance centre of cavity to surface if $H/D > 8$ (piezometer method) (cm)	L
r_d	Distance from cavity well where spherical flow starts (m)	L
r_o	Radius of sphere equivalent to cavity for $H > 8D$ (piezometer method) (cm)	L
r_w	Radius of well (m)	L
r_8	Radius $8d$ beyond which flow is supposed radial (piezometer method) (cm)	L
S	Maximum potential retention in a basin (mm)	L
	Seepage (mm)	L
	Spacing of individual corrugations (m)	L
S_r	Excess of water at soil surface (mm)	L
	Surface runoff (mm)	L
S_r'	Surface runoff produced by precipitation P' in unit hydrograph (mm)	L
SAR	Sodium adsorption ratio ($meq^{1/2}l^{-1/2}$)	$M^{1/2}L^{-3/2}$
SDW	Sum of days with waterlogging during certain period (d)	T
SDW_{30}	SDW at less than 30 cm depth (d)	T
SDW_{50}	SDW at less than 50 cm depth (d)	T
s	Hydraulic gradient	-
	Hydraulic gradient for drainpipe flow	-
	Drain slope	-
s_x	Standard deviation in Gumbel distribution	-
T	Return period (years)	T
	Transmissivity (KD) (m^2d^{-1})	L^2T^{-1}
TDS	Total dissolved solids (gl^{-1})	ML^{-3}
t	Time (s, h)	T
	Average time of storm (h)	T
	Time base length of hydrograph (h)	T
t_c	Concentration time (h)	T
t_d	Lag time between average time of storm and time for maximum discharge (h)	T
t_e	Elevation time or time to peak (h)	T
t_r	Recession time (h)	T

Symbol	Description	Dimension
t_1	Initial time during time period (s)	T
t_2	Final time during time period (s)	T
u	Constant in Gumbel distribution	-
	Wetted perimeter (m)	L
	Wet circumference of drain (m)	L
v	Vertical	-
	Average flow velocity (ms^{-1})	LT^{-1}
	Wind velocity (ms^{-1})	LT^{-1}
v_d	Flow velocity at outlet of delivery pipe of pumping station (ms^{-1})	LT^{-1}
v_i	Average flow velocity in section i of main drainage system (ms^{-1})	LT^{-1}
v_M	Maximum average flow velocity (ms^{-1})	LT^{-1}
W	Total flow resistance near drain (radial + entry) (dm^{-1})	TL^{-1}
W_r	Radial flow resistance (dm^{-1})	TL^{-1}
W_{fc}	Moisture content at field capacity (mm)	L
x	Horizontal coordinate	-
	Distance (m)	L
	Distance from drain (upstream) (m)	L
	Values of extremes in Gumbel distribution	-
$\bar{x}$	Average value in Gumbel distribution	-
x_0	Limiting value in Gumbel distribution	-
Y	Relative crop yield	-
y	Vertical coordinate	-
	Water depth in drainage channel (m)	L
	Depth of water level below groundwater in auger hole (cm)	L
	Water level below groundwater in piezometer (cm)	L
	Reduced Gumbel variable	-
$\bar{y}$	Average value of y in interval where $y > 3/4y_0$ (auger-hole method) (cm)	L
y_0	Depth of water level below groundwater in auger hole at time 0 (cm)	L
y_1	Depth of water level below groundwater in auger hole or piezometer at time 1 (cm)	L
	Water depth in drainage channel at extreme discharge (m)	L
y_2	Depth of water level below groundwater in auger hole or piezometer at time 2 (cm)	L
	Water depth in drainage channel at design discharge (m)	L
Z	Drain depth (m)	L
Z_c	Collector depth below soil surface (m)	L
Z_r	Average thickness of rootzone (m)	L
z	Groundwater depth (m)	L
	Design groundwater depth midway between drains at design discharge (m)	L
$\bar{z}$	Average depth of the water table (m)	L
z_c	Critical groundwater depth where $E = 0$ (linear model) (m)	L
z_h	Critical groundwater depth where $E = 0.4343E_0$ (exponential model) (m)	L
z_i	Vertical distance of layer i (m)	L
z_0	Groundwater level observations in piezometer laid on drainpipe (m)	L
z_1	Salt content in rootzone at start of certain period ($mm.dSm^{-1}$)	$T^3I^2M^{-1}L^{-2}$
z_2	Salt content in rootzone at end of certain period ($mm.dSm^{-1}$)	$T^3I^2M^{-1}L^{-2}$
$z_{6.5}$	Groundwater level observations in piezometer at 6.5 m from drain (m)	L

Symbol	Description	Dimension
$z_{12.5}$	Groundwater level observations in piezometer at 12.5 m from drain (m)	L
z_{25}	Groundwater level observations in piezometer at 25 m from drain (m)	L
α	Constant in Gumbel distribution	-
	Coefficient	-
	Coefficient in side slope (*v*:*h*)	-
	Angle of slope (radians)	rad
γ	Angle (radians)	rad
φ	Angle (radians)	rad
η_p	Pump efficiency	-
η_t	Transmission efficiency in pumping station	-
θ_l	Minimum soil water fraction for non-crop stress (m^3/m^3)	-
θ_{fc}	Soil water retained at field capacity (m^3/m^3)	-
θ_{wp}	Soil water retained at wilting point (m^3/m^3)	-
κ_i	Transformed permeability of layer *i* considering anisotropy (md^{-1})	LT^{-1}
λ	Coefficient	-
	Characteristic length of aquifer (m)	L
μ	Coefficient	-
	Drainable pore space	-
$\bar{\mu}$	Average drainable pore space	-
ζ_i	Transformed vertical distance considering anisotropy (m)	L
π	3.1416...	-
ρ	Density of water (kgm^{-3})	ML^{-3}
τ	Lag time (d)	T
ν	Kinematic viscosity (m^2s^{-1})	L^2T^{-1}
Φ	Function	-
	Coefficient	-
Ψ	Coefficient	-
ΔH	Head loss in drain (m)	L
ΔH_i	Head loss in drain in *i* (m)	L
ΔH_1	Head loss in first section of drain line with increasing diameter (m)	L
ΔH_2	Head loss in second section of drain line with increasing diameter (m)	L
Δh	Head loss (m)	L
	Average fall of groundwater table in certain period of time (mm)	L
	Head difference along canal caused by wind (m)	L
	Head loss along culvert	L
	Available head in weir (m)	L
	Total head loss in suction and delivery pipes of pumping station (m)	L
	Difference in piezometric head above trench bottom of interceptor drain (m)	L
ΔS_r	Increment of surface runoff (mm)	L
Δt	Increment in time (s)	T
ΔW	Time-step (d)	T
	Change of the moisture content of rootzone (mm)	L
Δx	Distance step (m)	L
Δz	Variation of salinity in the short term ($mm.dSm^{-1}$)	$T^3I^2M^{-1}L^{-2}$

Chapter 1
Introduction

NEED FOR AND BENEFITS OF LAND DRAINAGE

Drainage of agricultural land is one of the most critical water management tools for the sustainability of productive cropping systems, as frequently this sustainability is extremely dependent on the control of waterlogging and soil salinization in the rootzone of most crops. On some agricultural lands, the natural drainage is sufficient to maintain high productivity. However, many others require improvements in surface and subsurface drainage in order to optimize land productivity, while maintaining the quality of soil resources. As time passes, drainage requirements may change because of changes in the general socio-economic conditions, such as input and output prices, and more intensive crop rotations.

In rainfed and irrigated areas of the temperate zones (where waterlogging is the dominant problem in lands lacking natural drainage), proper drainage has improved soil aeration and land and rural road trafficability. Moreover, it has facilitated the lengthening of the potential crop growth period.

In the irrigated lands of the arid and semi-arid regions (where salinity problems dominate), in addition to the benefits described above, subsurface drainage has been essential for controlling soil salinity and reducing the incidence of erratic crop yields.

In the semi-humid and humid tropical regions, drainage development has been less than in the agroclimate zones mentioned above. However, salinity control is required during the irrigation season in the semi-humid tropics, as is waterlogging control during the rainfall season (e.g. in countries with monsoon rainfall). In addition, flood control is also often a necessary component of drainage projects in many of these areas in order to protect the safety and livelihood of the rural population more effectively. In plains in the humid tropics, occurrences of organic soils or acid-sulphate soils often present special problems whose resolution entails careful drainage.

The general goal in all agroclimate zones is to obtain a proper water table control necessary at the given time and under the given circumstances. Sometimes, special water control methods are required, e.g. in acid-sulphate soils and in peat soils, and in areas where rice is grown in rotation with dry-foot crops.

CURRENT CONTEXT OF LAND DRAINAGE

Land drainage works usually have public (or semi-public) and individual farmers' components. Especially in developing countries, drainage projects deal with the former component and often take place in deltaic areas, coastal fringes and river valleys where population is increasing rapidly and land use is intensifying. Projects are prepared, carried out and financed under the responsibility of a standing government organization or a specific rural water authority. Completed projects are operated and maintained by the government organization in charge of managing the existing systems. However, increasingly, self-financing authorities and water users organizations with farmer participation are becoming legally involved in the implementation and financing of the necessary operation and maintenance (O&M) activities of the lower tiers of public irrigation and drainage systems.

Modern drainage system planning and design should take into account a wide range of agricultural and non-agricultural values and consider a broad group of stakeholders. The publication *Reclaiming Drainage, Toward an Integrated Approach* (World Bank, 2004) provides sound guidance for facilitating wider planning and design.

Much of the existing drainage installation work has been done in developed countries. While about 27 percent of the agricultural land in developed countries is provided with some form of improved drainage, only about 7 percent of agricultural land in developing countries is supplied with drainage (Smedema and Ochs, 1998). Therefore, there is room for drainage development in the latter countries, because land productivity has to increase dramatically in order to enable rural incomes to rise.

The context of land drainage has changed considerably in recent decades owing to changes in agriculture policies, mainly in developed countries, and to new environmental and natural resource considerations.

In developed countries, food security generally means quality of efficiently produced safe food, and environmental issues are becoming a first priority, jointly with maintaining the rural environment. Therefore, no substantial horizontal expansions of new drainage developments are foreseen in these countries, but only consolidation of the existing agricultural areas and the rehabilitation of and/or technological improvements to outdated existing drainage systems in line with the changed socio-economic circumstances. As there is a good background of drainage information in these countries, the transfer of expertise and the evaluation of the performance of existing systems may be the predominant activities as far as drainage is concerned.

In developing countries, food security means food availability, which is not achieved satisfactorily in too many countries. Consequently, the enhancement of agricultural production to raise rural incomes and the reduction of crop failure risks are still the main priorities, but on a sustainable basis.

In arid and semi-arid regions, irrigation development is still required in order to achieve food security. Therefore, to achieve the continuous benefits from irrigation projects, new or more intensive drainage systems will be needed to control waterlogging and soil salinity, and to ensure the sustainability of production on irrigated lands. This is especially the case in areas where irrigation water availability and water quality decrease owing to urban, industrial and environmental developments (Croon and Risseeuw, 2005). Drainage will also be required in order to reclaim salt-affected soils and problem soils if new lands are needed for agricultural use. In already drained lands, evaluation of the performance of existing drainage systems will also be needed in order to determine the need for rehabilitation.

In addition, the installation of new drainage systems in the humid tropics is expected in the near future. However, little practical experience is available in much of the humid tropics. In these areas, crop diversification (through the introduction of dry-foot crops in areas where rice fields are traditionally the major land use) will require subsurface drainage in addition to the existing surface drainage facilities. Agriculture intensification, by growing vegetables and tropical fruit trees, will also need subsurface drainage in areas lacking natural drainage. If in irrigated lands drainage is closely related to irrigation, in the flat areas of the humid and semi-humid tropics land drainage must be a component of integrated flood management.

Environmental issues are becoming more important and, therefore, they should be considered in the planning and designing of new drainage systems and in the rehabilitation of existing ones. Water quality control must also be considered. Moreover, opportunities for enhancement, reuse and protection of water are paramount for an intervention for drainage to be considered successful and supported by stakeholders and the community of concerned citizens adjacent to a project. When planning or designing drainage systems, consideration must also be given to any other locally important environmental matters, such as the protection and enhancement of wetlands and wildlife habitats, and to matters related to community health.

These changes have brought concerns for the environment, and the recognition of the important need to integrate users into the planning, design, operation and maintenance process, and financing of the capital and recurrent costs of land drainage systems. In addition, it is necessary to integrate irrigation, drainage and flood control with important agronomic, environmental and socio-economic aspects. Such integration is intended to provide a proper balance between sustainable agriculture and the environment in rural areas. With proper planning, drainage can also contribute to restoring or maintaining environmental values.

In addition to the previously described changes, improvements in the technology and methods applied in drainage development have been made as a consequence of the experience gained and the research carried out in recent years. For example, computers and computer-trained people are available even in remote rural environments, and remote-sensing technologies are becoming adapted to identify waterlogged and salt-affected areas.

NEED FOR GUIDELINES AND COMPUTER PROGRAMS FOR PLANNING AND DESIGN

A land drainage project is frequently a component of another agricultural water management project where drainage practices may be required, e.g. an irrigation project. Then, integration of the different components of the land and water project is especially essential. In the drainage component of such broad development projects, the following phases may be distinguished:

- identification, characterization and priority ranking of the problem areas;
- planning and designing of the systems;
- implementation and control of the quality of the works;
- O&M;
- evaluation of the performance of the system.

Through this process, many essential decisions must be taken at different government levels on proposals made by planners and designers.

This publication considers only the first two items, with the emphasis on the technical aspects. Nijland, Croon and Ritzema (2005) provide guidelines for the implementation, operation and maintenance of subsurface pipe agricultural drainage systems, including the assessment of the quality of the installed works. FAO (2005) has also published guidelines for selecting and designing the most appropriate drainage materials (pipes and envelopes) for land drainage systems. A future FAO publication will cover the evaluation of the performance of existing drainage systems.

Although up-to-date text books on land drainage exist, such as ILRI (1994), Skaggs and Van Schilfgaarde (1999), and Smedema, Vlotman and Rycroft (2004), specific and concise guidelines and user-friendly computer programs for drainage design calculations (based on simple and limited input parameters) may facilitate the work of field drainage engineers in planning or designing drainage facilities.

These guidelines are intended to serve as a tool for integrated drainage planning and design, giving due consideration to sustainability, and to environmental and socio-economic factors. Therefore, this publication is not a comprehensive handbook as such; rather, it presents new guidelines and calculation tools developed under the current land drainage context. It is oriented to engineers with previous drainage background. Readers who might not be familiar with some background theory are referred to the recommended handbooks and the references quoted in this publication.

IMPORTANCE OF FOLLOWING A PLANNING AND DESIGN PROCEDURE

The evaluation and integration of alternative solutions and comprehensive planning are critical to the success of drainage projects. Drainage is only one part of the solution, and careful consideration of potential alternatives is necessary where developing new areas or improving existing agricultural lands. Comprehensive planning in a river basin is critical, especially for large or numerous small projects. The drainage options should be weighed carefully along with the other water management alternatives in order to achieve the socio-economic development and environmental protection desired in any project area.

As there are many unknowns and assumptions in areas with little or no experience in drainage, a flexible approach is required early in the planning and design process. This is so that adjustments can be made as necessary in order to address unforeseen items encountered during the investigations or design problems that develop as the construction work is in progress. This means that, where part of the system has been implemented, the assumptions should be verified systematically by monitoring and evaluation (M&E) under the responsibility of a permanent institution. Where it is shown that the system does not fulfil all the expectations, the design criteria or the methods applied can still be adjusted before the remaining area is constructed. Thus, good design procedures will result in efficient, cost-effective and easily implemented drainage designs.

In this way, experience is built up, which can finally lead to fixed design procedures that are adapted to the prevailing circumstances. Such design procedures for drainage, as well as for most civil works, are complex. In drainage design, it is important to start with a well-prepared but flexible plan that is developed within a framework of public participation and sound consideration of alternatives. Environmental, social, economic, health and physical factors must be considered in preparing the designs. The participatory procedures used in planning cannot be discarded during the design process. They must be continued and made a part of the design procedure in order to ensure a sound follow-up so that stakeholders are satisfied. The resulting drainage water management system should be easy to operate and maintain in accordance with the needs of the area.

SCOPE OF THIS PUBLICATION

The concept of this FAO Irrigation and Drainage Paper is to focus on the "what to do and when" in the main text while including the technical details of the "how to do" in the annexes.

Chapter 2 provides general information about environmental considerations that should be taken into account in drainage projects in order to mitigate the unfavourable impacts of drainage development on the environment and enhance the positive ones. Chapter 3 deals with the socio-economic aspects that must be considered in the planning and design of agricultural drainage systems. Chapters 4–8 address the technical aspects.

In the planning and design procedure, different phases can be distinguished. These range from the identification of the problem lands of an agricultural area and their further characterization, to the assessment of the technical, socio-economic and environmental feasibility of the systems planned to solve the waterlogging and salinity problems. Once this feasibility has been confirmed, the design of the drainage works can be completed. For these purposes, the first step in the procedure is the collection of the necessary field information (climate data, topographic maps, soil and hydrological data, etc). According to the specific objectives of the procedure phase, fieldwork is done at different levels of intensity, and maps are prepared at different scales. Chapter 4 contains a description of this process.

Two complementary drainage systems are usually distinguished to control waterlogging and salinity, where drainage is not adequately provided by nature and by the existing watercourses: (i) individual surface and/or subsurface field drainage systems to remove excesses rainfall or irrigation water from the individual fields; and (ii) an open public main drainage system that collects the water from the field drainage systems and carries it to an outlet. Both systems must be constructed or improved in order to ensure adequate land drainage and soil salinity control.

The public main drainage system consists of an outlet for the drainage water (an open connection, outlet sluice or pumping station) and a network of open channels to convey the water from the fields to this point. Without this main drainage system, field drainage cannot work properly. For this reason, this main drainage system for rather flat areas is described first in Chapter 5.

Where the soils are permeable enough, and water levels in the main drainage system are maintained at an adequate depth, a wide-spaced drainage system may be sufficient to maintain properly deep groundwater levels in the whole area. In some cases, the main drainage system can provide the required drainage. However, even where the normal groundwater level remains deep enough during wet periods, water may remain on the ground surface or on poorly permeable layers at shallower depths, where it forms perched water tables. Under these conditions, downward percolation can often be improved by deep ploughing or subsoiling to break up hardpans and other types of less pervious soil layers. In some exceptional cases, a single operation is sufficient, but regular repetition is required in others.

If these measures are not successful, a field drainage system must be laid out in order to remove this surface water. The same is the case where the main drainage system fails to remove sufficient groundwater. Field drainage systems can consist of shallow open waterways to remove water standing on the soil surface, or deeper drains to control high groundwater tables and to discharge salts. The latter are usually buried pipe drains.

Surface drainage systems are needed where soil infiltration rates are low and rainfall or irrigation water ponds on the ground surface. Such low infiltration rates are usually caused by the formation of a surface crust, to which some soils are very susceptible. Stagnation of this kind is usually first noted in small depressions and at the lower borders of irrigation basins. The problem can be reduced by smoothing the land to remove small depressions and by providing the surface with a consistent non-erosive slope for excess water to flow through furrows or shallow field ditches towards surface drainage outlets. In very flat areas, bedding systems are applied to create strips with less waterlogging in between furrows, which convey excess surface water to the ditched field borders. Surface drainage water collected in these ways can be discharged through protected points into the larger watercourses of the main drainage system. Such surface drainage systems are described in Chapter 6, as are methods to estimate peak discharges, which are needed to design the different components of the drainage system.

Deeper subsurface drainage is needed to prevent high groundwater tables that lead to both waterlogging and soil salinization, of which the latter is the main consequence of high water tables in arid environments. Waterlogging is caused by rainfall, snowmelt and, in dry periods, excess irrigation water. The way its control is achieved depends on the causes of the problem. Where the surface drainage system is capable of removing excess water, but the groundwater table is still too high, the soil is not permeable enough for sufficient flow to the surface system. This is a common feature in plains and in some sloping lands, and it requires additional measures for field drainage.

Another cause of high water tables is seepage. This is the lateral movement of excess water from leaky irrigation canals or from higher ground elsewhere, or the upward flow coming from deep artesian aquifers. Such seepage can be controlled at the source (e.g. by canal lining), on its way (by interceptor drains) or at the field itself (by drains or wells). However, drainage of areas recharged by seepage is often difficult and costly. In severe cases, it is usually better to leave such areas as wetlands.

Groundwater control can be achieved by open drains, buried pipes and wells. The function of these hydraulic structures is to accelerate the removal of excess groundwater and to maintain the water levels in the soil at such depths that they do not harm crop production and soil workability. Moreover, they should provide sufficient downward movement of water to prevent the capillary rise and subsequent accumulation of salts in the topsoil and to evacuate the salt that has entered the field with the irrigation water. The former requirement is dominant in humid areas, the latter in arid environments.

The different methods of subsurface drainage have their advantages and disadvantages. Buried pipes do not have most of the drawbacks of open drains, i.e. loss of land, maintenance problems, obstruction of farming operations, and weed growth. However, they need to be installed properly and maintained in good condition by adequate cleaning. The frequency of cleaning depends on the local circumstances. While some soils cause hardly any clogging of the pipes, other locations show such rapid clogging (often by iron compounds) that the pipes must be cleaned each year. This combination of soil properties and cleaning operations will lead to a certain "maintenance status" varying from "excellent" to "poor", depending on the degree of clogging. Subsurface drainage with buried pipes forms the main subject of Chapter 7.

Public or individual vertical drainage systems driven by pumping wells can be used to lower the groundwater level under special hydrogeological circumstances, i.e. a good aquifer that has sufficient contact with the shallow groundwater. However, it is only economic where the water obtained can be used for irrigation or for municipal supply. Moreover, it has the drawback that pumping aquifers usually leads to unwanted mobilization of salts from deep subsoil layers, which may subsequently cause salinity damage to the environment. Chapter 7 also gives a short description of well drainage and its consequences.

Finally, Chapter 8 describes the computer programs developed for calculating the design parameters of subsurface drainage systems. These programs have also been included in the CD–ROM accompanying this FAO Irrigation and Drainage Paper.

The annexes of this publication provide more detailed information, including technical background and appropriate equations used in the computer programs.

Chapter 2
Environmental considerations in drainage projects

INTRODUCTION

The results of the socio-economic benefits of creating conditions for sustainable agricultural production and reclaiming problem soils through land drainage are the increase in agricultural production through higher yields, the introduction of more rewarding crops and the obtaining of higher and less erratic farm income, in addition to a generally improved accessibility of the rural area and healthier living conditions.

Moreover, land drainage provides environmental benefits through the conservation and improvement of the quality of soil resources. In fact, irrigation development in arid and semi-arid regions without adequate drainage facilities has caused excessive salinization, resulting in extreme cases in severe deterioration of existing agricultural lands and making them unfit for agricultural production. Salinization is commonly reported as seriously affecting 20–30 million ha of the approximately 250 million ha of irrigated cropland. In addition, 250–500 000 ha are estimated to be lost from production every year as a result of salt buildup (FAO, 2002a). This area of salinization will grow if irrigation improvements to minimize water losses are not accelerated (Smedema and Ochs, 1998), and if the decreasing availability of good quality irrigation water in a number of important irrigated river plains and deltas in arid and semi-arid regions upsets soil salinity balances due to insufficient leaching (Croon and Risseeuw, 2005).

Salinization is controlled by leaching excessive salts from the crop rootzone. This is the case where the net water movement in the soil over the year is downward and of sufficient magnitude to evacuate the accumulated salts. Where natural drainage in the soil profile does not achieve this, artificial drainage is required in order to provide a sustainable irrigated agricultural system. The salts requiring leaching are present in irrigation water and are also brought to the rootzone by capillary rise from high water tables, especially where the groundwater is salty.

The excessive seepage from canals, reservoirs and overirrigated farmlands also often contributes to groundwater recharge causing waterlogging, which can contribute greatly towards the need for artificial drainage systems. Judiciously minimizing irrigation water losses is a prerequisite to the installation of economically efficient drainage facilities.

Thus, land drainage can also have a favourable environmental impact as well-designed and implemented drainage systems can control soil salinization. At the same time, soil degradation and land desertification are avoided.

In some cases, already salinized lands can be reclaimed and brought back to their original or even better productivity through leaching, drainage and appropriate soil management. For this purpose, sufficient seasonal rainfall or irrigation water availability in combination with a drainage system is often required, which can serve for permanent salinity control after the reclamation. From an environmental standpoint, the lack of proper drainage in these areas results in farming systems that degrade, causing numerous socio-economic problems that may eventually lead to desertification.

In humid temperate zones, drainage systems can facilitate the management of groundwater levels according to an environmentally desired regime. Notably in arid and semi-arid regions, public drainage systems may facilitate the evacuation of polluted drainage flows of urban and/or industrialized areas in order to avoid their mixing with fresh irrigation water supplies downstream.

In tropical areas, health problems are generally associated with vector-borne diseases, transmitted by insects that breed in stagnant water, and water-based diseases, transmitted by aquatic and semi-aquatic snails. Improvements in drainage conditions might have positive impacts on health if stagnant water is eliminated. In addition, lowering the groundwater table will facilitate sanitation and reduce the spread of diseases brought about by the absence or poor functioning of sanitation systems. Health issues related to drainage water management are described in FAO/ICID (1997).

However, drainage projects can have unfavourable side-effects on natural resources (soils and water) and on wetlands and the landscape. Potential negative environmental impacts of land drainage systems are numerous and include:

- Changes in hydrological peaks can affect downstream areas.
- The discharge of often saline drainage effluent can harm downstream areas.
- Soluble substances such as those causing eutrophication, remnants of pesticides and herbicides, and other pollutants or toxic substances (e.g. from urban and industrial areas) usually collect and concentrate in drainage water, notably in arid and semi-arid regions. They may enter the food chain through aquatic life and crops irrigated with drainage water.
- Disposal options for drainage water, such as evaporation ponds and outflow drains, and water treatment options, especially stabilization ponds and water desalination plants, can become sources of pollution and, thus, become hazardous.
- Banks of open drains can be eroded by water if they are not adequately designed and maintained. Moreover, even soils in flat areas can be eroded if surface runoff is not managed properly.
- Subsidence and irreversible desiccation of peat soils are common side-effects where such soils are drained improperly.
- Acid sulphate soils can form where lands of tropical swampy areas with soils rich in iron sulphate ($FeSO_4$) are drained.
- The draining of lands adjacent to wetlands or higher-lying cropped areas can have negative effects on their groundwater levels.
- Straight layouts of the main drainage system can alter natural watercourses and have a negative impact on the riparian natural vegetation and the landscape.

As some of the above environmental matters are related to farming operations and water management at the field and project levels, the application of good agricultural practices is the first action to consider for reducing unfavourable impacts. However, water conservation and recycling within the project area and the safe disposal of drainage water must also be considered. Therefore, care must be taken to consider thoroughly the ramifications resulting from any drainage system changes. Improvements in the system designs and the construction of remedial structures or adoption of more suitable alignments of main drains may have considerable effects on downstream users. Basin-wide evaluations may be required in order to reach agreement on appropriate effective and sustainable control measures.

As both socio-economic improvements and environment enhancement should be the goal of drainage projects, potential conflicts between rural development and environment should be avoided, mitigated or resolved. Therefore, care must be taken in the planning, design, implementation, operation and maintenance phases of drainage projects in order to ensure that negative environmental impacts, once determined, are minimized to politically acceptable dimensions.

Major environmental impacts of irrigation and drainage projects and mitigation measures are described in FAO/ODA (1995). In addition, the International Commission on Irrigation and Drainage (ICID) prepared an "environmental checklist" to assist in the identification of environmental effects of irrigation, drainage and flood control projects (ICID, 1993). This chapter focuses on the environmental aspects that should be considered in drainage planning and design. The following section discusses these issues briefly. Other aspects that can be controlled by management, especially concerning drainage water, have been considered in depth by FAO in other publications (FAO/ICID, 1997; FAO, 2002b).

ENVIRONMENTAL PROBLEMS

Changes in hydrological peaks

With time, hydrological changes in outflow can have negative or positive impacts downstream and can cause environmental concerns. The planning and design process must identify these potential changes from present conditions and their consequences. For example, in humid tropical and monsoon areas, the flood control and drainage component of planning and design are strongly interrelated. In these cases, main drains are usually multipurpose channels for flood control as well as for drainage water disposal, and they are sometimes used as irrigation canals in the dry season. Climate changes also influence the environment either directly or via changes in water flows.

Water quality management

As a water development project generally causes changes in water quality both inside the area and downstream, water quality control is an essential factor to be considered in drainage planning and design. At the outlet of the system, drainage water quality influences the receiving water, usually in a negative way. Attention is needed for downstream water intakes for irrigation, municipal water supplies, nature reserves and wildlife habitats, which may be extremely sensitive to water pollution. The choice of a place to discharge drainage water is a crucial decision in the process of water quality control.

In arid zones, in addition to many possible different pollutants, the salt content of drainage waters is a main cause of concern because soil salinity control is necessary and the leached salts are discharged through the subsurface drainage system. Salinity control is needed to prevent secondary salinization of soils caused by the salts added with the irrigation water or the salts accumulated in the rootzone through capillary rise of saline groundwater. Irrigation development in arid zones can also lead to mobilization of primary salts present in soils, subsoils and deep strata. This process should be avoided as much as possible because salt mobilization, in conjunction with diminished river flow resulting from water diversions for irrigation, leads to increasing salinity of river waters. Smedema and Shiati (2002) have described in detail the principal salt mobilizing mechanisms and irrigation-induced river salinization with some examples from India and the Islamic Republic of Iran.

To provide quantitative information on the salt content of drainage water, Table 1 shows average values of electrical conductivity (EC) and total dissolved solids (TDS) in drainage waters.

In some areas, drainage water with high levels of toxic trace elements, such as boron and selenium, might cause environmental problems. Suspended sediment in surface runoff is another water quality factor to consider.

Therefore, analysis of the soils and the present groundwater quality

TABLE 1
Classification of waters

Type of water	EC (dS/m)	TDS (g/litre)	Water class
Drinking and irrigation water	< 0.7	< 0.5	non-saline
Irrigation water	0.7–2.0	0.5–1.5	slightly saline
Primary drainage water and groundwater	2.0–10.0	1.5–7.0	moderately saline
Secondary drainage water and groundwater	10.0–25.0	7.0–15.0	highly saline
Very saline groundwater	25.0–45.0	15.0–35.0	very highly saline
Seawater	> 45.0	> 35.0	brine

Source: Adapted from FAO, 1992b.

must be made, as well as of the expected seepage water that may have a great influence on the future drainage water quality. The soils, for example, will lose most of their soluble salts within a few years, but seepage is a continuous process. If the seepage water is highly saline, a small amount will carry large quantities of salt into drains within the project area. This in turn will have a profound influence on the downstream water quality.

Generally, salinity is not a major problem in temperate regions. However, pollution from agricultural chemicals and in some cases manure applications requires careful control in spite of the fact that pollution tends to be diluted by frequent rainfall excesses, which is not the case in arid and semi-arid regions. Nitrogen (N), phosphorus (P) and pesticide residues are the major elements of concern. Considerable guidance on this subject has been provided by FAO (1996).

Therefore, while salt is a major concern in drainage water reuse in downstream agriculture, there are many other substances to be considered, such as:

- Domestic and agricultural wastes that may cause anaerobic conditions in the receiving waters. They are subject to a rapid biodegradation where enough oxygen is available, but this is not the case in almost stagnant water.
- Residues from manure, fertilizer and biodegradation of organic wastes in the water, mainly nitrates and phosphates, are favourable to agriculture, but they may cause algal blooms, rapid growth of aquatic weeds and ecological disturbances in downstream waters.
- Soluble toxic substances of natural origin. They occur in alluvial soils in some regions and are concentrated by reuse of the water and by its evaporation. Examples are: arsenic and fluoride in some groundwaters, making them unsuitable for drinking, boron in areas influenced by volcanism (toxic to fruit trees) and selenium in other regions (toxic to wildlife). Many of these elements are necessary for life in small quantities, but they may be present at dangerously high levels in some areas.
- Insoluble toxic substances of natural origin, such as heavy metals. They are almost absent in clear water, but they are adsorbed to suspended silt and clay and concentrated in bottom sediments from where they may move downstream in times of high discharges. Some processes can mobilize these adsorbed substances, after which they can become further concentrated through bio-accumulation.
- Residues from soluble and persistent pesticides, which endanger safe use of water for consumption by humans and animals. Persistent insoluble pesticides behave in much the same way as heavy metals.
- Single-celled organisms, bacteria and viruses that cause water-borne diseases. They are especially dangerous in drinking-water (where they can cause diseases such as cholera and typhus), while other species can spread human and plant diseases via the irrigation water (bilharzia, brown rot in potatoes, etc.).

In addition to controlling surface water quality, care must be taken that polluted waters do not reach deep freshwater aquifers that are used or may be used for municipal water supply. Unconfined aquifers and semi-confined ones with hydraulic heads below groundwater are vulnerable to such pollution. Where the aquifers are unconfined, saline water leakage from evaporation ponds, used for drainage water disposal, will move downward rapidly owing to its higher density. However, where the drainage water quality is good, recharge of aquifers may be beneficial. Designed recharge areas may even be appropriate in such a case.

Thus, the quality and quantity of water that may seep and cause recharge must be considered. However, in all cases, monitoring is critical to see that certain changed conditions do not create unexpected effects on the environment or groundwater resources.

Soil conservation

Even in flat areas, soil erosion by surface runoff is a potential risk, in particular at the outlets of the field surface drainage systems. The banks of open ditches are especially sensitive to water erosion where they are not covered by protecting natural vegetation. In addition to soil losses caused by erosion, the sediments transported by surface water are deposited in the system of open drainage channels, thereby reducing their hydraulic capacity.

Therefore, when designing and maintaining surface and subsurface drainage outlets, soil conservation measures must be considered in order to ensure the safe discharge of drainage water flowing from the field systems. Where singular drainage systems are used, the banks of the open ditches should be designed with an appropriate slope and be protected by natural vegetation. In arid zones, it is often difficult to maintain adequate plant cover (a reason for preferring composite drainage systems with subsurface pipe collectors). In this case, maintenance considerations concern primarily the main channels, especially at the outlets of these collectors.

Wetland and wildlife habitat areas

Existing valuable ecosystems must be respected where new systems are being built or additions made to existing projects. Important habitat lands are of worldwide concern and potential damages are numerous. Changes in existing water tables or in land use, and the pollution of wetlands, lakes and/or streams by disposed drainage water, can lead to loss of typical vegetation and fauna in nearby nature reserves or natural areas.

These ecological risks and opportunities entailed by drainage projects should be identified by environmental impact assessments (EIAs). Environmental plans should also discuss the protection and mitigation measures to be taken and formulate the works required to best protect these areas from negative influences or to enhance their existing quality. As many aspects considered in EIAs are not the direct competence of the drainage engineer, proper environmental specialists should assist in these efforts.

WATER CONSERVATION AND RECYCLING WITHIN THE PROJECT AREA

Management to control any drainage-related environmental problems must occur throughout the water development scheme and should not be confined only to the disposal point. The most efficient way to control water quality is to minimize the initial pollution from each field as the cumulative impacts are much more difficult and expensive to handle. Therefore, in order to prevent damages downstream from the outlet of the drainage system, a major effort must be made to minimize the degradation of water within the project area through sound field and farm water management.

In order to optimize agricultural production as well as to minimize pollution, farming practices should be in harmony with sound agronomic principles, such as the use of integrated pest management systems and controlling the use of fertilizers so they are provided as the plants need them. In addition, irrigation practice should prevent soil erosion. Therefore, care must be taken to prepare farmers to consider water quality impacts resulting from their operations. Irrigation and drainage designs must be compatible with the farming operations anticipated.

Inside the project area, management of drainage waters includes water conservation measures at the field level and reuse of drainage water at the farm and scheme levels where possible. These measures imply use of environmental criteria for adequate planning and design of drainage systems. However, there are often some practical complications, such as the implementation of a sound salt management system, which is required in arid and semi-arid zones in order to control soil salinization and degradation. These measures are sometimes difficult to implement because they require considerable management and discipline of the farmers.

At the field level, the main objective is to reduce the volume of drainage water and the salt and pollutant load discharged through the field drainage systems. This can be achieved by reducing surface runoff and deep percolation in the irrigated fields through improving on-farm irrigation management and by shallow water table management. Additional measures at the scheme level are reuse of drainage water, groundwater management, land retirement and dry drainage, and biological drainage. These measures are described briefly below. Additional detailed information is provided by FAO/ICID (1997), FAO/IPTRID (2002) and FAO (2002b).

Conservation and recycling of water within the project area will reduce the amount of drainage water to be discharged. However, this will be at the expense of its quality as salts, plant nutrients and other soluble substances will become more concentrated. The quantity of outlet water to be managed will be lower, but its quality will deteriorate. However, the amounts of persistent soluble pollutants discharged from the project area will remain almost the same unless they are reduced at the source, i.e. at field level. Therefore, field water management and sound crop husbandry practices are essential factors in controlling the quality of drainage water.

Improving on-farm irrigation management

Increasing the irrigation efficiency within the project area may reduce the amount of drainage water to be disposed of. Sound irrigation application is necessary in order to reduce surface runoff water losses. Deep percolation can be reduced if the amount of irrigation water applied effectively and uniformly only covers crop water requirements plus the leaching fraction necessary to control soil salinity.

In many irrigation schemes, there is room to improve irrigation water conveyance and application efficiency by:

- improving local and regional scheduling of irrigation supplies;
- improving the irrigation practice in order to eliminate surface runoff;
- ensuring uniform water application over all the field;
- adjusting the irrigation requirements to the actual evapotranspiration needs considering the soil moisture storage capacity, while ensuring the annual leaching requirement for salinity control;
- making optimal use of rainfall in the annual salt/water balance in order to reduce irrigation applications in the drier part of the year;
- improving the existing surface irrigation systems;
- changing to pressurized systems, such as sprinkler or drip irrigation.

While upgrading the irrigation management to save water, care should be taken to ensure a minimum leaching fraction to wash out the salts applied with the irrigation water. Moreover, in arid and semi-arid regions, continued availability of relatively fresh drainage water flows (stemming from inefficient irrigation practices in upstream areas) is gaining importance in an increasing number of downstream areas (tail ends) within contiguous irrigated perimeters (Croon and Risseeuw, 2005).

As soil salinity control is a key environmental factor, Annex 7 provides details on leaching requirements.

Shallow water table management

Where high water tables have relatively fresh water near the surface layers, as usually happens in humid areas, and soils are sufficiently pervious, water table levels can be managed to minimize drainage water quality problems before the water reaches a disposal point (water table management systems can also be used with less pervious clay soils but the risks and difficulties are greater). Specifically, water table management can contribute greatly towards the control of nutrient and pesticide pollution. A controlled high water table can also reduce the need for irrigation, thus saving freshwater for use elsewhere.

A shallow groundwater table can be maintained at different design depths if the water levels and the drainage discharge are controlled artificially, a technique known as controlled drainage. In dry periods, the drainage discharge is restricted and the groundwater level is kept close to the bottom of the rootzone. Thus, crops can benefit from groundwater supplied by capillary rise between the subsurface drains. In the wet season, the drainage system provides full discharge in order to maintain the water table low; thus, adequate aeration is achieved in the rootzone.

To prevent an excessive drawdown of the water table in the dry season, a constant supply of freshwater is required to replenish the soil water lost through crop evapotranspiration. Sometimes, a continuous supply by seepage maintains a sufficiently high water table if drainage is controlled. However, in the absence of natural seepage, the water table can be recharged by surface water conveyed through the main drainage system. To facilitate the water inflow into the soil through the subsurface drainage system (subirrigation), water levels in the collector system must be kept at a high constant design level. This is necessary in order to ensure sufficient hydraulic gradient to move water from the drains to the subsoil.

Water table management systems, as depicted in Figure 1, involve controlled drainage structures such as:

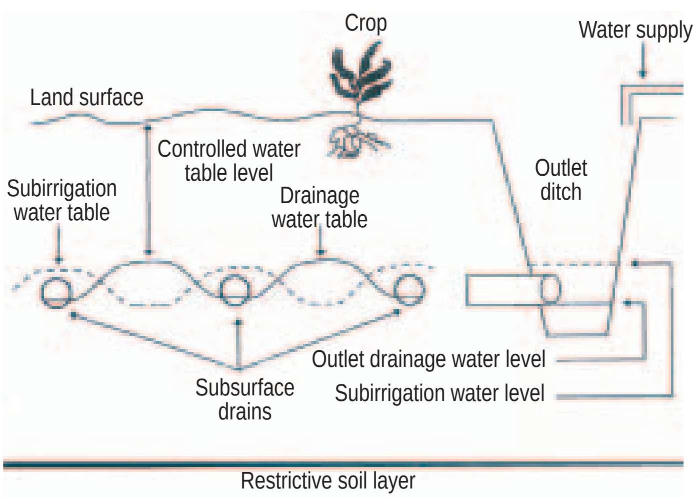
- plugs and elbows at the lateral outlet in singular drainage systems or at the collector outlet in composite drainage systems;
- water-level control weirs in the open main drainage system;
- automatic systems to control the water level in the pumping facilities.

Detailed designs for controlled drainage structures have been described by FAO (2005).

The quality of the groundwater is a key issue in water table management. Particularly in arid climates, maintaining a shallow groundwater table (with the exception of rice) can be very dangerous owing to the associated salinity hazard. If the groundwater is saline, salts transported by capillary rise will accumulate quickly in the rootzone. Therefore, such systems should not be considered unless the land is cropped permanently (e.g. the Nile Valley and Nile Delta). In areas where the accumulated salts can be leached by excess rainfall in the rainy season, e.g. in monsoon climates, or by surface irrigation after harvesting, care should be taken during the irrigation season to maintain soil salinity below the crop salt tolerance threshold.

Valuable research work has been developed on water table management. However, applying the results of research also requires judicious public water management capabilities. This task is frequently not easy and there are often conflicts, for example, where the available heads to infiltrate water are too small to be of practical use. On the other hand, if drainage is interrupted before the end of the rainy season and the drain base rises, by maintaining high water levels in the open drains, undesirable waterlogging can occur. Therefore, farmers sometimes prefer to pump the drainage water from the open drains during drought periods (as described below).

FIGURE 1
Water table management systems



Source: FAO/ICID, 1997.

TABLE 2
Quality of drainage water for use in irrigation

Quality	EC (dS/m)	Application
Very good	< 1	all crops
Good	1–2	most crops
Moderate	2–3	tolerant crops
Poor	3–6	tolerant crops, with ample leaching
Very poor	> 6	not recommended

Drainage water reuse

In the tail ends and fringes of an increasing number of irrigated areas in arid and semi-arid regions, where freshwater supplies are also required for other socio-economic developments, medium-quality water from open drainage channels can potentially be used for irrigation. Nowadays, drainage water is necessarily used for irrigation, either directly or after mixing with irrigation water of better quality, in order to compensate for decreasing freshwater flows.

The suitability of drainage waters for reuse depends greatly on the salts and pollutants carried by the water, on the crops to be grown and on irrigation practices. Table 2 gives an indication of the salt content of waters, measured as EC, in relation to their suitability for irrigated agriculture.

Waters with a low salinity can be reused for irrigation by pumping directly from the open drains. Where N compounds are present, they can be beneficial for crops as they form a valuable nutrient input resource and can result in reduction of fertilizer cost. However, excess nitrates prevent their use for other purposes, such as drinking-water for humans or livestock. This is particularly the case where the water is also polluted with agrochemicals such as pesticides and/or raw sewage water and process water spills of urban and industrial areas.

At higher salt content and/or pollution levels, drainage water may be blended with freshwater to provide an acceptable irrigation water quality. Another option is to use freshwater in periods when crops are salt sensitive and to use more saline drainage water when they are tolerant. As plants are generally relatively sensitive during germination and emergence, but become more tolerant during later stages of growth, it is imperative to keep salinity in the seed bed low at these early stages. However, problems of soil structure stability can occur if freshwater is applied after irrigation with drainage water with a high sodium content. The cycling option requires special infrastructure and considerable public water management efforts in order to realize it on a practical scale.

As the drainage water quality is reduced owing to increased salinity, more salt-tolerant crops must be used. FAO (2002b) provides data prepared by Maas and Grattan (1999) on the relative salt tolerance of various crops at emergence and during growth to maturity. To verify whether a water of a certain salinity can be used safely for a particular crop, an annual salt balance can be made to check that the salt in the soil profile does not accumulate or rise periodically above the acceptable salt level chosen for the crop.

Saline (not polluted) drainage waters can also be used to:

- irrigate halophytes where a proper system for salinity control is provided;
- maintain water levels in commercial fish ponds;
- secure temporarily minimum water levels in environmentally valuable brackish coastal end lakes;
- provide leaching for reclamation of salt-affected soils during the initial stage of the reclamation process.

Large volumes of drainage water, which are not suitable for the irrigation of dry-foot crops, may be used successfully for continuous refreshment of the standing water layer of rice grown on non-subsurface drained clay soils in the tail ends of the irrigation system of the Nile Delta, Egypt. Rice yields on lands with a topsoil salinity in the growing season of 3–5 dS/m increased by about 1 tonne/ha if frequent flushing of the standing water layer decreased the average salinity of the standing water layer with 1 dS/m (Egyptian–Dutch Panel for Land Drainage, 1977–79).

Reuse of drainage water inside a project area reduces the volume to be disposed of, but tends to concentrate salinity and pollutants although the total load of discharged pollutants may be slightly reduced. Ultimately, disposal of this reduced volume of drainage water outside the project area is inevitable.

In the case of domestic or industrial wastewater polluting agricultural drainage water, degradable and notably persistent organic pollution is a major problem, and water treatment is needed in order to achieve safe reuse. For irrigation of crops not used for direct consumption, treated wastewater can be used directly. For this purpose, treatment by conveying the water through constructed wetlands with reeds or rushes, or through stabilization ponds, is often sufficient. However, for most other purposes, especially for irrigation of vegetables, more sophisticated methods of treatment are required. This subject has been covered by FAO (1992a and 1997).

More details on drainage water reuse can be found in FAO (1985, 2002b) and FAO/ICID (1997).

Groundwater management

Pumping groundwater from tubewells can be an effective method for controlling waterlogging and salinity and it is widely used, e.g. in Pakistan. In freshwater areas, drainage wells are particularly valuable as the pumped water can be used for supplemental irrigation. Where the water pumped is slightly saline and cannot be blended with adequate quantities of fresh irrigation water, its reuse is seldom attractive. This is because the recycling of the saline water will gradually increase salinity levels in the soil and aquifer, thus causing the pumped water to become gradually more concentrated. Where the groundwater is salty, pumping a waste product is not economic unless there is a direct and safe disposal option.

Although often initially successful, tubewell pumping can result in upconing of saltwater from great depths, causing complex problems with irrigation and safe drainage water disposal. Therefore, it is important to consider the hydraulics of the deeper layers from which salts might be mobilized. A special construction of the wells (skimming wells) or double pumping of deep salty and shallow freshwater could prevent such upconing. However, sophisticated O&M practices are required. Moreover, with double pumping, the salty water discharge presents a new disposal problem. Systematic monitoring is a critical component for tracing water quality changes. Where such changes occur, adjustments in pumping rates are required in order to maintain a sustainability outlook. Therefore, subsurface horizontal pipe drainage should ultimately be considered in such areas as it will improve project sustainability and reduce the saltwater disposal problem considerably.

Chapter 7 provides technical details about vertical drainage.

Land retirement and dry drainage

In new irrigation developments, environmental side-effects from salt and trace-element mobilization might be largely avoided if areas with saline soils and soils rich in those elements are not irrigated. In existing projects, lands can be taken out of production (retired) for the same reasons, especially if substituted by newly irrigated land without such sustainability risks.

Dry drainage is a questionable concept involving the creation of sink areas of fallow land, e.g. uncultivated strips between cropped lands. These function as evaporation basins, drawing a flux of water and salt from adjacent irrigated crop fields. This already occurs spontaneously in salt-affected areas where abandoned agricultural fields act as salt sinks and in nature, for example in low-lying fringe lands of alluvial fans and deltaic areas situated in arid zones, such as in the Tunuyan fan in Mendoza, Argentina, the Garmsar area in the Islamic Republic of Iran, the Indus Plain in Pakistan, and Mesopotamia in Iraq.

A dry-drainage water disposal system has potential in areas where land is abundant and water is too limited to irrigate all the lands serviced by an irrigation system. It also has potential in cases where drainage outlets are not available (Gowing and Wyseure, 1992). However, dry drainage might have negative impacts on the surroundings if the barren salinized land causes saline dust storms or flooding with saltwater of the adjacent lands during a rare heavy rainstorm. Furthermore, deep aquifers may be affected by density currents. Therefore, retired and fallow lands should be managed properly with adequate salinity control by public or semi-public water management organizations, which can secure a sustainable form of land use, such as halophyte development for fuelwood production or nature protection.

Biological drainage

Biological drainage (biodrainage) is a concept based on plant evapotranspiration where tree belts are planted to remove excess soil water. Trees can absorb water from the unsaturated zone, thus diminishing deep percolation and subsequent recharge of the groundwater. They can also absorb water directly from the capillary fringe of the saturated zone. Thus, by lowering the groundwater level below their rootzone, lateral seepage can be intercepted or groundwater flow can be enhanced, and the water table of adjacent arable lands can be maintained at a depth suitable for crops. For biodrainage, highly evaporative plant species are recommended, such as some *Eucalyptus* varieties (Diwan, 1997), and in saline environments, salt-tolerant trees or shrubs, such as *Tamarix*.

In biodrainage systems, tree belts are planted systematically in different arrangements, according to the specific water control purposes:

- in areas with a deep water table to reduce percolation and recharge and thus prevent the rise of the water table;
- along a slope to intercept lateral seepage from highlands to lowlands;
- along irrigation canals to intercept lateral seepage due to leakage;
- in flat areas with a parallel layout similar to a subsurface pipe drainage system.

In dry lands where the water table is deep, tree plantations may be particularly useful to reduce recharge and be sustainable if the natural salt balance is not altered. In lands with subsoils rich in salts or harmful trace elements, the reduction in percolation can be especially useful by preventing their mobilization. However, as the introduction of non-native plants (especially of invasive species) may have a negative impact on the landscape, careful selection of the tree species is required.

Under certain conditions, tree belts can be effective at intercepting lateral seepage in sloping lands, and especially along irrigation canals, depending on the seepage amounts and the salt content of the canal water. However, the environmental impact on the landscape mentioned above should be considered. In addition to this, there may be risks of additional leakage from the canal as the hydraulic gradient increases owing to the lowering of the water table. Therefore, detailed benefit/cost analyses are recommended in order to compare the biodrainage option with other engineering alternatives, such as lining the canal or installing an interceptor drain.

Biodrainage systems may contribute to controlling waterlogging, while reducing the volume of drainage water to be disposed of by conventional drainage systems. However, they may not provide the long-term salinity control needed in arid climates, especially in irrigated lands. Salinity control is especially needed where the groundwater is shallow and saline, and salt buildup around the rootzone is inevitable. In this case, salts accumulated must be leached and conventional subsurface drainage systems may be required. As soil water salinity increases, plant evapotranspiration diminishes and the effect of biodrainage is reduced. In addition to these constraints, parallel tree layouts in flat irrigated lands to control shallow water tables may often be limited by the availability of land, which is generally scarce in irrigation schemes.

In summary, rootzone salinity control is critical in cropland areas served by such biological drainage systems. Careful design and management are essential as seepage rates, natural and artificial leaching, water availability, tree uptake efficiency, and maintenance can provide significant swings in effectiveness from season to season and year to year.

FAO/IPTRID (2002) have provided details on designing biodrainage systems and issues related to their implementation. This publication, which is based on several case studies, describes trends for future research and development (R&D) to address the current uncertainties mentioned above.

DRAINAGE OUTLET AND DISPOSAL TO AVOID OR MINIMIZE DOWNSTREAM EFFECTS

General remarks on outlet structures

The drainage water outlet is a critical point in any project, both from a viewpoint of downstream water quality and for the functioning of the project itself, because any flow stagnation in the conveyance channels causes problems upstream. Therefore, attempts must be made to prevent erosion and not locate outlet structures at points where heavy siltation may be expected. Moreover, the adverse downstream negative impacts on water supplies, fish, riparian habitats, wetlands and other valuable ecosystems must be minimized.

In some areas, particularly in humid and tropical zones, hydrological changes within the river system can also cause concerns related to the increase in flooding incidence or significant changes in base flows. Thus, river system hydrology related to some projects or groups of proposed projects requires hydrological evaluation.

This section considers only environmental aspects concerning the drainage outlet and disposal. Chapter 5 considers aspects of technical design, such as required levels and head differences.

In the planning phase, it is necessary to assess the volume and quality of the drainage effluent and the effects it will have on the downstream water systems. Considering the merits and drawbacks of alternative disposal routes, it must then be judged whether they are acceptable or not. On this basis, plans for remedial or mitigating measures must be formulated. Later in the design phase, the detailed environmental plan required for the selected disposal routes may be prepared.

Drainage water from the project must be disposed of in a safe way. Generally, it flows to the sea and tidal waters, to a lake or to a river.

Other possibilities for disposal of drainage water to minimize adverse impacts on downstream water resources are:

- evaporation ponds;
- constructed wetlands and related systems;
- the groundwater through recharge wells.

Each option has its advantages, drawbacks, problems and complications.

Disposal to the sea and tidal waters

In view of the large volume of seawater, the influence of a project on the sea or on marine environments cannot be large compared with other disposal sites. Although salts do not pollute the sea, other pollutants can alter coastal ecosystems, especially in estuaries. Discharge of drainage water directly into the sea is desirable from an environmental point of view, but it is usually restricted to coastal areas, unless long main drainage channels are dug, which serve also distant inland drainage catchment areas, e.g. the Left Bank and Right Bank Outflow Drain, Indus Plain, Pakistan, and the Bahr el Baqr Main Drain, Nile Delta, Egypt.

Discharge to the sea or associated waters is achieved:

- directly by gravity through outfall drains with a free outlet;

- through a flap gate or tidal sluice, discharging at low tide;
- through a pumping station.

Because free outlets (sometimes referred to as gravity outlets) may cause flooding of any upstream lands below the highest tides, this option is only possible where the drain base is significantly higher than the high-tide level, taking into account the head losses in the system. Even then, the heavier saltwater may creep considerable distances upstream along the bottom of the outflow channels. In addition to the intrusion of saline water, free outlets have other risks because they depend on the currents in the sea, are liable to be closed again by the moving sands along the coast (littoral drift), and can be eroded severely by wave action and currents along the coast. Protection against these forces is generally very expensive, where practically possible, as coastal jetties may be required and extensive dredging operations may be needed to maintain the free flow through the outlet.

Where water is discharged to the sea at levels that are below the highest high-water level, then there is a need for either pumping facilities or tidal structures in combination with "bossoms" with sufficient capacity to temporarily stock drainage water flows, below acceptable main drain levels whenever seawater levels impede free outflow. This is common in drainage catchment areas in flat coastal zones that may suffer seasonally from heavy rainfall, e.g. in monsoon climates.

Tidal sluices and flap gates can keep out high tides and provide discharge during the ebb tide. They are combined with dykes to keep out high outside-water levels. The tide sluices and gates close automatically at high water levels and they open at low tide. Therefore, the continuing upland drainage water flows need to be stored (e.g. in nearby topographic lows) below a pre-set level in order to avoid flooding cropped land during the period of sluice closure. However, although the flow of water is outward only, a discharging tidal sluice is still only partially able to prevent seawater intrusion by salty undercurrents flowing inland under freshwater in the channel.

Flap gates are placed in a small outlet sluice or in a culvert outlet through a dyke. They are commonly used for low discharges, whereas sluices are used where the discharge is higher. Both are efficient when the tides are strong as they need a certain tidal difference in order to work properly. The risk of restrictions below these structures due to sand movements noted for free outlets also applies to the proper operation of these structures.

Favourable locations are estuaries, where the tidal differences are usually greater than in the open sea, and coastal lakes, which are usually better protected from high sea extremes while offering suitable locations for the construction of "bossoms". Moreover, the risk that tidal sluices may be obstructed by moving sands along the coast is lower in estuaries. Where the coastal waters are shallow, high inland winds will cause storm surges, which may cause outlet stagnation for several days and sometimes even disastrous floods (the North Sea, the Gulf of Bengal and the Plate River).

The presence of periods where no discharge is possible requires storage possibilities above the sluice. These take the form of a large pond, large channels or a low-lying tract of land where flooding is allowed during critical periods (e.g. a storm surge in combination with a high design discharge).

Pumping stations are usually considered for coastal lands lying below a level where tidal sluices cannot provide proper relief and for inland topographic lows without a proper gravity outlet. They are used in combination with dykes that keep out the flooding waters. They require higher costs of investment and operation, but they do not allow saltwater intrusion into the outflow channel.

Disposal to a lake

In projects where drainage water is discharged to lakes and thus comes into contact with air for a considerable time, water will purify somewhat. Many lakes have an outlet that discharges into a river or into the sea. In these cases, the water quality consequences of a project may differ considerably from the case where drainage water is discharged to the sea or a river, especially where the lake is shallow or small. In order to protect the water level and the quality regime of the lake from excessive alterations, direct discharge by a waterway around the lake into the river or the sea may be an option.

Lakes without an outlet (tail-end lakes), often found in arid regions, behave in a different way. Here, the influences are far more pronounced. The delicate balance of the lake between inflow and evaporation is disturbed easily, and this has led in some cases to overflow or drying up with rapidly increasing water salinity (e.g. Qarun Lake, Egypt; the Aral Sea, Kazakhstan; and Manchchar Lake, Pakistan). In nearly all cases, the salinity of the lake increases more rapidly, and the larger amount of plant nutrients, silt and pollutants may affect its whole ecosystem unless compartmentalization provides relief. If this is not acceptable, alternative disposal sites, such as evaporation ponds, must be found.

The water level in lakes is rather constant, and special precautions against floods are seldom needed.

Disposal to a river

In many development schemes, disposal of drainage water is back into a natural river system directly or eventually through wetlands. In this case, the drainage water discharged from the project area is part of the water resource supply for downstream water users, and it will form a potential source of pollution of the river downstream of the discharge point.

A drainage outlet into a river alters its outflow regime (especially in small rivers). Salinity may affect downstream interests, and plant nutrients or pollutants may also exert their influence on ecosystems. Attention must be given to changes in river morphology caused by erosion and siltation. Large-scale constructions are sometimes undertaken in order to avoid pollution of a river with drainage outflow from very large projects (e.g. Right Bank and Left Bank Outflow Drain, Indus Plain, Pakistan) or urban and industrial developments (Bahr el Baqr Main Drain, Nile Delta, Egypt).

Most rivers have floods, sometimes seasonal, sometimes of shorter duration, but even so, they may last longer than a few days. If the outlet is open, the floodwaters will back up into the project area, which is usually not acceptable. Where it is protected by a sluice or flap gate, the normal upstream discharge may have the same effect. For simple cases, a computer program may provide some indication about these backwater effects (Chapter 5). One solution is an extended outlet channel with an outlet further downstream. In other cases, a pumping station is preferred. Much depends on the local circumstances, especially on the river gradient and the duration of high water levels blocking the drainage outflow.

Evaporation ponds

Evaporation ponds are sometimes used in arid climates for disposal of saline water in inland drainage projects where no other possibilities exist. Natural depressions are sometimes used, but artificial ponds are frequently constructed. Where possible, a number of cascading ponds are used to maintain a constant water level in order to achieve suitable environments for waterbirds.

In order to design evaporation ponds, the composition of the inflowing drainage water should be known and the inundated area must be calculated on the basis of the water balance needed to control the salt concentration in the pond. In this way, part of the cascading ponds can eventually be used to store water temporarily for reuse during dry periods, unless it needs to be disposed to nearby rivers when their discharges have increased sufficiently.

However, an evaporation pond disposal system has environmental drawbacks:

- It is inevitable that the ponds will become salt lakes, in which other pollutants will also concentrate.
- Toxicity levels can be reached by trace elements discharged to ponds where they are concentrated by evaporation.
- Some pollutants might be highly toxic to wildlife and could bio-accumulate through the aquatic food chain.
- Percolation water of poor quality might contaminate shallow groundwater and, later, deeper aquifers by density currents; then, impermeable pond liners must be used.
- Seepage from ponds might cause waterlogging and soil salinization in the adjacent areas.

In addition to these problems, there is a risk of saline dust storms if the pond falls dry when evaporation exceeds inflow.

In the absence of toxic elements, evaporation ponds may initially become interesting wetlands, although these are not sustainable. Where toxic elements are present, alternative or compensation habitats should be provided for waterfowl in order to minimize the negative impact on wildlife.

Therefore, evaporation ponds are only a disposal option where safe discharge to natural waterbodies is not possible. The technical, economic and environmental feasibility of this option depends on the water quality of the drainage water and on the topographical and geohydrological conditions of the available areas.

Details on the evaporation ponds constructed in the San Joaquin Valley, the United States of America, are described in FAO/ICID (1997).

Constructed wetlands and related systems

The drainage water volumes can be reduced and the water can be purified to some extent by guiding the drain flow through a series of artificial wetlands and related systems as discussed below.

Individual wetlands have different types of vegetation (planted or natural), such as grasses and reeds, and even trees, that will reduce the concentration of minor element constituents and provide sedimentation of soil particles and attached contaminants. The first wetland is planted with salt- and pollution-tolerant plants and the following ones have increasingly tolerant plants. These systems can be valuable for removing excess N, P, potassium (K), organic wastes and many other substances, especially those adsorbed to soil particles and those susceptible to biodegradation (FAO/ICID, 1997). For this purpose, water depth must be adapted to plant requirements and the water flow must be controlled in order to provide plants with enough time to take up pollutants. As the upstream basins evaporate a large part of the water, the low volume of concentrated drainage water remaining can be discharged to evaporation ponds or other safe disposal areas.

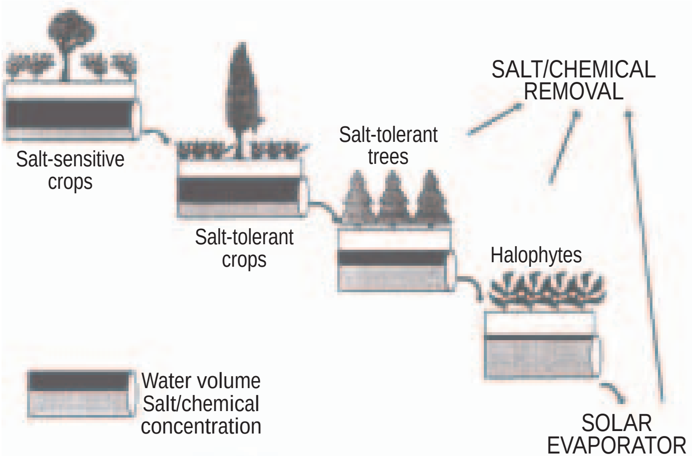
Artificial wetlands can sometimes be used to protect irrigated lands from wind-blown sands and desert encroachment, for wildlife, as shelter belts with salt-tolerant trees, etc. Another possibility is their use as fish ponds. However, where the fish is to be consumed, periodic monitoring for bio-accumulation of toxins is needed.

If the wetlands are not watertight and the aquifer is not confined, the underlying groundwater will be polluted by leakage. This may become a problem where the groundwater is valuable for purposes such as municipal water supply or irrigation. Therefore, wetlands require careful design (one that considers the topographic, soil and geohydrological characteristics of the site selected) and attentive management.

Where salinity is the main concern, related concepts that are being tested in a number of locations are saline agriculture/forestry systems and land application systems. In the former, drainage water is used to grow a series of increasingly salt-tolerant crops and other plants. With such systems (Figure 2), the drainage water is used progressively, without mixing with freshwater (or with minimal mixing during germination and early seeding stages), as follows:

1. salt-sensitive crops or fresh-water wetland where the water is still of reasonable quality;
2. salt-tolerant crops or wetland;
3. salt-tolerant trees or wetland;
4. halophytes, fish ponds, saline wetland, etc.;
5. solar evaporator.

FIGURE 2
Saline agriculture forestry systems



Source: FAO/ICID, 1997.

In all these compartments except the last one, sufficient natural or artificial drainage should be available in order to obtain a sustainable solution.

Land application systems for disposal of drainage water are often used to protect irrigated lands from wind damage and to control the advance of desert environments. Drainage water is applied to selected desert or to salt-tolerant tree belts planted for windbreaks. In large projects, internal tree belts can also be irrigated with the drainage water, but care must be taken to minimize impacts on adjacent vegetation that may not be salt tolerant. Where drainage water is used for irrigating even salt-tolerant trees, care must also be taken to ensure that no salt accumulation occurs over time. Drainage water application to desert land surrounding a project can help in protecting it from drifting sand and salt dust.

Groundwater recharge wells

Injection of drainage water into a deep confined aquifer through recharge wells is used in some areas to dispose of drainage effluent. The aquifer must be extensive and thick enough, and have a large storage coefficient and transmissivity in order to receive the injected water and it should not be overloaded. In addition, the drainage water must be free of silt and debris. Moreover, its quality should be compatible with the aquifer water in order to prevent the formation of precipitates, which might clog the well or reduce the hydraulic conductivity of the aquifer around the well.

The cost of this option is usually a major issue because transport facilities for the drainage effluent are required where the well is far from the drainage outlet. In addition, there are the costs for the construction, operation and maintenance of the well, and the monitoring costs.

There are other environmental drawbacks, such as pollution of valuable aquifers by waters of inferior quality. Therefore, if this option is considered, great care must be taken to determine the aquifer characteristics (hydraulic properties and water quality) in order to prevent potential negative effects.

Details on deep well injection and systems for continuous monitoring are described by FAO/ICID (1997).

OPTIONS FOR DRAINAGE WATER TREATMENT

Generally, drainage waters are chemically complex but their characteristics depend mainly on the origin of the drainage flow. Salinity is the main concern where subsurface drainage is the major component of the drainage effluent, especially in irrigated lands. However, in certain cases, trace elements may also be present in drainage waters.

Pollutants, such as pesticides and fertilizers, may be present where surface runoff from crop cultivated lands is relevant. Municipal or industrial wastewater is sometimes discharged into agricultural main drains. In these cases, organic compounds (and even heavy metals where factories are involved) may pollute the drainage water.

Adequate treatment of water to improve the quality of the drainage effluent is often expensive and requires careful O&M. Therefore, it is applied only where other disposal alternatives are not effective. However, processes to treat wastewater and water desalination can be used for drainage water where the treated water can be used in an economically viable way, e.g. as irrigation water.

Water can be purified by physical, chemical or biological methods, which are specifically appropriate for each type of pollutant. A wide variety of systems can be considered but thorough studies are necessary in order to design practical and effective systems. Therefore, before selecting the most adequate process, the specific characteristics of each case must be evaluated. FAO/ICID (1997) provides guidelines for the selection process, considering:

- the chemical composition of the drainage water to be treated and the range of volumes expected;
- the water quality criteria for the drainage effluent in accordance with the uses of the recipient waterbody and the regulatory constraints;
- the capital, operation and maintenance costs;
- the land requirements of each process and land availability.

FAO/ICID (1997) recommend carrying out pilot-scale tests before full-scale implementation, once the most promising treatment has been selected, after considering the advantages and disadvantages of the different processes considered in the evaluation.

Removal of organic compounds and nutrients

Stabilization ponds may be used to remove suspended soil particles, minor elements and organic pollutants where domestic wastewater is reaching outlet drains without proper treatment. Through aerobic processes, organic matter is decomposed by micro-organisms into carbon dioxide and water. Pond systems are normally installed in series and provide a slow natural purification. Careful control is necessary in order to ensure a proper residence time in each pond and to monitor effluents (Khouri, Kalbermattern and Bartone, 1994).

Biological treatment systems usually involve the use of bacteria in engineered reactor systems to effect the removal or change of constituents, such as organic compounds, trace elements and nutrients. Algae and wetland systems can also be used in some instances to replace reactors (FAO/ICID, 1997). Biological treatment is effective in removing organic matter, and also N and P in more sophisticated installations, but it is ineffective in reducing dissolved salts.

Water desalination

As salinity is a common problem in drainage waters, water desalination may be a treatment option to reduce the environmental impacts of disposal in inland drainage projects. There are several technically feasible technologies, such as thermal distillation (whereby water is converted into steam, which condenses into high quality water). Through membrane technologies, salts are separated from water either by means of an electric load application (electrodialysis) or by applying pressure to the saline water to force it to flow through a semi-permeable membrane that prevents most of the salts from passing through (reverse osmosis).

Reverse osmosis may be financially attractive to desalt drainage water for agricultural purposes, but only in special cases, such as in intensive horticulture with high-value crops, and where subsidies on capital costs are provided. However, water desalination may have certain environmental impacts caused by disposal of brines and residues from desalination, and emission of greenhouse gases.

More information on water desalination is available in an FAO publication (FAO, 2006).

Trace element treatments

Soluble selenium may be removed by reducing it to the insoluble elemental selenium form through anaerobic biological treatment. Artificial wetland cells have also been used in the United States of America at an experimental level to remove selenium biochemically (FAO, 2002b).

Adsorption of soluble pesticides

These compounds can be removed via adsorption onto granular activated carbon (FAO/ICID, 1997).

Removal of heavy metals

For non-degradable (persistent) substances, such as heavy metals, physical-chemical treatment of the drainage effluent is possible, such as precipitation by increasing the pH with lime or caustic soda to minimize solubility (FAO/ICID, 1997). However, the costs are usually prohibitive. Hence, application is restricted to special cases, such as the wastewater outlet of a factory. Sometimes such methods are even economically viable owing to the relatively high concentrations and value of the recovered materials (e.g. mercury, as used in chlorine production).

Chapter 3
Socio-economic and institutional aspects

SCOPE FOR DRAINAGE DEVELOPMENT

Drainage is a widely applied water management instrument in developed countries. However, in most developing countries, agricultural productivity and development have yet to reach the threshold level at which drainage becomes a viable investment. Other factors (e.g. lack of awareness among the farmers and policy-makers, institutional weaknesses, and non-conducive government policies) also account for part of the difference in development between developed and developing countries in the state of drainage (Smedema, 2002).

Government commitment to drainage development in many developing countries tends to be ad hoc and not sustained. Interest in drainage may be strong in a high rainfall year but declines when followed by a series of normal dry years. Public support will increase as levels of agricultural and rural development rise, and more farmers and rural communities start to appreciate the value of drainage and start to use their political strength to mobilize support for their cause. This process may be accelerated (through demonstration and education) by pro-actively raising awareness of the need for and impact of improved drainage (Smedema, 2002).

Owing to the lack of sufficient private-sector capacity, drainage development and management in almost all developing countries relies heavily on government initiative and support. However, for long-term sustainability, especially the assurance of proper maintenance, a strong involvement and commitment of farmers' communities is essential. The drainage development and management model that is generally most appropriate places responsibility for the main drainage system with the government or other public body, and that for the on-farm drainage systems with the farmers, with the role of the government limited mostly to creating the proper enabling conditions (policy/legal frameworks, incentives, research and technical assistance, etc.). However, widely tested successful institutional models for participatory drainage development and public/private partnerships have not yet been established (Smedema, 2002).

FARMER PARTICIPATION

The need for a participatory approach

In the planning and design of drainage systems, with their subsequent O&M routines and related activities (e.g. drainage water quality management), stakeholder participation is paramount to the successful long-term satisfactory O&M of the systems and acceptability by the users. Farmers are the primary stakeholders in most agricultural drainage projects. However, other interests such as environmental, community and road authorities are also important stakeholders, and their views should be considered. Design factors should also strive to carry through with the desires of the stakeholders and be in line with the capabilities of the farmers groups. Regardless of whether the project is a new development or concerns the rehabilitation of, modernization of or addition to an existing system, stakeholder participation is important for ensuring its success.

Drainage development activities have proved to be too expensive and too ineffective where carried out by governments or semi-government institutions with public budgets. Therefore, as farmers are the ultimate beneficiaries of drainage development and as their strong participation in local system O&M is paramount, it is necessary to involve them in the process as early as possible. Some participatory work should be initiated even before planning begins so that stakeholders are involved actively throughout the project development cycle.

Thus, in the planning stage, a general plan for the organization and financing of the O&M of the systems should be drawn up and the role farmers can play therein should be developed. It is difficult to organize a successful users association to operate and maintain a system if they have not had a sincere chance to shape the extent, type, scope and parameters of the facilities installed, or if they have not been involved in decisions regarding the financing of their recurrent and capital costs.

This approach means that farmers should be involved first in the identification of the need for new drainage developments or the rehabilitation and modernization of existing systems. Later, they can participate in the process of planning and design to fully operate and maintain the drainage systems, jointly with the irrigation systems in irrigated lands. They also should be presented with options and provided with discussions on the effect of each option on: production; diversification potential; environmental conditions; and social, economic and health issues. These activities are often undertaken in conjunction with other improvements and they normally require inputs from other professionals in order to be effective.

Through this process, farmers are informed about the options they have and are further involved in making the final decision. If any changes from the initial plan are required, farmers need to be kept informed.

Considerations for participatory planning and design

The system layout should consider the potential users groups and subgroups. A number of factors make it difficult for groups to work together, and sound thinking must go into the system layout in order to avoid among others:

- social conflict stemming from ethnic differences and other factors;
- physical development that would reduce the opportunity for communication between users dependent on one another;
- major differences in cropping systems;
- conflict potential related to infrastructure operation owing to different objectives.

Consideration of other related infrastructure, such as irrigation, flood mitigation works, and road systems, should also be part of the planning and design considerations. Successful operation of a drainage system requires facilities to be compatible, and participatory efforts should discuss the compatibility, strengths and weaknesses. It is not fair to expect users groups to operate and maintain a drainage system that creates problems for operation of irrigation, flood control and other interdependent facilities.

Designers must adopt a system to test the potential interaction scenarios for a variety of climate events. This is needed to assure users groups that operational conflicts will be minimal and to inform them of potential problems to watch for during unusual events. This type of information should all become a part of the O&M plans that should be worked out with users groups for the portion of the facilities that each group or subgroup is responsible for.

Participatory design procedures should be used in order to facilitate cost-effective O&M routines. Consultation with users and all stakeholders during the design process is critical to the future satisfaction of all organizations with the way the system is operated and maintained. Consultation means informing these groups and individuals as to what options they have and how the decision in selecting a particular option would affect their O&M cost, effectiveness, timing and complexity.

Drainage system modifications to facilitate participatory management

Monitoring and evaluation systems are critical to the successful management of drainage systems. Design of the M&E system is best done in conjunction with the designs for infrastructure modification or addition. All facets of the system that could affect sustainable production, the environment and social satisfaction should be considered. Monitoring plans should be developed during the design stage with the users groups that will operate the system, the supervising governmental organization, user group and/or other organizations that have stakes in the M&E of system performance. A system for prompt feedback to system operators and maintenance staff should be included. Timely modifications, adjustments and maintenance are critical for successful project operations.

Structure and facility modifications are sometimes necessary in order to enable improved participatory management. Controls and regulation equipment should be designed with the consent and ability of the users organization in mind. Similarly, measurement devices and monitoring equipment should consider the users organizations' information needs and operation abilities. The individuals operating the facilities should understand any automation facilities fully. Training to ensure that controls, devices and equipment are operated and maintained properly is just as important as having the facilities.

The establishing of beneficiaries' participated organizations in due time in conjunction with the transfer of O&M responsibilities for local existing irrigation and drainage systems is also important. Users organizations can be established at any time, but the earlier the better and preferably in combination with the planning, design and implementation of the water management system rehabilitation/improvement works. However, such organizations cannot take over existing systems effectively without user training and guidance during a start-up period. The financial obligations can be significant and these details must be worked out with potential users organizations prior to transferring the drainage facilities (and the responsibility for their O&M) to them. The training required to perform the functions being turned over to a users group is also critical. The individuals responsible for each aspect of operation or maintenance work must also know who they can call on as resource people if questions come up that they are not equipped to handle or if they need advice. The institutional capacity must be considered when work is being turned over to any organization for operation, maintenance, management and/or monitoring (FAO/ICID, 1997).

SOCIO-ECONOMIC INVESTIGATIONS

Procedures

Drainage design cannot be undertaken in a social vacuum. Consideration in the planning process must involve socio-economic impacts and costs associated with the design. Evaluations of impacts should be made when evaluating alternative designs and when considering changes in the design that are not in line with the original planning concepts. This is necessary as adjustments may have a significant impact on the economic evaluations that were conducted in order to justify the project at the planning stage. It can also have an effect on the social assessments made and on the predicted social impacts.

Agreements must be reached between the involved government organization or specific executing authority and its financiers, designers and the organized beneficiaries on the socio-economic procedures and considerations to be followed. These agreements should be reached early in the planning stage of project preparation, and the design organization must follow these agreements. Formalized contracts with sanctions for both sides of issues that relate to some of the most critical items may be considered as a part of these agreements.

Alternatives should be a part of the planning process. Similarly, alternative designs to accomplish established plans should be considered when preparing designs. It is important to follow established economic and social evaluation procedures. Guidance in economic evaluations has been provided by the Economic Development Institute of the World Bank (World Bank, 1991), the United States Bureau of Reclamation (USBR, 1984), the FAO Investment Centre (FAO, 1995), and the World Bank (Ochs and Bishay, 1992).

Costs and benefits

In most projects, a benefit/cost analysis culminating in a feasibility report is completed in the planning stage. Wherever possible, cost and benefits are quantified, with exception of certain socio-economic and environmental impacts (which are described qualitatively). In the design stage, cross-checks are made as adjustments are contemplated. Early in the planning and decision process, irreversible decisions are sometimes made that influence the cost and impacts of a project and may limit technical choices and alternatives in later stages of planning.

The costs associated with the implementation of drainage works are important and should reflect accurately investment costs and recurrent costs of O&M in relation to the quality or durability of the initial installation. Measures installed by cutting corners will inevitably increase the recurrent O&M costs of the facilities. Therefore, it is important to consider this balance during the planning and design stages of project development. It is also desirable to consider using cost optimization procedures or cost analysis for various aspects of the drainage work.

Costs are dynamic and change with time because of inflation and standard market factors. However, it is also necessary to consider changes in technology. Items such as trenchless drainpipe installation equipment, continuous rolls of plastic piping materials and geotextile envelope materials to protect drainpipe are examples of technology that, when used, have considerable potential for changing the construction costs of drainage systems.

The economic and financial benefits of agricultural drainage projects ("with project" case) relate primarily to increases in agricultural production, crop diversification opportunities and improved food security. These benefits need to be compared with the economic and financial benefits of the "without project" case, which in drainage projects to control waterlogging and soil salinity in arid and semi-arid regions often implies a significant decrease in yields. This decrease can be estimated with crop salt-tolerance data (FAO, 2002b). Thus, in these cases, an important enhancement of the incremental benefits is expected. Secondary benefits relate to items such as lower production costs, but may also include qualitative improvements, e.g. lower incidence of water-borne diseases and improved access to villages.

Quantifiable benefits as a whole should be based on current economic and financial farm prices for products sold or used locally as applied to the estimated production costs. The changes in costs of production, including for example O&M costs, can be deducted from the gross benefit in order to determine the net incremental benefit. The tests that may be used are the ratios of gross farm benefits to investment, and the net farm benefit to investment (Ochs and Bishay, 1992).

Economic efficiency is generally evaluated for a project by traditional economic measures such as the rate of return and the net present value. A conventional assessment should be made of the economic benefits of the directly productive elements of the project. The most common assessment method is to measure the project's internal economic rate of return. This may be defined as the discount rate for which the total present value of costs incurred during the life of the project is equal to the total present value of benefits accruing during the life of the project. In an investment project, costs are typically bunched at the beginning of the project, whereas benefits begin to accrue only after a lapse of time.

The application of a discount factor enables these costs and benefits to be compared on the basis of their present value. In order to calculate the internal economic return, it is necessary to construct a table showing the cost and benefit streams and the incremental income as they accrue each year during the life of the project. The cost streams used for economic analysis should include the capital costs of the project (including physical contingencies) as well as the incremental operating costs to farmers and any project authority or association. In an early phase of planning, maintenance costs are often estimated and based on a percentage of the capital costs, but care must be taken to ensure realism in these estimates. However, in subsequent stages, maintenance costs must be estimated in detail.

Where the technical life of a component in the project, such as a pumping plant, is less than the economic life of the project, provisions should be made for the cost of replacement (Ochs and Bishay, 1992). Where the internal rate of return is calculated, the replacement of different items at different times depending on the lifetime is included automatically.

Accurate data are necessary in order to provide proper evaluations. Sensitivity analyses should be included in the economic studies in order to be certain that the risks are understood when evaluations are presented to implementation or financing authorities as well as to the beneficiaries and stakeholders.

Life expectancy of drainage systems

The economic life of a land drainage system is an important factor in the economic evaluation for a project. Large drainage schemes, such as those built by the United States Bureau of Reclamation (USBR), predominately for salinity control, are based on a 100-year life expectancy, which could be called the technical life (USBR, 1984). Drainage systems in tropical and humid areas are sometimes built with a life expectancy as low as 25–30 years where they are primarily surface drainage systems. The anticipated actual technical life of a well-maintained pipe drainage system is usually 50–100 years.

However, the economic life of a project is more a consideration of the time at which a project will be renewed. The value of a project is greatest soon after construction and reduces steadily with time until the end of the economic life. The terminal value is generally considered zero. A project could have an economic life that coincides with its actual or expected design life, but future costs that occur after about 30 years are insignificant in economic terms. Thus, the economic life is generally taken to be 20–30 years (Smedema, Vlotman and Rycroft, 2004).

Cost recovery

Cost recovery considerations regarding investments in new drainage systems should not always be thought of in the same manner as for irrigation system installation. Major drainage facilities for a project area are normally considered a public good. This is because they benefit entire communities or regions and normally provide secondary jobs, resulting in poverty reduction in areas much larger than the actual project areas. Thus, they should receive more government financing. Therefore, the investment costs of public drainage facilities that protect numerous landholdings and provide a public good owing to control of salinization, waterlogging and flooding are not often required to have extensive cost recovery from the direct beneficiaries (Smedema and Ochs, 1998). The main drains are considered to be more of a regional benefit similar to public works such as roads, bridges, utilities and other infrastructure that provide incidental protection and secondary benefits. However, recurrent O&M costs of the public drainage system should be recovered as much as possible and in accordance with the level of benefit that accrues to individual project stakeholders.

Drainage facilities on private land and facilities for small groups of farmers are usually considered as private investments. The costs for constructing, operating and maintaining these smaller facilities should be recovered from the direct beneficiaries.

In areas with mature drainage systems that need to be repaired or rehabilitated, organizations of drainage boards or drainage districts become common. Drainage improvements in these areas are normally carried out using the normal cost-recovery procedures used for local irrigation project areas (Smedema and Ochs, 1998). Thus, the beneficiaries pay for improvements to their own systems and even the larger civil works that involve numerous landholdings and provide some incidental public good. In irrigated areas, where the irrigation district is normally the user organization responsible for drainage facilities, cost recovery is carried out for the drainage work and assessments are made for the beneficiaries that benefit directly from the work.

Operation and maintenance costs

O&M costs are recurrent and must be planned for prior to initiating any construction or improvement work. Planning and design should not be restricted to defining the technology of the systems. Consideration is also needed on how these systems are to be operated and maintained. System designs and institutional design of its O&M arrangements should be fully compatible.

An organization with the authority to perform the O&M that is required at the time it is required, with the financial capacity to carry the involved costs, and with the skills necessary to recognize the needs, should be in place when the construction is completed or segments of the work are transferred to it. The earlier this organization is established the better as it should be thoroughly familiar with the construction complications and the impact they may have on O&M.

Cash flow and accounting systems are critical to the successful performance of the local/regional organization responsible for O&M. Training of appropriate individuals is critical to carrying out this responsibility effectively. The cash flow and accounting systems must be transparent and understandable to all interested parties. Numerous appropriate cash flow and accounting systems are available, also in the rural communities throughout the world, and should be used.

Auditing of the financial and physical systems is also an important factor. The organization responsible must be certain this is done in an unbiased way. Financial audit refers to independent audit of the accounting work done and it should be undertaken at least annually. Physical M&E is also necessary to reflect the needs for special maintenance efforts, improvement work and operational changes. Conscientious monitoring and serious prompt evaluation of the monitoring data will help ensure system sustainability and operational success of the system in an efficient manner.

Social evaluations

The social impacts of drainage projects on the population can be both negative and positive. It is important to assess and evaluate, at least in qualitative terms, the probable social changes that will result from each alternative design. This is usually done on a project-wide basis and may include consideration of related infrastructure work, such as irrigation or flood control, that is being implemented at the same time.

Base studies may be needed to establish the existing social situation in an area. Social impact evaluations should include details necessary to establish potential benefits and losses. Sometimes these evaluations are done in conjunction with EIAs. *The ICID environmental check-list* (ICID, 1993) and *Environmental impact assessment for irrigation and drainage projects* (FAO/ODA, 1995) provide a good starting point.

Social scientists should be used to undertake these impact evaluations. They have valuable tools, such as matrix systems, that can help to ensure the accuracy and completeness of the evaluation. Social implications relate to the number of beneficiaries and losers from the drainage activities.

Important factors to consider include:

- impacts on living standards (lifestyle changes and cultural heritage);
- changes in the cost or value of land experienced by farmers;
- nutritional results anticipated;
- educational resultants;
- changes in the role of women;
- food security;
- poverty alleviation;
- health and disease issues (vectors, schistosomiasis, and water-borne);
- relocation or resettlement impacts (human migration potentials);
- services such as health and social services;
- equity issues (resulting social gains and losses caused by new facilities).

Chapter 4
Drainage studies and investigations

INTRODUCTION

Drainage studies and investigations may be carried out for different purposes and at different scales. The type of information required and its level of detail depend on the specific objectives of the phase of the project, the size of the area and the complexity of the problem. Other factors, such as the time and funds available to finance the study, are also considered. Two major groups may be considered: (i) master plans, at national, regional or river basin level; and (ii) drainage studies in a specific project area.

Drainage master plans are necessary in order to: (i) identify large areas with waterlogging and salinity problems; (ii) formulate priorities from a policy, technical, environmental and socio-economic point of view for developing specific drainage projects; and (iii) strengthen involved water management institutions and executive farmers organizations where required. The drainage plan is frequently a component of a regional development plan. Then, integration of the different components of the land and water project is essential. Sometimes, for large areas, such a regional master plan precedes more detailed studies, such as those listed below.

In a prioritized area, which should preferably be a hydrological catchment, studies and investigations are required for the planning and design of an appropriate drainage system. For large-scale drainage projects, three study levels are usually needed: (i) identification; (ii) feasibility; and (iii) detailed design.

The identification study includes a reconnaissance of the whole area in order to: (i) globally identify waterlogged or salt-affected lands; (ii) assess the natural drainage capacity of these problem areas; (iii) consider possible causes of the drainage problems and outline technical solutions; and (iv) give a first estimate of the benefits and costs, and of the socio-economic and environmental effects of the project.

The feasibility study refers to a properly identified potential drainage project area and elaborates on the selection of the most appropriate solution for the drainage problem in technical, environmental, socio-economic and financial terms.

The design study includes detailed investigations for areas with positive feasibility results, and aims at the final design of the drainage system and associated engineering works. This study consists mainly of soil investigations to determine the spatial variation of soil hydraulic characteristics and hydrological observations to characterize carefully the surface and groundwater flows.

In small-scale drainage projects, the feasibility and design phases can be combined into a single study.

Investigation and study costs should be minimized, especially where doubts exist whether the intended drainage project will be feasible, but also during the design study phase. In addition to financial reasons, organizational and topography reasons mean that it is not possible to provide each and every patch of land with the theoretically best drainage provisions. Therefore, depending on the level of sophistication of equipment, available materials, and contractors' experience and organization during the project implementation phase, the minimal size of subsurface drainage units with one specific drain depth and drain spacing may reach 200 ha in some developing countries. For similar reasons, public main drains cannot always follow an ideal alignment.

Consequently, field information should not be collected to reach an accuracy that exceeds the requirements of the implementation practices.

In addition to the above studies focused on new drainage projects, investigations are frequently made in already drained lands. Generally, these observations are made in order to: (i) check or establish drainage criteria; (ii) assess the performance of existing drainage systems; and (iii) identify poorly functioning features that may jeopardize the correct technical operation of the drainage system. Investigations are also needed to identify negative environmental impacts of the system (to formulate correction measures later), and to assess the socio-economic return of drainage investments. Observations are also made to modernize drainage systems, for example, by designing structures for controlled drainage. Moreover, investigations in drained lands can also provide useful information for drainage design as they permit checking of the soil hydraulic characteristics and criteria used.

FAO has already addressed the issue of investigations in drained lands (FAO, 1976). As FAO is planning a new Irrigation and Drainage Paper on evaluation of the performance of land drainage systems, this chapter describes only those investigations in drained lands useful to deriving drainage design factors and soil hydraulic characteristics for designing new drainage systems.

Drainage studies at their different scales generally require information commensurate with different grades of detail according to the level of the study. This information can be obtained from:

- aerial photographs and satellite images;
- topographic maps;
- land-use maps;
- climate data;
- soil investigations;
- hydrological studies;
- crop, land and water management data;
- EIAs;
- socio-economic evaluations;
- institutional considerations related to the implementation and subsequent recurrent O&M requirements of the drainage project.

Table 3 summarizes the characteristics of drainage studies and the specific information needed for each level of detail. Table 3 shows that the intensity of the study and the map scale increase from master plan to design, but that the area studied decreases accordingly. Depending on the level of readily available information, the costs of these studies average less than 3 percent of the costs of implementing the system where no expensive specific geohydrological studies are required. The benefits of a sound planning and design process are considerably greater.

The following sections describe the sequence of studies. Reference is made to the drainage investigation guidelines compiled in the FAO Irrigation and Drainage Paper *Drainage design factors* (FAO, 1980), which are still largely valid. Additional information on this subject is available in studies by the Van Aart and Van Alphen (1994), the USBR (1984), Madramootoo (1999), and Smedema, Vlotman and Rycroft (2004).

DRAINAGE MASTER PLANS AT NATIONAL OR RIVER BASIN LEVEL

National and regional drainage master plans are normally combined with other aspects of national and regional development, such as irrigation, flood mitigation, and land-use planning. However, they are sometimes specific in order to solve a severe problem, e.g. soil salinity.

After a brief description of the country or region (general information), especially focused on the economic aspects of rainfed and irrigated agriculture, the first phase

TABLE 3
Components of drainage studies

Type of study	Objectives of the drainage study	Maps (based on available topographical background)		Climate	Soils & land use	Hydrology	Crop, land & water management	Environment	Socio-economic analysis
		Content	Scale						
Master plans	Identification of drainage needs at national or river basin level; formulation of priorities	Agro-ecological areas	1:200 000 1:100 000	Type of climate; average data on precipitation & evapotranspiration	Available information on soil associations & land use	Available information on general groundwater & surface water flow	Available general information on agricultural systems & on irrigation & drainage development	Available information on environmental problems related to water management	Available information on drainage costs, returns & cost recovery; institutional development concerning land drainage
Identification	Identification of problem areas; assessment of drainage needs; pre-feasibility considerations	Land classification; mapping drainage subclasses	1:50 000 1:25 000	Climate characteristics: average precipitation & evapotranspiration	Soil reconnaissance study & land use; based on satellite image or aerial photographs interpretation	Reconnaissance of surface & groundwater hydrology; irrigation practices	Estimation of drainage coefficients & crop water management requirements	Assessment of environmental problems related to drainage & need of EIA	Estimation of benefits & costs of drainage developments in problem areas; information on water users organizations & involved government organizations
Feasibility study	Outline of main drainage systems; formulation of typology of field drainage systems; technical, socio-economic & environmental feasibility aspects	Soil & groundwater maps; layout of the main drainage system	1:10 000 1:5 000	Analysis of extreme rainfall & extreme droughts	Determination of soil hydrological & soil salinity characteristics; based on interpretation of time series of satellite images or aerial photographs	Characterization of groundwater flow; analysis of extreme discharges; availability & quality of irrigation water	First estimate of specific design discharges for channels & drains; critical time of ponding, waterlogging & soil salinity level in relation with crops	EIA of the drainage development	Calculation of the benefit/cost ratio & the internal rate of return; description of social impacts & gender issues
Design	Drainage systems design	Drainage systems layout	1:5 000 1:2 000	Analysis of extreme rainfall (results of the planning phase)	Average values of transmissivity of major soil units	Detailed groundwater flow (based on results of the planning phase)	Drainage design coefficients; required drain & groundwater depth	Detailed EIA of the drainage works & formulation of the mitigation plan	Budget of the designed drainage systems & auxiliary structures; assessment of O&M costs in relation with type of responsible organization
Investigations in drained lands	Check design drainage criteria & soil hydraulic characteristics	Layout of the observed drainage system	1:2 000 1:1 000	Current rainfall data	Soil hydraulic characteristics; soil salinity	Groundwater observations; drainage flows	Actual irrigation practice	Drainage water quality M&E	Ex-post determination of crop yields, crop schedule shifts & financial returns

of a national or regional master plan is to define large-scale agro-ecological areas. These areas are distinguished on a physiographic basis, considering climate, soils, hydrological conditions, and land-use and agricultural systems. In each area, current drainage development and drainage needs must be identified, considering separately: excess rainfall, irrigation losses, water shortages and soil salinity control. Finally, priorities concerning the various areas are indicated.

Generally, existing data form the basis of master plans. Therefore, they are formulated using the available information on: climate, soils, hydrology, land use, irrigation and drainage development, environmental problems and socio-economic aspects related to water management, and institutional development concerning agricultural drainage, with some additional information if needed. Checking on consistency and consolidation of the information is important and one of the main purposes of such plans.

Where funds are available for regional studies or at river basin level, a general reconnaissance is sometimes made in order to provide basic information on existing land use and to identify problem agro-ecological units. Remote-sensing techniques, with limited field observation support, are usually applied to locate the major soil associations and land systems of the area. Most field observations are visual. Information from geological maps is also valuable. The density of these observations is very low, about one site per 1 000–5 000 ha, according to the size of the area. Thus, the scale of the final maps may vary from 1:100 000 to 1:200 000. This basic information may be compiled into a geographical information system (GIS) by means of thematic maps that can be updated periodically. Details on the use of GIS for the planning and design of drainage systems are available in Chieng (1999).

The physical information is complemented with information on public-sector development as concerns drainage in education, research, training, project development and budgets. In this way, by comparing the drainage needs in each large-scale agro-ecological unit with the current drainage development, the master plan can be formulated and priorities defined. The master plan must include recommendations for planning, design and construction of new drainage systems and for capacity development, considering drainage as a component of integrated water resources management (IWRM). This plan may serve as a basis for the more detailed studies to follow.

IDENTIFICATION OF PROBLEM AREAS

In a specific zone, the first phase of a new drainage project is to identify the problem areas and to characterize the soil salinity and/or waterlogging problems, i.e. excess surface water, overirrigation and existence of perched water tables or shallow groundwater tables. In addition to this, the general characteristics of the existing drainage facilities must be described. Where possible, historical information and views of stakeholders and future beneficiaries should be sought and taken into consideration. In this way, the new surface and subsurface drainage needs and their related costs can be estimated. The drainage solutions identified for the affected areas are later developed in detail in the next planning phase.

Climate information

In the identification phase, the main features of the required climate data are the atmospheric components of the water balance: precipitation, evaporation and, in some areas, snowmelt. Temperature correlates roughly with potential evaporation, but there are better estimation methods for this quantity where long-term measurements are not available or are of doubtful quality. Once the potential evaporation is known, crop evapotranspiration, which is an essential component of the water balance, can be determined. Effective rainfall is also essential to formulating the salt balance.

For identification purposes, available average values of the above-mentioned climate parameters, their seasonal distribution and their spatial variability are sufficient.

Methods to determine evaporation, crop evapotranspiration, and evaporation of fallow land have been described by FAO (1977a and 1998). FAO (1974) has also provided guidelines to determine effective rainfall and the annual, seasonal and monthly water balances.

Landforms, soils and land use

Land-use and natural vegetation information, inferred from satellite images, aerial photographs and field visits, can give important clues for identifying waterlogged and salinized areas as some visible crop reactions (e.g. poor growth, patchy pattern, rolled leaves and pale colour) are indicative of soil salinity. Land-use information is also necessary to estimate the benefits obtainable if the drainage problems of the project area are solved. For identification purposes, available land-use data are generally used. Some countries have this kind of information on maps at scale 1:50 000. The main constraint is updating/validating this information. For this purpose, farmer participation is indispensable. Remote sensing (RS) supported by fieldwork is a useful tool to produce new maps or to update existing ones.

However, in order to differentiate those areas affected by waterlogging and salinity from salt-free lands with adequate natural drainage, soil mapping focused on assessing land drainability is frequently required. Existing soil maps, where available, are seldom appropriate for drainage purposes. This is mainly because conventional soil classifications are generally based on soil information restricted to some 150 cm, but layers below this standard depth are commonly relevant for drainage purposes. Therefore, observation depths should exceed those for conventional soil surveys and be at least 2–3 m. In addition, knowledge of the substrate down to the impervious layer is required. This can be derived from deep observations or from geological maps.

The physiographic approach for mapping soils (Veenenbos, 1972) is particularly suitable to identifying problem areas. This is because a close relationship exists between drainage conditions, soil salinity and geomorphology. The "Russian School" has stressed the relationship between geology, geomorphology, waterlogging and salinity (FAO/UNESCO, 1973). This approach has also been described by Bardají (1998) and applied successfully in Spain. Following this approach, once the landforms of the studied area have been mapped through a photo-interpretation study, field observations are made in order to find the causes of the observed phenomena. Generally, waterlogging and salinity occur in the lowest places and in locations where upward seepage occurs.

Aerial photographs (with scales from 1:20 000 to 1:40 000) and satellite data are of great help because the waterlogged and saline areas are often much better identified than on topographic maps. Moreover, photographs are usually more recent than existing maps, and satellite data are even more up-to-date, while time series of satellite images may reveal trends in problem development.

In this phase, reconnaissance studies at scales from 1:50 000 to 1:25 000, depending on the size of the project area, are sufficient. The observation densities depend on the intensity of the survey, but they can vary from one to four observations per 100 ha, localized mainly in the problem areas.

The main purpose of these observations is to determine: (i) the relative position of the mapping unit in the landscape; (ii) the salinity conditions; and (iii) those soil characteristics required to assess land drainability, such as the infiltration rate, the internal drainage and the transmissivity of the subsoil down to the impervious layer. These soil characteristics, in particular the hydraulic conductivity, are often estimated in this phase from two basic soil physical properties: soil texture and structure (Annex 1). Other features that can be observed in the soil surface, such as wet spots, white spots and puffs, and in the soil profile, such as mottling and the distribution of soil moisture, are useful to understanding land drainability. Average data of these soil characteristics should be obtained for each mapping unit.

Hydrology and irrigation practices

In the identification phase, the general pattern of the surface and groundwater flow should be determined. This can be obtained from the available hydrological information of the project area. It is sometimes necessary to prepare an inventory of data from the existing observation points, such as groundwater wells and permanent gauging stations of the watercourses network. Other field observations on natural vegetation and in particular on channels (presence of small mud volcanoes and failing side slopes) can indicate areas affected by seepage. About one observation per 100–200 ha is sufficient at this stage.

A description is usually required of the public irrigation water supply management and the field irrigation practices (where drainage water excesses are also thought to stem from overirrigation). In this respect, field visits to tail ends of the lower irrigation command areas may be revealing.

Information on the quality of the irrigation water and on salinity of the groundwater is also desirable at this phase.

With the hydrological, soil and irrigation-practice information, the areas affected by waterlogging and salinity can be mapped and the general characteristics of surface water and groundwater flow can be understood. For example, Figure 3 shows the soil map of an irrigation district in northeast Spain. This map identifies clearly the areas with excess water.

This map shows: the lowest part of an alluvial plain with fluvial terraces (T), the actual floodplain with levees (B), backswamps (D) with intermediate transitions (t), and an estuarine plain (LL). Additional landforms (R) have not been subdivided in this map because they are well drained. The main characteristics of the above mapping units are described in the map legend. The areas affected by waterlogging and salinity are restricted to the backswamps and transitions of the floodplain and the estuarine plain. Therefore, the planning of new drainage systems will focus only on these areas.

Soil salinity

As soil salinity is a major problem in the irrigated areas of the arid and semi-arid regions, mapping of soil salinity is usually an essential component of identification studies. Areas where soils with severe salinity predominate should be set aside and perhaps not be included in the reclamation project in order to minimize the mobilization of salts and reduce the negative impact on drainage water quality.

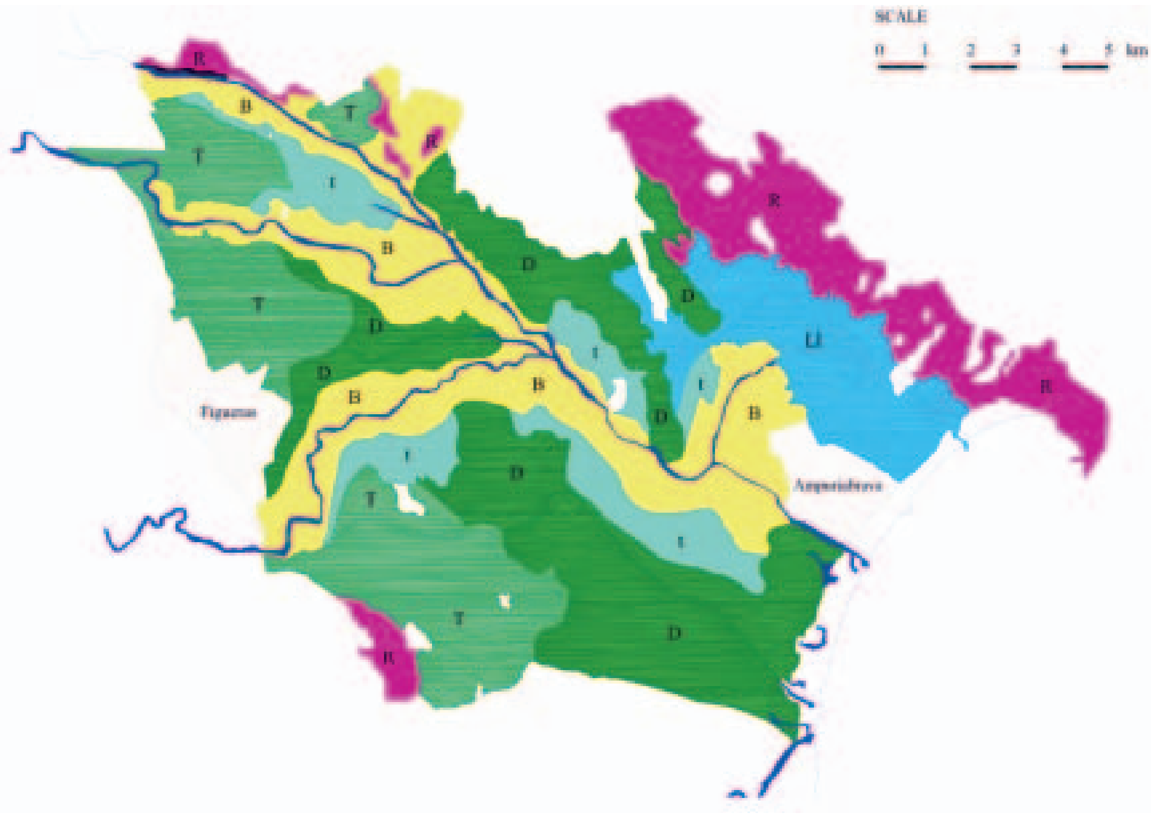
Soil salinity has usually been determined by laboratory analysis from soil samples. Generally, the electric conductivity (EC_e), anions, cations and pH are determined in the extract of the saturated paste. This extract is the reference soil water extract for measuring soil salinity, and the salt tolerance of crops is always expressed in terms of EC_e.

As determining the EC_e requires laboratory equipment for the preparation of saturated extracts, quick yet reliable field estimations of soil salinity can also be derived from EC measurements in 1:2 soil solutions ($EC_{1:2}$) or more frequently in 1:5 ($EC_{1:5}$). A 1:5 soil solution can be obtained after filtering the liquid resulting after mixing 100 g of dry soil with 500 g (≈ 500 ml) of water. The relationships between the field data expressed as $EC_{1:2}$ or $EC_{1:5}$ and the reference values expressed as EC_e are very site specific because they depend on soil texture and the presence of slightly soluble salts. Therefore, for accurate determinations, the recommendation is to obtain local correlations.

Where the salinity problem is extensive, costs can be reduced by direct field measurements of the EC. This can be done by means of a four-electrode probe, which can be used for measurements of the EC at depth intervals of about 25 cm down to a depth of 1 m. To obtain reference EC_e values from field measurements, calibration of the equipment is required. The EC probe can be useful for monitoring measurements made with similar moisture content as the EC value determined with the probe depends on, in addition to the soil salinity, the soil moisture content.

FIGURE 3

Simplified soil map of an irrigation district identifying areas affected by waterlogging and salinity



Symbol	Landform	Soil phase	Soil properties				Topo-graphy	Drainage				
			Effective depth (m)	Surface texture (m)	Subsoil	Salinity	Slope (%)	Soil drainage	Ground water level (m)	Impervious layer (m)	Hydraulic conductivity (m/d)	Natural drainage
B	Floodplain levees	Well-drained alluvial soils	> 0.9	Loam, sand-loam	Sand	Free	< 2	Moderately high	1.5–2.0	> 3.0	2–3	Sufficient
D	Floodplain backswamps	Poorly drained alluvial soils	> 0.9	Loam, clay-loam	Clay	Moderate	< 2	Moderate	0.5–1.0	1.0–2.0	1–2	Insufficient
t	Floodplain transitions	Moderate poorly drained alluvial soils	> 0.9	Loam, sand-loam	Clay	Slight	< 2	Moderate	1.0–1.5	2.0–3.0	2–3	Insufficient
T	Fluvial terraces	Well-drained alluvial soils	0.6 - 0.9	Sand-loam, loam-sand	Gravel	Free	< 2	Moderately high	> 2.0	> 2.5	-	Sufficient
Ll	Estuarine plain	Poorly drained alluvial soils	> 0.9	Silt, silt-clay-loam	Clay	High	< 2	Slow	< 1.0	2.0 - 2.5	3–5	Insufficient
R	Miscellaneous landforms with natural drainage	Well-drained colluvial and residual soils	Variable	Variable	Variable	Free	> 2 Variable	Moderately high	> 3.0	Variable	Medium high	Sufficient

Sources: Adapted from IRYDA, 1989a, and Martínez Beltrán, 1993.

Quick measurements of the soil salinity without the need to bore holes can be done with an electromagnetic sensor, which is easily managed on the soil ground. To determine the EC values at different soil depths, the sensor is situated at different heights above the ground, in the horizontal position for measurements to 1 m depth and in the vertical position for 2 m. Calibration is also required to relate the field values to the EC_e.

Additional details about direct field EC meters have been described by FAO (1999). As these sophisticated methods are expensive, their use is restricted to soil salinity measurements at the field level in large-scale projects.

RS is being used for soil salinity mapping in several countries with different approaches. For example, the Mexican Institute for Water Technology (IMTA) has applied RS techniques in several irrigation districts in northwest Mexico (Pulido Madrigal *et al*., 1999, 2000 and 2003b) by applying methods developed by Wiegand (2000). Soil salinity was determined indirectly through indicator crops, such as wheat, cotton and maize. The first step was to correlate global soil salinity (EC_g) with crop yields by field measurements and later crop yields with the spectrum values of the following bands of Landsat images: TM_2 (green, from 0.51 to 0.56 µm), TM_3 (red, from 0.62 to 0.76 µm) and TM_4 (infrared, from 0.7 to 1.5 µm). For example, the regression equation obtained in the Rio Fuerte Irrigation District through the spring–summer maize was: $EC_g = 5.1863 - 0.1896TM_2 + 0.2835TM_3 - 0.0724TM_4$ with a correlation coefficient of 0.85. A lower coefficient of 0.67 was obtained for cotton, which is more salt tolerant that maize and covers the ground less. Probably, better correlation would have been obtained if RS had been applied jointly with the physiographic approach described above and the regression analysis had been made independently for each type of soil. In these districts in Mexico, crop growth under full irrigation is affected mainly by soil salinity. This because other limiting factors, such as water management and agricultural practices, are less relevant when water is fully available. Detailed information on this methodology is available in Pulido Madrigal *et al*. (2003a).

Soil salinity mapping in the Rio Mayo Irrigation District, Sonora, Mexico, over an area of about 100 000 ha, with an electromagnetic sensor and RS was completed in about six months. Two satellite images were used at an average cost of US$0.35/ha. Figure 4 shows the salinity map of this irrigation district.

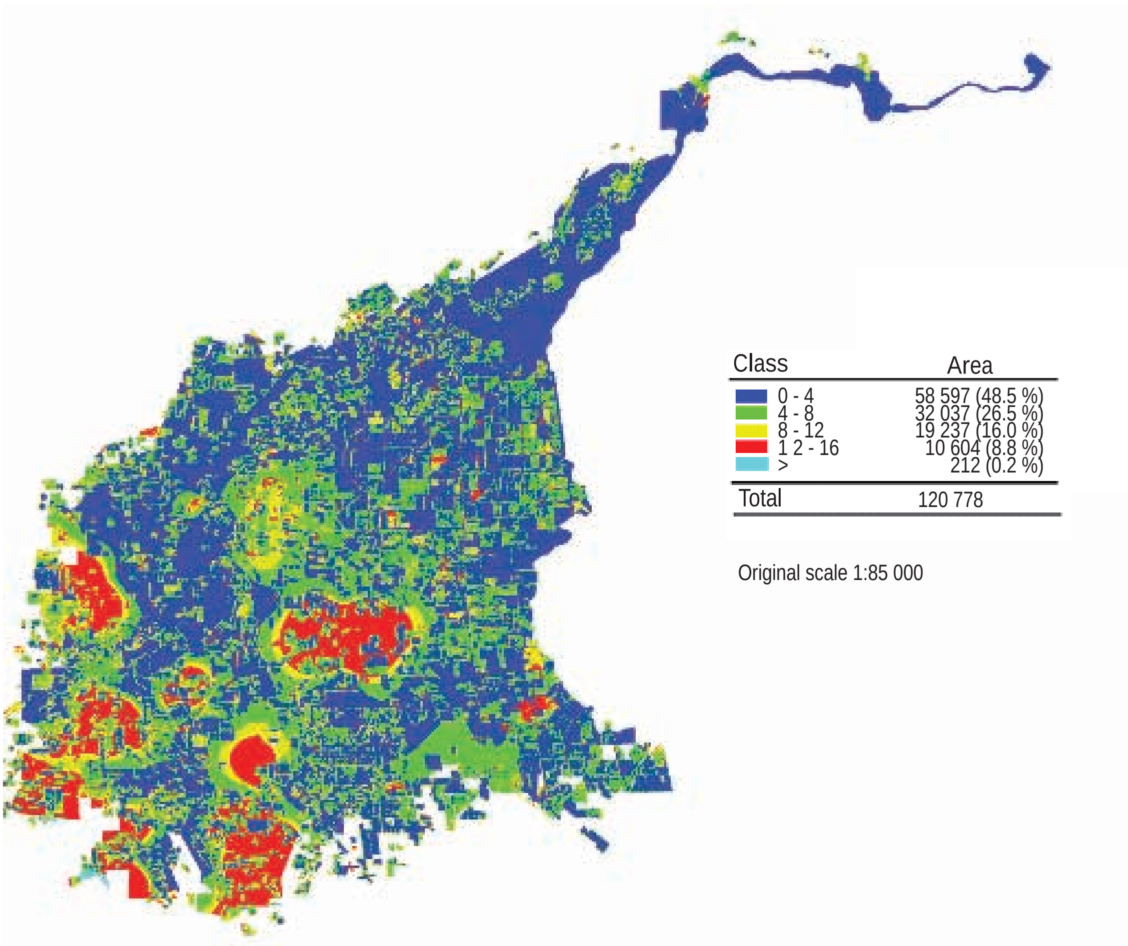
For areas where other factors in addition to soil salinity affect crop yield, Bastiaanssen *et al*. (1998a and 1998b) developed a biophysical approach called the Surface Energy Balance Algorithm for Land (SEBAL). This approach is based on the determination of the spatial distribution of the leaf area index (LAI). This reflects the agronomic practices and the variations of stomatal resistance in crops, which are related to the retention of moisture owing to osmotic potential in the rootzone (which depends on the soil salinity). SEBAL requires visible, near-infrared and thermal infrared satellite measurements in order to compute the bulk surface resistance for a cropped surface and the LAI. This approach has been applied in several irrigated areas, e.g. in Uzbekistan (FAO/IPTRID, 2005), and in other countries as Egypt and the Syrian Arab Republic.

Research has been done on developing direct methods of measuring soil salinity through spectrum analysis, but they have yet to be applied in large-scale irrigation projects.

Drainage development in the studied area

In many areas, the existing drainage systems consist of natural watercourses and main drains only. In such cases, a general description of the systems will suffice, highlighting the responsibilities and budgets of the involved public water management organization and the observed maintenance practices and conditions. However, where additional field drainage systems exist, it is necessary to describe the general technical characteristics of the existing surface drainage system (e.g. type, spacing between surface drains, and slopes) and of the subsurface drainage system (e.g. such as drain depth, spacing and drainage materials). In addition, information is needed on the availability of drainage materials and machinery, drainage costs, economic returns and

FIGURE 4
Soil salinity map of the Rio Mayo Irrigation District obtained by RS and measurements with electromagnetic sensor



Source: Adapted from Pulido Madrigal *et al*., 2003a.

cost recovery. Information on implemented drainage tenders and the availability of private contractors for production of materials and implementation of drainage works is also desirable.

The environmental impacts of the existing drainage systems must be identified at this level as must the need for an EIA to be carried out in the following planning phase. The existence of water users organizations and their participation in existing drainage projects is one of the institutional aspects to be considered at this level.

FEASIBILITY STUDY FOR PLANNING NEW DRAINAGE PROJECTS

Once the problem areas have been identified, the feasibility of reclamation must be investigated and the new public drainage systems required in order to reclaim the affected areas can be planned. The feasibility study involves technical, environmental and socio-economic aspects to see whether the project is viable. Where this is the case, it is followed by designing the main drainage system or the rehabilitation, renewal of extension of the existing one, and by defining the characteristics of the individual field drainage systems.

The feasibility study of a drainage project is based on climate data, soil and hydrological studies, and additional information on land use and crops, natural vegetation, and, in irrigated lands, on irrigation water management (including supply and quality aspects). Where some form of main drainage system exists, evaluation of its performance is required in order to assess the need for improvement. In addition, the environmental implications of the proposed systems and the socio-economic feasibility of the estimated investments ("with project" case) must be assessed against the situation that would develop were no project ("without project" case) implemented.

Topography and land use

In the feasibility stage, existing topographic maps (scale from 1:10 000 to 1:5 000) that have significant information on existing irrigation and drainage, rural roads and other infrastructure are used. In flat areas, where most drainage projects are developed, the contour interval (which is the difference in elevation between contour lines on the final map), should be at most 50 cm, but a contour interval of 20–25 cm is desirable as this will be required at the design stage.

In this phase, land-use data provided by existing maps, usually at scale 1:50 000, are often not sufficient or they are outdated. Therefore, additional field information is required. Farmers' information on crops and the cropping calendar, together with information on prevailing irrigation water supplies and availability, is essential at this stage.

Climate data

The general information on climate and water balances is already known from the identification stage. For feasibility, more detailed rainfall information is required. The extreme values to be expected once in 2–10 years (for agriculture) or once in 100–200 years (for inhabited places) or human safety (1 000 years or more) should be known.

Annex 2 includes rainfall analyses for establishing design discharges by applying Gumbel's method. Annex 23 provides the computer program for these calculations. More detailed information needed to calculate rainfall intensity is provided in Oosterbaan (1994). However, recent climate changes are tending to increase such extremes in many areas, and care is needed with such methods.

Soil information

The areas affected by waterlogging and salinity can be defined more precisely than in the identification phase through a more detailed soil mapping at scale 1:10 000, with 5–10 observations per 100 ha. In this phase, the cost-saving physiographic approach can also be applied to map the main landforms if this has not been done already or if more detail is required.

In each landform, the soil characteristics required for drainage planning and further design are measured, especially:

- the permeability or hydraulic conductivity (K) of the saturated soil;
- the infiltration rate (I_{nf});
- the internal drainage of the soil and the characteristics of soil hardpans or other layers impeding water percolation where present;
- the transmissivity (KD) (being the product of K and the layer thickness D) of the layers down to any impervious barrier;
- the drainable pore space (μ) of the layer where the groundwater level oscillates;
- sometimes, the hydraulic resistance (c) of a semi-pervious layer must be known, where vertically upward seepage from a semi-confined aquifer towards the rootzone is expected or where the reverse is true, i.e. deep percolation to deeper strata through such a layer.

There are often several soil layers in a profile, each with different K, D and μ values. They are denoted K_1, K_2, K_3, etc, and similarly for the D and μ values. Moreover, K can vary in different directions (anisotropy) and, in cracking soils, it depends on soil wetness.

Field observations to describe soil texture, mottling, consistency and moisture content, depth of groundwater, and the thickness of the permeable layer, are made by manual auger holes down to some 2–3 m depth in mineral soils and down to 5 m in soft materials (e.g. peat). As high-cost deep borings (down to 10–15 m) call for mechanical augering, they should only be made where strictly needed. Where the groundwater level is below 1–1.5 m, soil profile descriptions from observations in pits are also recommended in order to describe soil structure (a relevant soil property related to infiltration rate, permeability and storage coefficient) and to observe the presence of layers that may hamper soil water flow processes.

Permeability and infiltration rate

The soil permeability and the infiltration rate are measured directly in the field. The values obtained should be related to soil texture and structure described through the soil observations.

The K values are generally measured by the auger-hole method. The piezometer method to determine K is less often applied. Annex 3 describes the field methods for measuring K. Programs AUGHOLE and PIEZOM to calculate K, whose principles are described in Annex 23, are included on the accompanying CD–ROM.

The infiltration rate can be determined using infiltrometers. The lowering of water in a ring is measured. The disturbance caused by lateral seepage is avoided by pouring water into two concentric rings and measuring in the inner one. Placed on the surface, the top layers are investigated, whereas the permeability of deeper layers can be measured at the bottom of a pit.

Conventional "single ring" and "double ring" infiltrometers have the drawback of only measuring a small area and, consequently, there is large variation between nearby measurements. Moreover, the soil conditions existing at the beginning of the experiment (wet or dry, crusted or not) have a great influence. Therefore, estimations from observations of the presence/absence of stagnation water after rainfall may be useful. The impact of raindrops can be mimicked by a rain simulator, of which various types are available.

In irrigated fields, more accurate values for infiltration into the top layers can be derived from irrigation evaluations, for example by comparing the curves of advance and recession of the irrigation water or by measuring the drop of the standing water layer of inundated plots.

The infiltration rate can also be used as an estimate for vertical permeability in granular soils. However, extrapolation of unsaturated hydraulic conductivity to saturated conditions in structured soils, where the saturated flow mainly occurs through macropores, is less feasible.

To reduce the effect of the spatial variability of the soil, it is necessary to obtain a series of data for each type of soil, from which an average value can be derived. In view of the large variability encountered in many cases, determination of the median value (which is easily determined) or the geometric mean of the soil characteristics is usually preferred to the arithmetic mean.

Anisotropy

The auger-hole and piezometer methods measure predominantly the horizontal permeability, infiltrometers measure vertical permeability, whereas the inverse auger-hole method measures a combination. However, alluvial sediments tend to be layered horizontally, with a vertical conductivity K_v lower than the horizontal component K_h, whereas K_v is usually higher than K_h in cracked clays because of the development of vertical fissures. Where the permeability varies in different directions, the soil is called anisotropic. In these cases, it is necessary to estimate an isotropic K value equivalent to the actual anisotropic one. For clearly layered soils, a value of K_h/K_v of 16 is a better guess than neglecting the anisotropy, which means assuming a value of one.

Anisotropy is difficult to measure in the field because the usual methods provide the horizontal conductivity K_h only. The coefficient of anisotropy is defined by the relation between the horizontal and vertical conductivity of a soil layer. It can be estimated in a laboratory permeameter on undisturbed soil samples taken in both directions. However, as the size of the sample is usually small, the measurement can only be considered as an indication. A better choice is to compare the values obtained from infiltration tests (vertical) with those from auger-hole measurements (horizontal).

However, where this coefficient is known, drain spacing calculations can be made by using a model developed by Boumans (1979), to transform an anisotropic system into an equivalent isotropic one. This method has the advantages that the horizontal coordinates (such as drain spacing) remain constant and that multilayered soils can be handled. Annex 17 provides details on the application of this method.

This method is only applicable to vertical-horizontal anisotropy, which is widespread in drainage projects on alluvial soils. For other more complicated cases, it is necessary to refer to handbooks on groundwater flow (e.g. Childs, 1969; Raudkivi and Callander, 1976).

Drainable pore space

For non-steady drain spacing calculations, an additional input parameter is needed: the storage coefficient or drainable porosity of the soil. Direct field methods to determine μ are available. For example, the method developed by Guyon (Chossat and Saugnac, 1985) based on the relationship between the volume of water pumped from an auger hole and the drawdown of the water observed in four piezometers installed close to the hole. However, direct field determination is difficult, cumbersome and imprecise. Therefore, the drainable pore space is usually estimated.

Quite good field estimates of the drainable pore space can be made from observations of the rise in groundwater level. The μ value is the excess rainfall (expressed in metres or millimetres) divided by the groundwater rise (expressed in the same units) that would occur in the event of no discharge. Thus, a sudden rainfall of 20 mm giving a water table rise of 200 mm (without discharge) corresponds to a storage coefficient of 0.1. This method has been applied successfully in the Dutch polders, where μ values of 0.10 and 0.03 were obtained for clay soils with crack development and for dense silt soils, respectively.

An alternative is measurement of the drawdown of the groundwater after heavy rain in cases where the discharge can be measured, especially in drained lands (see Annex 8).

Cylinder tests can be carried out, but have a limited representativeness. Therefore, the drainable pore space is often estimated indirectly from *pF* curves or from *K* values. Annex 4 provides some information about the relationship between μ and *K*.

Natural and present hydrological conditions

Additional information is required on hydrological aspects, such as the recharge and discharge of the shallow aquifer, the discharge capacity of the existing main drainage system, and the options available to dispose of drainage water. Deeper aquifers are important if they are not completely confined. If under pressure, they cause upward seepage and often imply a considerable salt import into the rooted soil layers; if pressure is below the upper one, they contribute to drainage flows. All these conditions should be taken into account not only as quantitative aspects, but drainage water quality and the impact of disposal of drainage water on downstream water resources should also be considered.

Therefore, an integrated hydrological study is required in order to characterize the groundwater flow and its relationship with surface water under actual conditions before and sometimes after the drainage project. Where natural drainage is provided by a deep aquifer, it may sometimes be sufficient to render artificial drainage superfluous but its capacity may become insufficient if irrigation is introduced. If too much salty seepage is expected, the feasibility of the project is doubtful. Such future aspects associated with the project must be also considered.

Surface water study

First, the available outlet conditions and the water level regime of the receiving waterbody must be considered. This is because every drainage project will fail without a proper disposal site for the water. The question is whether there are sufficient outlet possibilities or whether they should be improved or even created, e.g. by means of a pumping station. Chapter 5 describes the outlet and disposal requirements in detail.

The outlet water levels and the quality of the future receiving waterbody must be determined in at least two critical periods, i.e. during maximum flows (when the levels are highest) and during minimum flows (when the salt and pollutant concentrations are maximal).

Second, the flow conditions of the existing open ditches and watercourses inside and around the project area must be known. The major issues to be considered are: the drainage conditions provided by the existing drainage network; the risk of flooding during the season of high flows (whether stemming from rain storms, periodic overirrigation customs or specific water management practices such as the release of the standing water layer of rice fields before crop harvest); and the relationship between surface water and groundwater.

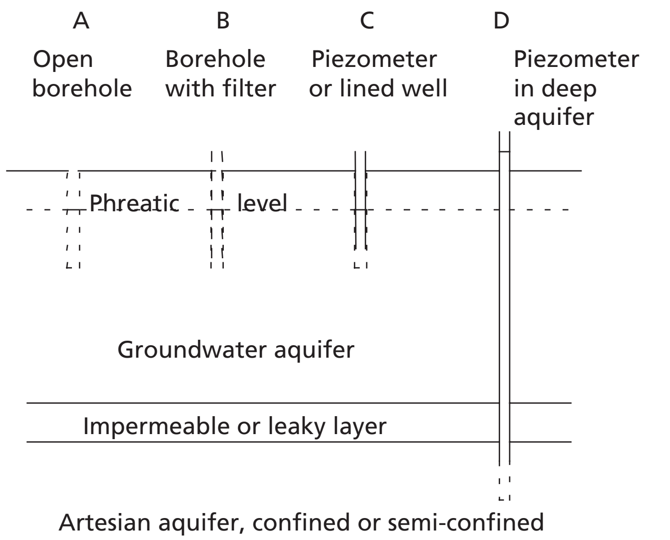
Finally, the influence of watercourses, lakes and the sea (if nearby) on groundwater must be determined in order to assess the amount of seepage and to formulate the water balance in the project area. In coastal areas, the danger of salt intrusion into fresh groundwater must be investigated.

Shallow groundwater study

An important item in the hydrological evaluation is the study of the shallow groundwater that the drainage system should control. The final product of this study is the isohypses map (isohypses being lines of equal water table height above a reference level). This map can be drawn once the hydraulic head data have been obtained by means of piezometric recording.

A piezometer, also known as a “lined observation well”, is a relatively short observation well provided with a closed standpipe to exclude influences from higher levels. Type C piezometers (Figure 5) are iron or plastic pipes with diameters of 25–50 mm. The bottom 10 cm are perforated and protected by a piece of cloth surrounded by

FIGURE 5
Observation points for groundwater depth (A–C) and piezometric level (D)



gravel or coarse sand. Type B observation wells are completely perforated pipes with a permeable envelope. These pipes are usually installed in a borehole with diameters ranging from 5 cm (in clay soils) to 8 cm. Type D piezometers, which generally have higher diameters, can also be laid by augering inside the pipe at 10–15-cm intervals and introducing the pipe afterwards. Thus, a good contact of the pipe with the surrounding soil is achieved. The high part of the buried pipe must be sealed with impervious clay to prevent leakage of surface water. About 30 cm of the pipe must be above the ground surface and tied to the soil with concrete. The upper end of the pipe is closed by a perforated plug to facilitate air circulation. The depth of the perforated area of the observation wells must exceed the usual lowest groundwater level.

Once the pipe is installed, groundwater is pumped and water levels are recorded in order to check whether the piezometer is working well. Once the installation has been completed, the elevation of the upper end is assigned by a detailed topographic survey, with reference to the mean sea level or a local basis level.

The main purpose of the isohypses map is to determine the direction of groundwater flow through the project and adjacent areas and its recharge and discharge zones. An isobaths map (isobaths being lines of equal depth of the water table below the soil surface) is sometimes needed to identify those areas where the groundwater table is above some acceptable threshold depth.

For a rapid reconnaissance of the groundwater flow in the feasibility stage, phreatic water level observations in the open boreholes (type A) drilled to obtain soil information are usually sufficient. However, in unstable soils (e.g. sands), type B is to be preferred. Water levels in existing wells and watercourses are also observed. All water levels are expressed in metres above or below the mean sea level (MSL).

Once the flow lines, which are perpendicular to the isohypses, and the recharge and discharge areas have been estimated on the draft map, more permanent observations of both B and C type are useful to measure the fluctuation of the groundwater level over a longer period. Type D pipes are needed for deeper aquifers, to find their possible influence on overlying layers.

Rows of observation points must be aligned along expected streamlines, but the exact location is not critical. For accessibility, it is useful to put them close to a rural road.

Detailed information can be useful:

- at the margins of the area, for flow towards or away from the project area;
- near existing watercourses, to see whether they are infiltrating or discharging.

The observation should be made perpendicular to these objects and in their neighbourhood.

The network density depends on: the level of the study, the complexity of the project area, and the resources available for drainage investigations. It may be low in the feasibility stage, but much denser in detailed studies for drainage design, where at least 5–10 piezometers per 100 ha are required.

From the soil elevation (for boreholes) or height of the top (for observation pipes) and the depth of the water level measurements, the hydraulic head can be calculated. Then, the isohypses map can be drawn by interpolation of the hydraulic heads between adjacent points, unless they are separated by discontinuities in landforms or in hydrology.

Different equipment is available to record the water level inside an observation well, ranging from flexible tapes with a special device to detect the water level by means of a sound or a light signal to submerged electronic water level recorders. Observation frequency depends on the groundwater level fluctuation. However, in climates with two distinct seasons, the recommendation is to repeat the observations in both the rainfall and irrigation periods. At least two records should be obtained in the shortest possible time interval, during the most critical periods of recharge:

- during periods of heavy rainfall, particularly if they coincide with critical phases of cropping (e.g. sowing or harvesting), when the need to ensure soil workability is a first priority.
- in the period of maximum irrigation requirements during the dry season or at the end of the irrigation season.

To support investigations regarding the detection of lateral seepage or deep upward flows, it is preferable to also measure the lowest water levels in a few selected observation wells.

The quality of the groundwater should also be assessed by taking water samples from the observation wells and piezometers. For the reconnaissance stage, quick measurements of the EC and pH may be done in the field. Alkalinity (pH > 8) can be found as a pink colour on adding a drop of phenolphtaleine. However, full ion and pollutants analyses require laboratory support. Guidelines for groundwater sampling (including retrieval techniques) can be consulted in Smedema, Vlotman and Rycroft (2004). Details for fieldwork for sampling can be found in the guidelines drawn up by the United Nations Environment Programme (UNEP) and the World Health Organization (WHO) on water quality monitoring programmes (UNEP/WHO, 1996).

The final groundwater map should be checked with the topographic and geomorphologic maps in order to determine potential inconsistencies. The streamlines and the recharge and discharge areas can be drawn on the isohypses map. The hydraulic gradient can also be measured. In addition, where the *KD* values are known or estimated, the amount of lateral seepage through the borders of the studied area can be estimated by applying Darcy's Law. For the feasibility study, a rough estimate suffices.

Geohydrological study

Shallow groundwater is often underlain by an aquifer, which may be thin or hundreds of metres thick. This aquifer can be:

- unconfined, in open contact with the shallow groundwater;
- semi-confined, with a resistive layer in between;
- confined, without contact with the shallow groundwater;
- under pressure, where the head is above the phreatic level;
- artesian, where the head in the aquifer is above the land surface.

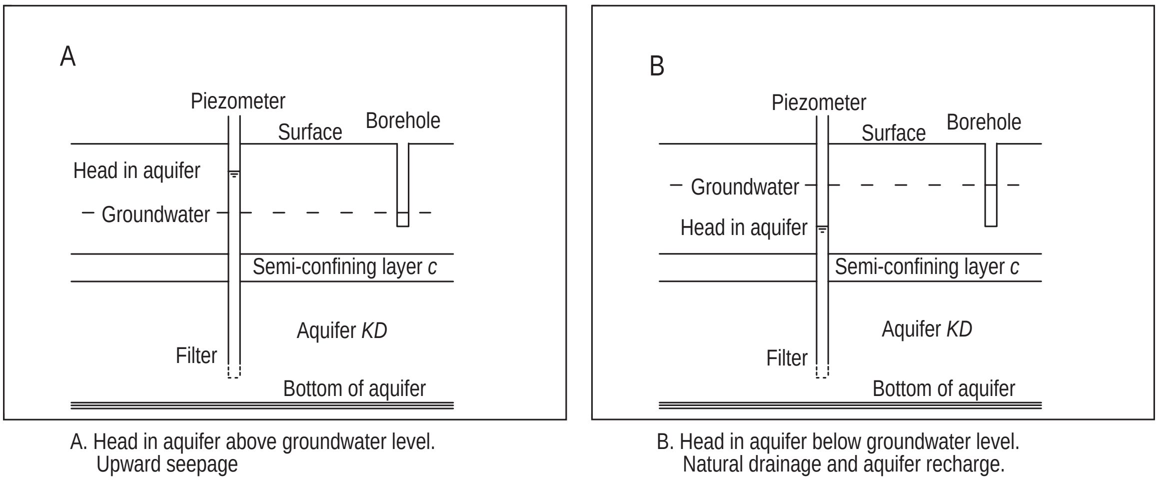
Where an unconfined or semi-confined aquifer is present below the shallow groundwater, geohydrological observations using piezometer batteries are required in order to determine whether there is upward or downward seepage from the aquifer or downward leakage towards it (Figure 6).

The magnitude and direction of the vertical flow depend on the hydraulic resistance to vertical flow *c* of the semi-confining layer and the transmissivity *KD* of the aquifer. These quantities can be found from pumping tests, which are described by Wesseling and Kruseman (1974), Boonstra and De Ridder (1994), and Kruseman and De Ridder (1994). Deep piezometers are installed above and below the semi-pervious layer in order to determine the vertical hydraulic gradient through the resistive layer (where the thickness of this layer is known). Annex 5 provides more details about these hydrological characteristics.

A piezometer network also allows the drawing of isohypses and isobaths for these deeper aquifers, to provide information about flow directions and heads. Comparison with the groundwater maps enables identification of the areas of possible upward and downward seepage in the present situation.

After the project has been executed, conditions will change. In newly irrigated areas, the aquifer may no longer be able to cope with the increased recharge. This may lead to rising groundwater levels in areas where natural drainage was initially sufficient. If the groundwater approaches the surface, waterlogging and salinization will follow. On the other hand, draining an area is usually followed by increased upward seepage. The

FIGURE 6
Influence of a semi-confined aquifer



groundwater observation network should be maintained at strategic sites in order to evaluate the design after the system has been constructed.

Once the necessary data are available, the amount of seepage or leakage can be calculated by applying Darcy's Law. Detailed information on groundwater investigations is available in De Ridder (1994).

Geohydrological studies are not always economic as in some drainage projects the high costs derived from the placement of deep piezometers and performing pumping tests cannot be justified. However, sometimes they are badly needed, e.g. in the presence of karstic limestones, where there are possibilities of natural discharge.

Hydrological conditions associated with the project

The need for artificial drainage may be assessed from a water balance in the saturated zone once the magnitudes of the inflows and outflows are known. For this purpose, in addition to percolation from the rootzone, the amount of seepage or natural discharge of the aquifer must be determined or estimated.

To determine the hydrological changes to be expected in the project area, it is necessary to follow an integrated approach. Improvements in the irrigation system and in the water management at the field level may reduce losses to drains and by deep percolation to aquifers, and consequently diminish the drainage needs and seepage problems. Costs and benefits of the different options of irrigation and drainage should be compared in order to select the most feasible solution.

Water balance

An important item of the agrohydrological study is the formulation of a water balance considering the present and future conditions of the project area. From the observations of the present and future conditions related to the selected designs, a water balance should be drawn up, involving the following components:

- rainfall, snowmelt and irrigation;
- evaporation and evapotranspiration;
- surface runoff;
- infiltration;
- capillary rise to the rootzone;
- deep percolation from the rootzone, including leaching for salinity control;

- upward seepage from watercourses, higher lands and deep aquifers under pressure;
- downward seepage (leakage) to drains, rivers and deep aquifers;
- natural drainage or lateral seepage to lower lying terrains.

Excess surface water

Excess water (from rainfall, snowmelt and irrigation) that stagnates on the ground surface can be determined from the water balance of the soil surface. The excess should be discharged through the surface drainage system. For short periods, the evaporation is small and the excess depends mostly on input by precipitation and irrigation and output by infiltration into the soil.

Heavy rainfall is the main source of water stagnating on the soil surface. Chapter 6 describes a simple method to estimate the amount of excess surface water and the magnitude of the infiltrated rainfall based on the water balance. In addition to the infiltration rate, the data for extreme rainfall obtained by applying Gumbel's method (Annex 2) are the main inputs to formulate the water balance at the soil surface. Climate change often increases the frequency of extremes and, thus, may affect the results adversely. A separate analysis of the last 20 years is needed to check indications as to whether any recent changes have occurred (although this period is too short to confirm a climate change).

Groundwater recharge by deep percolation

Most of the water that infiltrates into the soil is retained in the unsaturated zone and taken up by plant roots. The remainder percolates and recharges the groundwater table. Thus, the water balance of the rootzone can be used to estimate the amount of recharge.

The recharge of the groundwater table can have different origins, i.e.: percolation of excess rainfall infiltrated into the soil, percolation of irrigation losses (including the leaching requirements), and upward or lateral seepage stemming from irrigation works or from higher lying surroundings. The opposite of recharge is loss caused by the reverse processes.

Percolation of rainfall water can be estimated from the water balance described in Chapter 6. In temperate regions, the annual balance is sufficient in most cases. Table 4 gives an example from the Veluwe area, the Netherlands. This area is formed by wooded hills on sandy soils, where irrigation is not practised and discharges are not visible.

Comparing the chloride content of the percolating water (9 mg/litre) with the rain content (4 mg/litre), the concentration factor gave a similar value of 2.2 (Meinardi, 1974).

In irrigated lands, the groundwater table is recharged by leakage from the system of water conveyance, by non-uniform distribution of the irrigation water and by deep percolation from the irrigated fields. Seepage from irrigation canals is described in the following section. Normal losses of irrigation water at the field level must be anticipated as they influence drainage requirements of the same field. If these losses penetrate into deep aquifers, they may cause increased seepage elsewhere.

The options to determine the amount of recharge caused by percolation of irrigation losses depend on the availability of data and the field information that can be collected in time and with reasonable costs. In irrigated lands, recharge can be determined from the water balances at the ground surface and rootzone (if other components of the balances are measured in irrigated fields). In new irrigation developments, the balances may be calculated at the design stage.

TABLE 4
Example of annual water balance

Rainfall	800 mm
Evapotranspiration	450 mm
Recharge	350 mm

For planning purposes, average data from the literature are frequently used. Annex 6 provides details and examples of these procedures.

Percolation losses at the field level may be reduced by increasing the irrigation application efficiency. However, some amount of percolation is required in order to leach the salts accumulated in the rootzone. This fraction of water should be discharged through a subsurface drainage system if the natural drainage capacity is not sufficient. Therefore, in arid and semi-arid regions, the main purpose of drainage is to control secondary soil salinization.

In many irrigation projects, the amount of percolation from non-uniform application is enough to leach the salts added with the irrigation water. In overirrigated areas, it is possible to save water by improving the irrigation application efficiency while keeping the salt buildup in the rootzone under control.

However, in arid regions there is a trend to conserve water because it is becoming scarce. In these areas, additional water resources (groundwater, drainage water and treated wastewater) with restricted quality are being applied. Therefore, if more salts are added with irrigation and percolation is reduced, an accurate positive control of soil salinity is required. Moreover, the leaching requirements to maintain an appropriate level of salinity in the rootzone must be calculated, and the salt balance should be checked under these conditions. These requirements should be compared with the anticipated percolation in order, to ensure that water management (irrigation and drainage) is adequate to control soil salinity. Annex 7 provides technical details on leaching for salinity control and describes an approach for calculating the leaching requirements.

Seepage

Lateral seepage from adjacent lands and leakage from watercourses through shallow layers is also a component of the recharge of phreatic aquifers. It is usually confined to the neighbourhood of these sources. More important and spread over larger distances is upward seepage from a deep semi-confined aquifer under pressure. Such pressure is caused by recharge elsewhere at a higher elevation.

New irrigation developments often cause additional excess water problems such as upward or downward seepage in areas where they did not previously occur. Leaky reservoirs and irrigation canals cause an extra load on drainage systems. These inputs should receive careful attention in order to predict future seepage rates. Strong upward seepage from the irrigation system as well as from higher areas can make drainage of waterlogged areas difficult or even impossible. In less severe cases, the increased seepage may be discharged through the drainage system without causing problems. However, where the seepage water is salty, even small quantities will deteriorate the water quality in the drainage ditches enough to make its reuse for additional irrigation impossible.

Drainage projects aim to lower groundwater tables in cultivated areas. However, in doing so, they may increase existing local or regional groundwater gradients. This may also lead to increased upward seepage in the project area.

Seepage from the network of irrigation canals may be reduced by improvements in the irrigation system itself. Under certain conditions, seepage can also be controlled effectively by interceptor drains. However, it is sometimes only partly controlled and the recharge related to seepage is a component of the total recharge to be controlled by contiguous subsurface drainage systems. The continuous losses of water resulting from leakage from irrigation canals depend on such characteristics as the canal size and whether the canal is lined or unlined. In this latter case, the permeability of the soil around the canal is the major factor affecting leakage, which is sometimes diminished considerably by siltation (self-sealing). Losses from closed irrigation pipes are negligible.

The amount of leakage from canals can be derived from local measurements. One method is to measure water flow in two control cross-sections upstream and downstream of the canal reach considered. The difference between inflows and outflows is the flow lost along the reach considered. This flow is usually expressed as a percentage of the water flow per kilometre of canal, but the method involves a difference between two difficult measurements with often similar values. A better method is to install two temporary dams at both sides of a canal reach, and to measure the recession of the ponded water level in between. The water loss is expressed in cubic metres per kilometre, and as a percentage of the normal flow. Data on leakage can also be estimated from the information provided in the literature, where general data of seepage losses (expressed as a percentage of the canal flow per kilometre) of lined and unlined main and lateral canals are provided (FAO, 1977b and 2002b). However, although it is difficult to obtain reliable local measurements, they are preferable.

Natural drainage

Potential outflows from the saturated zone are lateral flow through the boundaries of the project area and leakage through the semi-pervious layer (where present) to an aquifer that has draining properties owing to outflow elsewhere. The magnitude of seepage and of these components of the natural discharge can be estimated if a water balance is made and the groundwater levels are measured at different times or by following the procedures described previously in the geohydrological study and particularly those in Annex 5.

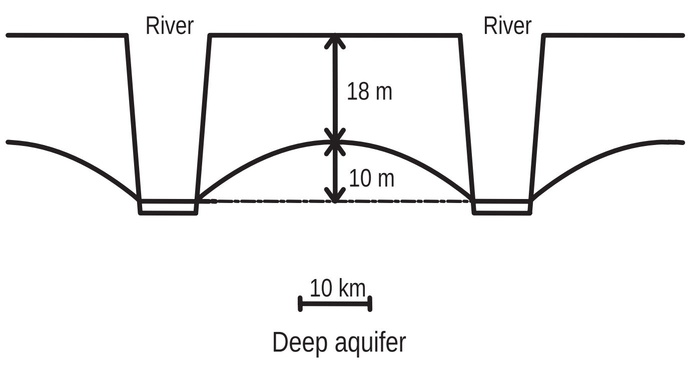
The existing aquifer capacity for natural drainage can be determined from the original situation. Some indication about natural drainage can be found by applying the method developed by Boumans, based on the assessment of the groundwater salinity, as described by Smedema, Vlotman and Rycroft (2004). A simple estimation can be made by measuring the fall of the water table in dry periods when there is no recharge and the water table is deep enough to ignore evapotranspiration. For example, if under natural conditions the water table is at depth of about 20 m and during a dry period of 200 days the drawdown of the groundwater level, measured in a deep piezometer, is 0.5 m and the μ value of a sandy layer is 10 percent, then the amount of water discharged by natural drainage will be about 50 mm, i.e. 0.25 mm/d. If after the introduction of irrigation a recharge of 1 mm/d is expected, natural drainage will not be sufficient and some artificial drainage will be required.

The existing recharge can be estimated and the corresponding head measured. Because for deep aquifers both are approximately proportional, the future head can be predicted from the post-project recharge. Drainage will usually be needed, but not if the water table remains deep enough.

As an example, Figure 7 shows the original situation in an area in which the recharge is estimated at 50 mm/year.

After irrigation, the recharge increased to 200 mm/year, and the equilibrium groundwater level would become 40 m above the water level in the river, which would mean complete waterlogging. In reality, the groundwater rises about 1 m/year in this area. When it has come close to the surface, the water is removed by

FIGURE 7
Original situation with 50 mm recharge per year



capillary rise and evaporation, an upward movement that leads to salinization. A similar process has occurred elsewhere, e.g. in the Pakistani Doabs (currently irrigated plateau lands between intersecting rivers) and in part of the Nubaria Desert reclamation area on the West Bank of the Nile Delta in Egypt.

Where such processes are to be expected, artificial drainage will be needed in the future. Although this can be postponed for 10–15 years, it will become necessary. As such a system is costly, the need and global future costs must be estimated (albeit approximately) at the feasibility stage.

Crops and crop drainage requirements

Once the natural conditions of the project area concerning climate, soils and hydrology have been determined, the next step in the feasibility study is to choose the appropriate cropping system adapted to the conditions foreseen after the drainage project implementation. It is then necessary to assess the requirements of the plants included in the selected cropping system as regards to ponding, rootzone waterlogging and to tolerance to soil salinity at the seedling, growth and harvest stages. Requirements for surface water and groundwater control relating to tillage and field trafficability associated with the cropping system are also needed in order to formulate land drainage criteria.

Concerning surface water ponding, the length of the critical period of crop inundation is the key issue. The period most critical is usually during spring and summer and in particular at the seedling stage. Ponding indices can be consulted in the literature (Smedema, Vlotman and Rycroft, 2004). Chapter 6 provides indications on the duration of critical periods for different crops.

Concerning control of waterlogging of the rootzone, the depth to the average highest water table midway between two drains is used for drainage design. Sieben (1964 and 1965) investigated an alternative method for the growth and production of notably winter wheat and barley. It showed that plant growth in the Netherlands is hampered by the duration and intensity with which groundwater levels exceed a crop-specific critical depth during the growing season. Similar conclusions were drawn from groundwater-level observations in sugar-cane fields in Peru (Risseeuw, 1976).

The value of the desirable average depth to the water table or the critical time for groundwater lowering is usually inferred from local experience, expertise or literature. Simulation models, such as SWAP (Van Dam *et. al.*, 1997) and DRAINMOD (Skaggs, 1999; Skaggs and Chescheir, 1999), can also be used, but they must be checked with local data. Local relationships between the average depth to the water table and crop yields and trafficability can also be estimated from observations in drained lands (Annex 9). Chapter 7 includes some indications on the desirable average depth to the groundwater table for different climate areas and crop systems.

Crop salt-tolerance information based on data by Maas and Grattan (1999) can be consulted in FAO (2002b).

Environmental procedures

Chapter 2 has described the environmental problems and opportunities of drainage projects. The EIA of the drainage project and the mitigation plan should be prepared in this planning stage with the aim of controlling the environmental effects on adjacent areas, in particular those situated downstream of the drainage outlet, and the negative impact of the drainage works in the project area itself.

Some specific items of EIAs and mitigation plans include:

- soil salinization and soil conservation;
- hydrological changes in downstream flow peaks, duration and low flows either as related to the existing conditions or as agreed upon;

- hydraulic issues, such as stability of outlet channels, sediment control, capacity of evaporation ponds, constructed wetlands, stabilization ponds and water treatment facilities;
- impacts of the hydraulic works on the landscape;
- water quality aspects, namely, salinization and pollution potentials and the protection for receiving bodies of water in line with agreed standards;
- saltwater intrusion into surface waters caused by open drainage outlets or river mouths (especially under tidal conditions), or into groundwater by excessive pumping from wells;
- impacts on existing wetlands, protected areas, and nature reserves situated nearby or downstream protected areas where changes in the water regime may alter the existing conditions;
- health factors related to water-borne diseases and sanitation requirements;
- social considerations, such as safety of the population during and after construction, relocation of individuals and facilities.

Any adverse effects should be compensated as much as feasible, and the needs and global costs of such mitigation plans must be assessed and included in the economic assessment of the project.

The EIAs for drainage systems are particularly critical owing to the wide range of the potential environmental issues described above. Designers must use the various details noted during the planning process in order to be certain that the facilities are in line with stakeholder expectations. The process for EIA preparation and discussions about the major environmental impacts of irrigation and drainage projects has been described by FAO/ODA (1995). This FAO Irrigation and Drainage Paper includes guidelines for preparing terms of reference for EIAs.

The feasibility report should include these environmental requirements and mitigation plans in its recommendations for the design stage.

Socio-economic evaluation

Governments and financing organizations require a socio-economic and financial evaluation of the drainage system planned in order to enable them to assess the overall feasibility of the project. At this stage, this can be done by comparing benefits with costs and by calculating the internal rate of return (Chapter 3). With regard to the assessment of the social impacts, a descriptive or comparative approach is preferred in order to evaluate the impact of the drainage project in comparison with the situation that would develop if no project were implemented.

The USBR recommends a simple method to estimate the benefit/cost ratio (USBR, 1984). Capitalized benefits over the life of the drainage system (100 years in USBR projects, at the current interest of capitalization considered) are compared with total costs, i.e. the costs of the system plus the estimated O&M costs. If the project area is cropped and drainage contributes to increase land productivity and ensure agriculture sustainability, the opportunity cost of the actual benefit without drainage must be considered in comparison with the drainage costs. In areas where the actual land productivity is negligible, such as with saline soils, the total value estimated for the future agricultural production is taken into account. In irrigated lands, in addition to the drainage costs, the irrigation costs should be included in this analysis. Drainage projects with a benefit/cost ratio greater than one are generally considered economically feasible. This method is useful for preliminary estimations made by drainage engineers. More complete economic and repayment analyses are required for large-scale drainage projects.

Example of the planning at feasibility level of a new drainage system in an irrigation sector

As an example of output of the soil and hydrological investigations described in the above sections, Figure 8 shows a simplified map of an irrigation section with the soil types and the isohypses map superimposed.

FIGURE 8
Detailed soil and isohypses map of an irrigation section situated in the river floodplain

Symbol	Landform	Soil texture		K (variation) (m/d)	Impervious layer		Depth groundwater level (m)	Hydrological conditions	Land use	Need for subsurface drainage		Need for surface drainage
		Ploughing layer	Subsoil		Depth (m)	Texture				L (m)	Z (m)	
B	Floodplain levee	Loam, Loam-sand	Sand-loam	2.5 (2.1-3.7)	> 3.0	-	> 1.5	Recharge area	Arable land	-	-	-
T	Floodplain transition	Silt-loam	Sand, Sand-loam	2.3 (2.3-3.6)	2.5-3.0	Clay-silt, gley	0.9 (0.6-1.3)	Transmission area	Arable land	30-35	1.3-1.4	-
D	Floodplain backswamp	Clay-silt	Clay-silt, pseudo-gley	1.4 (1.1-2.6)	1.8-2.0	Clay-silt, gley	0.5 (0.3-1.8)	Discharge area	Grassland	50-55	1.2-1.3	Land smoothing + outlets to open ditches
E	Estuarine plain transition	Clay-silt	Silt, clay	1.6 (1.1-2.6)	2.5-3.0	Clay-silt, gley	1.1 (0.8-1.8)	Transmission area	Arable land	30-35	1.3-1.4	-
F + El	Estuarine plain	Loam-clay-silt	Coarse sand, shells	3.4 (2.3-4.2)	2.0-2.5	Silt	1.0 (0.7-1.4)	Discharge area	Wetland	-	-	-

Source: Adapted from IRYDA, 1989b, and Martínez Beltrán, 1993.

From the map legend, it can be observed that no artificial drainage is required for the lands of the levee of the floodplain (B) because they benefit from natural drainage, the groundwater level is below 1.5 m and the soils are salt free. The isohypses map shows that groundwater is flowing from the river (the recharge area) through the aquifer below the levee towards the backswamp. On these lands, maize, wheat and sunflower are cropped on a sustained basis.

However, the soils of the transitional area between the levee and the backswamp (t) are affected by salinity because the groundwater level is frequently shallower than 1.3 m. To obtain a sustainable agriculture in these lands, similar to the levee lands, a subsurface drainage system is required in order to control the groundwater level and to discharge the salts leached by percolation water.

Natural drainage is even more restricted in the lowlands of the backswamp (D) and in the transitional areas to the estuarine plain (t'). The soils are fine textured, hydraulic conductivity is lower and the groundwater level is permanently above 0.8 m, stagnation being frequent after heavy rainfall. In this area, a surface drainage system is required in order to discharge excess rainfall and a subsurface drainage system is needed to control the groundwater level, if farmers wish to grow field crops and even if they wish to improve the existing grassland.

The estuarine plain (LL) is currently a wetland with silty saline soils. These are generally waterlogged owing to persistent shallow groundwater levels. As this land use must be maintained as a natural reserve, there is no need for artificial drainage in this area. However, as the main outflow drain, which conveys the drainage water from the agricultural lands, runs along this reserve, management of the drainage water quality in the adjacent agricultural area is an essential component of the planning phase and of the subsequent normal operational of this project.

Figure 9 shows the map of the irrigation section where drainage problems were identified and characterized in the previous stages (left bank of the Mugueta River shown in Figure 8). It can be observed that only some open ditches are situated in the low-lying areas identified in Figure 8. The drainage water is conveyed to the outside drains (rec Cagarrel and rec Madral). The studied area is protected from flooding of the outside drains by means of small dykes. In the highest part of the studied area and during the dry period, water is discharged in the outside drain (rec Cagarrel) by gravity, but pumping is required in the lowest part (rec Madral), especially after heavy rainfall.

However, there are no field drainage systems. Therefore, it is necessary to design new systems for the areas lacking natural drainage, and to complete the existing main drainage system with collector drains to convey the surface runoff and subsurface flow discharged by the new field drains.

An essential factor to be considered in this planning stage is the environmental impact of the irrigation section on the natural reserve, especially concerning the quality of the drainage water to be disposed in the outside drains. Consequently, the recommendation was to maintain grassland use (free of agrochemical applications) in the low-lying areas of the backswamps, with improved surface and subsurface drainage and controlled water levels.

Once the areas with different land-use and drainage needs have been mapped and their specific hydraulic characteristics have been determined, drainage requirements and costs can be assessed. In the legend to Figure 8, it can be observed that, at the feasibility stage, subsurface drainage requirement in terms of drain spacing (L) and depth (Z) are determined for each type of soil. This is done on the basis of the average values of their hydraulic characteristics and the drainage coefficients determined for the planned land use. In the backswamp areas, in addition to subsurface drainage, land smoothing and surface drainage outlets are required in order to remove excess rainfall.

The benefit/cost ratio estimated for the transitional lands between the levee and the backswamp areas (t) (where agricultural production could be similar to the production

FIGURE 9
Irrigation section with contour lines, road network and existing irrigation and drainage systems before the new drainage project

obtained in the levee lands (B) if groundwater levels are controlled) was about 5:1, considering an annual interest rate of 5 percent and 100 years of economic life of the subsurface drainage system. The benefit/cost ratio for improved grassland in the backswamp areas (D) was about 2:1.

In summary, the technical and economic feasibility of the drainage systems planned was confirmed. The environmental feasibility would also be positive if sound management of the water quality of the drainage water could be achieved.

DETAILED STUDIES FOR DRAINAGE DESIGN

Once the feasibility of the project has been established, the final design is made, either for a section or for the entire project area. At the beginning of this phase, consideration should be given to a stepwise implementation approach of the necessary project works.

This is especially important where the inferred causes of the drainage problems include inadequate water level management in the main public drainage system and/or irrigation water management practices at public system level, which generate unnecessary irrigation and rainstorm surface runoff. Rehabilitation and restoring sufficiently deep water levels in a main drainage system and/or reduced irrigation water spill losses may well reduce expensive field drainage requirements in areas with pervious subsoils, as well as avoid the construction of oversized pumping stations.

During the design and execution stages precise, topographic maps are indispensable. There should be a special focus on the careful measurement of the land elevation and on providing detailed information on the irrigation and drainage systems existing at the project area. The scale most frequently used in drainage projects for areas of 100–300 ha is 1:5 000, but scales of from 1:2 000 to 1:10 000 are also used depending on the size of the project area. On maps at a scale of 1:5 000, the common contour interval is 20–25 cm, but 10 cm might be required in extremely flat lands and 50 cm can also be worked with in sloping lands. Where no detailed maps are available and the survey is too expensive to base the design on the existing maps, once the first design is completed, the levels in the drain lines and collector lines must be measured in detail (one point every 20–25 m) and the design adjusted as necessary.

Figure 9 shows an example of topographic information. In addition to the contour lines separated each 50 cm, this map shows the existing irrigation and drainage systems (and the road network).

At the design stage, the first action is to identify precisely the outlet site and to select the type of disposal structure. Then, the layout of the new main drainage system is determined. Where only supplementary drains are required, they are inserted in the existing network. Later, the field drainage systems are designed. However, in order to determine the dimensions of the main drains, the surface and subsurface drainage coefficients must be known.

The outlet site should be located in the lower part of the project area, as identified in the topographic and isohypses maps. At this site, the average and maximum outside water levels must be determined. By comparison with the design inside water levels, the type of outlet structure (gravity, tidal gate or pumping station) is chosen. In addition, the discharge and water quality regime in the receiving waterbody must be known in order to design the water quality management practices to reduce the environmental impact of drainage water disposal.

Chapter 5 and Annex 10 describe the basic concepts and hydraulic formulae that are required for detailed design of the components of the main drainage systems, including auxiliary and outlet structures. Chapter 8 and Annex 23 describe a computer calculation program for determining the backwater effects on the main drainage system.

Surface drainage systems are described in Chapter 6, with details on methods to estimate the design discharge in Annexes 11–16.

For subsurface drainage, the first decision is whether to follow a steady-state or a non-steady-state flow approach. The former is easier and usually sufficient, but the latter may be necessary for sensitive crops under heavy rainfall, and for irrigated high-value crops in arid and semi-arid regions. In any case, the critical period or season should be defined as this is when groundwater and/or soil salinity conditions usually have their most negative impact on crop production.

The criteria for the steady-state flow approach are:

- The design discharge (in millimetres per day) is usually the same for an entire project or large parts of it.
- The desired groundwater depths under critical conditions. They may differ for different cropping systems.

For the non-steady-state flow approach:

- The critical time of ponding (especially for sensitive crops).

- The required lowering of the groundwater within a given time after the soil profile has been saturated by a heavy irrigation turn or rainstorm.

Chapter 7 and Annexes 17–19 describe the principles for detailed subsurface drainage design, including drainage equations and drainage design criteria. Computer calculation programs are also described in Chapter 8 (in more detail in Annex 23).

Once the drain depth has been selected, drain spacing is calculated by applying, in the most appropriate equation and in the specific computer program, the design criteria and the average values of permeability and thickness of the pervious layers. In non-steady-state calculations, the value of the drainable porosity is also used.

Detailed studies are now required, especially for those that have a profound influence on drainage design, i.e. the drainage design criteria and the soil hydraulic characteristics: the hydraulic conductivities; the thickness of the permeable layer (or layers); and – in the non-steady case – the storage coefficient. Soil stability estimations at design drain depth are also important for predicting the need for drainage envelopes or special drainage installation techniques. As they often vary considerably, detailed observations are required, e.g. one per 5–10 ha, depending on the spatial variability.

At a detailed level, less use of photo-interpretation is made. However, soil maps (from 1:5 000 to 1:10 000) based on landforms are also used. These comprehensive maps are much more convenient than thematic maps with contour lines for each characteristic.

Care must be taken in the design phase to consider the environmental issues identified in the planning phase and to develop designs that follow the agreed environmental plans.

Considerations on future maintenance needs must be taken into account at this stage as the institutional setup of maintenance has its consequences for the designs (see Chapter 3).

The project document should further include:

- a short text making reference to the outputs of the planning phase, in particular to the design criteria;
- annexes with calculations of the parameters of the different components of the system;
- the environmental mitigation plan;
- the detailed designs of the drainage works on the basis of the foregoing criteria;
- maps with the layout of the system and the start and end level of each drain or section of drain;
- the necessary appurtenant works, such as bridges, weirs, sluices and pumping stations;
- a bill of quantities of the materials needed;
- the technical specifications and procedures for implementation of the drainage works;
- the construction costs assessment;
- the O&M costs in relation to the organizational aspects of its implementation;
- where applicable: special irrigation development and/or crop husbandry practices to be adopted in order to meet specific environmental requirements in relation to the disposal of drainage water to downstream areas.

Example of a map with the layout of a designed drainage system

Figure 10 shows the drainage system designed to control waterlogging and salinity in the problem areas of the irrigation section mapped in Figure 9.

Figure 10 shows that a subsurface drainage system with laterals laid at an average depth of 1.4 m and spacings of 30 m plus seven pipe collectors (C1–C7) has been designed in order to control waterlogging and soil salinity in most of the left bank of the Río Mugueta floodplain.

FIGURE 10
Layout of the drainage system designed for an irrigation section

In the low-lying backswamp, subsurface drains have been designed at an average depth and spacing of 1.2 m and 50 m, respectively. In this area, the existing open drains (D1 and D2) are retained in order to receive surface runoff in addition to the subsurface drainage discharge.

To cut off seepage from adjacent high-lying areas and from the nature reserve, interceptor drains (I1 and I2, respectively) have been designed. The latter will also function as the collector drain of the affected laterals.

The existing main drain (DP1) is retained as are the existing gravity outlets to the Cagarrel outside drain (S1 and S2) in order to dispose of subsurface drainage water during most of the year, especially during the irrigation season. However, a new main drain has been designed (DP2) to collect the drainage discharge from pipe collectors C4 to I2, and surface and subsurface runoff of the whole irrigation section, when gravity outlets S1 and S2 are not able to dispose of water because of outside high water levels during peak water flows. Therefore, DP2 ends in a pumping station located in the lowest point of the project area (S3). During periods of extreme rainfall, water could be pumped from this station to the Madral outside drain with a direct outlet into the sea.

Once the drainage works have been implemented, M&E systems are needed to verify whether the systems are operating physically, environmentally and socio-economically as planned and designed. Monitoring plans should be developed as designs are being prepared, and the evaluation system must relate directly to the items to be monitored. The evaluations must be made in a timely manner so that adjustments or additions can be made where potentials for environmental or physical degradation are noted.

INVESTIGATIONS IN DRAINED LANDS

In already drained lands, investigations focusing on the relationship between drainage discharge and hydraulic head are useful for deriving soil hydraulic characteristics for a specific type of soil. In this case, a large volume of soil is contributing to the outflow of the drains, in contrast to the small volume investigated by the methods described in the previous sections.

The specific discharge is calculated from the drainage flow measured at the drain outlet. The different components of the hydraulic head required to overcome the different resistances to the subsurface flow towards the drains can be measured by means of piezometer readings in tubes laid at different distances from the drain, as shown in Figure 11.

Annex 8 provides details on the procedures for determining the hydraulic conductivity, the radial resistance and the drainable pore space in drained lands.

Measurements of drain discharge in drained areas are also useful for determining drainage coefficients under actual conditions. Discharge hydrographs (Figure 12) can be used to assess the peak discharge and the average drainage coefficient during a non-steady-state drainage period.

The relationship between the depth of the groundwater level and crop yields and wetness conditions of the top layer affecting soil trafficability is also useful for estimating the optimal depth of the groundwater table. For example, Figure 13 shows the relationship between the relative yield (Y) of maize and alfalfa and the average depth of the water table (Z). It indicates that an average depth of 85 cm might be critical for these crops, although temporary shallower

FIGURE 11
Piezometer line to determine the components of the total hydraulic head in a drained soil

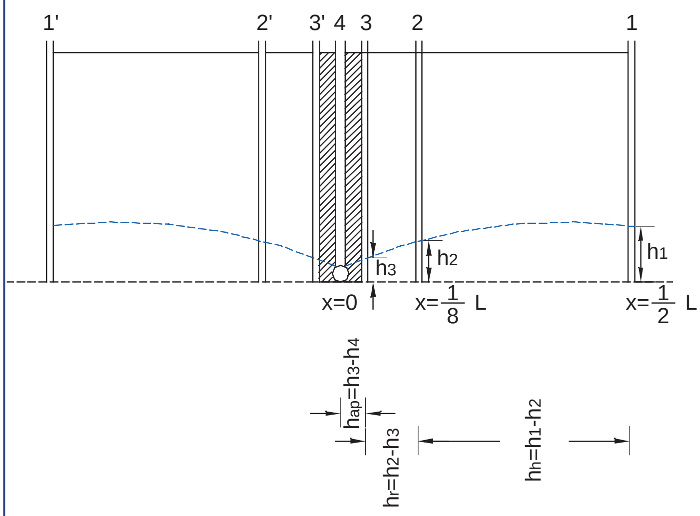
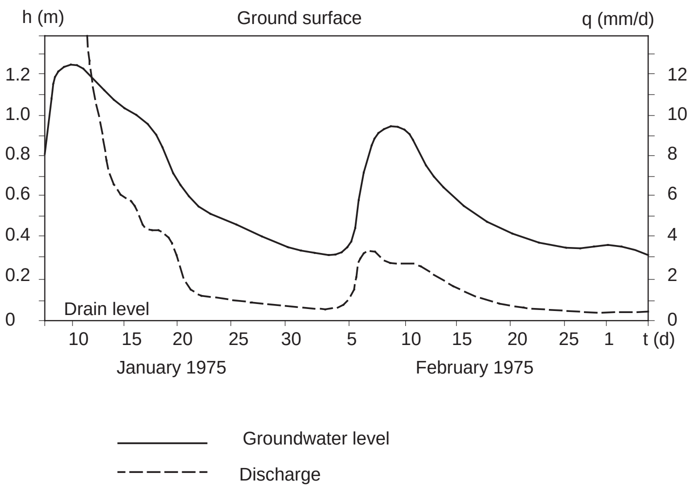


FIGURE 12
Example of hydraulic head (h) and drain discharge (q) hydrographs



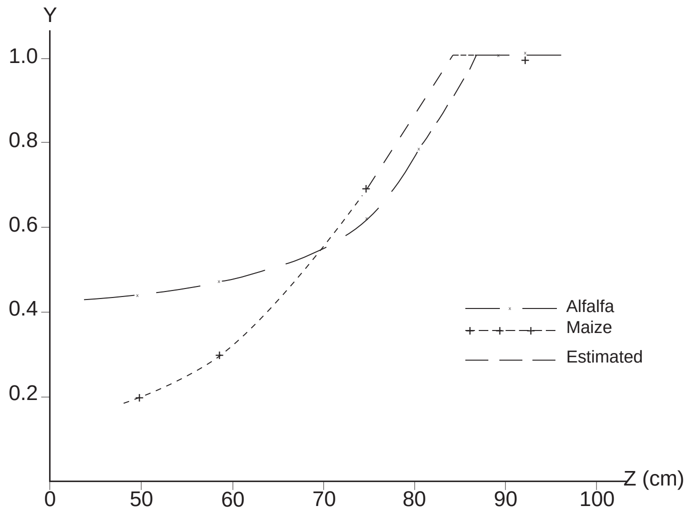
Source: Adapted from Martínez Beltrán, 1978.

depths are tolerated by most field crops, e.g. 30 cm in one day.

Annex 9 provides additional details on procedures for assessing drainage criteria in drained lands.

In summary, sound drainage design criteria can be formulated for new projects with similar characteristics to those of the observed lands. In recently drained areas, the basic data used for drainage design can be checked in the same way. If the performance of a newly installed drainage system is monitored as early as possible, the resulting information can be used to adjust the partly estimated *KD* values and drainage coefficients for similar areas still under construction.

FIGURE 13
Relationship between the average depth of the water table and maize and alfalfa yields



Chapter 5
Main drainage and disposal systems

INTRODUCTION

In any project involving land drainage, it is advisable to work from the downstream part of the system in an upstream direction, that is:

1. It is necessary to provide a suitable outlet for the drainage water, either by gravity or by pumping.
2. If this outlet alone is insufficient, a main system of open drainage channels and ditches must be constructed to convey the drained water to the outlet (in this chapter, channel indicates an open drain that is of larger dimensions than a ditch, whereas canal is used specifically for irrigation).
3. If this main system cannot provide adequate control of groundwater levels in the fields, a system of field drains is needed, forming a detailed drainage system, consisting largely of subsurface pipe drains. Chapter 7 describes such subsurface drainage. Open trenches are sometimes used for groundwater control instead, e.g. in drainage of heavy land and in the humid tropics. However, the use of open trenches for subsurface drainage is not generally recommended, as they hamper agricultural operations, reduce the cultivable area and increase the maintenance burden.
4. If these field drains are still not able to cope with water stagnating on the surface, additional measures (e.g. raised beds with superficial ditches or landforming) should be taken. Chapter 6 describes such surface drainage.
5. The division between public main open drainage networks and detailed open subsurface drainage systems is somewhat arbitrary. Ditches serving a few land users are sometimes classed with the former and sometimes with the latter depending on local circumstances, traditions, and direct responsibility for maintenance.

Where any element of the main drainage system is not functioning properly, all upstream facilities cannot fulfil their purpose. Thus, a good outlet and a well-designed and well-maintained main drainage system are prerequisites for adequate field drainage.

Apart from these features dealing with the removal of a certain quantity of drainage water, the quality of the drainage water should also be considered. Within the project area, water quality governs the possibility for its reuse for irrigation, and outside downstream users and downstream ecology may be affected. These quality aspects are becoming increasingly important in drainage projects. Therefore, the layout of the system must minimize its negative environmental impacts.

Chapter 2 has considered briefly the quality aspects of drainage waters. The FAO Irrigation and Drainage Paper on the management of agricultural drainage water in arid and semi-arid areas (FAO, 2002b) considers these factors in detail.

This chapter focuses on the main system, first providing a general description and then adding design requirements and criteria. Annex 10 provides technical details for calculations.

Programs for tree-shaped systems with steady flow, based on the Manning formula, are available at many institutions, waterboards and engineering firms. An example is the HEC-2 program developed by the United States Army Corps of Engineers (USACE, 1990). For non-steady-state flow and network structures, more sophisticated methods are needed, such as the program DUFLOW (STOWA, 2000), based on numerical solutions of the Saint Venant equations, and the HEC-RAS program, which has been adopted by the United States National Resources Conservation Service. Details on these programs, including procurement addresses and Web sites, are given in Smedema, Vlotman and Rycroft (2004). In this paper, the program BACKWAT (Annex 23) describes backwater effects.

STRUCTURAL ELEMENTS

Where the position and hydraulic characteristics of the outlet are known, the following decisions are of concern in the layout and design of the various elements of the upstream system.

The layout will be considered first, followed by a description of the various structures belonging to the main drainage system, such as:

- channels and ditches – they require alongside facilities (tracks, agreements with adjacent land users / landowners) for inspection and maintenance;
- bridges and aqueducts;
- culverts and siphons;
- weirs and drop structures;
- sluices, gates and main pumping stations at the outlet, or any intermediate pumping stations may be considered to belong to this category because they form part of the outlet works.

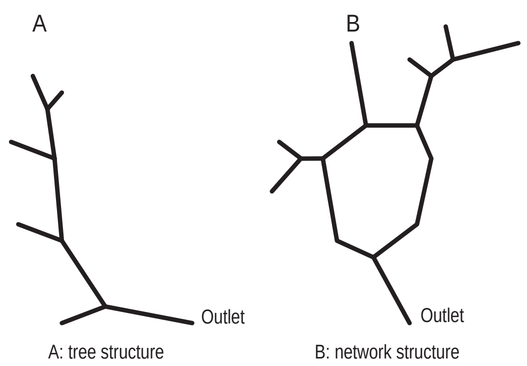
Attention is needed in order to prevent bank erosion, especially at points where surface runoff collects or field drainage systems are connected with the open channels of the main system.

Layout

The projected main drainage system usually has a branching-tree configuration (Figure 14) in which every drop of drainage water has only one way to reach the outlet. However, more complicated network structures are sometimes found, usually remnants from former natural drainage systems.

The network depends greatly on the size of the area, its topography, the existing watercourses and the form of its borders. In a system composed of buried field drains, collector pipe drains, ditches and larger waterways, the length of each successive order determines the distances of the next. Thus, the distances of the first open channels (usually ditches) depend on the lengths spanned by the subsurface drainage system. There is a tendency to replace the first open ditches with buried pipes, thus reducing the density of open waterways and consequently saving on maintenance (a costly operation).

FIGURE 14
Structural types of a drainage system



Another element for the choice of layout is the future maintenance of the main system and its organization. The smaller elements can be maintained by a farmer or a local farmers group, be it by hand or by machine. The larger elements can be maintained mechanically by an organization in which the stakeholders participate and can have indirect influence, e.g. a water board.

Within the project area, there

may be protected natural reserves. These should be left untouched by the main drainage elements. The channels should keep enough distance from these areas to avoid influencing the underground water currents to and from the reserve area. Opportunities for improving ecological values sometimes exist in important areas not protected as reserves. Some special drainage with water table management may improve the habitat or ecological values considerably. These potential options should be discussed with stakeholders.

Villages and towns and agriculture-based industrial zones in the project area are best provided with a dedicated connection to the public main drainage system to facilitate controlled disposal of polluted water and minimize the risk of improper reuse. Where possible, such urban waters should be treated.

The location of the drainage channel network depends on the topography. In undulating terrain, the watercourses follow the valleys and, thus, the pattern is irregular. However, in flat land, a rectangular layout is usually designed, with exceptions owing to the shape of the project boundary and natural watercourses, or to slight differences in elevation. Existing waterways are often enlarged, but sometimes they are replaced by a new and wider spaced network of larger channels. These channels should follow the natural drainage paths where possible.

As layout and location of elements are highly determined by local circumstances, it is not possible to give more detailed information about these points.

Channels and ditches

Open waterways or channels form the principal part of a system that conveys the outflow from the fields to the outlet. There are two types of layout (Figure 14):

- a tree structure, where this path is fully determined (e.g. from ditch via a small watercourse into an ever larger one, until the outlet is reached);
- a network structure, in which more than one route is available and where the path depends on the local gradients. The branches of such a network are cross-connected (anastomosis).

Networks, such as the elaborate system of channels of the Mekong Delta, are beyond the scope of this publication as this type is seldom used in new projects. Special calculation methods for flows through networks are available, but they are complicated.

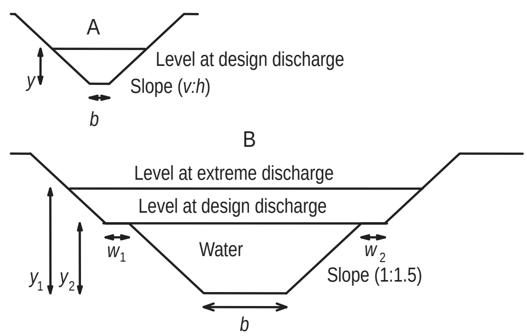
In most projects, the tree structure is chosen and a straightforward method of calculation is allowed.

The cross-section of open channels (Figure 15) is usually trapezoidal for small drains, and sometimes with a double trapezoid for larger ones.

The side slopes are indicated in this publication as the ratio of vertical to horizontal (*v*:*h*) and depend mostly on the type of soil (see Table A10.2). Some points to consider are:

- Steep slopes save on excavation costs, and such channels occupy less agricultural land, but slopes that are too steep result in bank failures.
- Local experience is the best guide for safe channel side slopes.
- Any slope failures usually occur shortly after construction – later, the bank vegetation has a stabilizing effect.

FIGURE 15
Cross-section of an open ditch (A) and a large channel (B)



- Vegetation (especially submerged plants) obstructs the water flow. Thus, regular maintenance is required. In particular, woody vegetation on the banks must be kept short.
- Lateral groundwater seepage promotes slumping of channel banks. In places with strong seepage, it is necessary to either adopt flatter slopes, provide suitable vegetation cover, or cover the banks with permeable but heavy materials. Geotextiles covered with loose stones are useful in such cases.
- Trapezoidal profiles are designed and built. However, after many years, they change into more parabolic forms, often with steeper slopes above the usual water levels, covered by vegetation kept short by mowing. In areas with arid climates, the vegetation remains sparse and is confined mainly to the area near and below the water line.

The ratio of water depth (v:h) to bottom width (y:b) should be kept between certain limits (see Table A10.1). The calculation of the expected flow rates for the assumed channel dimensions and gradient is based on Manning's formula (Equation 1 in Annex 10). Where the calculated flow rate is too high, the calculation should be repeated with a milder gradient and/or a different y:b ratio.

Erosion in watercourses should be avoided. At design discharge, the flow velocity must be limited to safe values (see Table A10.2). At low flows, meandering of the small remaining stream must be prevented as it can undercut the banks. Both can be achieved by placing weirs at appropriate points, so that sufficient water depth is maintained. Another option is to limit the bottom width of the ditch. This is helpful if weed growth is expected to increase with weirs owing to shallow water depths during extended low flow periods. Parabolic ditch bottoms or small base flow drains in the ditch bottom area have also been used successfully in some cases. Special attention is needed at places where elements of the detailed field drainage systems spill into open waterways (Chapters 6 and 7).

Open drainage channels need regular maintenance. This is because they are susceptible to choking by the growth of aquatic plants and silting up by sediments brought in by uncontrolled surface runoff. In contrast to most irrigation canals, which carry turbid waters, plant growth in open drains is more intensive. In large channels, a water depth of at least 1 m (1.5 m is better) will hamper the growth of reeds, although submerged and floating plant species may still thrive. In ditches, growth is retarded where they periodically fall dry. A minimum cross-section is often prescribed in order to secure sufficient discharge capacity, especially for small waterways (which may become rapidly overgrown with weeds). Such a minimum cross-section has a bottom width of 0.5–1 m and a water depth of 0.30–0.50 m at design discharge, and preferably zero in dry periods. These dimensions can vary according to the machinery available for construction and maintenance.

Although these measures are of some help, periodic maintenance is always needed, especially before the onset of the season in which drainage requirements are highest (usually the end of the wet season; in the case of rice in arid regions it may be the pre-harvest period). Special equipment is available for mechanical cleaning of these open watercourses. This equipment is specific for two maintenance operations: desilting and deweeding. When cleaning an open drain, care should be taken to avoid making the side slopes steeper – so reducing the risk of bank failure.

Bridges and aqueducts

Where roads and railways cross main waterways, bridges are needed. Irrigation canals usually cross by means of aqueducts. Those that leave the cross-section of the waterway intact have no influence upon the flow in the channel. However, if they are narrower, notably in flat areas, special formulae for flow through openings are used to limit backwater effects (Equation 6 in Annex 10). Erosion of the channel under the bridge should then be avoided by not allowing high flow velocities.

Culverts

Culverts are necessary where an open drain crosses a farm entrance or a rural road. One metal, concrete or reinforced plastic pipe buried at least 50 cm deep is commonly used where the water flow is less than 0.5 m^3/s, and two such pipes for discharges up to 1 m^3/s. Their diameter depends on the amount of flow and on their length, but a minimum diameter of 300 mm is often recommended in order to facilitate cleaning. Where the flow is higher, large-diameter pipes, box-type culverts or bridges are used.

Calculations for culverts are based on the hydraulics of flow through openings and friction in pipes. Culverts are usually overdimensioned because they are less able to cope with extraordinary large discharges and to avoid floating debris that may clog them. Whereas open channels may be bank-full or even overflow their surroundings, culverts may be washed out completely and road connections broken. Annex 10 describes methods for this overdimensioning.

Coarse debris can clog small culverts easily. Hence, they need regular inspection and cleaning. A trash-rack at the entrance can be useful in waterways that carry this type of pollution (dead vegetation, or even dead cattle in some regions). A floating beam can hold back floating vegetation. However, both require regular cleaning.

Weirs and drop structures

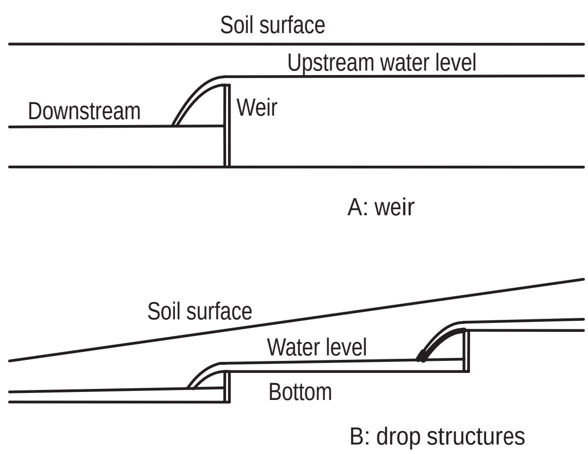
Weirs are used to separate different water levels that would otherwise lead to deep excavations upstream or to an excessive flow velocity and erosion. They can be adjustable or have a fixed crest level (Figure 16). This crest can be sharp or broad, in which case a different coefficient is used for design (Equations 7 and 8 in Annex 10). There are various kinds of weir, belonging to two groups:

- Fixed weirs. These are the simplest type, but their width may not be ample enough to handle heavy discharges. In this case, "long nose" ("duck bill") weirs may be a solution.
- Movable weirs. These are of different types varying from planks or stop logs resting in grove side-walls to self-adjusting valves acting on upstream water levels or forming part of a remotely controlled system.

Drop structures (Figure 16) are used in sloping lands where the bottom gradient must be smaller than the ground slope to prevent erosion. They are necessary to maintain the permissible flow velocity, and to dissipate the excess head. Where the energy drop exceeds 1.5 m, inclined drops or chutes should be constructed; and where it is less, straight drop structures are preferred.

Weirs and drop structures cause an improvement in water quality through aeration. The overflowing water falls into the downstream section, thereby increasing its oxygen content. As a consequence, degradable organic substances break down more rapidly.

FIGURE 16
Weir and drop structures



Pumping stations and sluices

The capacity of pumping stations and sluice structures is determined by the expected flow under unfavourable circumstances, i.e. a design discharge combined with high water levels outside the project. Compared with sluices, pumping stations are not much affected by outside floods, but they are less flexible. Whereas sluices increase their capacity at high internal levels, the capacity of pumps is almost fixed. Tables A11.1–11.3 show examples of a pumping station operating under extremely adverse conditions.

A pumping station is needed where the outer water level is either always or for long periods above the desired inner water level. Drainage by pumping is often only necessary in the rainy season; then a sluice or a gate is combined with the pumping station to allow discharge at low outside levels.

A pumping station consists of: an approach channel, which enables uniform flow; a sump, where drainage water collects; a suction pipe; a pump or group of pumps; and delivery pipes with outlets to the receiving waterbody. Generally, the peak flow is discharged through several pumps and the base flow through only one. However, other combinations may be used according to the circumstances. An additional standby pump is usually included for safety reasons and to enable repair of one of the other pumps without losing design capacity.

Three types of pumps are commonly used according to the drainage flow to be elevated and the lift. Where the flow is less than 200 litres/s even if the lift is high, radial pumps are recommended because they have greater flexibility in relation to flow variations. Axial pumps are most suitable for water flows of up to 1 m^3/s and low lift (2–4 m). Where the outer level is almost constant, and the water transports vegetation or other debris, an Archimedes screw is appropriate. The choice of pump type also depends on local conditions such as availability, experience, maintenance possibility, and prices. These pumps are driven by diesel engines or by electricity where enough power is available on the spot. Where the electricity supply is sufficiently reliable, notably in periods when rainstorms occur, electrically driven pumps are preferable to diesel pumps because they require less attention (no fuel supply organization), maintenance and management, and they can be automated easily. The design of pumps and pumping stations is discussed in Wijdieks and Bos (1994).

Where pumping stations are essential in a drainage network, then they require competent operation and solid preventive maintenance routines. Annex 10 gives calculation methods for the dynamic head and power needed (Equations 9 and 10 in Annex 10).

For sluices (and flap gates), the tides or high outside levels may hamper the drainage discharge. In combination with the storage possibilities inside, this leads to solutions that are highly dependent on local conditions. These require special calculations, based on: numerical estimation of the storage and the levels inside; the water levels outside; and the flow through the sluice opening when the inside levels are higher. Although Annex 10 includes a formula to calculate the discharge rate (Equation 6 in Annex 10), this type of calculation will not be treated in this publication. However, firms in hydraulic engineering and hydraulic institutes often have calculation methods and programs available for such cases. Details on the design, construction and maintenance of tide sluices and gates are given in De Vries and Huyskens (1994).

Connection with field drainage systems

Chapters 6 and 7 consider drainage of individual farm fields. Structures to conduct this surface and subsurface water safely into the main drainage system without causing erosion are necessary. These usually consist of rigid pipe sections for subsurface drain outlets as well as for many surface water exit points. Pipe outlets should discharge above the water level of the receiving channel in order to facilitate visual inspection and to prevent bank erosion. An alternative is protection of the bank below the pipe outlet by chutes lined with grass, concrete or rock. For large outlets, special structures are made. Their dimensions and construction materials depend on local circumstances and on the amount of water to be discharged. Details of these auxiliary structures of subsurface drainage systems are described in FAO Irrigation and Drainage Paper No. 60 (FAO, 2005).

Attention must be given to the effect of these drainage connection structures on mechanical cleaning operations, especially where they occur at many points. In this respect, chutes and special constructions are more convenient than protruding pipes.

DESIGN REQUIREMENTS AND CRITERIA

Water level requirements

The capacity requirement of the main drainage outlet system is that it maintains sufficiently low water levels under unfavourable conditions. This means that in wet periods occurring with a frequency of once in 5–10 years, it must provide an adequate outlet for the field drainage systems so that these outlets still have free discharge into the main system or – if that is not always and everywhere possible – are only submerged temporarily and slightly.

The water levels are governed by the following data:

- specific discharge (drainage coefficient);
- design discharges of channels;
- hydraulic gradients and geometry of channels;
- head differences for culverts, bridges, weirs, sluices and pumps.

For the design, the channel system is divided into sections in parts that are small enough to be considered homogeneous in discharge and gradient, so that within each section the bottom width and water depth will be the same. Bridges, culverts, weirs, sluices and pumps are treated as separate structures.

Specific discharge

Main drainage system discharges are generated by various field drainage processes. Of these, the surface drainage processes are usually the most critical.

The specific discharge is the rate at which excess water must be removed by the system without difficulty. In humid climates, it is the runoff that occurs on average from rainfall with a frequency of once in 5–10 years, increased with water from other sources (e.g. seepage). It is usually expressed in millimetres per day (to be comparable with rainfall) and is later converted into a drainage coefficient expressed in litres per second per hectare for further calculation. A less probable precipitation event is sometimes taken (once in 50–100 years) in order to check the safety of the system and the extension and duration of the flooding under extreme circumstances (yet longer times where human lives are at stake). Gumbel's method (Annex 2) may be used to predict such rare phenomena from a limited amount of data. For humid areas, there are several methods to estimate the discharge intensity for design (Chapter 6). However, the effects of climate change must be kept in mind.

Under arid conditions, not more than 1.5–2 mm/d is usually required for salt control and irrigation losses (Chapter 7). However, where rainy seasons occur (e.g. monsoons), this coefficient may be much higher.

In principle, expected seepage should be added. However, in humid areas, it is usually negligible compared with the rainfall. However, in arid regions, the drainage coefficient is low and seepage can be of comparable magnitude or even higher. Moreover, where seepage is saline, then the soil salt balance of the rootzone may be affected significantly in arid regions.

Impermeable surfaces, such as areas with bare rocks, asphalt road, buildings and horticulture under glass or plastic, have a large specific discharge. This is because infiltration in the soil is impossible. In agricultural areas, the influence of these areas is usually of minor importance. However, built-up and covered areas tend to become more extensive over time, especially where large cities come into existence or where covered horticulture or orchards with intensive surface drainage systems become widespread. Problems may arise in periods with exceptionally intense rainfall. Where such problems occur, they may lead to a revision of the drainage system in a distant future. This involves mainly an increase in the open water storage by allowing certain low-lying areas to be flooded (wetlands, or retention basins) in critical periods as compensation for loss of soil and land surface water storage.

Design discharges of channels

In areas with rainfall, all discharges from upstream sources must be added in order to calculate the necessary transport capacity at the end of each section below the stated highest admissible water levels as well as for any other construction belonging to the drainage system, taking into consideration the effects of retention and different travelling times in the discharging subdrainage catchments.

The design flow for return periods of 5–10 years is determined at representative places:

- at the outlet of contributing smaller channels;
- at the beginning and end of each of channel section;
- at other constructions;
- at the final outlet.

At these places and also in channel sections in between, control points are located where characteristics such as surface elevation and other data are measured.

In reality, the flow rate changes with time, and storage in the channels will cause the discharge process to be non-steady. Nevertheless, in many cases, the channel storage is relatively small in comparison with the storage in a pre-wetted soil (low percentage and some 10–20 percent, respectively). Therefore, the storage leading to non-steady effects is mainly located in the subsurface drainage system of the fields and not so much in the main system (provided most outflow is via the groundwater). Moreover, the channels are often short enough to ignore outflow retardation by travelling discharge waves, so that steady-state calculation is often a good approximation. However, in cases where surface runoff is important, the storage in the fields is much smaller and, consequently, the design discharges for the main system become far higher.

Non-steady-state calculations for runoff normally begin at the upper end of open drains and proceed downstream. Determinations on the time of the runoff peak, its shape and duration are used to calculate the size of outlet drains. Generally, the empirical method of the unit hydrograph is applied (Annex 14). In this case, the shape of each contributing area and the slope of the watersheds enter into the channel-sizing equation.

For steady-state calculations in a short channel section, the flow is taken as flow from upstream sections plus the inflow into that proper section. Both flows are calculated as the product of the specific discharge (q in litres per second per hectare) multiplied by the contributing area (A in hectares) in order to obtain the flow (Q in cubic metres per second). This gives a slight overestimation, but the difference is on the safe side.

A reduction is often applied to the upstream flow from large areas (Q). It is in the form of an exponent n (< 1). Where the local rainfall is patchy, the area for considering reduction is above some 1 000 ha. Where the local rainfall is widespread, reduction may be applied if the upstream area is larger than 50 000 ha. Recommended area reduction factors (n) can be consulted in Smedema, Vlotman and Rycroft (2004).

In irrigated areas of arid regions where rainfall is negligible, the accumulation of discharges from different parts of the system is not necessary unless flooded rice is grown. In this case, a high drainage discharge capacity is required at the end of the growing season. This is because all farmers want to evacuate the remnants of the standing water layer in a relatively short time. As not all fields are irrigated at the same time in non-rice areas, the peak discharges from the different sections do not occur at the same time. Thus, the peak discharge from the entire system is less than the accumulated peak discharges from the sections. FAO (1980) has provided values for the multiplying factors to determine the design discharge for collector drains as related to the fraction of the area that is irrigated simultaneously.

Exceptional discharge

In order to check the safety of the system during discharges with return periods of 50–100 years (which may occur at any time), the calculations are sometimes repeated with a value of q of 1.5–2 times higher than the design coefficient. For such rare occasions, the water levels may become much higher than normally allowed, but disasters such as serious floods or severe damage must be avoided.

Hydraulic gradients and head differences

The hydraulic gradient of a channel section is the slope of the hydraulic energy line along the channel. At low velocities (< 0.5 m/s), this line is almost equal to the slope of the channel water surface. It must be more or less parallel to the slope of the land along the channel. Initially, the average hydraulic gradient available for gravity discharge can be chosen to be approximately the same as the surface gradient. Where the terrain is completely flat, it is necessary to choose a small hydraulic gradient. This must be enough to allow sufficient water flow. However, considering the need to avoid erosion, the flow velocity should not exceed 0.5 m/s. In silty soils, it may be as low as 0.20 m/s.

The bottom slope of an open-channel section should normally be equal to the average hydraulic gradient. Values of 0.05–0.1 per thousand are common in flat areas (even lower where the area is extremely flat). To create a higher gradient in these cases, discharge by pumping from one section into another could be considered. However, the capitalized operation costs may easily exceed the saving on channel dimensions.

Longitudinal profile

In a longitudinal profile of a channel, the level of the strip of land along the top of the banks and the water levels to be tolerated at design discharge should be indicated, together with the location of buildings and confluences. Sudden changes in the gradients should be avoided and, where necessary, occur only at the limits of a section. Where sudden water level changes are required by the topography (to avoid deep excavations or excessive flow velocities), weirs are needed. Their location follows from land surface measurements.

Head losses caused by weirs and other structures must be shown in the channel hydraulic profiles. Weirs and culverts cause differences in head between their upstream and downstream ends, and weirs in particular lead to backwater effects that may be noticeable far upstream in flat country.

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

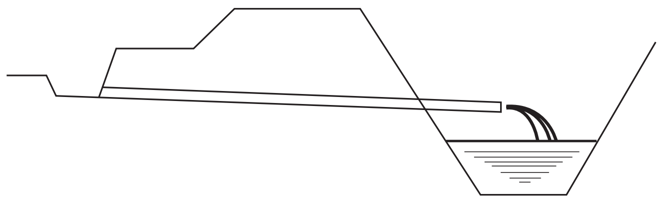


FIGURE 18
Layout of a field for irrigation and surface drainage through furrows

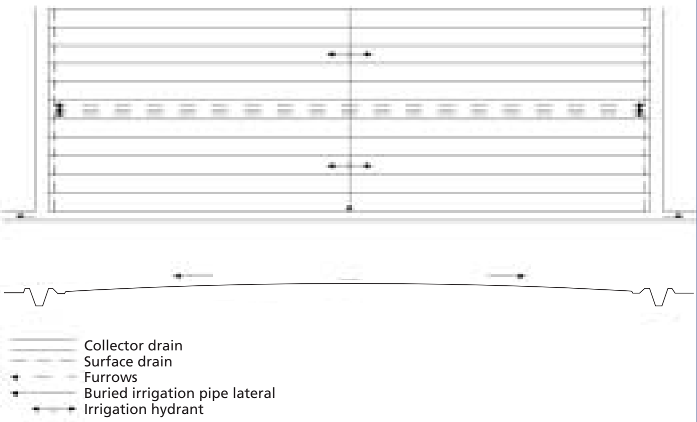
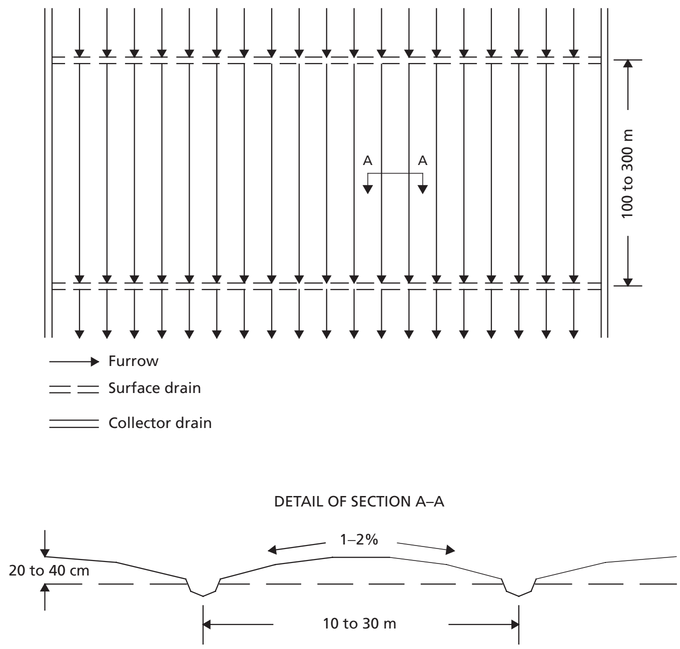


FIGURE 19
Layout of a drainage system with convex beds and furrows



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.

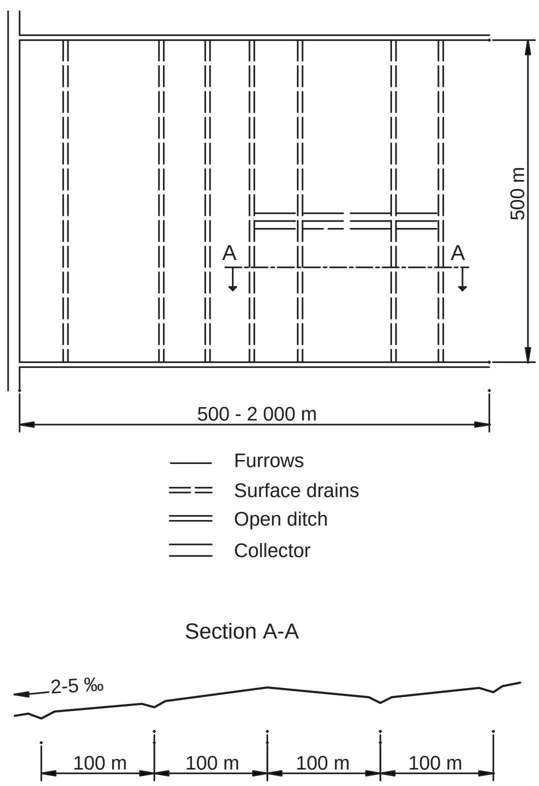
Parallel surface drains at wide spacings

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.

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

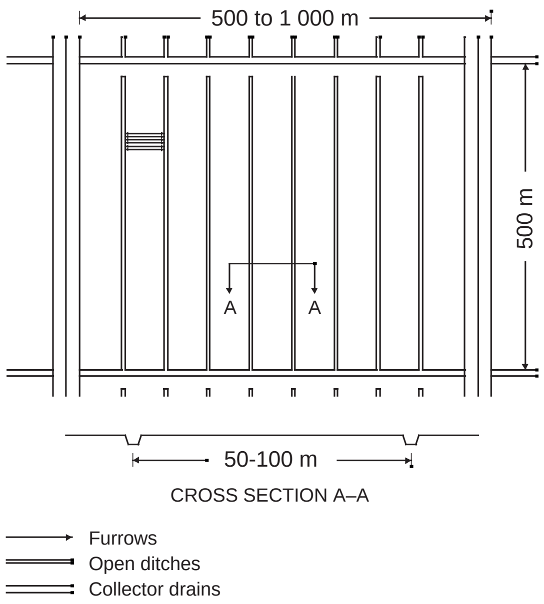
FIGURE 20
Layout of a parallel field drainage system



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.

FIGURE 21
Layout of a drainage system with parallel small ditches



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.

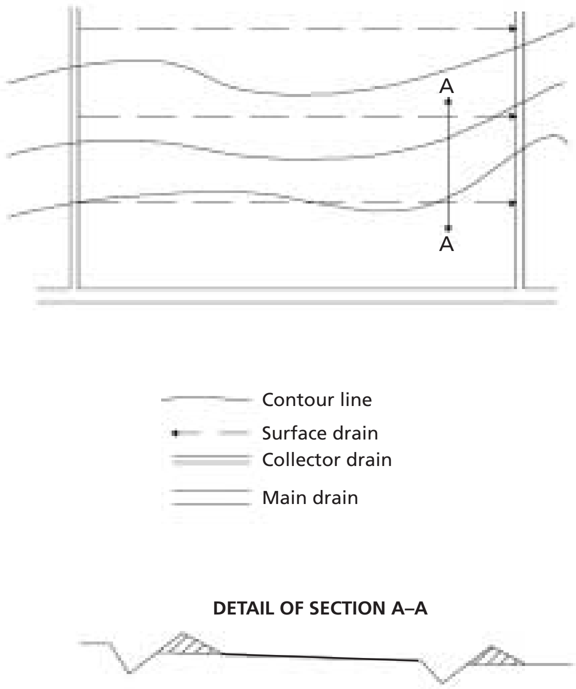
SYSTEMS FOR SLOPING AND UNDULATING LANDS

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.

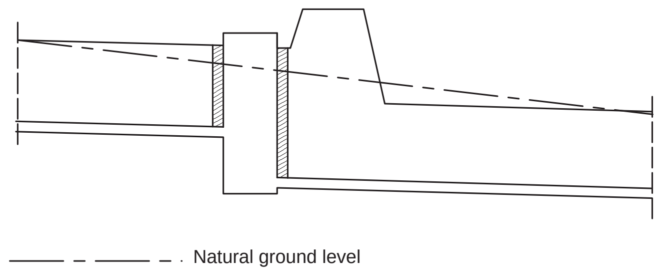
The type of interceptor drains used depends on the relative amounts of runoff and seepage. The former usually dominates, in which case open ditches are needed. Their side slopes, especially the upstream one, must be very flat in order to prevent erosion, and grassed waterways are often useful. A grassed 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

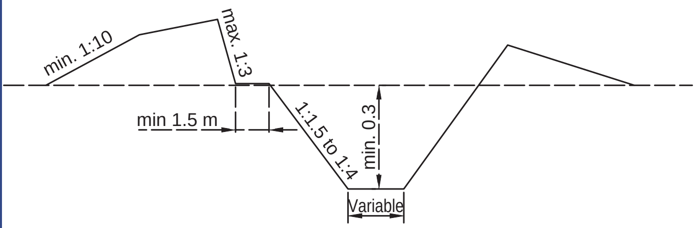


CROSS-SECTIONS OF SURFACE DRAINS

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.

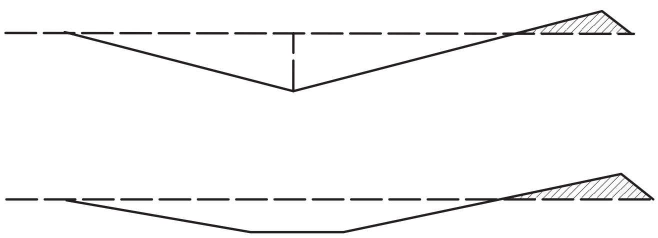
The cross-sections of ditches are usually trapezoidal (Figure 25) although small ones may be V-shaped. Their dimensions vary according to: the expected runoff, the necessity for open water storage, the capacity to be passable for machinery, the risks of bank erosion, and the available means for maintenance.

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

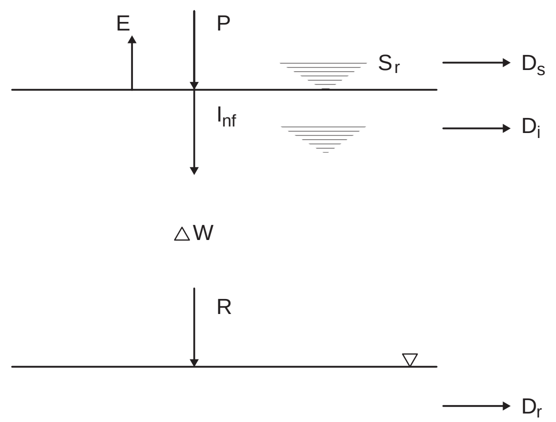
DESIGN DISCHARGES

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):

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

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):

$$P_6 / P_{24} = 0.5\text{–}0.7 \qquad (2)$$

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

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):

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

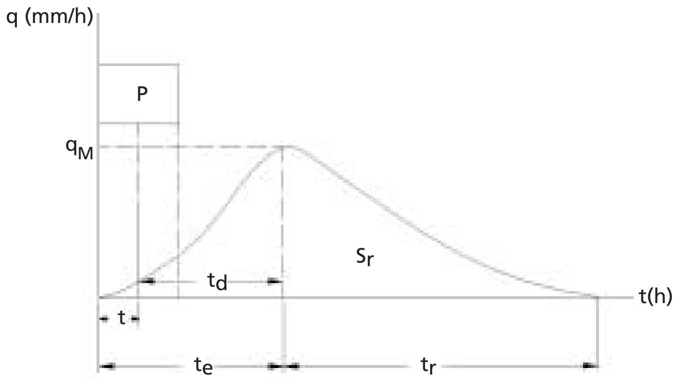
$$P_{T10} / P_{T1} = 1.7\text{–}2.5 \qquad (5)$$

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).

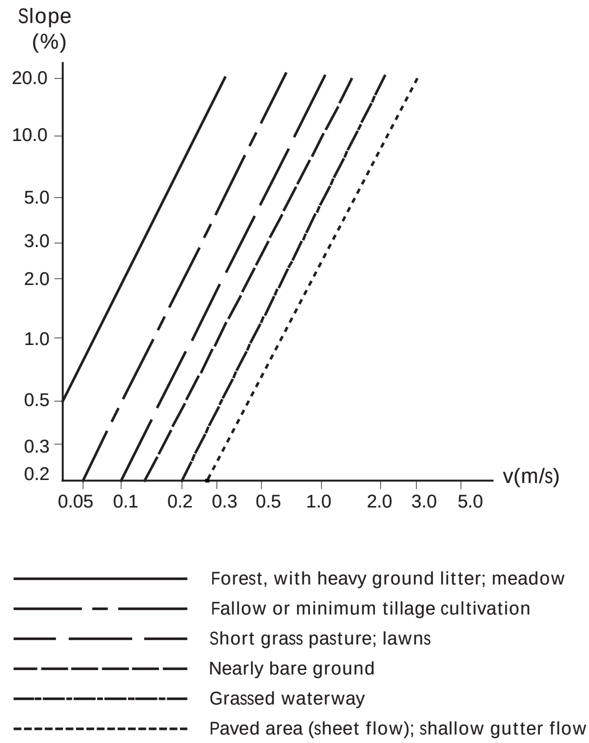
After a certain amount of precipitation (*P*), the specific discharge of surface drainage water at the outlet of the basin (*q*) increases progressively during the elevation time or time to peak (t_e). Once the maximum value (q_M) is reached, the specific discharge decreases progressively during the 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.

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).

FIGURE 29
Flow velocity on the ground surface



Source: Wanielista, 1978.

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

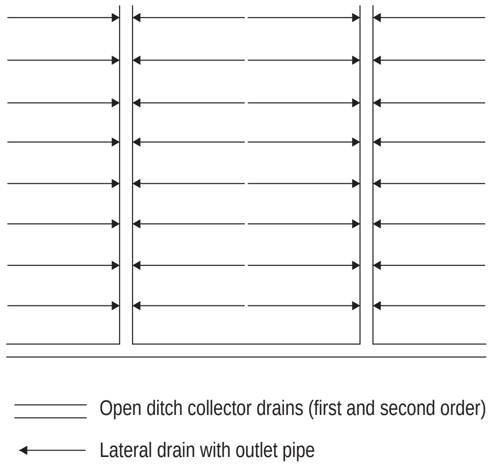
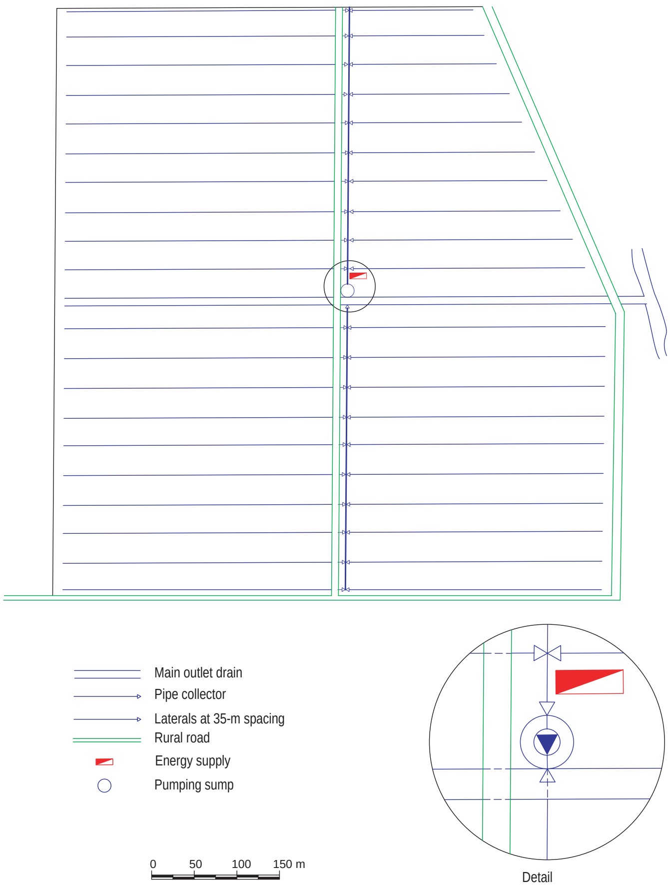


FIGURE 31
Layout of a composite subsurface drainage system with central sump pumping



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

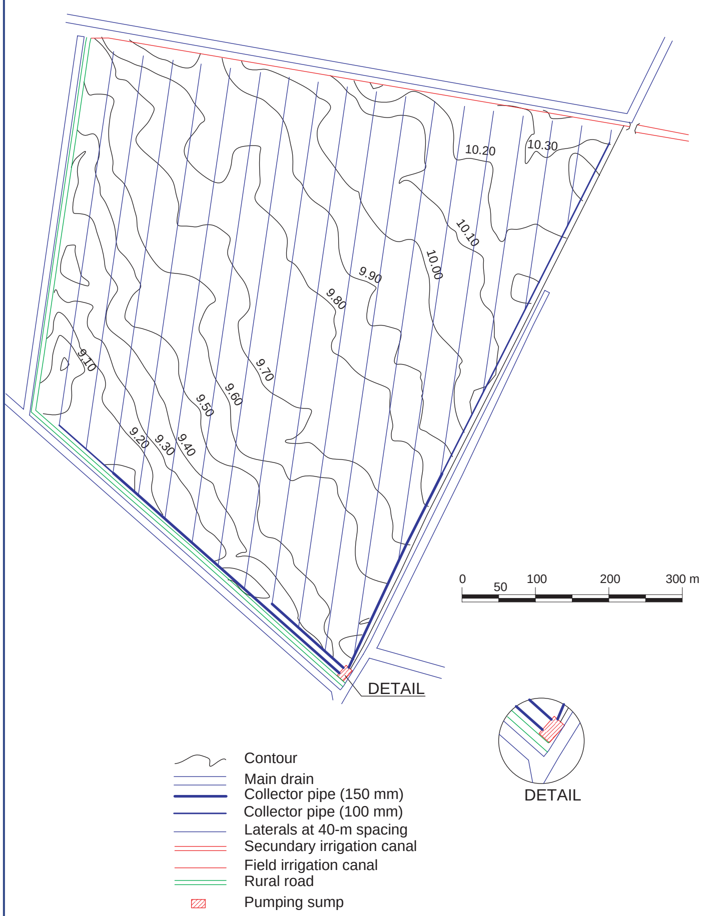
- The depth of field drains is usually greater than in the temperate zones and, consequently, large excavations would be required if open ditches were used as field or collector drains.
- The excess rainfall is generally negligible; as a consequence, drainage rates are low (although often very salty) 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.

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



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.

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

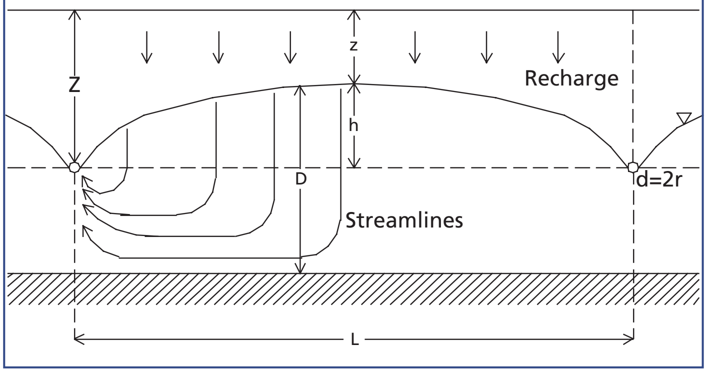
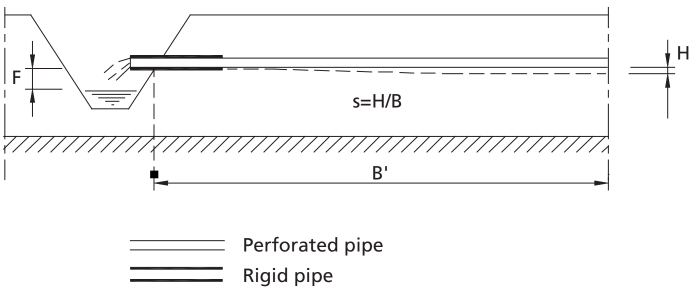


FIGURE 34
Longitudinal section of a lateral drain



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:

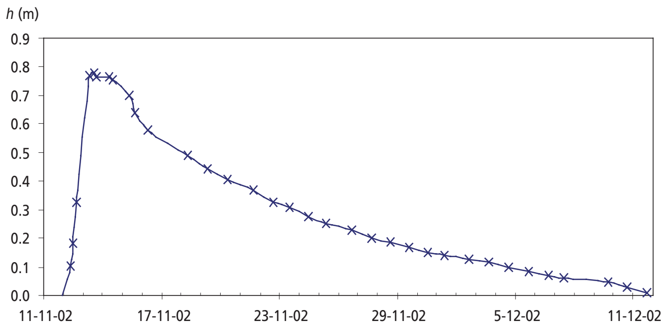
- for horticulture, a lowering after complete inundation of 0.30 m in 4–6 hours;
- for most crops in hot climates, a lowering or 0.30 m in 1 day;
- in cool climates, a lowering of 0.20 m in 1 day.

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.

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

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



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.

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).

FIGURE 36
Outlet of a lateral drain into a collector drain

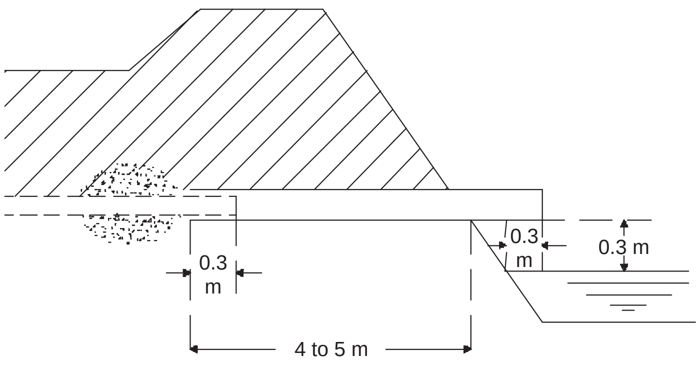
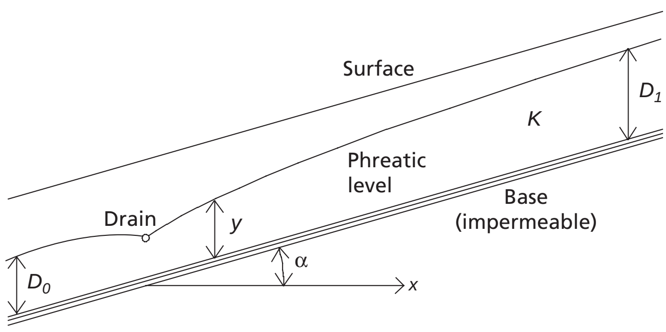


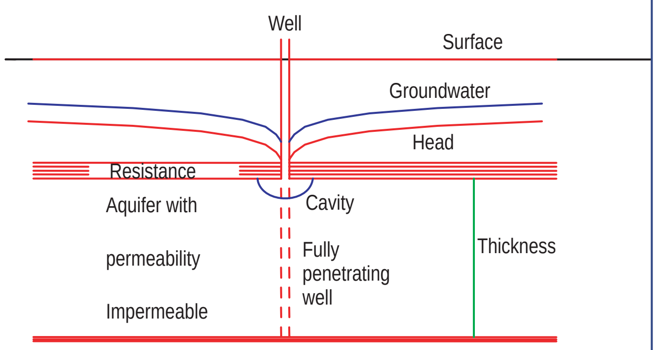
FIGURE 37
Interception drainage on a hill slope



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 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.

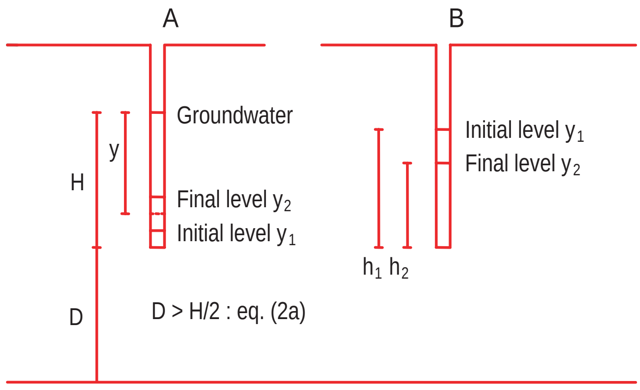
Method

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:

$$K = C\frac{dy}{dt} \qquad (1)$$

in which:

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

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

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

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);

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

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:

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

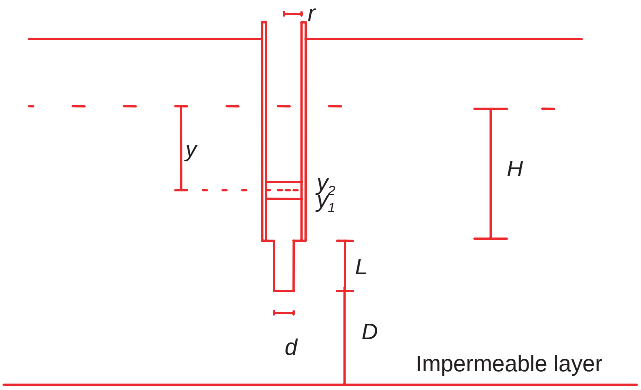
which was derived analytically by integration of the following differential equation:

$$\frac{dh}{h + \frac{r}{2}} = -\frac{2K}{r}dt \qquad (5)$$

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 $H = 8d$:

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

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

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

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:

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

where:

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

$$r_8 = 8d + L/2 \quad (9b)$$

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

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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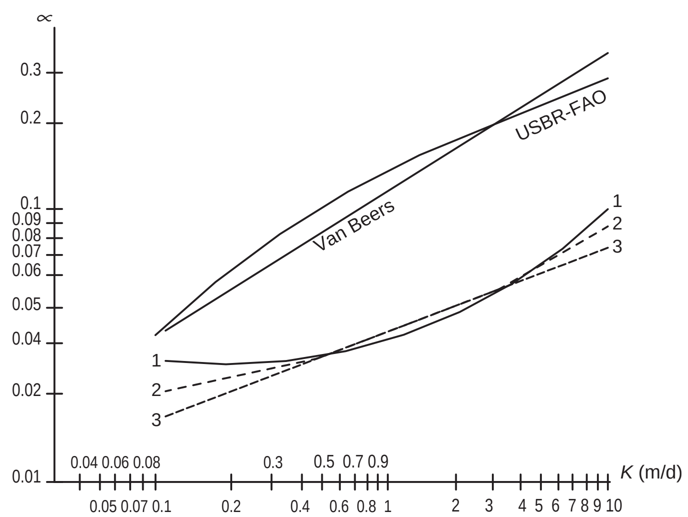
Annex 4
Determining drainable soil porosity

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:

$$R = I_n - \Delta W = (I - E - S_r) - \Delta W \tag{1}$$

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:

$$R = I_n - ET_c = (I - E - S_r) - ET_c \tag{2}$$

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:

$$R = I(1-e_a) - (E + S_r) \qquad (3)$$

being:

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

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:

$$\theta_{fc} - \theta_i \approx \frac{1}{2}(\theta_{fc} - \theta_{wp}) \qquad (5)$$

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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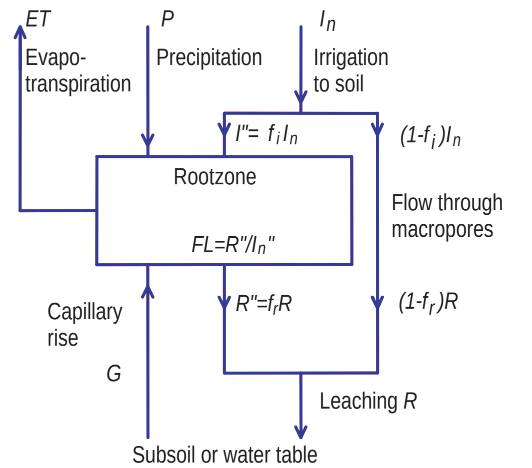
Annex 7
Leaching for salinity control

THE WATER AND SALT BALANCES

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.

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



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:

$$LF = \frac{R}{I_n} \quad (1)$$

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

$$LF = \frac{f_r R}{f_i I_n} \quad (2)$$

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

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

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

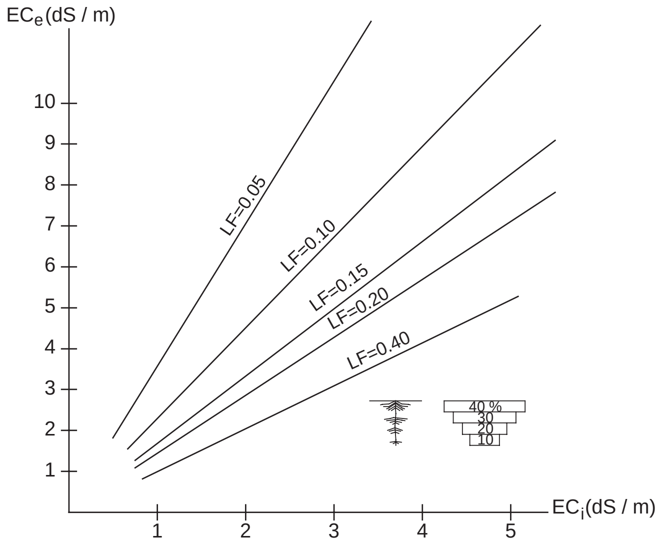
The data in the example in Annex 6 show that farmers apply a net irrigation of about 87 mm during the peak irrigation season, and that about 21 mm of this amount percolates below the rootzone. It was also determined that about 6 percent of the infiltrated water flows directly through cracks without mixing 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).

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



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).

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:

$$\Delta z = z_2 - z_1 = \frac{f_i\, I_n\, EC_i - f_r\, R EC_1}{1 + \dfrac{f_r\, R}{2W_{fc}}} \tag{6}$$

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:

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

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

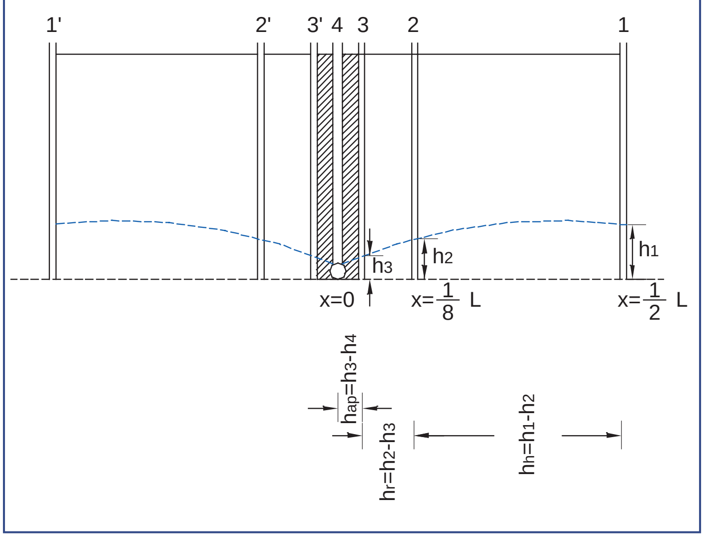
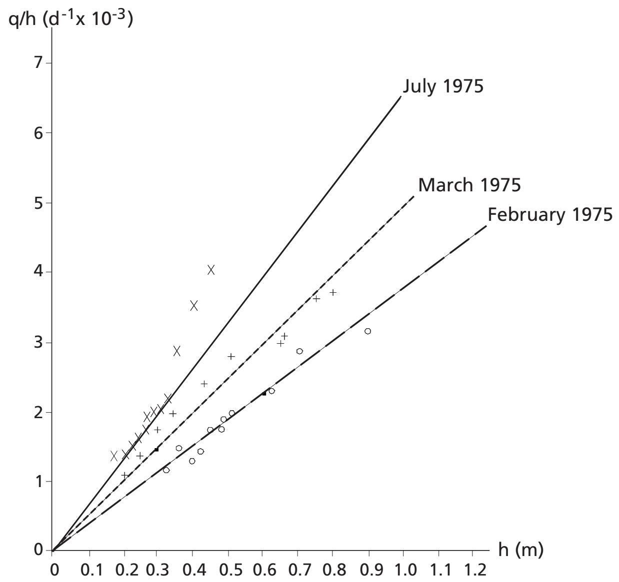


TABLE A8.1
Determination of *KD* values from groundwater-level observations in a drained soil with a sandy substratum

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:

$$q_t = \frac{3.46K}{L^2} h_t^{\ 2} \tag{2}$$

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:

$$tg\gamma = \frac{3.46K}{L^2} \tag{3}$$

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:

$$W_r = \frac{h_r}{qL} \tag{4}$$

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)	$z_{6,5}$	z_0	$h_2 = 1.8 - z_{6\cdot5}$	$h_3 = 1.8 - z_0$	h_r	q	W_r
		(m)	(m)	(m)	(m)	(m)	(mm/d)	(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:

$$\mu = \frac{D_r}{\Delta h} \tag{5}$$

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)

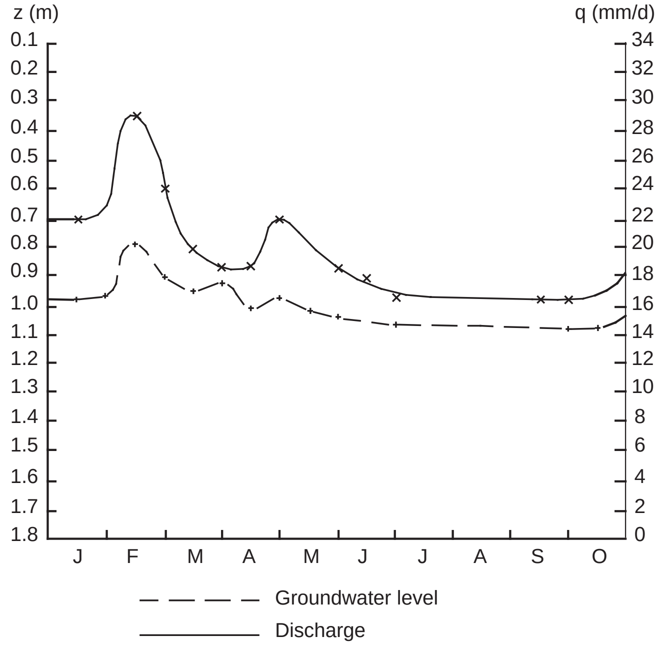
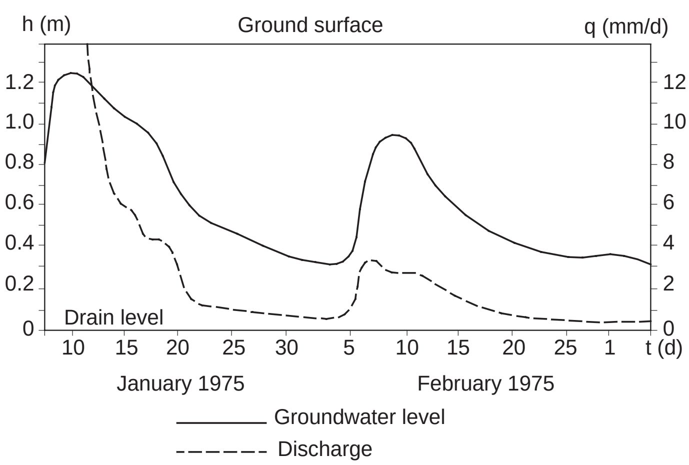
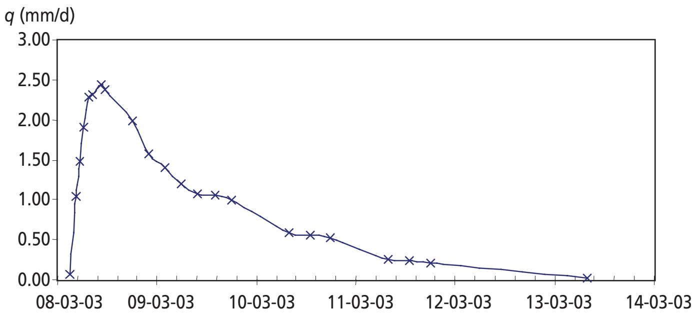


FIGURE A9.2
Drawdown of a water table after irrigation to reclaim saline soils



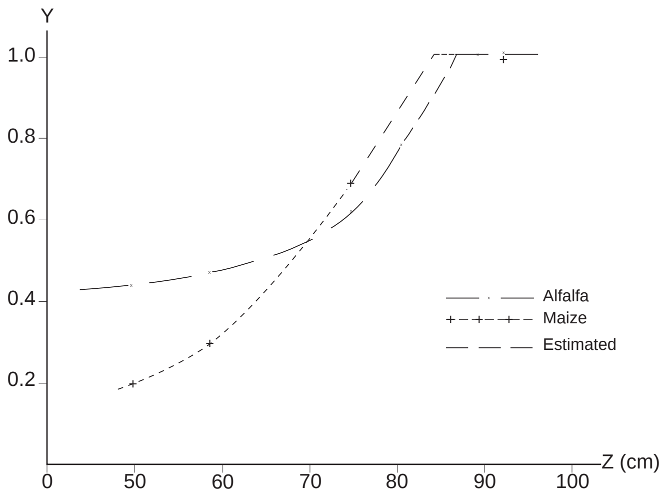
Source: Adapted from Martínez Beltrán, 1978.

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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Annex 10
Calculations regarding elements of the main drainage system

OPEN CHANNELS AND THEIR CROSS-SECTIONS

For open channels, Manning's formula is widely used:

$$v = K_m R^{2/3} s^{1/2} = \frac{1}{n} R^{2/3} s^{1/2} \quad (1)$$

being:

$R = A / u$;

$A = by + \alpha y^2$;

$u = b + 2y\sqrt{1+\alpha^2}$;

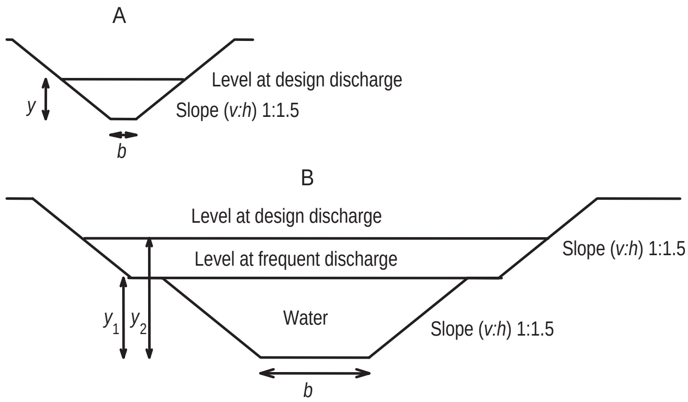
$v = Q / A$: average flow velocity over the cross-section A;

where (see Figure A10.1):

A = cross-sectional area of flow (m^2);
b = bottom width (m);
K_m = $1/n$ = roughness coefficient ($m^{1/3}/s$);
n = $1/K_m$ roughness coefficient ($s/m^{1/3}$);
Q = discharge (m^3/s);
R = hydraulic radius (m);
s = hydraulic gradient (-);
u = wetted perimeter (m);
v = average flow velocity (m/s);
y = water depth (m);
α = coefficient in side slope (v:h) 1:α.

The roughness coefficient K_m depends on factors such as the irregularities of the drain bed and side slopes, amount of vegetation, irregular alignment and hydraulic radius of the open drain. Values range from 50 for large channels in bare earth, to 20 for open drains two-thirds choked with vegetation, to less than 10 for entirely choked ones. Table A10.1 lists design values for normally maintained channels. For the coefficient K_m, the following equations for such open waterways (with some vegetation) are used, in which it is supposed that the channels have been cleaned before the onset of the wet season (so that they are in a reasonable condition).

FIGURE A10.1
Cross-sections of an open ditch (A) and a large channel (B)



If $y < 1$ then $K_m = 32 y^{0.4} - 2.0$ (2a)

else $K_m = 30 y^{0.125}$ (2b)

TABLE A10.1
Design parameters for open drains

Drain size	Water depth y (m)	Ratio $b{:}y$	Soil texture	Manning's K_m ($m^{1/3}s^{-1}$)	n
Small	< 0.75	1–2	sandy	20	0.050
			clayey	15	0.067
Medium	0.75–1.5	2–3	sandy	30	0.033
			clayey	20	0.050
Large	> 1.5	3–4		40–50	0.020–0.025

Sources: Adapted from ILRI, 1964; and from Smedema, Vlotman and Rycroft, 2004.

TABLE A10.2
Maximum average water velocity and bank slopes for open ditches

Soil type	v_M (m/s)	Bank $v{:}h$
Heavy clay	0.60–0.80	1:0.75 to 1:2
Loam	0.30–0.60	1:1.5 to 1:2.5
Fine sand	0.15–0.30	1:2 to 1:3
Coarse sand	0.20–0.50	1:1.5 to 1.3
Tight peat	0.30–0.60	1:1 to 1:2
Loose peat	0.15–0.30	1:2 to 1:4

Source: Adapted from ILRI, 1964.

The ratio of bottom width (b) to water depth (y) should remain preferably within certain limits (Table A10.1). Where this ratio is known, the required cross-section can be calculated with the above formulae.

The average flow velocity v over the cross-section should not be so high that erosion of the bottom or banks occurs. Table A10.2 gives some values for the maximum average flow velocities and also the recommended side slopes for trapezoidal cross-sections.

For safety, it is advisable to check the behaviour of the system at a larger discharge. At 1.5–2 times design discharge, some inundation may be allowed to occur in low places, but disasters and extensive inundation should not occur.

Depth and freeboard

The depth of a drainage channel equals:

$$Z_c = F + y \tag{3}$$

where:
F = freeboard (m);
y = water depth (m);
Z_c = collector depth below soil surface (m).

The freeboard F must be such that at design discharge the outlets of any subsurface drains, including pipe collectors, are just above or equal to the drainage-channel water level, although a slightly higher water level can be tolerated temporarily. This usually leads to water levels of 1–2 m below the land surface at design discharge. In arid regions, drain outlets should remain above the water level, although they may become temporarily submerged after an infrequent rainfall has caused large surface runoff volumes to the open drain.

Wind effects

Similar to shallow seas, long canals (> 10 km) may be subject to storm surges when strong winds blow in the direction of the waterway. However, in most situations, such wind effects are negligible.

An estimate for storm surges at sea, but also for all kinds of waterways, is:

$$\frac{dh}{dx} = \Phi \frac{v^2}{gy} \quad \text{or} \quad \Delta h = \Psi \frac{v^2}{y} B \tag{4}$$

where:
B = length of waterway, in wind direction (km);
v = wind velocity (m/s);

g = acceleration of gravity (m/s²);
h = head (m);
x = distance (m);
y = water depth (m);
Δh = head difference along canal, caused by wind (m);
Φ ≅ 4.10^{-6} = coefficient;
Ψ ≅ 0.0004 = coefficient.

For seas and estuaries, the calculation must be numerical, using sections of the same, or almost the same, depth.

Normal flow and inundation

Where the water level downstream is lower than the upstream water level of an outflow, channel flow occurs. Depending on the conditions, this channel flow may be streaming or shooting. This is governed by the Froude–Boussinesq number:

$$Fr = \frac{v}{\sqrt{gy}} \qquad (5)$$

where:
Fr = Froude–Boussinesq number;
g = 9.81 = acceleration gravity (m/s²);
y = water depth (m);
v = flow velocity (m/s).

For streaming water, it is required that $Fr < 1$; while for $Fr > 1$, shooting occurs.

Streaming water is supposed to obey Manning's formula (Equation 1).

If the water level downstream becomes higher than the land surface, overflow and inundation occur.

Backwater effects

Backwater curves occur near the downstream end of a channel, where it joins other watercourses with a higher water level or within the reach with a backwater curve effect upstream of weirs. Upstream, the water will reach a constant equilibrium depth in accordance with a given flow. However, near the downstream end, the water level will come under the influence of the fixed downstream level and form a curve upwards or downwards (Figure A10.2) depending on whether this level is higher or lower than the water level corresponding with the upstream equilibrium depth. Complications arise when the land is inundated or when the channel overflows.

The program BACKWAT is based on these considerations. This program calculates the equilibrium depth by iteration. The calculations start at the downstream end, where the water level is given. They are numerical, with steps in water depth of a given size. The water depth diminishes inland if the curve is convex, and increases inland if concave (Figure A10.2). In the latter case, overflow may occur upstream.

FIGURE A10.2
Convex and concave backwater curves

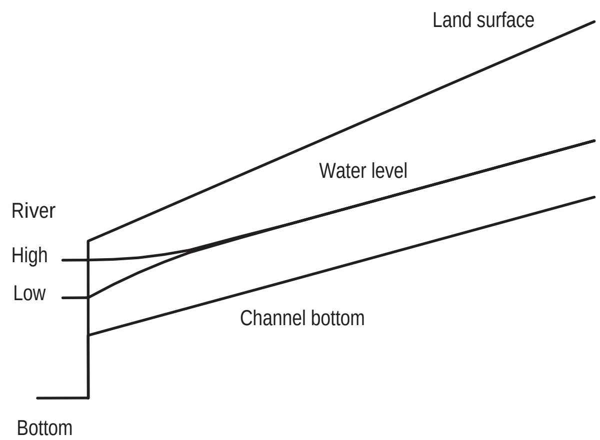
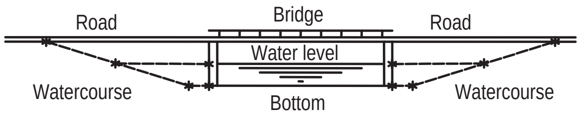


FIGURE A10.3
Bridge (non-obstructing)



If shooting occurs, the program terminates.

CULVERTS AND BRIDGES

For culverts, there are two types of head losses, caused by:
- convergence of streamlines at the entrance – these losses are not recovered at the exit;
- friction losses, occurring at the walls of culverts.

For the former, laws for flow through openings apply. The hydraulic section of a culvert can be calculated using:

$$Q = \mu A \sqrt{2g\Delta h} \quad (6)$$

where:
A = area of the hydraulic section (m^2);
g = 9.8 m/s^2 is the gravity acceleration;
Q = design discharge (m^3/s), preferably increased by a safety factor;
μ = coefficient that depends on the shape of the entrance and at the exit;
Δh = head loss along the culvert (m).

The design discharge is often taken some 25–50 percent higher than for the upstream drainage channel. This is because the flexibility of culverts to accommodate for higher flows without causing structural damage is less than for open waterways. The values of μ are about 0.7 for long culverts (20–30 m) and 0.8 for short culverts (< 10 m) (ILRI, 1964). Head losses of 5 cm for small structures and 10 cm for large ones are generally taken (Smedema, Vlotman and Rycroft, 2004). In order to calculate the cross-section of the structure, in addition to the wet section A, a minimum of 10 cm of clearance should be added.

The friction losses in culverts are of minor importance for the usual short passages under rural roads. For longer culverts, the head losses for friction must be added. Manning's formula is often used, with a K_m of 60–70 for smooth and 30–40 for corrugated walls.

Bridges are often constructed in such a way that the watercourse passes freely underneath, in which case they have no influence (Figure A10.3). If the channel is narrowed by the bridge, Equation 6 may be used, with μ = 0.8–0.9 (Smedema, Vlotman and Rycroft, 2004). Friction losses can be ignored as the influence of the short length of the narrow passage is small.

WEIRS AND DROP STRUCTURES

The width of freely discharging rectangular weirs and drop structures is calculated with the formula:

$$Q = \frac{2}{3}\mu b h \sqrt{\frac{2}{3}gh} = 1.7\mu b h^{3/2} \quad (7)$$

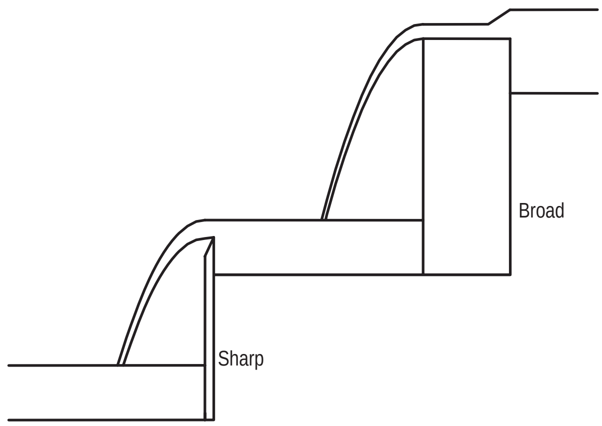
where:
b = crest width (m);
g = 9.8 m/s^2 is the gravity acceleration;
h = head above the crest level (m);
Q = discharge (m^3/s);
μ = contraction coefficient.

For submerged discharge the following equation may be used:

$$Q = \mu b h_2 \sqrt{2g(h_1 - h_2)} \qquad (8)$$

where:
h_1 = upstream water head (m);
h_2 = downstream head (m);
$\Delta h = h_1 - h_2$ = available head (m).

FIGURE A10.4
Crest form of weirs



The values of the coefficients in Equation 7 and 8 are mostly determined by the width/shape of the weir crest (broad or sharp, as shown in Figure A10.4) and by the nature of the approach flow (degree of streamline contraction and entry turbulence). For similar weirs, the μ values are in principle the same for both equations. Values for semi-sharp crested weirs commonly used in drainage channels (e.g. stop-log weirs) are generally in the order of 1.0–1.1 (Smedema, Vlotman and Rycroft, 2004). For sharp-crested weirs, the higher values of μ should be used.

OUTLET STRUCTURES

Sluices and flap gates

The discharge rate through a sluice or flap gate can be calculated with Equation 6, being in this case b the width of the sluice and μ a coefficient from 0.9 to 1.1. The water depth h_2 should be increased by 3.5 percent if the sluice discharges directly into the sea, because of the heavier saltwater outside (Smedema, Vlotman and Rycroft, 2004). The outside water heights vary with tides or floods, so that at high levels discharge is not possible and water must be stored inside. Therefore, the calculations must be numerical, in time steps, for water level and storage conditions that are typical for the location involved.

Pumping stations

The capacity of a pumping station is determined by the total discharge from all sources: rainfall, irrigation excess, seepage, municipal and industrial wastewaters, etc. However, it is not simple to estimate the simultaneous occurrence of all these events. In contrast to open watercourses, pumps have a rather inflexible capacity, so that some reserve is usually added.

A pumping station often has to run at full capacity for short periods only. Most of the time it has to remove the "base flow" from more permanent sources, of which seepage and tail-end losses from irrigation systems are the main ones. More than the strongly variable inputs from rainfall, these flows determine the number of pumping hours per year and, consequently, the costs of operation.

In order to cope with the variable capacity needed in different periods, more than one pump is usually installed, of which one to remove the base flow and one or more to cope with larger discharges and the design discharge at critical periods.

In order to select the most appropriate capacity arrangement and type of pump, some design parameters should be calculated, namely: the base, usual and maximum discharge, the lift and the dynamic head, and the power requirement.

The lift equals the static difference between inside and outside water. The dynamic head may be calculated using:

$$h = h_s + \Delta h + \frac{v_d^{\,2}}{2g} \tag{9}$$

where:
g ≈ 9.8 m/s^2;
h = total head (m);
h_s = lift or static head (m);
v_d = flow velocity at the outlet of the delivery pipe (m/s);
Δh = total head loss in the suction and delivery pipes (m).

Consideration should be given to the head-increasing effect of choking of trashracks that usually protect the inlet section of drainage pumping stations from the entrance of floating debris such as mown aquatic weed, plastic, and branches, if timely cleaning of these racks is not secured.

The power requirement may be calculated using:

$$P = \frac{\rho g Q h}{\eta_t \, \eta_p} \tag{10}$$

where:
h = total head (m);
P = power required (kW);
Q = discharge rate (m^3/s);
η_t and η_p are the transmission (0.90–0.95) and pump efficiencies, respectively;
p = density of water ≈ 1 000 kg/m^3.

The η_p values can vary for axial pumps from 0.65 for 1-m lift to 0.80 for 2.5–3.0-m lift; for radial pumps from 0.6 for 1-m lift to 0.80–0.85 for lifts of more than 4.0 m (Smedema, Vlotman and Rycroft, 2004); and Archimedes screws may have an efficiency of 65–75 percent (Wijdieks and Bos, 1994).

Some correction factors may be also considered in Equation 10 in order to take account of the elevation of the site and safe load (Smedema, Vlotman and Rycroft, 2004).

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Wijdieks, J. & Bos, M.G. 1994. Pumps and pumping stations. *In* H.P. Ritzema, ed. *Drainage principles and applications*, pp. 965–998. 2nd edition. ILRI Publication 16. Wageningen, The Netherlands, ILRI.

Annex 11

Example of the batch method for flat lands

The batch method for flat lands is described by means of an example for water distribution from an extreme rainfall, with data from the Ebro Delta in northeast Spain, where the climate is Mediterranean and extreme rainfalls are common in autumn. Although the rainfall period extends for several consecutive days, an exceptional rain of about 100 mm may fall in one day for a return period of 5 years. The following days are rainy but the amount of precipitation decreases progressively. These autumn rainfalls may affect irrigated rice fields during harvesting operations. On the left bank of the Ebro Delta, flat areas of 2 200–3 000 ha are served by drainage pumping stations managed by the local water users association. The farm in this example is served by a station with four Archimedes screws, each able to remove 9.5 mm/d, so that the maximum total capacity of the pumping station is 38 mm/d. During the irrigation period, only one of the pumps usually discharges about 5 mm/d, mainly surface drainage water from the rice fields. Table A11.1 shows the results of calculations based on the above data.

Although the rice fields are drained before harvesting by the existing surface drainage systems, the soil is almost saturated and storage can be considered negligible. However, about 25 mm can be stored in the channel system. On rainy days in autumn, evaporation can remove about 3 mm/d from the area.

It is assumed that, on the first day, the full pumping capacity of the station has to be started, evaporation is negligible and, therefore, only about 25 mm can be removed. The excess 75 mm cannot be stored in the soil and in the channels, so inundation occurs in the rice fields. In the following days, the four available screws work day and night. Subsequently, the inundation storage and the water in the channels are drained. These conditions are suitable for the rice field requirements.

However, in some areas of the Ebro Delta, vegetables are grown in fields with surface and subsurface drainage facilities. Heavy autumn rainfalls may affect crops such as tomato and lettuce severely. Table A11.2 shows the water distribution of extreme rainfalls for a 10-year return period with the existing shared pumping facilities. It is assumed that in these irrigated lands where the groundwater table is controlled by a subsurface drainage system, the soil becomes completely saturated after storing about 50 mm.

Even with all four pumps working fully, inundation cannot be avoided on two days. In addition to this, pumping should continue to lower the water level in the channels in order to allow the subsurface drainage system to drawdown the water level, at least

TABLE A11.1
Water balance of a rice field in a flat area

Day	Rainfall	Evaporation	Pumped water	Excess rainfall	Storage in:			
					Soil	Channels	Inundation	Total
	(mm/d)							
1	100	-	25	75	-	25	50	75
2	21	3	38	55	-	25	30	55
3	4	3	38	18	-	18	-	18
4	-	3	15	-	-	-	-	-
5	-	3	5	-	-	-	-	-

TABLE A11.2
Water balance of a vegetable field in a flat area

Day	Rainfall	Evaporation	Pumped water	Excess rainfall	Storage in:			
					Soil	Channels	Inundation	Total
	(mm/d)							
1	125	-	25	100	50	25	25	100
2	29	3	38	88	50	25	13	88
3	4	3	38	51	50	1	-	51
4	-	3	30	18	18	-	-	18
5	-	3	15	-	-	-	-	-

TABLE A11.3
Example of water balance for a 6-hour period

Hour	Rainfall	Evaporation	Pumped water	Excess rainfall	Storage in:			
					Soil	Channels	Inundation	Total
	(mm/h)							
1	53	-	1	52.0	20	15	17.0	52.0
2	27	-	1.6	77.4	35	20	22.4	77.4
3	14	-	1.6	89.8	45	25	19.8	89.8
4	6	-	1.6	94.2	50	25	19.2	94.2
5	3	-	1.6	95.6	50	25	20.6	95.6
6	1		1.6	95.0	50	25	20.0	95.0

25 cm in one day. Inundation for two days could be tolerated by tomato and lettuce in the Ebro Delta, providing that they are grown on beds between surface drainage furrows. However, as the pumping requirements are higher than for standard rice field needs, individual pumping stations may be needed in farms with surface and subsurface drainage systems where vegetables are grown jointly with rice (as the actual shared pumping facilities were designed mainly for covering rice field requirements).

The pumping capacity should also be increased if the critical period is less than 24 hours as it is frequently needed to cultivate more sensitive crops. If heavy rain falls in the first three hours, soil storage may be limited by soil infiltration, which is usually highest at the beginning. However, it soon decreases, becoming later almost constant until the soil is saturated completely. In the example of Table A11.3, water distribution is shown with pumping capacity and channel storage similar to the previous example.

In this example, inundation reaches its maximum value after about 2 hours. After this time, it decreases slightly, but stagnation occurs in the following hours. If the critical period is about 6 hours and the excess rainfall should be removed during this time interval, the pumping capacity should be increased substantially or less sensitive crops should be cultivated. Consequently, in certain areas of the Ebro Delta, where horticultural crops are grown, in addition to the pumping stations for subsurface drainage water, independent pumping stations with a higher capacity discharge surface drainage water during the critical periods of heavy rainfall.

Annex 12
Cypress Creek formula

PRINCIPLES

The Soil Conservation Service (now called the Natural Resource Conservation Service) of the United States Department of Agriculture developed a simple formula called the Cypress Creek equation (NRCS, 1998):

$$Q = qA^{5/6} \qquad (1)$$

where:

Q = design discharge (m^3/s) – not peak discharge as some flooding can take place;

q = $0.21 + 0.00744P_{24}$ = drainage coefficient related to the drainage area and the magnitude of the storm (cubic metres per second per square kilometre) (Ochs and Bishay, 1992);

P_{24} = 24-hour excess rainfall (mm) – the excess rainfall can be calculated with the CN graph, but considering that the CN method was developed for free drainage conditions; for storm periods longer than a day, the total rainfall excess is divided by the length of the storm period in days (Ochs and Bishay, 1992);

A = area served by the drain (square kilometres).

The equation was developed for the eastern portion of the United States of America. It is basically applicable for humid flat lands covering less than 5 000 ha, with conditions similar to the areas for which was developed.

Table A12.1 shows drainage coefficients for the east of the United States of America.

TABLE A12.1
Typical drainage coefficients for humid areas

	Drainage coefficient ($m^3s^{-1}km^{-2}$)
Coastal plain cultivated	0.59
Delta cultivated lands	0.52
Cool northern cultivated	0.48
Coastal plain pasture	0.39
Cool northern pasture	0.33
Delta and coastal rice lands	0.30
Semi-humid northern cultivated	0.26
Semi-humid southern range lands	0.20
Coastal plain woodlands	0.13

Source: Adapted from ASAE-EP 407.1, 1994.

REFERENCES

ASAE-EP 407.1. 1994. Agricultural drainage outlets - open channels. *In: American Society of Agricultural Engineers book of standards*, pp. 728–733. St. Joseph, USA.

Natural Resource Conservation Service (NRCS). 1998. Water management (drainage). *Chapter 14 of Part 650 Engineering Field Handbook*. Washington, DC. 160 pp.

Ochs, W.J. & Bishay, B.G. 1992. *Drainage guidelines*. World Bank Technical Paper No. 195. Washington, DC. 186 pp.

Annex 13
Statistical analysis of measured flows

PRINCIPLES

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. For example, the occurrence probability can be calculated with the following formula:

$$P = \frac{m}{N+1} \tag{1}$$

where:
P = probability;
T = $1/P$ = return period (years);
m = order number in the data series;
N = number of total data available.

Example

Equation 1 has been applied in the example shown in Table A13.1.

With the data of Table A13.1, the maximum discharge for a return period of up to 20 years can be determined (98.3 m^3/s in this case), which is sufficient to design the main drainage system. Where a higher return period is required in order to design special structures, the design discharge can be estimated by extrapolation, once the

TABLE A13.1
Frequency analysis of drainage flows (for *N* = 19)

Year	Q_M (m^3/s)	*m*	Q_M (m^3/s)	*m*	$P = \frac{m}{N+1}$	*T = 1/P* years
1967	85.1	4	98.3	1	0.05	20
1968	50.1	17	90.2	2	0.10	10
1969	48.2	18	85.3	3	0.15	
1970	68.3	10	85.1	4	0.20	5
1971	60.4	13	80.7	5	0.25	
1972	55.2	14	80.6	6	0.30	
1973	80.7	5	78.4	7	0.35	
1974	90.2	2	78.3	8	0.40	
1975	85.3	3	76.7	9	0.45	
1976	61.3	12	68.3	10	0.50	2
1977	98.3	1	61.5	11	0.55	
1978	78.4	7	61.3	12	0.60	
1979	80.6	6	60.4	13	0.65	
1980	36.7	19	55.2	14	0.70	
1981	50.2	15	50.2	15	0.75	
1982	61.5	11	50.2	16	0.80	
1983	50.2	16	50.1	17	0.85	
1984	78.3	8	48.2	18	0.90	
1985	76.7	9	36.7	19	0.95	1

Source: Adapted from Smedema, Vlotman and Rycroft, 2004.

available data are plotted on a probability paper, for example by using the normal distribution. However, this type of calculation is based on historical data, and runoff may change with changes in land use.

REFERENCES

Smedema, L.K., Vlotman, W.F. & Rycroft, D.W. 2004. *Modern land drainage. Planning, design and management of agricultural drainage systems*. Leiden, The Netherlands, A.A. Balkema Publishers, Taylor&Francis. 446 pp.

Annex 14
Unit hydrograph

PRINCIPLES

This method, developed by Sherman (1932), is based on the proportionality principle: the surface runoff hydrograph produced by certain amount of rainfall (*P*) can be obtained from the hydrograph of other storm of equal duration (*P'*) by multiplying the ordinates of the latter hydrograph by the following conversion factor:

$$a = \frac{S_r}{S'_r} \tag{1}$$

where:

a = conversion factor;

S_r = amount of surface runoff produced by precipitation *P* (mm);

S'_r = amount of surface runoff produced by precipitation *P'* (mm).

This method is also based on the concept that the base length (*t*) of a hydrograph depends on the duration of the storm, but is independent of the amount of rainfall and surface runoff, as shown in Figure A14.1. The recession time ($t - t_d$) is almost constant. This is because it only depends on the physical characteristics of the basin.

For practical applications, it is advisable to convert the available hydrographs to unit hydrographs, namely, hydrographs for precipitations of 1 or 10 mm. Thus, for the project basin, a series of unit hydrographs can be obtained for different rainfall durations. In order to determine the hydrograph for the design rainfall, the unit hydrograph with a time basis similar to the design rainfall is selected.

FIGURE A14.1
Proportionality principle of the unit hydrograph

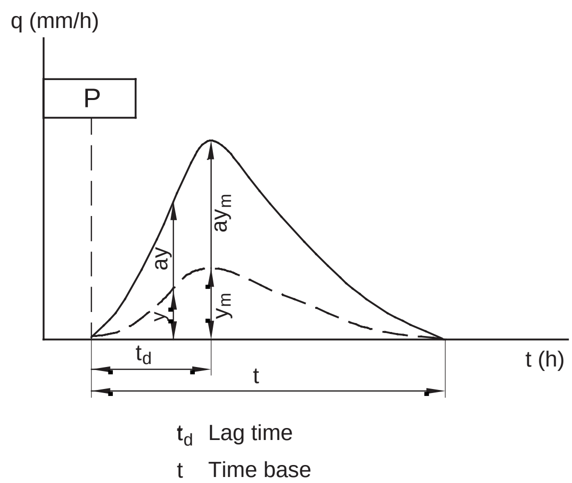
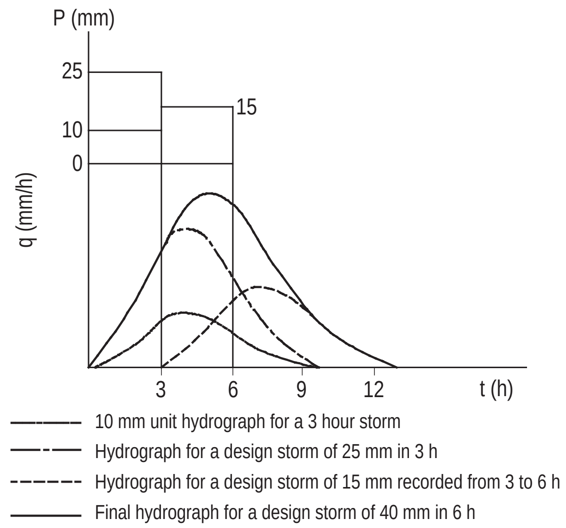


FIGURE A14.2
Example of the unit hydrograph method



Example

In Figure A14.2, the hydrograph for the surface runoff produced by a rainfall of 40 mm accumulated in 6 hours, of which 25 mm was accumulated in the first 3 hours, has been determined from the unit hydrograph available for a rainfall of 10 mm in 3 hours. It is assumed that all rain becomes surface runoff.

The hydrograph for the first 3 hours is obtained from the 10-mm unit hydrograph by applying a conversion factor ($a = 2.5$). For the following 3-hour period, a conversion factor ($a = 1.5$) is used. The final hydrograph is obtained by superimposing both hydrographs. It can be observed that the peak discharge will be produced 5 hours after the beginning of the storm.

REFERENCES

Sherman, L.K. 1932. Streamflow from rainfall by the unit-graph method. *Eng. News Rec.*, 108: 501–505.

Annex 15
Rational formula

PRINCIPLES

The rational method assumes that, in small agricultural basins, the maximum flow of surface water in the outlet is for a rainfall with a duration equal to the concentration time. Then, the maximum discharge depends on the rainfall intensity, the surface area and the hydrological conditions of the basin:

$$Q_M = \frac{CIA}{360} \quad (1)$$

where:

Q_M = maximum discharge for a return period equivalent to the design rainfall (m^3/s);
C = coefficient for surface runoff;
I = rainfall intensity during the concentration time (mm/h);
A = area of the basin (ha).

For the return period selected, rainfall intensity is assumed: (i) constant during the time interval considered; and (ii) equal to the ratio between the accumulated rainfall and the concentration time. Where only the amount of rainfall in 24 hours is known, the value of the precipitation accumulated in the concentration factor can be estimated, first by using an appropriate coefficient for the 6-hour rainfall (P_6/P_{24} = 0.5–0.7), and then with the coefficients of the rainfall distribution model described in Chapter 6 of the main text.

SURFACE RUNOFF COEFFICIENT

The runoff coefficient can be estimated directly through the indicative values of the Soil Conservation Service (SCS, 1972) shown in Table A15.1.

Example

The rational method has been applied to estimate the maximum discharge of surface water at the outlet (point D) of a farm of 85 ha shown in Figure A15.1.

In order to estimate the concentration time at point D, three sections have been considered from the most distant point from the outlet (point A): section AB (furrows), section BC (open collector drain), and section CD (the main drain).

Assuming values of the water velocity of 0.15 and 0.35 m/s along the furrows and the open ditches, respectively, Table A15.2 shows the concentration time t_c for each section as calculated using:

$$t_c = \sum_{i=1}^{n} \frac{l_i}{v_i} \quad (2)$$

where:

FIGURE A15.1
Example of drained farm with a system of furrows and open ditches

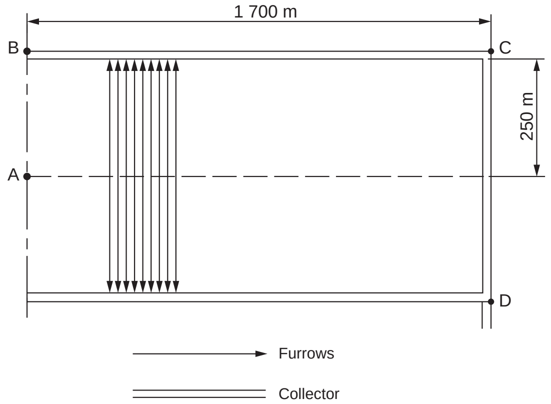


TABLE A15.1
Indicative values of the surface runoff coefficient for agricultural land

Land use	Slope (%)	Soil infiltrability		
		High	Medium	Low
Arable land	< 5	0.30	0.50	0.60
	5–10	0.40	0.60	0.70
	10–30	0.50	0.70	0.80
Pasture	< 5	0.10	0.30	0.40
	5–10	0.15	0.35	0.55
	10–30	0.20	0.40	0.60
Forest	< 5	0.10	0.30	0.40
	5–10	0.25	0.35	0.50
	10–30	0.30	0.50	0.60

Source: Adapted from Smedema, Vlotman and Rycroft, 2004.

TABLE A15.2
Estimates of the concentration time

Section	Length (m)	Slope (%)	Difference of elevation (m)	Water velocity (m/s)	t_c (h)
AB	250	0.10	0.25	0.15	0.46
BC	1 700	0.15	2.55	0.35	1.35
CD	500	-	-	0.35	0.40
AD	2 450		2.80		2.21

t_c = concentration time (s);
l_i = distance of section i (m);
v_i = average water velocity in section i (m/s).
The concentration time can also be estimated using the Kirpich formula:

$$t_c = \frac{K^{0.770}}{3080} \tag{3}$$

In this case:
l = distance AD = 2 450 m;
h = difference of elevation between A and D = 2.8 m;
s = h/l = average slope between A and D = 0.00114;

$$K = \frac{l}{\sqrt{s}} = \text{constant} = 72\ 471.98 \text{ (m)};$$

t_c = concentration time = 1.79 h.

The values obtained for t_c are around an average value of 2 h, which can be used for further calculations. If during this time the accumulated rainfall for a return period of 5 years is 64 mm, the rainfall intensity is about 32 mm/h.

The runoff coefficient according to Table A15.1 is about 0.3. Then, the maximum flow at point D is about 2.3 m^3/s, as calculated with Equation 1.

REFERENCES

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Soil Conservation Service (SCS). 1972. Hydrology. *National Engineering Handbook Section 4*. Washington, DC, USDA.

Annex 16
Curve Number method

PRINCIPLES

The Curve Number (CN) method is based on the conceptual interpretation of the hydrological process during a rainfall period. Initially, no surface runoff (S_r) is produced while rainfall is intercepted by vegetation and water infiltrates into the soil (I_a). When rainfall exceeds this initial interception, overland flow begins while soil infiltration continues (I_{nf}). Once the soil is saturated, any amount of excess rainfall (P) produces surface runoff (Figure A16.1).

Figure A16.2 shows the relationship between the precipitation accumulated and surface runoff during a rainfall period.

The amount of S_r is zero if the accumulated rainfall is lower than the I_a value. Once this threshold value has been exceeded, the S_r function takes a curve shape up to the saturation point where S_r is equal to P. From this point, the S_r function becomes a straight line with unit slope (a = 45 °). If this line is extended to cut the x-axis, a point is achieved that represents the maximum retention potential (S). The S value depends on the physical characteristics of the basin and on the soil moisture content before the rainfall period.

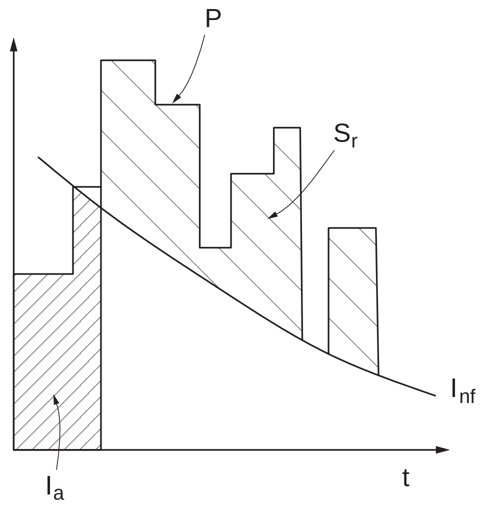
Once overland flow starts, the water balance on the soil surface is:

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

where:

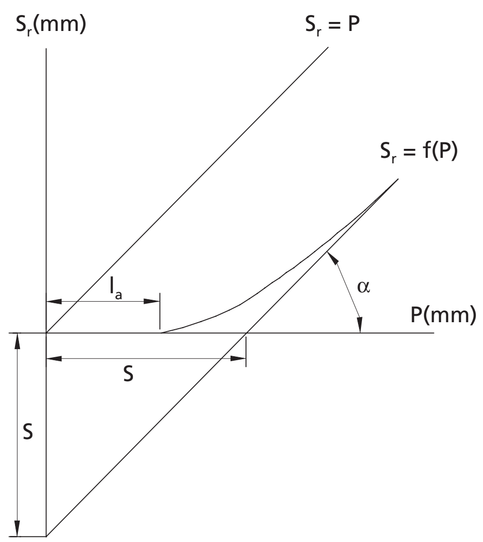
I_{nf} = actual infiltration while surface runoff is produced (mm);

FIGURE A16.1
Surface runoff during a rainfall period



Source: Adapted from Boonstra, 1994.

FIGURE A16.2
Relationship between precipitation and surface runoff



I_a = amount of water intercepted and infiltrated into the soil before overland flow occurs (mm);
P = amount of accumulated rainfall (mm);
$P - I_a$ = maximum potential of surface runoff (mm);
S_r = accumulated surface runoff (mm).

This method, developed by the Soil Conservation Service (SCS), assumes that the relationship between the actual surface runoff and its maximum potential value is equal to the rate between the actual infiltration and the maximum potential retention. The latter is approximately equal to the accumulated infiltration after runoff has started (Figure A16.2):

$$\frac{S_r}{P - I_a} = \frac{(P - I_a) - S_r}{S} \tag{2}$$

where:
S = maximum potential retention (mm).
Surface runoff can be then expressed as:

$$S_r = \frac{(P - I_a)^2}{(P - I_a) + S} \tag{3}$$

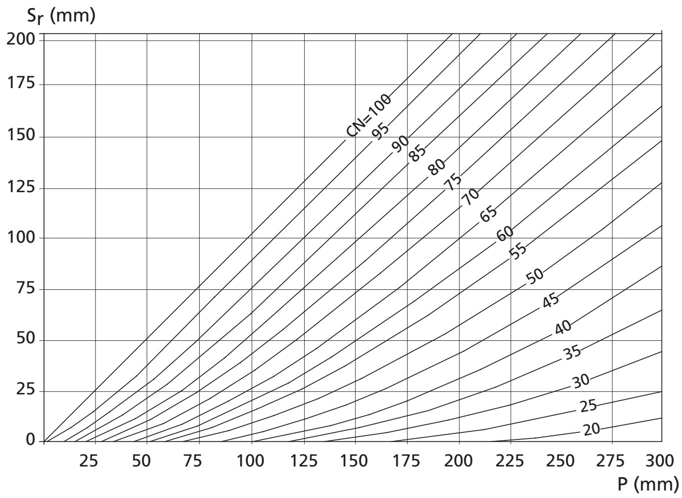
Equation 3 has been simplified by assuming that the value of the potential retention is constant during a storm and the initial interception is about 20 percent of the maximum potential retention ($I_a = 0.2S$). Thus, surface runoff depends only on precipitation and the maximum potential retention:

$$S_r = \frac{(P - 0.2S)^2}{P + 0.8S} \quad \text{for} \quad P > 0.2S \tag{4}$$

The SCS formulated a new undimensional parameter, named the Curve Number (CN), to assess the capacity of a basin to produce surface runoff after certain precipitation. This parameter is a hydrological characteristic of the basin, which depends on the maximum potential retention:

$$CN = \frac{25400}{254 + S} \tag{5}$$

FIGURE A16.3
Function surface runoff – precipitation for different Curve Numbers



Source: Adapted from Boonstra, 1994.

By combining Equations 4 and 5, one expression can be obtained to calculate the accumulated surface runoff from the amount of rainfall and the CN. Figure A16.3 shows the function S_r/P in the graph developed by the SCS (1972) for different CN values.

Thus, in a basin characterized by a certain CN, the amount of surface runoff produced by a design rainfall can be estimated by means of Figure A16.3 or through Equations 4 and 5.

ESTIMATION OF THE CURVE NUMBER

The CN value depends on:

- the natural vegetation and the current land use;

- the hydrological soil characteristics, especially the infiltration;
- the agricultural practices;
- the previous soil moisture content.

This method does not consider land slope because lands with gradients of more than 5 percent are not cultivated in the United States of America. However, classes for different slopes can be considered in a specific project (Boonstra, 1994).

The CN value increases progressively as retention decreases, the maximum value being 100 where retention in negligible. Table A16.1 shows the CN values established by the SCS (1972) for average soil moisture conditions before the design storm, considered as Class II.

In Table A16.1, the term straight rows means rows along the land slope. The hydrological condition essentially depends on the vegetation density. Condition is poor where meadows are intensively used or the grass quality is low, or where field crops are in the initial stage of growing. Otherwise, condition is good for densely vegetated meadows and for field crops covering the soil surface well.

In addition to the average soil moisture conditions considered in Table A16.1 for Class II, the SCS defined two additional classes (I and III), taking into account the amount of precipitation in the five-day period before the design storm (Table A16.2).

If the antecedent soil moisture condition differs from Class II, the equivalent CN values for Class I or Class III can be estimated by using the conversion factors developed by the SCS (1972) and shown in Table A16.3, once the CN value has been determined for Class II.

TABLE A16.1
CN values Class II

Land use	Practice	Hydrological condition	Soil infiltrability			
			High	Medium	Low	Very low
Fallow	Straight row	Poor	77	86	91	94
Row crops	Straight row	Poor	72	81	88	91
		Good	67	78	85	89
	Contoured	Poor	70	79	81	88
		Good	65	75	82	86
	Contoured/terraced	Poor	66	74	80	82
		Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	Contoured/terraced	Poor	61	72	79	82
		Good	59	70	78	81
Close-seeded legumes or rotational meadow	Straight row	Poor	66	77	85	89
		Good	58	72	81	85
	Contoured	Poor	64	75	83	85
		Good	55	69	78	83
	Contoured/terraced	Poor	63	73	80	83
		Good	51	67	76	80
Pasture range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
		Fair	25	59	75	83
		Good	6	35	70	79
Meadow (permanent)		Good	30	58	71	78
Woodland		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77

Source: Adapted from Boonstra, 1994.

TABLE A16.2
Classes for previous soil moisture conditions

Class	*P* in the previous 5-day period	
	Dormant season	Growing season
	(mm)	
I	< 13	< 36
II	13–28	36–53
III	> 28	> 53

Source: Adapted from Boonstra, 1994.

TABLE A16.3
Equivalent CN according to the antecedent soil moisture classes

Class	CN										
I	100	78	63	51	40	31	22	15	9	4	0
II	100	90	80	70	60	50	40	30	20	10	0
III	100	96	91	85	78	70	60	50	37	22	0

Source: Adapted from Boonstra, 1994.

In order to estimate the average CN value of a basin, all the sections with different hydrological conditions, land use and agricultural practices should first be mapped. Then, the respective CN is assigned to each independent section. Last, the weighted average is calculated according to the surface area of each section.

Example

In this example, the CN method has been applied to estimate the amount of surface runoff produced by an extreme rainfall of 125 mm in 24 hours, determined for a return period of 10 years, in a basin of 4 740 ha, where the current land use is rainfed agriculture and forest. This was the previous stage to calculate later the maximum water flow at the outlet of the main watercourse draining the basin.

The first step for this calculation was to estimate the concentration time of the basin with the Kirpich formula (although this formula was developed for small agricultural basins). For a watercourse with a length of 15.5 km and a difference in elevation between the most distant point from the outlet and the outlet itself of 299.4 m, the t_c value is 2.5 hours.

The second step was to assess the rainfall distribution during the first 6 hours of the storm. This period of 6 hours was selected, because the concentration time is less than 6 hours. It was assumed that during the first 6 hours, 60 percent of the one-day precipitation occurred, i.e. 75 mm. The rainfall distribution during this period can be estimated by the WMO model for time intervals of 0.5 hours, as shown in Table A16.4.

In order to estimate the weighted average CN for the whole basin, the area was split into six sections with homogeneous land use and hydrological conditions by superimposing the land-use map and the soil map. The physical characteristics of these sections are described in Table A16.5, where the individual CN, estimated for Class II, were assigned to each section.

The weighted average CN for the basin as a whole is 69 for Class II (Table A16.5). However, the previous soil moisture conditions are more similar to those of Class III as in the area studied extreme rainfalls are frequent in autumn. Therefore, it is more adequate to use the equivalent CN for Class III, i.e. 85 according to Table A16.3.

TABLE A16.4
Distribution of the total precipitation in a period of 6 hours

Time (h)	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0
Rainfall distribution (%)	2	8	15	22	60	70	78	84	88	92	96	100
Accumulated rainfall (mm)	1.5	6.0	11.3	16.5	45.0	52.5	58.5	63.0	66.0	69.0	72.0	75.0

TABLE 16.5

Physical characteristics and CN values of the hydrologically homogeneous sections

Section	Surface area (ha)	Soil type	Land use	Agricultural practice	Infiltrability	CN
1	762	Shallow soils on shale rock	Pasture		Low	79
2	1 566		Woodland & pasture		Medium	69
3	1 161	Terraced deep soils	Vineyard		Medium	71
4	990	Terraced deep soils	Field crops	Straight rows	High	59
5	30	Terraced soils	Dense field crops		Low	76
6	231	Moderately shallow soils with slopes > 2%	Pasture		Low	74
Basin	4 740					69

TABLE A16.6

Estimation of the amount of surface runoff for CN = 85

Time (h)	0.5	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0
Accumulated rainfall (mm)	1.5	6.0	11.3	16.5	45.0	52.5	58.5	63.0	66.0	69.0	72.0	75.0
Accumulated runoff (mm)			0.1	1.1	16.1	21.5	26.0	29.5	31.9	34.4	36.8	39.3
ΔS_r (mm)			1.0	15.0	5.4	4.5	3.5	2.4	2.5	2.4	2.5	

The maximum potential retention for this CN is 44.8 mm (Equation 5). With this value, the surface runoff produced for the design rainfall can be calculated with Equation 4 or estimated by means of Figure A16.3. Table A16.6 shows the results.

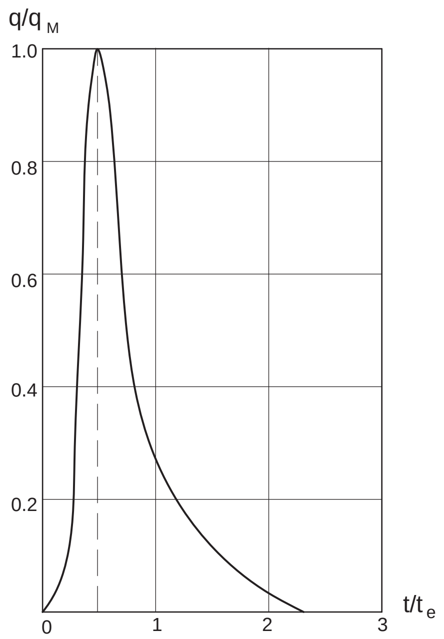
HYDROGRAPH OF THE SPECIFIC DISCHARGE

The dimensionless unit hydrograph developed by the SCS can be used to calculate the maximum specific discharge of surface runoff and the maximum water flow. In this hydrograph, time is expressed as a function of the elevation time, and discharge is related to its maximum value. Figure A16.4 shows this hydrograph and a table with average values.

From numeric integration of this hydrograph, the following expression can be obtained for the maximum specific discharge:

FIGURE A16.4

SCS dimensionless unit hydrograph



t/t_e	q/q_M
0	0
0.25	0.12
0.50	0.43
0.75	0.83
1.00	1.00
1.25	0.88
1.50	0.66
1.75	0.45
2.00	0.32
2.25	0.22
2.50	0.15
2.75	0.105
3.00	0.075
3.25	0.053
3.50	0.036
3.75	0.026
4.00	0.018
4.25	0.012
4.50	0.009
4.75	0.006
5.00	0.004

Source: Adapted from Boonstra, 1994.

$$q_M = 2.08 \frac{S_r}{t_e} \quad (6)$$

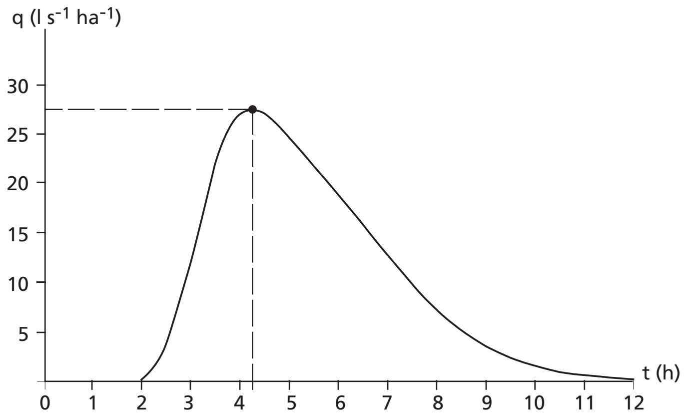
where:

q_M = maximum specific discharge (litres per second per hectare);

S_r = amount of surface runoff (mm);

t_e = elevation time (h).

FIGURE A16.5
Example of hydrograph for the total specific discharge



The t_e value can be estimated from the concentration time ($t_e \approx 0.7t_c$).

Example

The elevation time (t_e) in the basin of the previous example is about 1.75 hours. With this value, in Table A16.7 the maximum specific discharge (q_M) for each increment of surface runoff (ΔS_r) has been calculated with Equation 6. In Table A16.7, the distribution of the specific discharge has also been determined by applying the tabulated values of the undimensional hydrograph represented in Figure A16.4 to the q_M values.

The hydrograph for the total specific discharge (Figure A16.5) was obtained by superimposing the partial hydrographs obtained with the results of Table A16.7.

TABLE A16.7
Calculation of the partial specific discharges q_M and the total discharge q_t

			t (h)									q_t (l s-1 ha-1)
			1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	
			ΔS_r (mm) (see Table A16.6)									
			1.0	15.0	5.4	4.5	3.5	2.4	2.5	2.4	2.5	
t (h)	Undimensional hydrograph		$q_M = 2.08\ S_r/t_e = 1.19S_r$									
	t/t_e	q/q_M	1.2	17.9	6.4	5.4	4.2	2.9	3.0	2.9	3.0	
0.0	0.00	0.00										
0.5	0.29	0.17										
1.0	0.57	0.54										
1.5	0.86	0.91										
2.0	1.14	0.93	0.20									0.20
2.5	1.43	0.72	0.65	3.04								3.69
3.0	1.71	0.48	1.09	9.67	1.09							11.85
3.5	2.00	0.32	1.12	16.29	3.46	0.92						21.79
4.0	2.29	0.21	0.86	16.65	5.82	2.92	0.71					26.96
4.5	2.57	0.14	0.58	12.89	5.95	4.91	2.27	0.49				27.09
5.0	2.86	0.09	0.38	8.59	4.61	5.02	3.82	1.57	0.51			24.50
5.5	3.14	0.06	0.25	5.73	3.07	3.89	3.91	2.64	1.62	0.49		21.60
6.0	3.43	0.04	0.17	3.76	2.05	2.59	3.02	2.70	2.73	1.57	0.51	19.10
6.5	3.71	0.03	0.11	2.51	1.34	1.73	2.02	2.09	2.79	2.64	1.62	16.85
7.0	4.00	0.02	0.07	1.61	0.90	1.13	1.34	1.39	2.16	2.70	2.73	14.03
7.5	4.29	0.01	0.05	1.07	0.58	0.76	0.88	0.93	1.44	2.09	2.79	10.59
8.0	4.57	.008	0.04	0.72	0.38	0.49	0.59	0.61	0.96	1.39	2.16	7.34
8.5	4.86	.005	0.02	0.54	0.26	0.32	0.38	0.41	0.63	0.93	1.44	4.93
9.0	5.14	.003	0.01	0.36	0.19	0.22	0.25	0.26	0.42	0.61	0.96	3.28
9.5	5.43		0.01	0.18	0.13	0.16	0.17	0.17	0.27	0.41	0.63	2.13
10.0	5.71		0.01	0.14	0.06	0.11	0.13	0.12	0.18	0.26	0.42	1.43
10.5	6.00			0.09	0.05	0.05	0.08	0.09	0.12	0.17	0.27	0.92
11.0	6.29			0.05	0.03	0.04	0.04	0.06	0.09	0.12	0.18	0.61
11.5	6.57				0.02	0.03	0.03	0.03	0.06	0.09	0.12	0.38
12.0	6.86					0.02	0.02	0.02	0.03	0.06	0.09	0.24
12.5	7.14						0.01	0.01	0.02	0.03	0.06	0.13
13.0	7.43							0.01	0.02	0.02	0.03	0.08
13.5	7.71								0.01	0.01	0.02	0.04
14.0	8.00									0.01	0.02	0.03

Figure A16.5 shows that about 4 hours after of the beginning of the design storm the maximum specific discharge is expected, its value then being about 27 l s^{-1} ha^{-1}. With this surface drainage coefficient, each section of the main drainage system can be dimensioned. At the outlet of this basin of 4 740 ha, the maximum estimated flow will be about 128 m^3/s.

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Annex 17
Formulae for steady-state flow to drains

This annex gives formulae for the calculation of open or covered parallel drain spacings for use for different soil profiles.

FLOW ABOVE DRAIN LEVEL; THE ELLIPSE EQUATION

The ellipse equation (Figure A17.1) is valid for a single layer above drain level (Van der Ploeg, Marquardt and Kirkham, 1997).

Where an impermeable layer is present at drain level, the phreatic groundwater table between two drains has an elliptic shape. The resulting formula for the drain spacing then equals:

$$L^2 = \frac{4Kh^2}{q} \qquad (1)$$

where:
h = groundwater elevation mid-way drains (m);
K = permeability above drain level (m/d);
L = drain spacing (m);
q = design discharge (m/d).

The ellipse formula is used in the programs for the flow above drain level, either as the only discharge or in combination with flow through deeper layers.

FIGURE A17.1
Steady-state flow above drain level– the ellipse equation

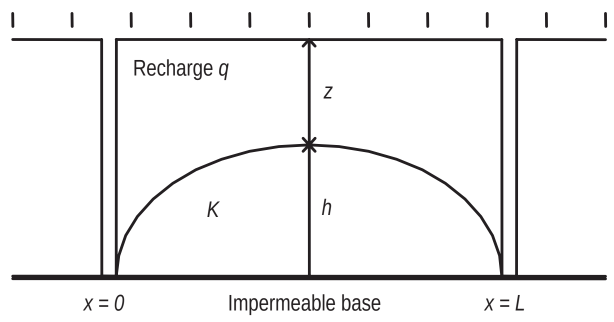
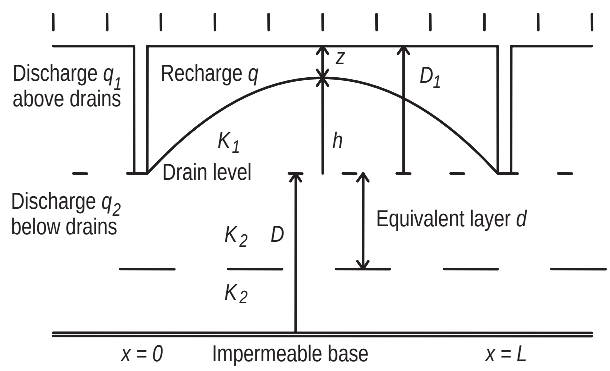


FIGURE A17.2
Steady-state flow above and below drain level – the Hooghoudt equation



FLOW ABOVE AND BELOW DRAIN LEVEL; THE HOOGHOUDT EQUATION

The Hooghoudt approach (Hooghoudt, 1940) considers a soil that is either homogeneous above and below the drain level or consists of two layers with different properties above and below drain level (Figure A17.2). Hooghoudt's formula for calculating drain spacings under steady-state flow assumptions is:

$$L^2 = \frac{4K_1h^2 + 8K_2dh}{q} \qquad (2)$$

where:
d = f(D_2,L,r) = effective thickness of lower layer (m);
D_1 = thickness of the layer above drain level (m) – mentioned in Figure A17.2;

D_2 = real thickness of the layer below drain level, down to the impermeable subsoil (m);
K_1 = permeability above drain level (m/d);
K_2 = permeability below drain level (m/d);
r = effective drain radius (m).

Inputs for Equation 2 are D_2, h, K_1, K_2, q and r, of which D_2 may be infinite. Because d depends on the required distance L, iteration is necessary.

Hooghoudt's method for calculating drain spacings is valid for a two-layered soil profile: one layer above and one below drain level. The latter not only offers resistance to horizontal flow, but also radial resistance that occurs near the drain, where the streamlines are converging.

In this approach, the flow pattern is replaced by horizontal flow through a thinner layer; the actual thickness D_2 of the layer below the drains is replaced by the equivalent layer d without radial resistance (Figure A17.2). For steady-state flow, this is allowed, but errors may occur in non-steady cases.

The equivalent layer d, which is a complicated function, is used as a substitute correction for the radial resistance caused by the convergence of streamlines near the drain. It is smaller than the real thickness D_2 of the lower layer and was tabulated by Hooghoudt. Subsequently, nomographs were based on these tables (Van Beers, 1979). However, for computer applications a series solution is more effective. The following series solution may be used to find d:

$$d = \frac{\pi L / 8}{\ln \frac{L}{\pi r} + G(x)} \qquad x = \frac{2\pi D_2}{L} \tag{3}$$

$$G(x) = \frac{4e^{-2x}}{1\left(1-e^{-2x}\right)} + \frac{4e^{-6x}}{3\left(1-e^{-6x}\right)} + \frac{4e^{-10x}}{5\left(1-e^{-10x}\right)} + \frac{4e^{-14x}}{7\left(1-e^{-10x}\right)} + \ldots \tag{4}$$

which converges rapidly for $x > 0.5$.

For smaller values of x, Dagan's formula results in the expression:

$$G(x) = \frac{\pi^2}{4x} + \ln \frac{x}{2\pi} \tag{5}$$

These formulae are well-suited for computer application.

ERNST EQUATION

The Ernst method (Ernst, 1956) for calculating drain spacings allows two-layered profiles with a horizontal boundary at arbitrary level but not necessarily at drain depth (Figure A17.3). If homogeneous, layers 1 and 2 are supposed to be of equal composition ($K_2 = K_1$ and $an_2 = an_1$).

In this method, the flow is divided into three parts, each of which is calculated:

- a vertical flow to the aquifer, with a vertical head loss h_v;
- a horizontal flow to the vicinity of the drain, with horizontal head loss h_h;
- a radial flow towards the drain, with radial head loss h_r.

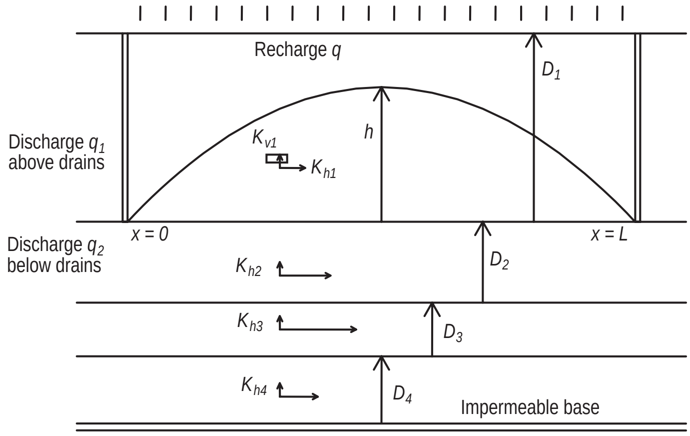
The total head loss in the soil h is:

$$h = h_v + h_h + h_r \tag{6}$$

The theory gives rise to a quadratic equation in L.

THE TOKSÖZ–KIRKHAM ALGORITHM

Figure A17.3
Drainage of a multilayered anisotropic soil profile – the Toksöz–Kirkham and Ernst methods (for Ernst, $K_1 = K_2$ and $D_4 = 0$)



Toksöz and Kirkham (1971a and 1971b) devised a general theory for determining drain spacings in multilayered soils with arbitrary horizontal boundaries (Figure A17.3). It consists of a set of complicated hyperbolic functions that depend on the number and thickness of layers considered.

The method calculates the flow through 1–3 different layers below drain level (Figure A17.3). It uses the following definitions:

- The layer above drain level has permeability K_1. It is not considered in the theory, but the resulting flow can be calculated by Equation 1.
- The first layer below drain level has permeability K_2 and thickness D_2.
- The second layer below drain level has permeability K_3 and thickness D_3.
- The third layer below drain level has permeability K_4 and thickness D_4.
- The drain spacing is L, the drain radius r, the recharge intensity q, and the head midway h.

Distances a, b, c and s are defined as:

$$a = D_2 \qquad b = D_2 + D_3 \qquad c = D_2 + D_3 + D_4 \qquad s = \frac{L}{2} \tag{7}$$

The following auxiliary quantities are calculated:

$$\alpha_m = \frac{K_4}{K_3}\frac{1}{\cosh\frac{m\pi(b-a)}{s}} \qquad \beta_m = \frac{K_4}{K_3}\coth\frac{m\pi b}{s} \qquad \gamma_m = \tanh\frac{m\pi(c-b)}{s} \tag{8.a}$$

$$\delta_m = \tanh\frac{m\pi(b-a)}{s} \qquad \eta_m = \frac{\sinh\frac{m\pi a}{s}}{\sinh\frac{m\pi b}{s}} \qquad \rho_m = \frac{K_3}{K_2}\coth\frac{m\pi a}{s} \tag{8.b}$$

$$\mu_m = \frac{\cosh\frac{m\pi a}{s}}{\sinh\frac{m\pi b}{s}} \qquad \varepsilon_m = \frac{K_3}{K_2}\frac{1}{\sinh\frac{m\pi a}{s}} \tag{8.c}$$

Furthermore:

$$S_m = me^{\frac{m\pi a}{s}} \tag{9}$$

$$T_m = \frac{1}{m}\frac{\varepsilon_m}{(1+\delta_m\,\rho_m)(1+\beta_m\,\gamma_m) + \alpha_m\,\gamma_m\,(\rho_m\,\eta_m - \mu_m)} \tag{10}$$

$$U_m = 1 - S_m\,T_m\,\delta_m - S_m\,T_m\,\gamma_m\,(\delta_m\,\beta_m + \alpha_m\,\eta_m) \tag{11}$$

The head h midway between drains is determined from:

$$h=\frac{2sq}{\pi(K_2-q)}\left\{-\ln\sin\frac{\pi r}{2s}+\sum_{m=1}^{\infty}\frac{1}{m}\left[-1+\coth\left(\frac{m\pi a}{s}\right)\right]\left[\cos\frac{m\pi r}{s}-\cos(m\pi)\right]U_m\right\} \quad (12)$$

Combination with the ellipse equation for flow above drains requires an iterative solution.

These formulae are suited for computer applications.

INFLUENCE OF ANISOTROPY

In many soils, permeability depends on the direction of flow. Considerations here are confined to horizontal layering and vertical cracks. The former results in a permeability that is larger in the horizontal than in the vertical direction, the latter in the reverse.

In such cases, where the axes of the anisotropy coincide with the horizontal and vertical x and z axes, the following rules may be used (Boumans, 1963):

- An "anisotropy factor" an_i is defined for each layer i as:

$$an_i=\frac{K_{hi}}{K_{vi}} \quad (13)$$

with K_h horizontal and K_v vertical permeability of layer i.

- Hydraulic heads and discharges remain the same.
- Horizontal distances remain the same.
- Vertical distances z_i in layer i (especially thickness D_i) are transformed to:

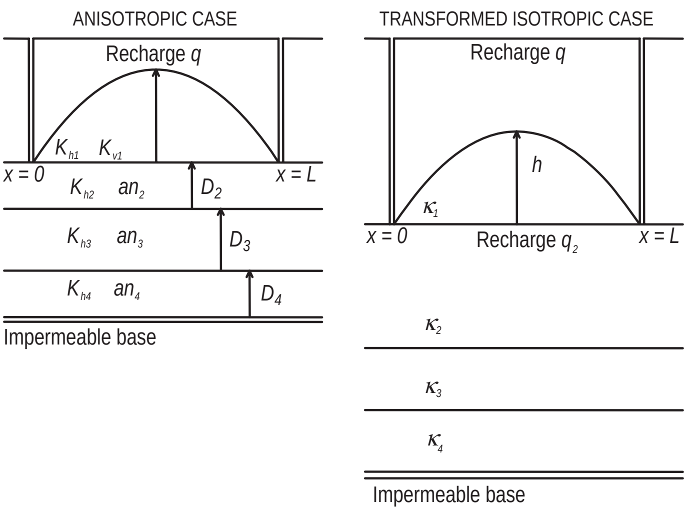
$$\zeta_i = z_i\sqrt{an_i} \quad (14)$$

- The permeability is transformed to:

$$\kappa_i=\frac{K_{hi}}{\sqrt{an_i}} \quad (15)$$

In this transformed isotropic system (Figure A17.4), all formulae for steady-state flow are valid. The resulting spacing L is horizontal and, consequently, it remains unchanged.

FIGURE A17.4
Anisotropic and transformed isotropic system



For flow above drains, a different approach is used. Here, the vertical permeability K_{v1} of the first layer is used to find the head loss between maximum head h and drain level and, consequently, the corrected head h_c (the head at drain level) as:

$$h_c=h\left(1-\frac{q}{K_{v1}}\right) \quad (16)$$

With this corrected head, all subsequent calculations are executed.

The program SPACING is based on the above theory. However, the Ernst equation is not included. In cases where it is applicable, it gives practically the same results as the more general Toksöz–Kirkham algorithm.

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Annex 18
Drainage under vertical seepage

INFLUENCE OF VERTICAL SEEPAGE

Artesian seepage (upward flow from deeper layers) is caused by groundwater flow from higher areas. The sources may be nearby (e.g. irrigated lands on higher grounds) or far away (through aquifers under pressure recharged in hills or mountains). Water escaping from such aquifers causes upward flow to the rootzone. Drainage of such seepage areas is often difficult. In many cases, temporary or even permanent wetness and salinization occur.

Two main methods have been proposed for drain spacing design under these conditions:

- Vertical drainage is a good solution under special hydrological conditions. Therefore, where there is no previous experience in the region, a careful hydrogeological survey is needed.
- Relief wells are another possibility where the aquifer is under pressure.

Where neither of these solutions is applicable, drains need to be laid at a narrower spacing than normal. In this case, a formula developed by Bruggeman (Van Drecht, 1983; Bruggeman, 1999) can be used. However, in severe cases, where the drain spacing must be greatly reduced, it is often better to leave the area as a wetland.

BRUGGEMAN'S FORMULA FOR ARTESIAN CONDITIONS

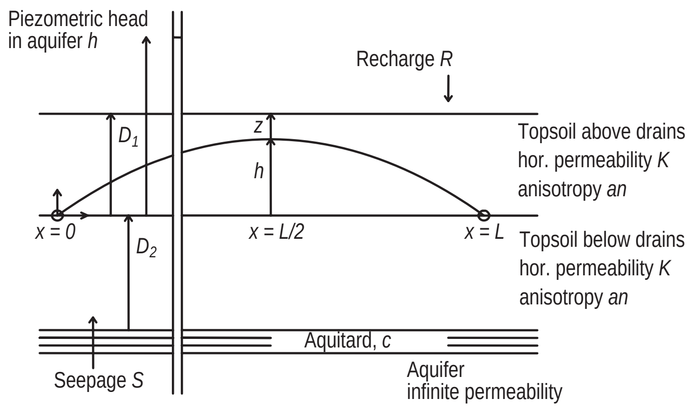
For horizontal drainage under artesian conditions, Bruggeman's method may be used. This calculates flow below drain level under the following circumstances (Figure A18.1):

- a moderately permeable top layer, in which the drains are located, overlies a highly ("infinitely") permeable aquifer;
- between the top layer and the aquifer a semi-confining layer (aquitard) occurs;
- the artesian head in the aquifer may be above drain level as well as below (in the latter case, natural downward drainage will occur);
- the artesian head is not influenced by the drainage system.

The final condition is seldom respected in large projects. Such works usually exert a profound influence on the underlying aquifer. This limits the applicability of the method to rather small areas. In large projects, combination with a geohydrological model of the aquifer is indispensable. The model SAHYSMOD (ILRI, 2005) can be used for this combination. It also allows an analysis of the salt balance.

Because flow above drain level is not considered in the Bruggeman formulae, the ellipse equation can be used to calculate this part of the flow.

FIGURE A18.1
Drainage under artesian conditions – the Bruggeman method



Spacings are to be calculated for two cases:

- high recharge by heavy rain or irrigation, in combination with a criterion for groundwater table depth under such wet conditions;
- zero recharge, with a criterion for a design groundwater depth under dry conditions, deep enough to avoid permanent wetness in humid climates and salinization in arid regions.

For the latter, groundwater should remain below a critical depth.

Bruggeman derived the following algorithm for two-dimensional flow below drain level under artesian conditions (Figure A18.1):

$$Q_2 = \frac{\left[\left(c + \frac{D_2}{K_{2v}}\right)\left(1 - \frac{u}{L}\right) - u\Sigma_1\right]\left(P - q_1\right) + h}{\frac{c_b}{u} + \frac{\left(c + \frac{D_2}{K_{2v}}\right)}{L} + \Sigma_1} \quad (1)$$

$$\Sigma_1 = \frac{a_B L^2}{\pi^3 u^2 K_{2v}} \sum_{n=1}^{\infty} \frac{1}{n^3} \sin^2\left(\frac{n\pi u}{L}\right) F(n,0) \quad (2)$$

$$F(n,z) = \frac{(n\alpha_1 + 1)\, e^{n\alpha_2} + (n\alpha_1 - 1)\, e^{-n\alpha_2}}{(n\alpha_1 + 1)\, e^{n\alpha_3} - (n\alpha_1 - 1)\, e^{-n\alpha_3}} \quad (3)$$

$$a_B = \sqrt{\frac{K_{2v}}{K_{2h}}} \qquad \alpha_1 = \frac{2\pi K_{2v}\, c}{a_B\, L} \qquad \alpha_2 = \frac{2\pi\left(D_2 - y\right)}{a_B\, L} \qquad \alpha_3 = \frac{2\pi D_2}{a_B\, L} \quad (4)$$

At drain level, where $y = 0$ and $\alpha_2 = \alpha_3$:

$$F(n,0) = \frac{(n\alpha_1 + 1) + (n\alpha_1 - 1)\, e^{-2n\alpha_3}}{(n\alpha_1 + 1) - (n\alpha_1 - 1)\, e^{-2n\alpha_3}} \quad (5)$$

The flux density is:

$$q_2 = \frac{Q_2}{L - u} \quad (6)$$

where:

c = resistance of semi-confining layer (d);
c_b = entry resistance of drain ($c_b = 0$) (d);
D_2 = thickness of layer below drain level (m);
h = head midway, at drain level (m);
h_a = head in artesian aquifer, above drain level (m) (in Figure A18.1);
K_{2h} = horizontal permeability below drains (m/d);
K_{2v} = vertical permeability below drains (m/d);
L = drain spacing (m);
q_2 = flux density below drain level (m/d);
Q_2 = flux below drain level, per metre of drain (m²/d);
R = recharge by precipitation or irrigation excess (m/d) (in Figure A18.1);
u = wet circumference of drain (m);
y = vertical coordinate, positive downward (m).

For artesian conditions and a two-layer profile (one of which is below drain level), the design program ARTES was developed. It is based on Bruggeman's algorithm, in combination with flow above drain level according to the ellipse equation.

It also requires general design criteria. These are followed by the soil properties, which now include the hydraulic head in the underlying artesian aquifer and the vertical resistance of a semi-confining layer between the aquifer and the two top layers mentioned.

An approximation is to use Hooghoudt's formula with the expected seepage from below added to the recharge from above. In most cases, the difference in spacing is negligible in practice (less than 5–10 percent). However, there are exceptions, especially where the resistance of the semi-confining layer is low and part of the drainage water passes through the aquifer.

ARTES uses the Bruggeman's method except in the rare cases where this procedure is not convergent or is otherwise doubtful. Then, the Hooghoudt approximation is given, together with a warning.

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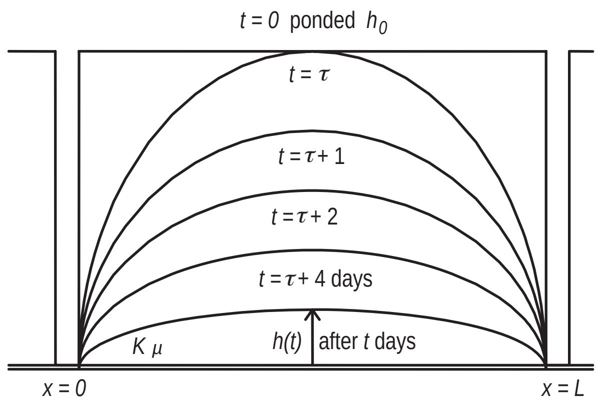
Annex 19
Formulae for non-steady-state flow to drains

FLOW ABOVE DRAINS – THE BOUSSINESQ SOLUTION

FIGURE A19.1
Non-steady-state flow above drain level – the Boussinesq solution



In 1904, Boussinesq found a solution for non-steady-state (transient) flow to drains lying on an impermeable subsoil layer (K_2 = 0), as occurring after heavy rain or irrigation. Boussinesq's equation (Boussinesq, 1904; Guyon, 1966; Moody, 1967) describes the fall in the water table after recharge. Where the initial shape of the groundwater between the drains follows a special curve (nearly an ellipse), it retains this shape during the drainage process because the head diminishes proportionally everywhere. It can be shown that, soon after the end of the recharge event, the shape of the groundwater table becomes almost elliptical, and during its lowering, the curve becomes flatter, but retains its shape (Figure A19.1).

If the soil surface is ponded and the soil profile is completely saturated at the beginning, the theory is not valid for short times. The lowering of the water table reaches the mid-point between drains only after some lag time τ, being the time to approach Boussinesq's pseudo-ellipse, after which a phreatic surface of constant shape is approached. The lag time τ is approximately:

$$\tau \cong \frac{\mu L^2}{CKh_o} \quad (1)$$

where:
C = 38, this is an empirical constant derived from numerical experiments;
Z = drain depth (m);
h_0 = initial head midway the drains, equal to drain depth (m);
K = permeability above drain level (m/d);
L = drain spacing (m);
μ = storage coefficient;
τ = lag time (d).

Boussinesq's formula is a solution of the non-linear differential equation:

$$\mu \frac{\partial h}{\partial t} = K \frac{\partial}{\partial x}\left(h \frac{\partial h}{\partial x} \right) \quad (2)$$

Based on this solution, Guyon proposed the following formula for calculating drain spacing (with $\tau = 0$), valid for Boussinesq's pseudo-ellipse:

$$L^2 = \frac{4.5 K h_0 h (t - \tau)}{\mu (h_0 - h)} \tag{3}$$

where:
h = hydraulic head midway, at time t (m);
h_0 = initial head midway between drains (at time $t = 0$) (m);
K = soil permeability (m/d);
L = drain spacing (m);
t = time (d);
τ = lag time (d);
μ = storage coefficient.
The factor 4.5 is an approximation of an expression that yields 4.46208…

If the lag time τ has to be considered, the L value may be calculated with the following formula, obtained by combining Equations 1 and 3:

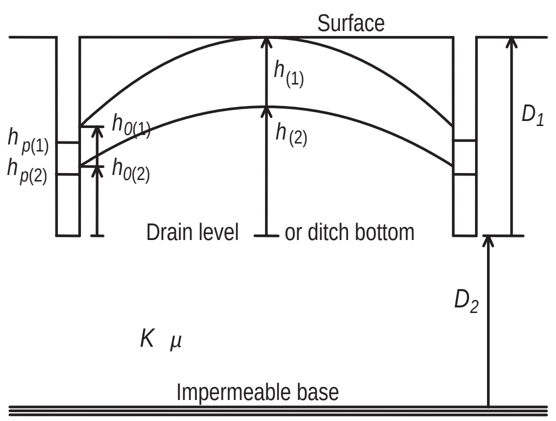
$$L^2 \left[1 + \frac{4.5 h}{38 (h_0 - h)} \right] = \frac{4.5 K h h_0 t}{\mu (h_0 - h)} \tag{4}$$

Equation 4 is the non-steady-state flow equivalent of the steady-state flow ellipse equation. The program NSABOVE, which is based on this equation, describes the flow to drains lying on an impermeable soil layer. The shapes of the water table closely resemble semi-ellipses of decreasing height.

FLOW ABOVE AND BELOW DRAINS – NUMERICAL SOLUTION

Analytical approximations (Glover–Dumm, and Kraijenhoff van de Leur) can be used to calculate drain spacings where $h << D_2$. However, these solutions do not consider radial resistance and resistance near the drain. Therefore, numerical methods are preferable because they are easier to handle and are accurate enough for practical purposes. Moreover, evaporation losses, which vary with the depth of the phreatic level and also the effect of outflow restrictions, can readily be incorporated. The latter are caused by the radial resistance concentrated near the drain and the limited capacity of the collecting system.

FIGURE A19.2
Non-steady-state flow above and below drain level



(1) - (2): successive stages

For drains lying above an impermeable soil layer (Figure A19.2), the flow below the drain level must be considered through a layer with a transmissivity KD_2. The permeability K is the same above and below drain level ($K_1 = K_2 = K$) and D_2 the thickness of the layer below drain level.

After a heavy rain, the water levels in the watercourses and the head in the pipes will be higher than designed. This will in turn restrict the outflow from the soil until equilibrium is reached. In view of the turbulent flow in pipes, their behaviour is supposed to follow a square-root function – at four times the design head, the outflow will be twice the design discharge. It is further supposed that, at design discharge, no water is standing above the drain (h_p = 0).

The outflow is further restricted by the radial and entrance resistance near the drain. This quantity is given as W_r in the program SPACING and here denoted as resistance W. It causes a head loss proportional to the flow.

Evaporation aids in lowering the groundwater, but it decreases rapidly with increasing groundwater depth. For this relationship, there are two options:

- linear reduction to zero at a given groundwater depth;
- exponential reduction with a given "characteristic" groundwater depth where E = 0.4343E_o.

These principles form the framework of the programs NSDEPTH and NSHEAD to check calculated drain spacings under non-steady-state flow.

Principle for numerical solution

The principle for numerical solutions is that both time and (horizontal) space are divided into discrete elements and steps. In each element, the water balance during one time-step is:

$$\mu \Delta h \Delta x = (Q_{in} - Q_{out})\Delta t \tag{5}$$

where:

Q_{in} = flux entering an element, per metre of length (m^2/d);
Q_{out} = flux leaving an element, per metre of length (m^2/d);
x = distance (m);
Δh = fall of groundwater table (m);
Δt = time-step (d);
Δx = distance step (m);
μ = storage coefficient.

To develop this principle into a calculation program, both explicit and implicit methods are possible. The programs use the first approach although the risk of instability requires small time-steps Δt.

Differential equation

For flow below the drain level in the area D_2 (Figure A19.2) and a permeability K being the same above and below drain level ($K_1 = K_2 = K$), Equation 2 becomes:

$$\mu \frac{\partial h}{\partial t} = K \frac{\partial}{\partial x}\left[(h + D_2)\frac{\partial h}{\partial x}\right] \tag{6}$$

where:

D_2 = thickness of layer below drains (m).

The explicit finite difference expression for Equation 6 is:

$$h_{i,j+1} = h_{i,j} + \frac{K\Delta t}{\mu(\Delta x)^2}\left[\left(\frac{h_{i+1,j} + h_{i,j}}{2} + D_2\right)(h_{i+1,j} - h_{i,j}) - \left(\frac{h_{i,j} + h_{i-1,j}}{2} + D_2\right)(h_{i,j} - h_{i-1,j})\right] \tag{7}$$

where:

h = hydraulic head (m);
i = index for distance step;
j = index for time step;
x = distance (m);
Δt = time-step (d);
Δx = distance step (m).

In the model based on this equation, the drain spacing L has been divided into 20 equal parts. Index i = 0 represents the left-hand boundary; and i = 10 is a plane of symmetry that forms the right-hand boundary (midway between drains). Therefore, index $i = 11$ is the highest used. In the drainpipe, the head is h_p, near the drain it is h_o.

Boundary conditions

The initial condition ($j = 0$) is a constant head everywhere between the drains (i.e. groundwater at the soil surface):

$$h_{i,0} = h_{init} \qquad i = 1,11 \tag{8}$$

The right-hand boundary condition simulates symmetry at $i = 10$:

$$h_{9,j} = h_{11,j} \tag{9}$$

The left-hand boundary is more complicated. Here, two types of resistance against flow are present:

- a linear resistance W (d/m) against total flow (from both sides), being the sum of the radial resistance (caused by convergence of streamlines near the drain) and entry resistance for flow into the drain;
- a non-linear resistance, caused by the limited capacity of the outflow system (usually the drainpipes). Here, flow is turbulent and proportional to the square root of the available head.

For the one-sided flow q_0 (in cubic metres per day per metre of drain) converging towards and entering into the drain:

$$|q_0| = \frac{h_0 - h_p}{2W} \tag{10}$$

where (Figure A19.2):

h_0 = head near drain (m);
h_p = head in drainpipe (m);
q = flux density to drain (m/d);
$|q_0|$ = flux to drain (absolute value), one-sided (m^2/d);
$qL/2$ = flux, one-sided (m^2/d);
W = total resistance near drain (radial + entry) (d/m).

For the pipe flow, the outflow system has been designed to discharge a given steady flux density q (in metres per day) at a given head h_{des} (usually the slope multiplied by the pipe length).

For larger discharges, there is a need for an extra head h_p caused by insufficient pipe capacity. Thus, for one-sided flow, originating from width $L/2$:

$$|q_0| = \frac{qL}{2}\sqrt{\frac{h_p + h_{des}}{h_{des}}} \qquad \text{if} \qquad h_p > 0 \tag{11}$$

where:

h_{des} = design head for outflow system (m).

Finally, for horizontal flow in the first compartment:

$$|q_0| = K\left(\frac{h_1 + h_0}{2} + D_2\right)\frac{h_1 - h_0}{\Delta x} \tag{12}$$

where:

h_1 = head in first compartment (m).

Equalizing Equations 10–12 yields two equations in the unknown h_0 and h_p.

The upper boundary receives a sudden large input at t = 0, that saturates the entire soil profile. For t > 0, evaporation may help in lowering the water table, but it is dependent on the groundwater depth. Two options are available in the model:

- linear decrease with groundwater depth z;
- exponential decrease.

The linear case is characterized by the "critical depth" z_c:

$$E = E_0 \left(1 - \frac{z}{z_c}\right) \quad \text{for} \quad z < z_c \tag{13a}$$

$$E = 0 \quad \text{else} \tag{13b}$$

where:
E = actual evaporation from groundwater (m/d);
E_0 = potential evaporation from groundwater (m/d);
h_{init} = initial head = drain depth (m);
z_c = critical depth where E = 0 (linear model) (m);
z = groundwater depth (h_{init} - h) (m).
The exponential case is characterized by the characteristic depth z_h:

$$E = E_0\, e^{-\frac{z}{z_h}} \tag{14}$$

where:
z_h = depth where E = 0.4343E_0 (exponential model) (m).

Solution for h_0 and h_p (W > 0)

The relation:

$$\frac{h_0 - h_p}{2W} = K\left(\frac{h_1 + h_0}{2} + D_2\right)\frac{h_1 - h_0}{\Delta x} \tag{15}$$

leads to the quadratic equation:

$$h_0^2 + \left(\frac{\Delta x}{KW} + 2D_2\right)h_0 - \left(h_1^2 + 2D_2 h_1 + \frac{\Delta x}{KW}h_p\right) = 0 \tag{16}$$

The solution for the head in the drain is:

$$h_0 = -U + \sqrt{U^2 + V} \tag{17a}$$

where:

$$U = \frac{\Delta x}{2KW} + D_2 \tag{17b}$$

$$V = h_1^2 + 2D_2 h_1 + \frac{\Delta x h_p}{KW} \tag{17c}$$

The relation for the head near the drain is found as follows:

$$\frac{h_0 - h_p}{2W} = \frac{qL}{2}\sqrt{\frac{h_p}{h_{des}} + 1} \tag{18}$$

Equation 18 leads to the quadratic equation:

$$h_p^{\,2} - \left(2h_0 + \frac{q^2L^2W^2}{h_{des}}\right)h_p + \left(h_0^{\,2} - q^2L^2W^2\right) = 0 \qquad (19)$$

with solution:

$$h_p = h_0 + \frac{q^2L^2W^2}{2h_{des}} - qLW\sqrt{\frac{q^2L^2W^2}{4h_{des}^2} + \frac{h_0}{h_{des}} + 1} \qquad (20a)$$

$$h_p = \max\left(h_p, 0\right) \qquad (20b)$$

Iteration starts with Equation 17, with $h_p = 0$ in (17c). The value of h_0 obtained from Equation 17 is used in Equation 20 to find a new h_p value, which is inserted in Equation 17, etc., until convergence is sufficient.

The process is repeated before each time-step. With $h_{1,j} = h_0$ and $h_{2,j} = h_1$ Equation 7 is used to find the new values for the next time-step.

The index F is used as a criterion for stability of explicit numerical calculations:

$$F = \frac{KD'\Delta t}{\mu\left(\Delta x\right)^2} \qquad (21)$$

where:

$D' = D_2 + h_0 =$ maximum initial thickness (m).

FIGURE A19.3
Stable solution at F = 0.1

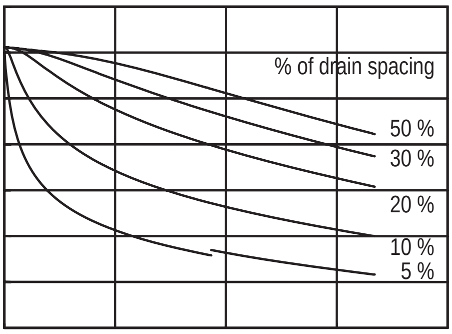
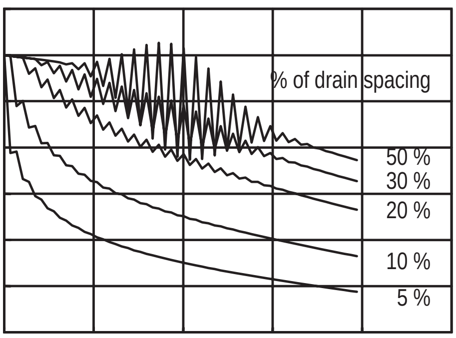


FIGURE A19.4
Instability at F = 0.6



The explicit method is valid for small time-steps and index F only. The characteristic:

$$F = \frac{K}{\mu}\frac{\Delta t}{\left(\Delta x\right)^2}\left(D_2 + h_{init}\right) \qquad (22)$$

should be less than 0.5 in order to avoid instability (Figure A19.3), and preferably be 0.25 or less (about 0.1) for sufficient accuracy. Figure A19.4 shows an example of instability.

The methods described, for flow above and below drain level through layers with the same K and μ values have been used in the programs NSDEPTH and NSHEAD. These programs check whether the three values for $|q_0|$ from Equations 10, 11 and 12 are indeed equal.

Finally, the water balance is checked. Errors should not exceed 5 percent. If difficulties arise, a smaller time-step is usually helpful.

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Annex 20
Diameters of drainpipes

PRINCIPLES

Drains are collecting systems. Along their length, the discharge and the flow velocity increase gradually. Therefore, the gradient of the hydraulic head is zero at the beginning, and will increase downstream.

Most drains are laid with a certain slope, and this slope is usually taken as a basis for calculating the required diameter. However, not the drain slope, but the total head loss is the basic design parameter. At the upstream end, the hydraulic head should remain at a certain depth below the soil surface, and this depth determines the available head with respect to the drain outlet, irrespective of the pipe length. The slope is not important, as illustrated in the following example. A drain 200 m long with an outlet 1.50 m below surface and a slope of 0.2 percent, without water standing above the upper end, loses 0.40 m in height along its length. Thus, it will control the upstream water table at a depth of 1.10 m. However, the same will be the case for a horizontal drain (slope zero) of the same length and outlet depth if it loses 0.4 m in head over its length owing to friction.

As an example, at the design discharge intensity *q* (metres per day – for pipe flow, *q* is recalculated and expressed in metres per second), the drain is running full at the outlet and the head at the beginning has a design value *H* (m) above the outlet. The drain itself has a slope, and the slope is such that no water is standing above the drain at its beginning (Figure A20.1). If the slope is less – and also when the drain is horizontal (Figure A20.2) – there is water above the drain at the upper end.

From a hydraulic point of view, the drain is functioning equally well in both cases. Sometimes "self-cleaning" is used as an argument for having the drain slope. However, in

FIGURE A20.1
Sloping drain

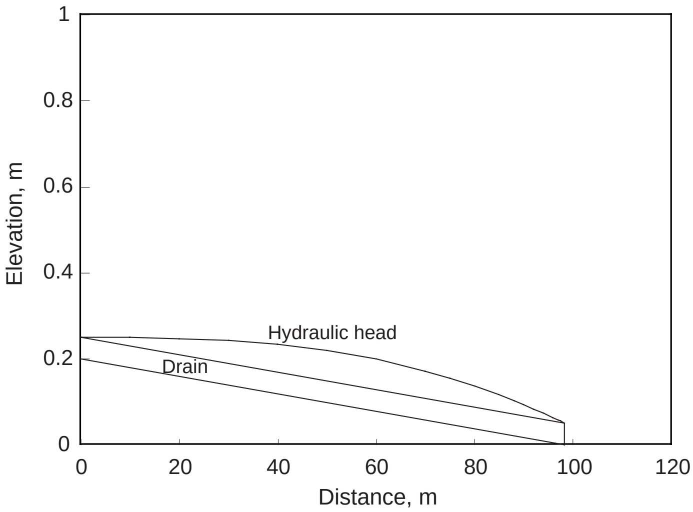
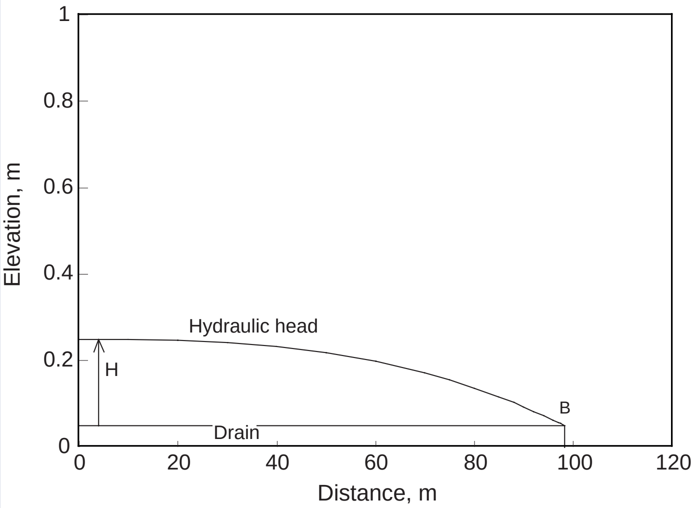


FIGURE A20.2
Horizontal drain



flat lands, drain slopes are seldom more than 0.5 percent and often far less. At such low slopes, the flow velocity is not enough to move sediments.

However, in practice, a slope for the pipe is usually prescribed. Horizontal drains are seldom encountered, except in subirrigation projects where drains are used for discharge in wet seasons and for recharge during droughts.

In the following, the system of Figure A20.1 is considered exclusively. Calculation of the diameter of horizontal drains (Figure A20.2) with formulae for sloping ones (Figure A20.1) sometimes shows small differences, but they are always on the safe side.

The available head loss at design discharge and the amounts of water to be drained under that condition form the basis for calculations concerning required drain diameters. These calculations are based on the laws for pipe flow, which differ for smooth and corrugated pipes.

Both smooth and corrugated pipe drains collect water along their length. As a consequence, the flow is not constant, but it increases gradually from zero at the upstream end to a maximum at the outflow. Introducing this variable Q corresponds with integration of the expressions for laterals and collectors. In laterals, Q increases continuously; in collectors, flow occurs stepwise, namely where the collector is joined by another lateral. However, provisional calculations show that in practice this makes almost no difference, provided that the laterals are of equal length.

SMOOTH PIPES

Non-perforated pipes made of glass, metals, PVC, PE and similar materials may be considered as “hydraulically smooth”. Pipes that are perforated or made of ceramics or cement are “technically smooth”, in which case they obey the same laws, but with a slightly different roughness coefficient. Corrugated pipes are “hydraulically rough”.

Basic equations

For smooth pipes, the Darcy–Weissbach equation is valid:

$$\frac{dh}{dx} = \frac{\lambda}{d}\frac{v^2}{2g} \qquad (1)$$

where:

$$\lambda = a\,\mathrm{Re}^{-0.25} \qquad \text{(Blasius)} \qquad (2a)$$

$$\text{or} \quad \frac{1}{\sqrt{\lambda}} = \log_{10}\left(\mathrm{Re}\sqrt{\lambda}\right) - 0.8 \qquad \text{(Nikuradse)} \qquad (2b)$$

$$\mathrm{Re} = \frac{vd}{\nu} \approx 10^6\, vd$$

with:

a = coefficient;
d = pipe diameter (m);
g = acceleration of gravity = 9.81 m/s^2;
h = hydraulic head (m);
Re = Reynolds’ number for pipes;
v = flow velocity (m/s);
x = distance along pipe (m);
λ = coefficient;
ν = kinematic viscosity ($\approx 10^{-6}$ m^2/s).

Both expressions for λ give comparable results (Table A20.1, for a = 0.3164). Because Equation 2b requires iteration, Equation 2a is normally used.

TABLE A20.1
Comparison between λ-Blasius and λ-Nikuradse

Reynolds' number	λ-Blasius	λ-Nikuradse	% difference
2 000	0.0473	0.0495	4.6
5 000	0.0376	0.0374	-0.6
10 000	0.0316	0.0309	-2.4
20 000	0.0266	0.0259	-2.7
50 000	0.0212	0.0209	-1.2
100 000	0.0178	0.0180	1.1

TABLE A20.2
Values for the *a* coefficient in Blasius' formula

Type of pipe	*a* coefficient	Remarks
Smooth, plastic, metal, glazed	0.3164	Non-perforated or well jointed
Technically smooth	0.40	Perforated, cement, ceramics
Corrugated plastic laterals	0.77	Zuidema, from field data[1]

[1] Theoretically not allowed for hydraulically rough pipes, but in accordance with field data for small-diameter corrugated drains.

Completely smooth laterals and collectors do not exist. Smooth plastic pipes contain perforations; ceramic and baked clay ones have joints and are not always aligned. For such "technically smooth" drains and collectors, the *a* coefficient in Equation 2a was taken as 0.40 instead of 0.3164. Table A20.2 shows values used for the *a* coefficient, as found in the literature.

Smooth laterals

Drain laterals collect additional water all along their length. At any point x, measured from their upstream end, the discharge Q and the velocity v are:

$$Q = qLx \qquad \text{and} \qquad v = \frac{4Q}{\pi d^2} = \frac{4qLx}{\pi d^2} \tag{3}$$

where:
L = drain spacing (m);
q = design discharge (m/s);
Q = drain discharge (m³/s).

Accordingly, the flow velocity v varies along the length and so does the Reynolds' number.

Inserting v in the basic equations (Equations 1 and 2a) leads to:

$$\frac{dh}{dx} = \frac{4\sqrt{2}}{\pi^{7/4}} \frac{a\nu^{1/4}(qL)^{7/4}}{gd^{19/4}} x^{7/4} \tag{4}$$

and integrating between $x = B_{i-1}$ and $x = B_i$:

$$\Delta H = \frac{16\sqrt{2}}{11\pi^{7/4}} \frac{a\nu^{1/4}q^{7/4}L^{7/4}\left(B_i^{11/4} - B_{i-1}^{11/4}\right)}{gd^{19/4}} = F_s\left(B_i^n - B_{i-1}^n\right) \tag{5}$$

with:
B_{i-1}, B_i = begin, end of a drain section (m);
F_s = calculation coefficient for smooth pipes;
n = 11/4;
ΔH = head loss in the drain (m).

In drains consisting of one pipe size only, $B_{i-1} = 0$. However, the full expression will be needed later for drains with increasing pipe diameters downstream (multiple drains). The head loss ΔH in the drain must be less than or at most equal to the design head loss over the entire drain length, H.

If $B_{i-1} = 0$, the permissible drain length B for this design head equals:

$$B = \left(\frac{11\pi^{7/4}}{16\sqrt{2}} \frac{gHd^{19/4}}{av^{1/4} q^{7/4} L^{7/4}} \right)^{4/11} \tag{6}$$

and the minimum diameter required for a given drain length B is:

$$d = \left(\frac{16\sqrt{2}}{11\pi^{7/4}} \frac{av^{1/4} q^{7/4} L^{7/4} B^{11/4}}{gH} \right)^{4/19} \tag{7}$$

The maximum drain spacing allowed at a given diameter amounts to:

$$L = \left(\frac{11\pi^{7/4}}{16\sqrt{2}} \frac{gHd^{19/4}}{av^{1/4} q^{7/4} B^{11/4}} \right)^{4/7} \tag{8}$$

For hydraulically smooth, new, collecting pipes the required head can be calculated with:

$$H = \frac{(qL)^{7/4}}{59.77^{7/4} \; d^{19/4}} \frac{4}{11} B^{11/4} \tag{9}$$

where conversion of units, physical and mathematical parameters, and integration have caused the numerical constants. An alternative formula for technically smooth pipes is: $Q = 89d^{2.714} s^{0.571}$ (FAO, 2005), where $Q = qLB$ and $s = H/B$. It gives almost the same results as the above formulae with a = 40.

In Equations 6–9:

a = Blasius coefficient;
B = drain length (m);
d = inside diameter (m);
g = acceleration of gravity (m/s²);
H = head loss in drain (m);
L = drain spacing (m);
q = specific discharge (m/s).

Smooth collectors

Where the laterals are of equal length, the same formulae may be used for designing collectors with added flows at each lateral connection. Now, L_c is the mutual distance between collectors and B_c the length of the collector (L_c is the symbol for collector spacing and B_c for its length. If the laterals are perpendicular to the collectors and the laterals flow from one side only L_c equals their length B. If inflow is from both sides, $L_c = 2B$). For collectors, both are substituted for L and B in the formulae for laterals. The difference from lateral design is that the flow into collectors is discontinuous, in contrast to laterals, where inflow may be considered as continuous along the pipe. However, where more than five laterals are involved, the "discretization error" caused by the inflow of the separate laterals may be ignored in practice.

In the case of unequal lengths of the contributing laterals, the collectors must be calculated section-wise, in which case the discontinuous inflow is accounted for.

CORRUGATED PIPES

Basic equations

Most authors calculate flow through corrugated pipes with Manning's equation:

$$Q = K_m \, A R^{2/3} s^{1/2} \tag{10}$$

where:

$A = \pi \left(\dfrac{d}{2} \right)^2$ = area of cross-section (m^2);

K_m = $1/n$ = Manning coefficient ($m^{1/3}s^{-1}$);

$R = \dfrac{A}{u} = \dfrac{d}{4} = \dfrac{r}{2}$ = hydraulic radius (m);

s = slope of H;

$u = 2\pi \left(\dfrac{d}{2} \right)$ = wet circumference (m).

The formula for smooth pipes is sometimes used for corrugated pipes, but with a much larger constant a (Zuidema and Scholten, 1972), whereas other authors (e.g. Van der Beken, 1969, Van der Beken *et al.*, 1972) introduce an equivalent "sand roughness" to account for the influence of the corrugations.

Manning's K_m for corrugated pipes

In Manning's equation, the constant K_m depends mostly on the spacing, depth and shape of the corrugations S and also on the diameter d. The K_m values for corrugated pipes are compiled in Table A20.3. The narrower the corrugation spacing S, the larger K_m. According to Irwin (1984) and Boumans (1986):

$$K_m = 70 \qquad \text{for } S < 0.01 \text{ m} \qquad (10 \text{ mm}) \tag{11a}$$

$$K_m = 18.7 d^{0.21} S^{-0.38} \qquad \text{for } S > 0.01 \text{ m} \qquad (10 \text{ mm}) \tag{11b}$$

where:
d = inner pipe diameter (m);
S = spacing of individual corrugations (m).

Equations 11a and 11b for K_m are used in the programs for corrugated pipes. For safety reasons, the maximum value is taken as 65 instead of 70.

Corrugated laterals

If for full flowing pipes, Equations 3 and 10 are solved for Q:

$$qLx = K_m \frac{\pi}{4} d^2 \left(\frac{d}{2} \right)^{2/3} s^{1/2} = K_m \frac{\pi d^{8/3}}{4^{5/3}} \left(\frac{dh}{dx} \right)^{1/2} \tag{12}$$

The head loss ΔH between points B_1 and B_{i-1} can be obtained by integrating Equation 12 between these points:

$$dh = \frac{4^{10/3} \, (qL)^2}{\pi^2 \, K_m^2 \, d^{16/3}} x^2 \, dx \tag{13}$$

TABLE A20.3
K_m values for corrugated pipes

Country	Material	Drain diameter *d* Outer (mm)	Drain diameter *d* Inner (mm)	Rib spacing *S* (mm)	K_m value
Netherlands	PVC	65	57	6.25	70
		80	72	6.25	74
		100	91	6.25	78
		160	148	7.50	80
Germany	PVC	60	52	6.30	69
		100	91	8.30	70
		125	115	8.30	73
		380	307	50.00	46
Unite States of America	PE	129	100	18.00	53
		196	171	20.00	57
United Kingdom	PP	265	225	33.00	50
		350	305	50.00	45

$$\Delta H = \frac{4^{10/3}(qL)^2\left(B_i^3 - B_{i-1}^3\right)}{3\pi^2 K_m^2 d^{16/3}} = F_c\left(B_i^n - B_{i-1}^n\right) \text{ with } n=3 \tag{14}$$

with:

F_c = calculation coefficient for corrugated pipes.

As mentioned above, in drains consisting of one pipe size only $B_{i-1} = 0$. For corrugated pipes, integration of Manning's equation results in:

$$H = \frac{4^{10/3}(qL)^2 B^3}{3\pi^2 K_m^2 d^{16/3}} \tag{15}$$

For corrugated pipes with small corrugations an alternative formula is (FAO, 2005): $Q = 38d^{2.667}s^{0.5}$. For corrugated pipes with a diameter of more than 200 mm and large corrugations an alternative formula is (FAO, 2005): $Q = 27d^{2.667}s^{0.5}$ Both give almost the same results as those mentioned in the text.

Where the design head H is given, and $B_{i-1} = 0$, the other values (e.g. d or L) are readily derived from Equation 15. Thus, the permissible length B is:

$$B = \left(\frac{3\pi^2 K_m^2 d^{16/3} H}{4^{10/3}(qL)^2}\right)^{1/3} \tag{16}$$

Corrugated collectors

If the collectors have the same spacing L_c, the same formulae may be used for their calculation, substituting their spacing L_c and length B_c for L and B. If they do not have the same spacing, calculations have to be made separately for each section of the collector. The spacing of laterals, and, thus, the distances of inflow points along the collector, has only little effect, provided that more than five laterals are involved.

MAINTENANCE STATUS AND REDUCTION FACTORS

The problem of clogging of drainpipes

In practice, drains are seldom completely clean. This is because some siltation always occurs, notably during and shortly after construction owing to the entrance of soil particles from the yet unsettled soil and/or envelope around the pipe when relatively large amounts of water enter. A layer of sediment usually forms over time. This sediment should be removed by maintenance, where it reduces the transport capacity of the pipe too much. Siltation may be also caused by other materials, e.g. iron oxides. Moreover, plant roots as well as certain animals may enter into drainpipes and hamper their proper functioning. Detailed information about the problem of clogging of pipes and envelopes is given in FAO (2005).

Siltation differs greatly from place to place and even in the same drainpipe. In particular, sunks in the alignment of the pipe cause siltation problems. Therefore, drain installation design and construction practices should take care to avoid the presence of such vulnerable stretches.

Entry of soil and plant roots can be prevented largely by a good envelope around the drains, by construction at sufficient depth, or by using non-perforated pipes for the stretch that crosses under a row of trees. However, for clogging by chemical precipitates, such as iron, this is not the case.

In addition to the effectiveness and durability of the drain envelope, the clogging of drains is connected with cleaning operations and their frequency. Drainpipe maintenance frequency depends on soil conditions and other circumstances. It is hardly needed for well-constructed drains surrounded by a stable soil or by an envelope and without iron precipitation phenomena, whereas in others deterioration is rapid. The latter is often the case under artesian seepage, which often induces ochre deposition, and in acid sulphate soils (cat clay soils and cat sands), where precipitation of iron compounds is also common.

Therefore, the design usually allows for a certain amount of clogging, which depends on the geohydrological and soil conditions at drain level and on the anticipated frequency of inspection and cleaning.

Maintenance status

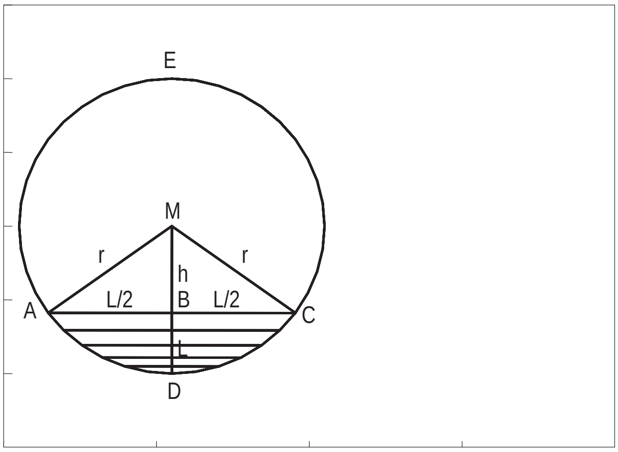
To take account of the aspects described above, the "maintenance status" is used as a parameter in the programs for calculating drain diameters. As mentioned above, maintenance status is a combination of:

- local circumstances (envelope materials, soils, ochre formation, etc);
- maintenance operations (frequency, intensity, availability of adequate equipment, etc).

Maintenance status has little to do with a specified rate of cleaning, but it is an indication of the state of cleanliness in which the drains can be kept under the given conditions. Under certain conditions, almost no maintenance is needed to realize a "good" maintenance status. This is the case with well-constructed drains in stable soil layers. In other conditions, much effort is required to keep it "fair", as is the case with unstable silt soils and where iron clogging is a severe problem.

This means that under an expected "poor" maintenance status even frequent cleaning is not sufficient. Hence, larger diameter pipes should be used than under an "excellent" status. Therefore, a reduction should be applied to the described formulae, by multiplying K_m with a correction factor f (e.g. f = 0.8).

FIGURE A20.3
Drain with sediment layer



Manning's K_m for drains with sediments

Figure A20.3 shows a drain *AECD*, with radius *r*, which is partly filled with sediment *ABCD*. The thickness of this layer *BD* is *l*, and the distance *BM* from the centre M is *h*.

For a clean pipe, Manning's formula can be written as:

$$Q = K_m A(r/2)^{2/3} s^{1/2} \tag{17}$$

A correction for the sediment layer is obtained as follows.
The angle $\angle AMC$ is φ, so $\angle AMB = \angle BMC = \varphi/2$.
The thickness of the layer is:

$$l = r - h = r\left[1 - \cos\frac{\varphi}{2}\right] \quad \text{and} \quad \varphi = 2\arccos(1 - l/r) \tag{18}$$

The area available for water flow A' is:

$$A' = \pi r^2 - \left(\varphi\frac{r^2}{2} - h\frac{L}{2}\right) = \pi r^2 - \left(\varphi\frac{r^2}{2} - r^2\sin\frac{\varphi}{2}\cos\frac{\varphi}{2}\right) = r^2\left(\pi + \frac{\sin\varphi}{2} - \frac{\varphi}{2}\right) \tag{19}$$

where the angle φ is expressed in radians.
Thus, the reduction factor for diminished area (A' instead of A) is:

$$f_1 = \frac{A'}{A} = \frac{r^2\left(\pi + \frac{\sin\varphi}{2} - \frac{\varphi}{2}\right)}{\pi r^2} = 1 - \frac{\varphi - \sin\varphi}{2\pi} \tag{20}$$

The hydraulic radius was $R = r/2$ and becomes:

$$R' = \frac{r\left[\pi + \frac{\sin\varphi}{2} - \frac{\varphi}{2}\right]}{2\sin\frac{\varphi}{2} + 2\pi - \varphi} \tag{21}$$

Thus, the reduction factor for *R* is:

$$f_2 = \frac{R'}{R} = \frac{2\pi + \sin\varphi - \varphi}{2\sin\frac{\varphi}{2} + 2\pi - \varphi} \tag{22}$$

Therefore, the drain discharge is reduced to:

$$Q' = K_m f_1 A(f_2 r/2)^{2/3} s^{1/2} = K'_m A(r/2)^{2/3} s^{1/2} \tag{23}$$

The correction factor for K_m is:

$$f = f_1 f_2^{2/3} \quad \text{and} \quad K'_m = fK_m \tag{24}$$

Table A20.4 shows the *f* values calculated for different fractions of sediment height and area. These values are represented in Figure A20.4.

Categories according to maintenance status

For the reasons discussed above, maintenance can only be specified in a global way. From the data in Table A20.4, the following choices were made with respect to maintenance status by distinguishing five categories. These categories have been defined in terms of the relative height of sediments in the drainpipes (Table A20.5). Table A20.5 shows the influence of maintenance status on the flow in partially clogged drains.

The maintenance status should be envisaged in the design stage. As only a rough classification is possible, the categories in Table A20.5 have been distinguished, for which the corresponding f values have been used in the programs. For these maintenance groups, the f factors will be used in the programs for drainpipe design. The f values are valid for Manning's equation. To avoid unnecessary complications, the programs also use these values in the Darcy–Weissbach approach for smooth pipes. The K_m values are multiplied by f to obtain "corrected" values K'_m, and the coefficients a must be divided by f^2 to obtain "corrected" values for a_c:

$$a_c = \frac{a}{f^2} \tag{25}$$

ZUIDEMA'S METHOD FOR CORRUGATED LATERALS

From numerous observations on existing corrugated laterals, Zuidema and Scholten (1972) found good agreement with Blasius' formula where a larger a coefficient was taken. They recommended using the value a = 0.77 for these pipes. This method is included as an option in the programs. It appears that the results obtained in this way are similar to or slightly more conservative than those for Manning's equation with "narrow rib spacing" (in the programs K_m = 65) and with a correction factor f = 0.923, corresponding to "good maintenance".

DRAIN LINES WITH INCREASING DIAMETERS

The above considerations refer to drains composed of one pipe diameter only. Long laterals and collectors usually require pipes of successively larger dimensions. Because of the rapid increase in prices with size, it often pays to replace the upstream part of the

FIGURE A20.4
Correction factors for Manning's K_m

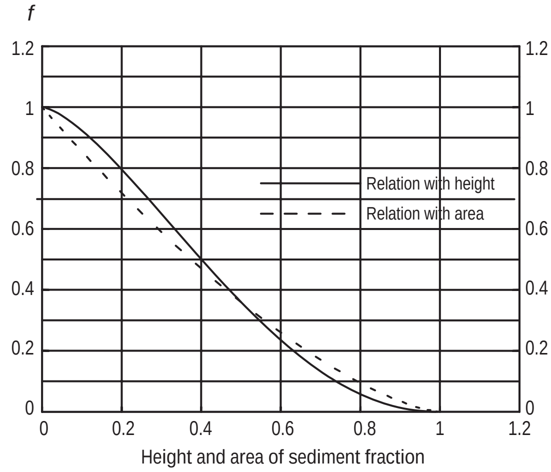


TABLE A20.4
Correction factor f for pipes with sediment

Fraction of sediment height	Fraction of sediment area	Factors		
$l/2r$	$1 - A'/A$	f_1	f_2	f
.050	.019	.981	.986	.972
.100	.052	.948	.961	.923
.150	.094	.906	.930	.863
.200	.142	.858	.894	.796
.250	.196	.804	.854	.724
.300	.252	.748	.810	.650
.350	.312	.688	.764	.575
.400	.374	.626	.715	.501
.450	.436	.564	.664	.429
.500	.500	.500	.611	.360
.550	.564	.436	.556	.295
.600	.626	.374	.500	.235
.650	.688	.312	.441	.181
.700	.748	.252	.382	.133
.800	.858	.142	.259	.058
.850	.906	.094	.196	.032
.900	.948	.052	.131	.013
.950	.981	.019	.066	.003
.990	.998	.002	.013	.000

TABLE A20.5
Flow reduction in partially clogged drains

Maintenance status	Cross-section clogged (%)	Reduction factor for flow f
New pipe	0	1.000
Excellent	5	0.972
Good	10	0.923
Fair	20	0.796
Poor	40	0.501

FIGURE A20.5
Drain line with two diameters and a given slope; transition point *B* at the drain

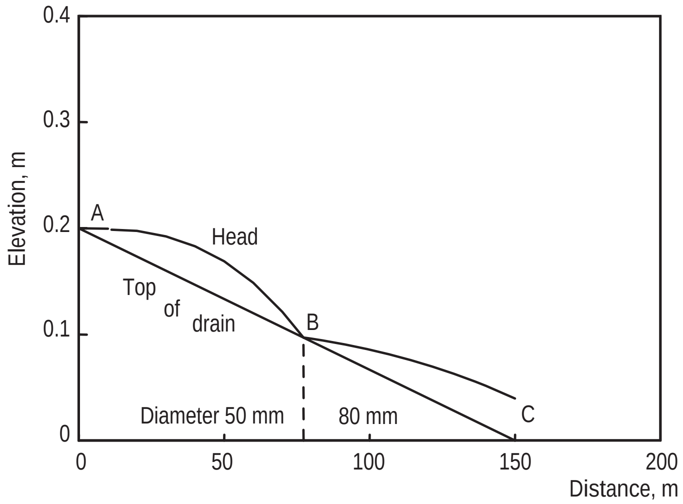
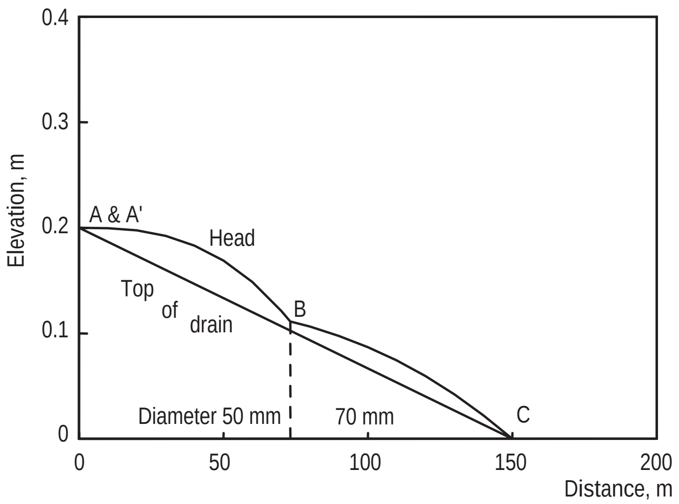


FIGURE A20.6
Drain line with two diameters and a given slope; transition point *B* above the drain



system – where the flows are still small – by a section of smaller size pipe, and use gradually larger ones downstream. The following sections consider drains of two sizes.

Effect of drain slope

Where a multiple drain is running full and slopes over the entire head ("full slope"), the head at the transition cannot fall below the top of the drain at that point. This is illustrated by Figure A20.5, where the transition point *B* lies at the top of the drain.

In Figure A20.5, it may be observed that drain *AC*, with given slope (0.20/150 m/m) consists of 50-mm and 80-mm pipes. *B* is a critical point determined by the head loss in the first section. The drain is running full and the head is not allowed to fall below the top of the drain. At *C*, some head is still available. Thus, the system is not very efficient. The consequence is that the drain has excess capacity and that the available head is not used entirely for water transport. The outlet at *C* could even be "drowned" to satisfy the design head at point *A*.

Figure A20.6 gives an example where this is not the case, because the transition point *B* lies above the drain and the full available head is used.

Drain *AC*, with given slope, consists of 50-mm and 70-mm pipes. *B* is not critical and the hydraulic grade line lies above the drain, at the intersection of the curves *AB* and *BC*. No extra head is available at *C*.

In the programs, attention has been given to these aspects.

Given slope

It is supposed that the drain slope equals $s = H_t/B_t$ so that – at design discharge – there is no water above the upper end of the drain (Figure A20.5).

The first section *AB* has a length B_1, governed on the one hand by Equation 5 or 14; on the other, by the given slope. From the latter, it follows that the head loss in this section equals:

$$\Delta H_1 = H \frac{B_1}{B} \tag{26}$$

where H is the design head.

Inserting ΔH_1 in Equation 5 or 14, with $B_0 = 0$ (first section), and rearranging, leads to:

$$B_1 = \left(\frac{H}{F_1 B}\right)^m \tag{27}$$

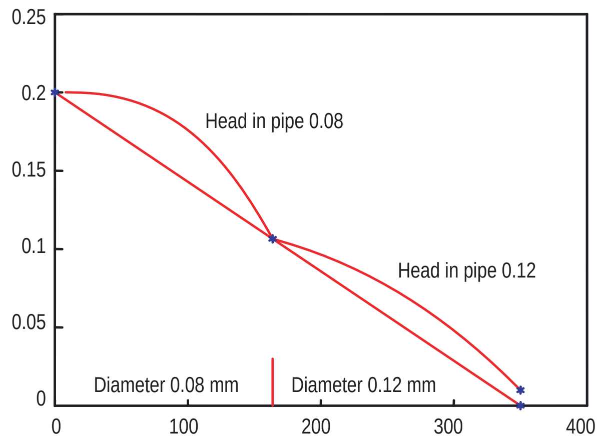
For smooth pipes, $F_1 = F_s$ with $d = d_1$ and $m = 4/7$; for corrugated pipes, $F_1 = F_c$ with $d = d_1$ and $m = 1/2$

If B_1 exceeds the total length B, the first section is already sufficient to meet the requirements. In this case, a combination with narrower pipes might be used.

The second section, with diameter d_2, causes a head loss ΔH_2, for smooth drains according to Equation 5, for corrugated pipes to Equation 14. The factors F_s or F_c are now calculated with $d = d_2$.

FIGURE A20.7

Hydraulic heads in a full drain, composed of pipes with different diameters



The total head loss $\Delta H_t = \Delta H_1 + \Delta H_2$ must be smaller than or at most equal to the required H. If greater, a second section with larger diameter must be chosen or another combination be tried.

Hydraulic heads along the drain

The available head along the drain depends on the distance x from the beginning. In the programs, they are expressed as head above outlet level. Thus:

$$H_x = H - \Delta H_i \qquad i = 1,2 \tag{28}$$

For the first section, where $x \leq B_1$, H_x is calculated from:

$$H_x = H - \Delta H_1 = H - F_1 x^n \tag{29}$$

and for the second, where $x > B_1$:

$$H_x = H - \Delta H_1 - \Delta H_2 = H - (F_1 - F_2) B_1^n - F_2 x^n \tag{30}$$

For smooth pipes:
$F_1 = F_s$ with $d = d_1$ and $n = 101/4$;
$F_2 = F$ with $d = d_2$ and $n = 11/4$.
For corrugated pipes:
$F_1 = F_c$ with $d = d_1$ and $n = 3$;
$F_2 = F_c$ with $d = d_2$ and $n = 3$.

The considerations given above form the basis of the program DRSINGLE for the design of smooth and corrugated laterals and collectors consisting of one section. For two or more sections with different diameters and also of different types, the program DRMULTI can be used. Figure A20.7 shows a longitudinal profile along such a drain. At the upstream end, no water is standing above the drain, and at the outlet downstream there is still some head available. This indicates that the proposed combination is sufficient to carry the design discharge.

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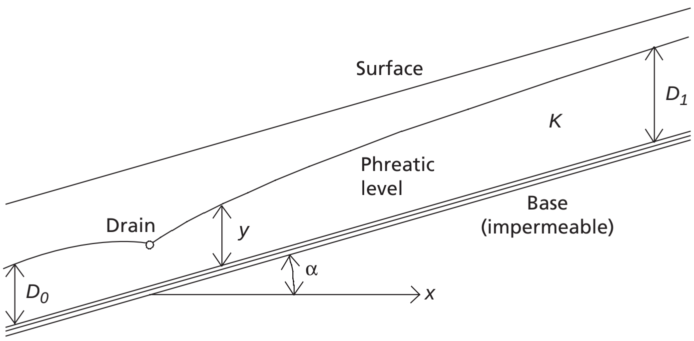
Annex 21
Interceptor drains

FLOW FROM SURROUNDINGS

Inflow from higher places and from leaky irrigation canals can sometimes be captured by interceptor drains, especially where it passes through relatively shallow aquifers. Such drains can take the form of pipes or open ditches. In the latter, the stability of the side slopes is often problematic if large amounts are to be captured. Better solutions are gravel-filled trenches provided with a suitable pipe of sufficient capacity to carry the discharge.

FIGURE A21.1
Glover–Donnan analysis of an interceptor drain



HILLSIDES

An analysis of the interception of flow from hillsides of uniform slope was given by Donnan (1959), as represented in Figure A21.1.

The flow from upstream, per metre of length, is:

$$q_1 = KD_1 tg\alpha \tag{1}$$

and downstream:

$$q_0 = KD_0 tg\alpha \tag{2}$$

The drain discharges, per metre of length, is:

$$q = q_1 - q_0 \tag{3}$$

where:
q_1 = upstream flow per metre of length (m^2/d);
q_0 = downstream flow per metre of length (m^2/d);
K = permeability (m/d);
D_1 = upstream thickness of flow (m);
D_0 = downstream thickness (m);
α = angle of slope (rad).

In this analysis, the downstream flow has a thickness D_0, which is entirely governed by the distance of the drain above the impermeable base (which is governed by the drain depth).

The upstream thickness varies from D_0 near the drain to D_1 far upstream. A given thickness y appears at a distance x from the drain:

$$x = \frac{1}{tg\alpha}\left[D_1 \ln \frac{D_1 - D_0}{D_1 - y} - (y - D_0)\right] \qquad D_0 < y < D_1 \tag{4}$$

where:
x = distance from drain (upstream) (m).

On hill slopes, hydrological conditions are often much more complicated. Wet or saline spots caused by seepage may sometimes be protected by an interception drain laid at the upper end of the affected field.

This formula ignores the radial resistance encountered in the convergence of the stream lines onto the drain. Because of this resistance, D_0 has to be increased, with the resulting head Δh.

In a homogeneous soil, this radial resistance can be estimated by Ernst's formula:

$$W_r = \frac{1}{\pi K} \ln \frac{2D_0}{\pi d} \tag{5}$$

and

$$h_r = (q_1 - q_0) W_r \tag{6}$$

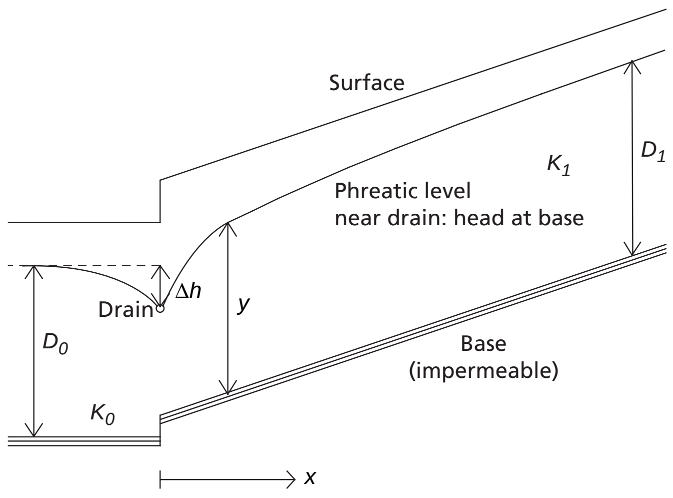
where:
d = effective diameter of drain (m);
W_r = radial resistance (d/m);
h_r = extra head from radial resistance (m).

In the described case of a homogeneous soil and a constant angle α, this increase in D_0 will usually be slight. However in the cases described below, the consequences can be considerable.

In most cases, an interceptor drain will be laid if: the slope decreases, the depth of the impermeable base becomes less, or the permeability decreases. At places where these occur, hillside flows tends to come too close to the surface and cause waterlogging, eventually followed by soil salinization. Based on the above theory, the program INCEP gives the required effective diameter of the drain, necessary to diminish the radial resistance to a sufficiently low level. It is valid for a non-layered soil (Figure A21.2), and allows jumps in thickness and permeability at the drain. The arithmetic averages of thickness and permeability are used in order to calculate the radial resistance.

The capacity of pipes for interceptor drains must be calculated separately from the discharge per metre, their length and their longitudinal slope. The programs DRSINGLE and DRMULTI can be used for this purpose. The largest value from both calculations (for effective diameter and for capacity) must be taken.

FIGURE A21.2
Definition sketch for program INCEP



Conditions become far worse where the drain cannot reach well-permeable subsoil and remains within a less permeable top layer, a case covered by program INCEP2. Then h_r soon reaches such high values that a single interceptor drain is not sufficient, and a wide ditch or even regular drainage is needed.

The program INCEP2 supposes that the drain trench or open ditch has a flat bottom that is located in the topsoil and receives the flow from the permeable subsoil (Figure A21.3). In this case, the exact solution can be found by complex transformation. An excellent approximation for this case is obtained by calculating the parallel lines flow between the border with the permeable subsoil and the ditch bottom with Equation 7, using a correction factor of 0.88.

$$b = a\left(\frac{q}{K_1 \Delta h} - 0.88\right) \quad \text{for} \quad b/2 > a \qquad (7)$$

where:
a = distance to more permeable subsoil ($K_1 < 0.1K_2$) (m);
b = width of drain trench or ditch bottom (m);
K_1 = permeability of topsoil (m/d);
K_2 = permeability of subsoil ($K_2 > 10\ K_1$) (m/d);
q = upward flow (m²/d);
Δh = difference in piezometric head above the trench bottom (m).
INCEP2 provides both solutions for b.

LEAKY CANALS AND UPSTREAM FIELDS

The same principles apply for interceptor drains catching leakage from irrigation canals of losses from upstream fields.

For leaky irrigation canals, the best way is to reduce the water losses by lining. Where that is impossible, and damage is occurring by nearby waterlogging or salinization, interceptor drains are a second option. Then, the incoming flow per metre, q_1, is half of the losses from the canal. These losses can be estimated by measuring the fall in water level in an isolated section.

However, these losses are proportional to the difference in head between the canal water and the nearby groundwater. Therefore, drainage will increase both head and inflow (Figure A21.4). Lowering the groundwater increases the flow with a factor h_2/h_1.

The incoming inflow can be calculated if the original loss and the factor h_2/h_1 and q_0 are determined:

$$q_1 = q_0 (h_2 / h_1) \qquad (8)$$

where:
q_0 = original outflow from canal (m²/d);
q_1 = outflow from canal after interceptor drainage (m²/d);
D = thickness of aquifer (m);
h_1 = hydraulic head in the canal (m) above original groundwater level;
h_2 = hydraulic head in the canal (m) above drain level.

On the other hand, losses from upstream irrigated or rainfed lands will not be influenced by interceptor

FIGURE A21.3
Drain trench or ditch in low-permeable topsoil

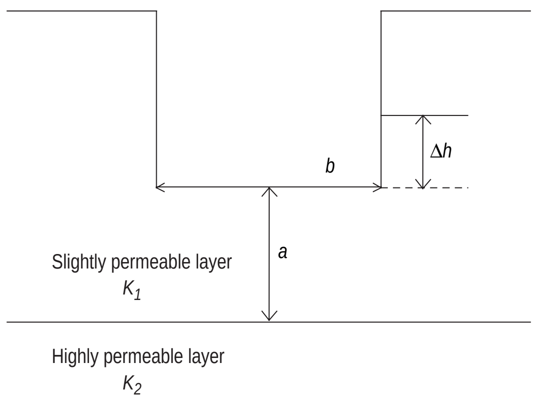


FIGURE A21.4
Leakage from a canal

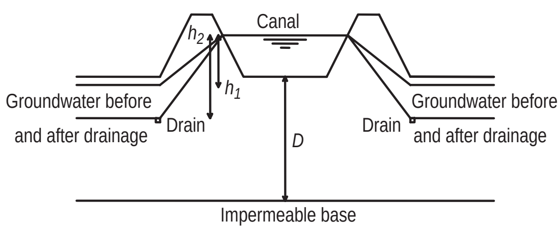
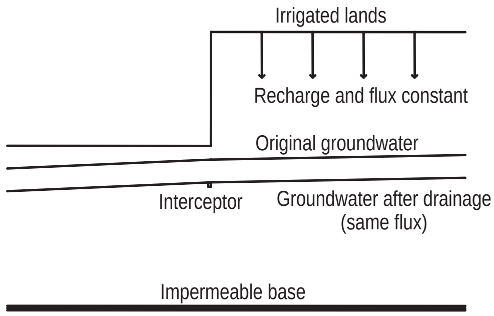


FIGURE A21.5
Cross-section of flow from higher lands with interceptor drainage



drainage. This is because these losses are a component of the upstream water balance, as can be observed from the cross-section shown in Figure A21.5.

These types of losses can be estimated from water balances or by applying Darcy's Law to the resulting groundwater current.

Where the canal or field losses are known, the programs INCEP and INCEP2 can be used to find the necessary trench width for the interceptor drain.

RESULTS

In many cases, the width is such that a regular drainage is to be preferred, for which the program ARTES gives some guidelines. Alternatively, a wide ditch can be considered, especially at intermediate values for the required width. However, as side slopes tend to become unstable under such circumstances, it is often necessary to stabilize them. This can be achieved by covering the side slopes with a gravel cover or by making a wide, gravel-filled trench provided with an outlet pipe.

REFERENCE

Donnan, W.W. 1959. Drainage of agricultural lands using interceptor lines. *J. Irri. Drain. Div. Proc. ASAE*, 85, IR 1:13–23.

Annex 22
Drainage by vertical wells

INTRODUCTION

"Vertical drainage" is possible under favourable geological circumstances:

- a good aquifer underneath;
- an aquifer containing water with a low salt content, so that the water can be used;
- not too large resistance between soil and aquifer.

Figure A22.1 gives a sketch of the method.

Two types of wells are considered: those fully penetrating the aquifer; and non-penetrating "cavity" wells. They are supposed to form a large array of squares (Figure A22.2) or triangles (Figure A22.3). In Figures A22.2 and A22.3, for one well, the flow region and the sphere of influence are indicated.

This method is mainly used in arid regions where use of the water for irrigation has often led to serious overpumping. In some areas, the lowering of the water levels in the aquifer has led to attraction of salty water from elsewhere, often from deeper layers, sometimes from the sea. In the long run, in an arid climate, salt will inevitably accumulate. However, this process is usually very slow, owing to the large amount of water stored in an aquifer. Thus, vertical drainage may be a temporary solution to a high water table situation.

Nevertheless, the method can be used to control groundwater levels. This is illustrated by the following (steady-state) theory.

FIGURE A22.1
Flow to a well in a semi-confined aquifer

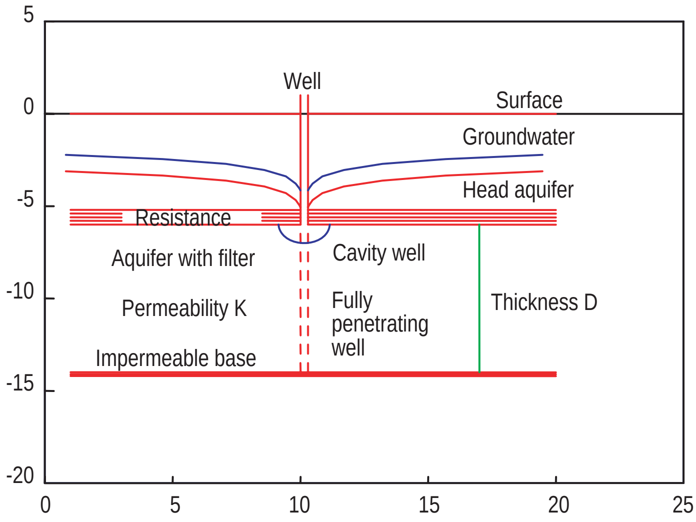
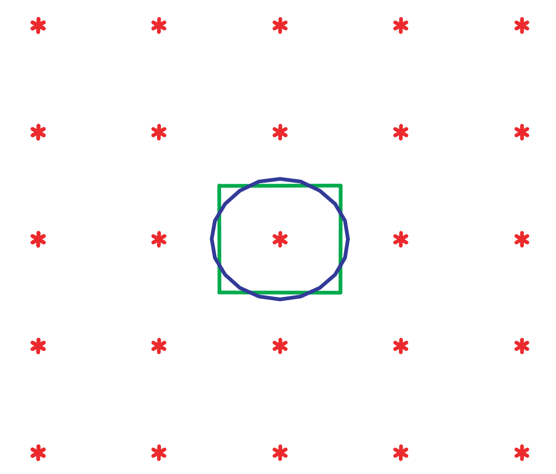


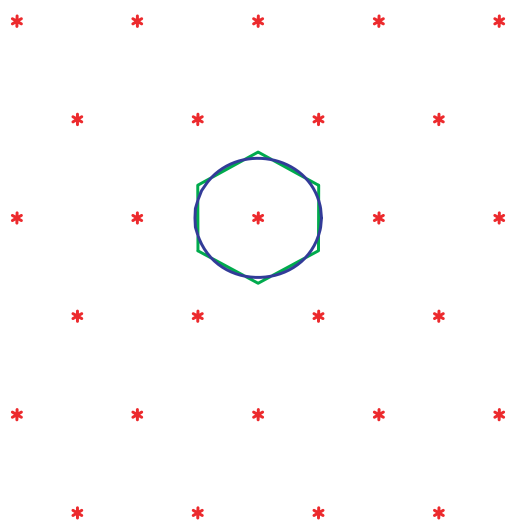
FIGURE A22.2
Quadratic well net



FULLY PENETRATING WELLS

An area is drained by an array of evenly spaced deep wells tapping an aquifer (Figure A22.1). This array may be quadratic or triangular and contains a large number of wells that penetrate the entire aquifer. Each of them drains an equivalent

FIGURE A22.3
Triangular well net



square (Figure A22.2) or a hexagon (Figure A22.3), depending on the array pattern (quadratic or tri-angular, respectively). This outer limit is approached by a circle of equal area, with radius *R*, and the flow is cylindrical towards the well. The entire well-field is very large and exchange of water with the surroundings may be ignored. Recharge is from the surface.

The aquifer is overlain by a relatively thin layer of low permeability, which separates it from the shallow phreatic water. It offers a certain resistance to flow between groundwater and aquifer, but does not prevent it entirely. Thus, pumping lowers not only the hydraulic head in the aquifer, but also the shallow groundwater level.

The aquifer has a permeability *K* (metres per day) and a thickness *D* (metres), and, thus, a transmissivity $T = KD$.

Between the aquifer and the groundwater is a semi-permeable layer of low vertical permeability *K'* and thickness *d´*. This leads to a certain resistance $c = d'/K'$, which is considered independent of the water levels. If *K'* is in metres per day and *d'* in metres, *c* is in days.

Through this layer, the aquifer is recharged by rainfall or irrigation, with an intensity *q* (metres per day).

A first estimate about the square spacing of wells is that it should be of the order of a characteristic length of the aquifer system:

$$\lambda = \sqrt{KDc} \tag{1}$$

where:

$c = \dfrac{d'}{K'}$ = resistance of semi-confining layer (d);

D = thickness of aquifer (m);
d' = thickness of semi-confining layer (m);
K = permeability of aquifer (m/d);
K' = permeability of semi-confining layer (m/d);
λ = characteristic length (m).

Greater insight is obtained from formulae describing the lowering of the groundwater when an aquifer is pumped by a network of wells under the following conditions (Figure A22.1):

- the wells are fully penetrating and tap the aquifer over its entire depth;
- between groundwater and aquifer, there is a layer of low permeability that gives a certain resistance to vertical flow, but still allows its passage;
- there is equilibrium between the amounts pumped and the recharge (steady state);
- no water is entering the well-field laterally from outside.

The yield of each well Q_w is taken to be positive, as is the flow Q towards the well. According to Darcy's Law and taking absolute values for Q, for the flow in the aquifer:

$$|Q| = 2\pi rKD\frac{dH}{dr} \tag{2}$$

On the other hand, the rainfall or irrigation excess should create the same flow:

$$|Q| = \pi\left(R^2 - r^2\right)q \tag{3}$$

so that both expressions for Q are equal, provided that there is no lateral inflow from around the well-field.

Finally, the vertical resistance c of the layer between groundwater and aquifer leads to a recharge:

$$q = \frac{h(r) - H(r)}{c} \tag{4}$$

where, in these equations:

h = groundwater level (m);
H = head in aquifer (m);
q = recharge (m/d);
$|Q|$ = flow towards well, absolute value (m^3/d);
Q_w = discharge of well, absolute value (m^3/d);
r = distance from well centre (m);
r_w = radius of well (m);
R = radius sphere of influence of well (m);
in which Q, h and H are functions of r.

At the watershed boundary with other wells, $r = R$ and $Q = 0$. At this critical point, h should have a prescribed maximum level. If h and H are expressed with respect to soil surface, the groundwater should be at a certain depth (e.g. 2.0 m), so that $h(R) = -2.0$. Then, with a given recharge q and resistance c, $H(R)$ can be calculated from Equation 4.

Then, it follows from the basic equations that:

$$2\pi KDr\frac{dH}{dr} = \pi q\left(R^2 - r^2\right) \quad \text{or} \quad dH = \left(\frac{qR^2}{2KD} - \frac{qr^2}{2KD}\right)\frac{dr}{r} \tag{5}$$

Integration gives for the head H in the aquifer:

$$H(r) = H(R) - \frac{qR^2}{2KD}(\ln R - \ln r) + \frac{q}{4KD}\left(R^2 - r^2\right) \tag{6}$$

R is taken as the radius of a circle with the same area as the quadrangular or triangular region served by one well.

Under these conditions, the following equation is valid for the groundwater height h:

$$h = H - qc \quad \text{for} \quad r_w \le r \le R \tag{7}$$

Midway between the surrounding wells, the groundwater table should be lowered to the required depth, but it will be deeper near the well. The head in the aquifer is lower than the groundwater level because of the resistance between the two. If more water is being pumped than the recharge, there will be overpumping, leading to a gradual depletion of the aquifer. Although this is usually not sustainable, overpumping can be a temporary solution for water scarcity ("groundwater mining"), high groundwater tables, and soil salinization.

For a quadratic pattern (Figure A22.2) with well spacing distances L, the area A served per well is:

$$A = L^2 = \pi R^2 \qquad \text{or} \qquad R = \frac{L}{\sqrt{\pi}} = 0.564L \tag{8}$$

For a triangular array (Figure A22.3), the region drained by a well is hexagonal, where:

$$A = \frac{1}{2}L^2\sqrt{3} = \pi R^2 \qquad \text{or} \qquad R = \frac{L\sqrt[4]{3}}{\sqrt{2\pi}} = 0.525L \tag{9}$$

CAVITY WELLS

In some areas, wells are made by removing sand from the aquifer by heavy pumping. A washed-out cavity is formed at the top of the aquifer, which remains intact during the following period of less heavy abstraction (Figure A22.1, in blue). Compared with fully penetrating wells, they encounter an extra resistance, but their diameter is larger, although the actual size is rarely known.

The cavity is supposed to be a half-sphere with radius r_w. In its vicinity, the flow is spherical and an extra resistance occurs. This effect is estimated by assuming that the flow to such non-penetrating wells breaks down as follows:

- cylindrical flow from the outer limit R to a distance r_d from the well, so that Equation 6 can be used for $r > r_d$; arbitrarily, r_d can be taken as the lowest value of D or R;
- spherical flow from distance r_d to the spherical cavity with radius r_w.

For r_d, arbitrarily:

$$r_d = D \tag{10}$$

where:

D = thickness of aquifer (m);

and $D < R$.

For very thick aquifers or a very dense network, D can become larger than R. Then, for $D > R$:

$$r_d = R \tag{11}$$

The cylindrical part of the flow is described by Equation 6 for $r_d < r \leq R$.

The head in the aquifer is calculated (or approximated) by:

$$H(r) = H(r_d) - \frac{qR^2}{2K}\left(\frac{1}{r} - \frac{1}{r_d}\right) + \frac{q}{2K}(r_d - r) \qquad r_w \leq r \leq r_d \tag{12}$$

There are several assumptions involved, but the greatest uncertainty lies in the unknown diameter (thus, radius r_w) of the cavity. Although this is an approximation, the errors are small enough for practical purposes.

APPLICABILITY OF THE METHOD

If more water is being pumped than the recharge, there will be overpumping, leading to a depletion of the aquifer. Moreover, an equilibrium abstraction will also not be sustainable in an arid region. This is because its use for irrigation will lead ultimately to a harmful accumulation of salt in the aquifer. However, both overpumping and equilibrium abstraction may be used as temporary solutions for water scarcity, high groundwater, and soil salinity. The time horizon depends on the local circumstances and requires further study.

The program WELLS is based on these considerations. The differences between fully penetrating and cavity wells relate to an extra radial resistance in the vicinity of the latter (red and blue lines in Figure A22.1). This extra resistance is caused by flow to a sphere instead of a long cylinder.

Annex 23
Computer programs for drainage calculations

GENERAL CONSIDERATIONS

The programs first mention their name and purpose. Then, the following three questions appear:

Notation of decimals

The use of the decimal separator in your country, point or comma, is requested. Answer 1, 2 or 9. If a comma, a warning is given to ENTER all decimal data with a point as separator. Using a comma would lead to serious errors. Answer the question with 9 if you like to quit.

Project name

A project, or a section of it, must be indicated by a name of at most four characters, which will form part of the output filename. The limited length allowed is because of the limited size of filenames under DOS.

Certain rules must be followed:

- The program asks for a project name, put between single quotation marks. A maximum of four characters are allowed between those quotes, so that abbreviations are often needed (e.g. 'proj' for project). It is advisable to divide large projects into sections and use section names (usually one or two characters) as the project name. The single quotes indicate that the name is entered as a character string, even if it is a pure number ('23').
- Project names with less than four characters are padded with minus signs in order to obtain filenames of equal length. Thus, 'A2' automatically becomes 'A2--'.
- When the session is finished and the program closed, the data are saved in a file. The filename has two characters indicating the kind of program, followed by this project name and the extension TXT, for example, file SPA2--.TXT for program drain spacings (SP) with project name entered as 'A2'.
- However, as new data become available, this existing file cannot be used again, because this project name is already occupied. If tried, a warning is given that the name is already in use and that a new name must be given. Thus, it is advisable to end with a number, so that (for example) project 'A2' can be followed later by 'A3', where both cover the same area 'A'.

Location

After this short indication for the project (or part of it), the program asks for the location within. Each project file can store observations from different locations, which are indicated by a name of at most ten characters (letters and numbers).

Again, the name must be between single quotation marks. The location can be a plot number ('123', ' C14'), a name ('Johnson', 'Bahawalpur'), or a combination ('7aq2n4').

If processed in the same session, the data for several locations within the same project are combined into one file, which contains the name of the project (A2--- .TXT in the earlier example). This project file contains all locations treated and is closed automatically at the end of the session. As mentioned above, the name cannot be used again.

All project files obtained are listed in a file LIST**, beginning with LIST, followed by two characters for its kind (LISTSP.TXT contains all drain spacing [SP] calculations made).

Output files

For each project, the results are written to a file, the name of which is mentioned by the program.

If reading in DOS, take care to copy this indication literally, including the signs -, --, and --- used if some of the four positions are blanks (project 'A' leads to file A---.TXT, and project 'AB' to file AB--.TXT).

Under Microsoft Windows, this difficulty is avoided. Just double-click the icon.

GUMBEL'S METHOD

GUMBEL, for estimating extreme values

After the three general questions (notation of decimal, project name, and location), the program moves on to specifics:

- Input of the extreme values (e.g. the highest three-day precipitation in a given month, in millimetres) from keyboard or from data file. They are processed using Kendall's method.
- The return period (T) related to hydrological data (usually in years). The program gives the expected values.
- End the series of T with 999. A graph appears on screen with the data on the vertical axis, and the Gumbel distribution on the horizontal, with the data plotted according to Kendall. The Kendall line is shown in red. The graph is useful to visually detect upward or downward trends, which make the prediction less valid and indicate that the method may not be applicable in this case: too low if upward, too high if downward.
- Leave the graph with ENTER.

Continuation, output and example

The process can be repeated in a new case belonging to the same project. With another project or END, the files are closed and the results written to file GU****.txt, where GU stands for "Gumbel" and **** is the abbreviated project name. These filenames are mentioned in LISTGU.TXT.

Figure A23.1 gives the output for extremes of total precipitation occurring during 1 to 7 successive days (1d to 7d) in an area in eastern Spain. The climate is Mediterranean, with heavy rainfall in autumn.

PERMEABILITY MEASUREMENTS

AUGHOLE, for permeability from auger-hole measurements

After the three general questions (notation of decimal, project name, and location), the program moves on to specifics:

- Which unit is chosen? Answer 1, 2 or 3. Recommended is 2, the use of centimetres, in contrast to most other programs.
- Diameter and depth of the auger hole in the chosen units?
- Location of the impermeable base?
- Groundwater present of no? This determines the method: normal or inverse (less reliable).

Normal method

For the "normal" method, the initial depth of the water in the hole is measured after equilibrium. Then, some water is pumped out and the position of the water table is given at different times:

- Equilibrium groundwater depth?
- Water depth at time t_1?
- Water depth at time t_2? (should be less).
- Time interval t_2 - t_1 in seconds?

Inverse method

In dry soils, the groundwater may be too deep to measure the permeability of the upper layers. In this case, the inverse method can be used. Water is poured in, and its lowering is measured over time. The method is less reliable and should be used only if there is no other possibility. Moreover, some soils swell slowly and have a lower permeability in the wet season.

Option “no groundwater” is followed, and the fall of the water level and the time interval are entered.

Continuation

The resulting permeability appears on screen.

Next items:

- Same or new auger hole or END? The first option allows another measurement in the same auger hole, e.g. in the subsequent interval. The other two finish the calculation and show the mean value and its standard deviation on screen.
- The next item can be in the same project or not. In the first case, the existing project file is continued. Otherwise, it is closed and the filename mentioned on screen as AU****.txt where AU denotes “auger hole” and **** is the abbreviated project name.
- This name is also added to the listing LISTAU.TXT, mentioning all existing auger-hole files.
- If “Other project or END” is selected, new names are required for project and location; “END” returns the user to the initial screen.

FIGURE A23.1
Printout of program GUMBEL

```
****** Gumbel Distribution ******
=======================================================
project: Pego; location: P-1d; case: Pego01.txt
return period  value
   2.0    111.3565
   5.0    188.6375
  10.0    239.8043
  20.0    288.8846

****** Gumbel Distribution ******
=======================================================
project: Pego; location: P-2d; case: Pego02.txt
return period value
   2.0    136.6437
   5.0    232.8728
  10.0    296.5849
  20.0    357.6990

****** Gumbel Distribution ******
=======================================================
project: Pego; location: P-3d; case: Pego03.txt
return period value

****** Gumbel Distribution ******
=======================================================
project: Pego; location: P-4d; case: Pego04.txt
return period value
   2.0    162.4361
   5.0    273.7000
  10.0    347.3664
  20.0    418.0289

****** Gumbel Distribution ******
=======================================================
project: Pego; location: P-5d; case: Pego05.txt
return period value
   2.0    171.9708
   5.0    283.2239
  10.0    356.8832
  20.0    427.5389

****** Gumbel Distribution ******
=======================================================
project: Pego; location: P-6d; case: Pego06.txt
return period value
   2.0    177.9110
   5.0    284.9797
  10.0    355.8684
  20.0    423.8667

****** Gumbel Distribution ******
=======================================================
project: Pego; location: P-7d; case: Pego07.txt
return period value
   2.0    186.2486
   5.0    291.2184
  10.0    360.7175
  20.0    427.3827
```

FIGURE A23.2
Printout of program AUGHOLE

```
****** Calculation of K from auger hole data ******
 ===================================================
project: OFL1; location: Swifterb; case: OFL101.txt

  diameter depth groundwater depth of  position of
   cm      cm    depth cm   base cm   hole bottom
 ----------------------------------------------------
   8.0    150.0     50.0    200.0     above base
 ----------------------------------------------------
 number   water level cm     time      K   stand.err.
  meas.      1       2        s      m/d   of mean
 --- direct method ---------------------------------
    1     85.0     83.0     20.0     .63
    2     80.0     78.0     24.0     .60
    3     70.0     68.0     31.0     .67
                         --------------------
                          mean   .63   .02
```

Example

In the project OFL1, at location Swifterb, an auger hole of 8 cm in diameter and 150 cm deep is made. The impermeable base is at a depth of 200 cm. Groundwater establishes a water level in the hole at a depth of 50 cm. Several measurements are taken after lowering to 90 cm below the surface. This gives $K = 0.63$ m/d, as shown by Figure A23.2.

PIEZOM, for permeability from piezometer measurements

After the three general questions (notation of decimal, project name, and location), the program moves on to specifics:

- Which unit is chosen? Answer 1, 2, or 3. Recommended is 2, the use of centimetres, in contrast to most other programs.
- Diameters of protection pipe and cavity in the chosen units?
- Length of protection pipe and cavity in the chosen units?
- Location of the impermeable base?
- Equilibrium groundwater depth below top of pipe?

Then, some water is pumped out and the position of the water table is given at different times:

- Water depth at time t_1?
- Water depth at time t_2? (should be less).
- Time interval $t_2 - t_1$ in seconds?

The "inverse method" is not included.

Continuation

The resulting permeability appears on screen.

Next items:

- Same or new piezometer hole or END? The first option allows another measurement in the same piezometer, e.g. in the subsequent interval. The other two finish the calculation and show the mean value and its standard deviation on screen.
- The next item can be in the same project or not. In the first case, the existing project file is continued. Otherwise, it is closed, and the filename mentioned on screen as PZ****.TXT where PZ indicates "piezometer" and **** is the abbreviated project name.
- This name is also added to the listing LISTPZ.TXT, mentioning all existing piezometer files.
- If "Other project" is selected, new names are required for project and location. "END" returns the user to the initial screen.

Output

The output is similar to that of AUGHOLE. Figure A23.3 gives an example.

CALCULATION OF DRAIN SPACINGS

SPACING, for drainage under "normal" (non-artesian) conditions

After the three general questions (notation of decimal, project name, and location), the program moves on to specifics:

- How is the size of drains expressed, (as diameter, as radius, as width of open ditches)? ENTER 1, 2 or 3.
- The size itself, in metres? Divide centimetres by 100 and always use a point for the decimal.
- The design discharge, in metres per day. Divide millimetres per day by 1 000.
- The required groundwater depth at this recharge, in metres below surface.
- The depth of drains (pipes or ditch bottoms), in metres below surface.

These general data appear on screen. If correct, ENTER 1; else 9 to restart the questions. Then:

- The number of layers distinguished: the first above drain level, the remaining strata below.
- Their thickness. That of the first is known, being the drain depth; for the others, it must be given.
- Their anisotropy. As this will seldom be available, it is advisable to use 1 above drain level, and below 4 if not clearly layered and 16 if so. This is a better guess than neglecting anisotropy.
- Their permeability, as measured by auger hole or piezometer or estimated from profile characteristics.

The soil data are shown and, if correct, the necessary calculations are made.

FIGURE A23.3
Printout of program PIEZOM

```
****** Calculation of K from piezometer data ******
 ========================================================
project:d; location: da nang; case: d---01.txt
 ========================================================
Piezometer
   diameter        length      groundw.  position
 pipe  cavity  pipe   cavity   depth     bottom
  cm     cm     cm      cm      cm        cavity
 ---------------------------------------------------------
  8.0    5.0   200.0    25.0    40.0     above base
 ---------------------------------------------------------
 number  water  depth cm  time      K    stand.err.
  meas.    1        2       s      m/d   of mean
 ---------------------------------------------------------
    1    120.0    115.0    12.0    3.29
    2    115.0    110.0    13.0    3.25
                      --------------------
                     mean 3.27   .02
```

Continuation

The project can be continued and then the data for the new location are added to the same file. If a new project is taken or the existing one is ended, the files are closed and the filename is mentioned on screen and added to LISTSP.TXT. Any new project needs another name.

Output and example

The results are visible on screen and put on file SP****.TXT, where SP denotes “spacing” and **** the abbreviated project name. Figure A23.4 gives an example of the output for project ‘aa’, location ‘amandabad’. The radial resistance W_r can be used as input in the programs NSDEPTH and NSHEAD.

FIGURE A23.4
Printout of program SPACING

```
******Drain spacings, steady state******
      Artesian influences not significant
 ========================================================
project: aa; location: amandabad; case: aa--01.txt
************  GENERAL INPUT DATA for SPACING  ************
 effective diameter of drain              .08  m
 design discharge of drain                .015 m/d
 design groundwater depth midway           .30  m
 design head above drain level            1.20  m
 design drain depth                       1.50  m
*********************************************************

*****************  Soil data **********************
 thickness layer 1, above drains     1.50  m
 thickness layer 2, below drains     2.00  m
 anisotropy factor layer 1           1.00  --
 anisotropy factor layer 2           4.00  --
 horiz. permeability layer 1         1.00  m/d
 horiz. permeability layer 2         2.00  m/d

-------------------  Results  ----------------------
 available head                     1.20  m
 radial resistance Wr                .97  d/m
 flow above drains/total flow        .20  --
 drain spacing L-Hooghoudt         43.     m
*********************************************************
```

NSABOVE, for drain spacing at non-steady flow above drain level only

After the three general questions (notation of decimal, project name, and location), the program moves on to specifics:

- Thickness of permeable layer (equal to drain depth or ditch bottom).
- Pipe drains or ditches. For pipes and dry and almost dry ditches, the Boussinesq approach is followed; for water-holding ditches, the Schilfgaarde method is used.
- For pipe drains and nearly dry ditches, there is choice between an "elliptic" initial situation, where the shallowest depth is midway between drains, or a total ponding of the entire area.
- In the elliptic case, the initial groundwater depth midway is asked (in ponding it is zero everywhere). In the Schilfgaarde method, the shape is initially elliptic.
- The required groundwater depth at time *t* and the value of *t*.
- For water-holding ditches, the (constant) water depth must be specified.
- If these data are correct, the soil characteristics are required: the permeability and the available storage (moisture volume fraction between saturation and field capacity).
- Calculations are made and the resulting drain spacing appears on screen.
- If initially ponded, a "lag time" is mentioned, an estimation of the time span between total saturation and the first lowering midway between drains.

FIGURE A23.5
Printout of program NSABOVE

```
****** Non-steady flow above drain or ditch bottom ******
 ==========================================================
project: a; location: a1; case: a---01.txt
 ******************************* Drains **********
 drain depth                  1.40 m
 depth impermeable base       1.40 m

Properties of permeable layer
 permeability (horiz.=vert.)    2.00 m/d
 storage coefficient             .12 --

----------------------  Results  -----------------
 groundw.depth          at t=  .00 d        .00 m  [everywhere]
 groundw.depth midway at t= 1.00 d        .20 m
 drain spacing . . . . . . . . .             L      19.     m
 estimated lag time                          .41 d
*********************************************************

****** Non-steady flow above drain or ditch bottom ******
 ==========================================================
project: a; location: a2; case: a---02.txt

**************** Ditches Schilfgaarde **********
 ditch water depth below surface       .80 m
 ditch bottom depth below surface    1.40 m
 depth impermeable base              1.40 m

Properties of permeable layer
 permeability (horiz.=vert.)          2.00 m/d
 storage coefficient                   .12  --

----------------------  Results  -----------------
 groundw. depth midway at t=   .00 d      .00 m  [elliptic]
 groundw. depth midway at t= 1.00 d      .20 m
 ditch spacing . .     L-Schilfgaarde     22.   m
 estimated lag time                       .00 d
*********************************************************
```

Continuation

The process can be repeated in a new case belonging to the same project. With another project or END, the files are closed and the results written to file NA****.txt, where NA stands for "Nonsteady Above" and **** is the abbreviated project name. These filenames are mentioned in LISTNA.TXT.

Output and examples

Figure A23.5 gives results at two locations in project 'a', of which location 'a1' has pipe drains, location 'a2' water-holding ditches. In the first case, the surface is considered ponded at the beginning; in the second case, the water table is initially elliptic. The difference in "lag time" to reach a nearly elliptic shape explains most of the difference in drain spacing.

NSDEPTH and NSHEAD, for drains above impermeable base

NSDEPTH gives the depth of the groundwater below surface, NSHEAD gives the head above drain level.

After the three general questions (notation of decimal, project name, and location), the programs move on to specifics:

- The permeability (equal above and below drain level), in metres per day.
- The storage coefficient, as volume fraction.
- The drain depth, in metres below surface.
- The thickness of the layer below the drains, in metres.
- The initial groundwater depth, the same everywhere: ponded or specified. If ponded, it is automatically zero; if specified, the initial depth is required.
- The radial resistance W_r near the drain (d/m). An estimate can be obtained from the program SPACING. The entrance resistance, met by flow into the drain, is ignored. For ditches, it is near zero; for good working drains, it is negligible, of the order of 0.1 d/m.

For abnormally high discharges, the outflow system can be handled by the pipes and ditches, but at higher heads and water levels. The following data allow an estimate:

- The design discharge of the outflow system, in metres per day. Divide millimetres per day by 1 000.
- The design head loss in this system, in metres. At high discharges, higher head losses are to be expected, leading to higher levels in this system.

After a heavy rain (or snowmelt), evaporation may help to lower the groundwater tables, but the influence diminishes the deeper they are. The following items allow an estimate:

- The potential evaporation, in metres per day. Divide millimetres per day by 1 000.
- The relationship of potential evaporation with groundwater depth, linear or exponential.
- The depth where evaporation becomes zero (linear) or the characteristic depth where it is reduced to 1/e times the value at the surface (exponential).

Check the input. If correct, continue with:

- Proposed drain spacing, in metres.
- Number of days to be calculated.
- Time-step for the calculation (lower than a given maximum), in days.

NSDEPTH shows the resulting groundwater depths on screen, with t is the time, d_p the groundwater level in the drainpipe, d_0 the groundwater level near the drain and d_1–d_{10} the depths between the drain and midway, where d_0 is drain and d_{10} is midway. Finally, d_{11} is equal to d_9 (symmetry).

If unsatisfactory, other drain spacing can be taken. A slow retreat in d_p values suggests an insufficient main system or unsatisfactory performance of the drainpipe. Large differences between d_p and d_0 indicate a considerable influence of the radial resistance W_r.

NSHEAD is similar, but it gives the heads above drain level instead of the depths.

Continuation

After ending with 999, the process can be repeated for a new case belonging to the same project. With another project or END, the files are closed and the results written to file ND****.txt or NH****.txt, where ND stands for "Nonsteady Depth", NH for "Nonsteady Head" and **** is the abbreviated project name. These filenames are mentioned in LISTND.TXT and LISTNH.txt.

Output and examples

Figure A23.6 and A23.7 show examples from NSDEPTH and NSHEAD for project aa, location aa1. The first shows the groundwater depths as function of time, the second the heads above drain level. Together they form the drain depth of 1.50 m. The initial depth of the water table was 0.2 m below surface, giving the initial head as 1.30 m.

FIGURE A23.6
Printout of program NSDEPTH

```
****** Non-steady flow, groundwater depths ******
 ==========================================================
project: aa; location: aa1; case: aa--01.txt

****** GENERAL INPUT DATA for NSDEPTH ****
 soil permeability                      2.000  m/d
 storage coefficient                     .150  ---
 drain depth below surface              1.500  m
 thickness soil below drain level       2.000  m
 initial groundw. depth below surface    .200  m
 radial resistance Wr                    .500  d/m
 outflow system, design capacity         .0100 m/d
 outflow system, design head             .500  m
 max. evaporation                        .0050 m/d
 groundwater depth where E=.43E0         .500  m
******************************************************

**** Results of NSDEPTH, non-steady depth ****

******* Depths below soil surface *******
t=time, dp=depth in drain, d0=outside drain
d10=midway, d0-d11=proportional distances from drain

 Drain spacing       L      20.00 m
 Radial resistance Wr        .50 d/m
```

t	dp	d0	d1	d2	d3	d4	d5	d6	d7	d8	d9	d10	d11
.00	.39	.21	.20	.20	.20	.20	.20	.20	.20	.20	.20	.20	.20
.15	.47	.29	.26	.25	.23	.22	.22	.22	.21	.21	.22	.22	.22
.30	.50	.33	.30	.28	.26	.25	.24	.23	.23	.23	.22	.22	.22
.45	.53	.36	.33	.31	.29	.28	.26	.25	.25	.24	.24	.24	.24
.60	.55	.38	.36	.33	.31	.30	.28	.27	.27	.26	.26	.26	.26
.75	.58	.41	.38	.36	.34	.32	.31	.29	.29	.28	.28	.28	.28
.90	.60	.43	.40	.38	.36	.34	.33	.32	.31	.30	.30	.30	.30
1.05	.62	.45	.42	.40	.38	.36	.35	.34	.33	.32	.32	.32	.32
1.20	.64	.47	.44	.42	.40	.38	.37	.36	.35	.34	.34	.34	.34
1.35	.66	.49	.47	.44	.42	.40	.39	.38	.37	.36	.36	.36	.36
1.50	.67	.51	.49	.46	.44	.42	.41	.40	.39	.38	.38	.38	.38
1.65	.69	.53	.51	.48	.46	.44	.43	.42	.41	.40	.40	.40	.40
1.80	.71	.55	.52	.50	.48	.46	.45	.44	.43	.42	.42	.42	.42
1.95	.73	.57	.54	.52	.50	.48	.47	.46	.45	.44	.44	.43	.44
2.10	.75	.59	.56	.54	.52	.50	.49	.48	.47	.46	.46	.45	.46
2.25	.77	.61	.58	.56	.54	.52	.51	.49	.48	.48	.47	.47	.47
2.40	.78	.63	.60	.58	.56	.54	.52	.51	.50	.50	.49	.49	.49
2.55	.80	.65	.62	.60	.58	.56	.54	.53	.52	.52	.51	.51	.51
2.70	.82	.66	.64	.61	.59	.58	.56	.55	.54	.53	.53	.53	.53
2.85	.83	.68	.66	.63	.61	.59	.58	.57	.56	.55	.55	.55	.55
3.00	.85	.70	.67	.65	.63	.61	.60	.59	.58	.57	.57	.56	.57

ARTES, for drainage under artesian conditions

After the three general questions (notation of decimal, project name, and location), the conditions are mentioned and the program moves on to specifics:

- How is the size of drains expressed (as diameter, as radius, as width of open ditches)?
- The size itself, in metres? Divide centimetres by 100 and always use a point for the decimal.
- The design discharge, in metres per day. Divide millimetres per day by 1 000.
- The required groundwater depth at this recharge, in metres.
- The required groundwater depth if there is no recharge (important for salinization in times that there is no irrigation and no rainfall). This depth must be greater than the former.
- The depth of drains (pipes or ditch bottoms), in metres below surface.

These general data appear on screen. If correct, ENTER 1, else 9 to restart the questions.

Then, data are required about soils and hydrology:

- The thickness of the top layer of low permeability, above and below drain level. Above, it is already given by the drain depth and mentioned as such. Below, it must be entered or estimated. However, where the thickness below is only a few decimetres, it is better to put the drains somewhat deeper, so that they tap the underlying aquifer. This avoids many problems with seepage.
- The anisotropy above and below drain level. Often this is unknown. If not visually layered, put 1 above and 4 below, else 16 below.
- The horizontal permeability above and below, in metres per day, as can be measured by auger hole or piezometer.
- The resistance between top layer and aquifer, in days. This is thickness divided by permeability of the layer between top layer and aquifer. A minimum is 25–50 days, a thin layer of tight clay has already 1 000–5 000 days. If unknown and no clay or compressed peat interferes, input 200 or try several values to see the effect.

- The hydraulic head in the aquifer in metres, above drain depth in cases that upward seepage occurs. For negative seepage (natural drainage), input negative values.

These data appear on screen. ENTER 1 if correct, 9 otherwise. If correct, the necessary calculations are made.

Continuation

The project can be continued and then the data for the new location are added to the same file. If a new project is taken or the existing one is ended, the files are closed and the filename is mentioned on screen and added to LISTAR.TXT. Any new project needs another name.

Output and example

The results are visible on screen and put on file AR****.TXT, where AR denotes "artesian" and **** the abbreviated project name. The smallest drain spacing is critical and should be taken. The filename is mentioned on screen and added to LISTAR.TXT.

As an example, Figure A23.8 describes a seepage area under irrigation in project 'a', location 'adana'. If irrigated, downward water movement causes removal of salts, but if no irrigation is given the situation is critical, because of upward movements. Therefore, the drain spacing should not exceed 17 m, the smallest spacing given.

FIGURE A23.7
Printout of program NSHEAD

```
****** Non-steady flow above drain or ditch bottom ******
 =======================================================
project: aa; location: aa1; case: aa--01.txt

****** GENERAL INPUT DATA for NSHEAD ****
******* all heads above drain level *******
 soil permeability                     2.000   m/d
 storage coefficient                    .150   ---
 thickness of soil below drain level   2.000   m
 initial groundwater head              1.300   m
 radial resistance  Wr                  .500   d/m
 outflow system, design capacity       .0100 m/d
 outflow system, design head            .500   m
 max. evaporation                       .0050 m/d
 groundwater depth where E=.43E0        .500   m
****************************************************

**** Results of NSHEAD, non-steady flow ****

******* Heads above drain level *******
t=time, hp=head in drain, h0=outside drain
h10=midway, h0-h11=proportional distances from drain

 Drain spacing    L     20.00 m
 Rad. resistance Wr      .50 d/m

  t    hp   h0   h1   h2   h3   h4   h5   h6   h7   h8   h9   h10  h11

 .00 1.11 1.29 1.30 1.30 1.30 1.30 1.30 1.30 1.30 1.30 1.30 1.30 1.30
 .15 1.03 1.21 1.24 1.25 1.27 1.28 1.28 1.28 1.29 1.29 1.29 1.29 1.29
 .30 1.00 1.17 1.20 1.22 1.24 1.25 1.26 1.27 1.27 1.27 1.28 1.28 1.28
 .45  .97 1.14 1.17 1.19 1.21 1.22 1.24 1.25 1.25 1.26 1.26 1.26 1.26
 .60  .95 1.12 1.14 1.17 1.19 1.20 1.22 1.23 1.23 1.24 1.24 1.24 1.24
 .75  .92 1.09 1.12 1.14 1.16 1.18 1.19 1.21 1.21 1.22 1.22 1.22 1.22
 .90  .90 1.07 1.10 1.12 1.14 1.16 1.17 1.18 1.19 1.20 1.20 1.20 1.20
1.05  .88 1.05 1.08 1.10 1.12 1.14 1.15 1.16 1.17 1.18 1.18 1.18 1.18
1.20  .86 1.03 1.06 1.08 1.10 1.12 1.13 1.14 1.15 1.16 1.16 1.16 1.16
1.35  .84 1.01 1.03 1.06 1.08 1.10 1.11 1.12 1.13 1.14 1.14 1.14 1.14
1.50  .83  .99 1.01 1.04 1.06 1.08 1.09 1.10 1.11 1.12 1.12 1.12 1.12
1.65  .81  .97  .99 1.02 1.04 1.06 1.07 1.08 1.09 1.10 1.10 1.10 1.10
1.80  .79  .95  .98 1.00 1.02 1.04 1.05 1.06 1.07 1.08 1.08 1.08 1.08
1.95  .77  .93  .96  .98 1.00 1.02 1.03 1.04 1.05 1.06 1.06 1.07 1.06
2.10  .75  .91  .94  .96  .98 1.00 1.01 1.02 1.03 1.04 1.04 1.05 1.04
2.25  .73  .89  .92  .94  .96  .98  .99 1.01 1.02 1.02 1.03 1.03 1.03
2.40  .72  .87  .90  .92  .94  .96  .98  .99 1.00 1.00 1.01 1.01 1.01
2.55  .70  .85  .88  .90  .92  .94  .96  .97  .98  .98  .99  .99  .99
2.70  .68  .84  .86  .89  .91  .92  .94  .95  .96  .97  .97  .97  .97
2.85  .67  .82  .84  .87  .89  .91  .92  .93  .94  .95  .95  .95  .95
3.00  .65  .80  .83  .85  .87  .89  .90  .91  .92  .93  .93  .94  .93
```

WELLS, for vertical drainage

Vertical drainage requires special conditions and is seldom a durable solution as it usually leads to overpumping and mobilization of salts from elsewhere. However, if required, a first estimate for well spacings can be obtained, based on steady-state equilibrium.

The program starts with the three general questions (notation of decimal, project name, and location) and then moves on to specifics:

- The minimum groundwater depth at the points furthest from the wells.
- The type of well, fully penetrating the aquifer or cavity well.
- The spacing of wells, in metres.
- Their diameter, in metres.
- The permeability of the aquifer and its thickness, in metres per day and in metres, respectively.

FIGURE A23.8
Printout of program ARTES

```
****** Drainage under artesian conditions ******
=======================================================
project: a; location: adana; case: a---01.txt
************* GENERAL INPUT DATA for ARTES ***********
 effective diameter of drain                .10   m
 design recharge R (by rain or irrig.)      .005 m/d
 design grw. depth midway at R             1.40   m
 design grw. depth midway at R=0           1.80   m
 design drain depth                        2.40   m
 design entrance resist. into drain         .00   d
*******************************************************

 ************* Data for case a---01.txt ******************
Properties of top layer
 thickness above drain level        2.40   m
 thickness below drain level        5.00   m
 anisotropy above drain level       1.00   --
 anisotropy below drain level       4.00   --
 hor.perm. above drain level         .20   m/d
 hor.perm. below drain level         .40   m/d
Hydrology
 resistance of aquitard           200.00  d
 hydraulic head in aquifer          2.00  m
 recharge (by rain or irrig.)  R=   .005 m/d

-------------- Results of case a---01.txt -----------------
 recharge (by rain or irrig.)   R =      .0050 m/d
 seepage (neg. if downward)              .0048 m/d
 spec. discharge above drain level       .0023 m/d
 spec. discharge below drain level       .0075 m/d
 head midway, at drain level             .98     m
 groundwater depth midway               1.39     m
 drain spacing . .    . .L-Brug. =     19.       m
- - - - - - - - - - - - - - - - - - - - - - - - - - - -
Values for recharge R=0
 recharge (by rain or irrig.)            .0000 m/d
 seepage (neg. if downward)              .0061 m/d
 spec. discharge above drain level       .0010 m/d
 spec. discharge below drain level       .0051 m/d
 head midway, at drain level             .60     m
 groundwater depth    midway            1.80     m
 drain spacing . . .    L-Brug. =      17.       m

* * * Take SMALLEST value for spacing L * * *
*******************************************************
```

- The recharge (by rain or irrigation losses), in metres per day. Divide millimetres per day by 1 000.
- The resistance of the overlying layer, either directly (in days) or from its permeability and thickness.
- The shape of the network (quadratic or triangular arrangement of wells).

The input is shown. If correct, the heads far from and near the well are given on screen. These heads are expressed with respect to the soil surface, because there is no drain level in this case.

Continuing gives a table with expected aquifer heads at various distances, again with respect to the soil surface.

Continuation

The project can be continued and then the data for the new location are added to the same file. If a new project is taken or the existing one is ended, the files are closed and the filename is mentioned on screen.

Output and example

The results are visible on screen and put on file WN****.TXT, where WN denotes well network and **** the abbreviated project name. The filename is mentioned on screen and added to LISTWN.TXT. Any new project needs a different name.

An example is given in Figure A23.9.

DRAIN DIAMETERS
DRSINGLE, for single drain

After the three general questions (notation of decimal, project name, and location), the program moves on to specifics:

- Type of drains: options are available for laterals and collectors. The latter are characterized by greater spacing, and often also greater length.
- Type of pipe: smooth (theoretical) (1); technically smooth (in practice) (2); or corrugated (two options, general (3) or according to Zuidema for small pipes, [maximum diameter 0.12 m]). Option “general” (3) will ask for the spacing of corrugations.
- Maintenance status, that is the amount of sediment to be expected in this soil under usual maintenance. In some soils, drains will keep clean, even without or with infrequent maintenance; in others, the pipes will clog with iron hydroxides,

sediments, or roots, even with regular (e.g. annual) cleaning. The first will have a good status, the second a poor one. The quantity must be estimated from earlier experience. Where unknown, try 3.

- Required items: length, diameter, maximum spacing allowed, head loss in drain, all in metres and maximum specific discharge (discharge divided by area served) in metres per day.
- According to this choice, all other quantities except the unknown will be required. The result is shown on screen and all data are written to file.
- ENTER to continue. The program calculates the results and asks for a new item or to end.
- Same project, other one, or end? The first option allows another measurement in the same project. The others finish the calculation.

Continuation

In the "same project" case, the existing project file is continued. Otherwise, it is closed, and the filename is mentioned on screen as DS****.TXT, where DS denotes "Drain, Single" and **** is the given project name. All these names are collected in the file LISTDS.TXT.

If "Other project" is selected, new names are required for project and location. With "END", the user returns to the initial screen.

Output and example

Figure A23.10 is an example for a collector of 1 000 m in length in an arid area.

DRMULTI, for multiple drain

The different materials of a multiple drain, consisting of sections with different diameters or materials (cement, smooth or corrugated plastic) must be specified, together

FIGURE A23.9
Printout of program WELLS

```
=====================================================
 Drainage by array of wells, steady state
 project: b; location: babel; case: b---01.txt

Fully penetrating well
Requirement on groundwater depth
  min. depth                    2.00    m
Well
  diameter                       .20    m
Aquifer
  permeability                 10.00    m/d
  thickness                    40.        m
  recharge                      .0030  m/d  [3.0 mm/d]
System
  aquifer transmissivity  400.    m2/d
  overlying resistance    200.    d
  characteristic length   283.    m
Network
  quadratic,  spacing     200.     m
  influence radius        113.     m
  discharge  per well     120.     m3/d (equilibrium)
  head aquifer, limit       -2.60   m
  head aquifer, well        -2.91   m

  radius m   head m    [ surface=0. ]
           groundwater   aquifer
     .10      -2.31       -2.91
     .11      -2.31       -2.91
     .28      -2.26       -2.86
    1.01      -2.20       -2.80
    2.99      -2.15       -2.75
    7.15      -2.11       -2.71
   14.71      -2.07       -2.67
   27.17      -2.05       -2.65
   46.28      -2.02       -2.62
   74.07      -2.01       -2.61
  112.84      -2.00       -2.60
 ---------------------------------------------------
```

FIGURE A23.10
Printout of program DRSINGLE

```
 ****** Dimensions of single drain ******
=====================================================
project: abba; location: Saltabad; case: abba01.txt
---------------------------------------------------
  Drain pipe design: Single diameter
 ---------------------------------------------------
 Collectors
  Technically smooth pipe, a-Blasius=0.40
   Maintenance status: good

 Input data
  Drain length          1000.00    m
  Collector spacing      300.00    m
  Design head loss          .30    m
  Design spec. disch.       .0030 m/d

 Results
  Min. inner diameter       .200   m
---------------------------------------------------
```

with the available diameters, total length and spacing. The program then calculates the length of the different sections.

After the three general questions (notation of decimal, project name, and location), the program moves on to specifics:

- Type of drains (laterals, collectors, or interceptor drains).
- For laterals and collectors, data are asked for allowed head loss in drain and specific discharge; for interceptors allowed head loss and inflow per m' length (obtained from INCEP or INCEP2).
- The number of different sections is required.
- Type of pipe used in each section: smooth (theoretical) (1), technically smooth (in practice) (2), or corrugated (two options, general (3) or according to Zuidema for small pipes [maximum diameter 0.12 m]). Option "general" (3) will ask for the spacing of corrugations.
- Maintenance status for the entire drain. This is the amount of sediment to be expected in this soil under usual maintenance. In some soils, drains will keep clean, even without or with infrequent maintenance; in others, the pipes will clog with iron hydroxides, sediments, or roots, even with regular (e.g. annual) cleaning. The former will have a good status, the latter a poor one. The quantity must be estimated from earlier experience. Where unknown, try 3.
- Diameter of each section.
- For laterals and collectors: spacing and length; for interceptors: their length only.

Results

The necessary calculations are made and the result appears on screen, first for two sections only. Then:

- ENTER to see a graph showing the head at design discharge and the slope of the drain.
- ENTER again to leave the graph.

If more than two sections are being considered, this procedure is repeated for all sections involved: lengths of all sections on screen, followed by a graph. Then:

- ENTER to continue.
- Same project, other one, or end? The first option allows another measurement in the same project. The others finish the calculation.

Continuation

In the "same project" case the existing project file is continued. Otherwise, it is closed, and the filename is mentioned on screen as DM****.TXT, where DM denotes "Drain, Multiple" and **** is the given project name. All these names are collected in the file LISTDM.TXT.

If "Other project" is selected, new names are required for project and location. With "END", the user returns to the initial screen.

Output and example

Figure A23.11 gives an example for laterals of 350 m in length in a humid climate.

MAIN DRAINAGE SYSTEM

BACKWAT, for backwater effects in the outlet channel of the main system

If an open channel of the main drainage system discharges via an open connection or sluice into a river, lake or sea, fluctuations in outside water level will influence the level in that channel. Especially high outside levels have an unfavourable and sometimes disastrous effect. Apart from a steady-state influence, also non-steady effects can be important in such cases. However, to form an idea of such effects, a steady-state approach is useful in cases where storage of water inland is not too important and the fluctuations are relatively slow.

For such situations, the program BACKWAT gives a solution. Thus, travelling waves cannot be calculated. Therefore, application is limited to downstream sections and sections above weirs that are of not too great length and that receive a constant flow from upstream.

Both high and low outside levels are covered, and data about positive of negative backwater curves are given.

FIGURE A23.11
Printout of program DRMULTI

```
****** Dimensions of multiple drain ******
 =====================================================
project: ba4; location: Balsa34; case: ba4-01.txt
==============================================================
  Drain pipe design
 -------------------------------------------------------------------
 Number of sections:  2
 Pipe type for lateral
   section 1: corrugated, Zuidema (a-Blasius= .77), diameter .05 m
   section 2: corrugated, Zuidema (a-Blasius= .77), diameter .08 m
   section 3: corrugated, Zuidema (a-Blasius= .77), diameter .12 m
   maintenance status: good
 Input data
   design head loss          .20   m
   discharge intensity       .010 m/d
   spacing of laterals     50.0    m
   length of laterals      350.0    m
 Output data
   length of section 1:       .00   head loss  .0000
   length of section 2:  163.64   head loss  .0935
   length of section 3:  186.36   head loss  .0966
   length of drain    :    350.00  real loss     .1901   allowed  .2000
 -------------------------------------------------------------------
 Number of sections:  3
 Pipe type for lateral
   section 1: corrugated, Zuidema (a-Blasius= .77), diameter .05 m
   section 2: corrugated, Zuidema (a-Blasius= .77), diameter .08 m
   section 3: corrugated, Zuidema (a-Blasius= .77), diameter .12 m
   maintenance status: good
 Input data
   design head loss           .20   m
   discharge intensity        .010 m/d
   spacing of laterals      50.0    m
   length of laterals      350.0    m
 Output data
   length of section 1:   45.69    head loss   .0261
   length of section 2:   86.95    head loss   .0497
   length of section 3:  217.36    head loss   .1026
   length of drain    :    350.00   real loss     .1784   allowed  .2000
==============================================================
```

Program

After the three general questions (notation of decimal, project name, and location), the program moves on to specifics:

- Dimensions of watercourse: bottom width in metres, side slopes. The results are shown on screen and can be corrected if necessary.
- Longitudinal profile: length of section, land and bottom elevation, first upstream and then downstream, in metres.
- Water elevation downstream, in metres. The results are shown on screen and can be corrected if necessary.
- Discharge from upstream, in cubic metres per second. Correction is possible. The program gives the equilibrium depth far upstream. As a check, the discharge is recalculated.
- The step size in water depth, in metres, to be used in the numerical calculations.

The program shows the results. ENTER returns to "step size" so that another value may be tried. Indicating END at this stage (type 9) leads to a question about the next item.

Next item and example

Same project, other one, or end? The first option allows another measurement in the same project. The others finish the calculation and ask for a new project filename for another abbreviated filename.

In the "same project" case, the existing project file is continued. Otherwise, it is closed, the filename mentioned on screen and added to LISTBW.TXT. If "Other project" is selected, new names are required for project and location. With "END", the user returns to the initial screen.

FIGURE A23.12
Printout of program BACKWAT

```
****** Backwater curves ******
=====================================================
project: aa ;location: adana; case: aa--01.txt
Backwater curves

 Watercourse
  bottom width        5.00 m
  side slopes    1:    2.00
     (1 vertical: 2.00 horizontal)

 Elevations
  length of section     2000. m

  land upstream         6.00 m
  land downstream       3.00 m

  bottom upstream       4.00 m
  bottom downstream      .00 m

  water downstream      2.00 m

  land slope            1.500 o/oo
  bottom slope          2.000 o/oo


 Discharge from upstream =   10.000 m3/s

 Equilibrium depth upstream    1.144 m
 Calc. discharge Q             9.998 m3/s

 distance   depth   water & land level   Q-calc
      0.    2.000   2.000    3.000   10.000
     28.    1.950   2.006    3.042   10.000
     56.    1.900   2.013    3.085   10.000
     85.    1.850   2.021    3.128   10.000
    115.    1.800   2.030    3.172   10.000
    145.    1.750   2.040    3.217   10.000
    176.    1.700   2.051    3.264   10.000
    208.    1.650   2.065    3.311   10.000
    241.    1.600   2.081    3.361   10.000
    275.    1.550   2.100    3.413   10.000
    312.    1.500   2.124    3.468   10.000
    351.    1.450   2.152    3.526   10.000
    394.    1.400   2.188    3.591   10.000
    442.    1.350   2.234    3.663   10.000
    499.    1.300   2.297    3.748   10.000
    571.    1.250   2.391    3.856   10.000
    679.    1.200   2.558    4.019   10.000
   1073.    1.150   3.296    4.610   10.000
```

An example is given by Figure A23.12.

INTERCEPTOR DRAINS INCEP and INCEP2, for homogeneous profiles and for a less permeable top layer

Interception drains are needed in places where waterlogging occurs in undulating terrain, especially to protect the downstream fields. This waterlogging is usually caused by a decrease in slope, a change in the soil profile or an abrupt lowering of the surface. In other cases, it is caused by leakage from irrigation canals and watercourses, or from higher lands. The program allows changes of this kind for a profile of permeable soil on an impermeable base. It calculates the width of a drain trench or ditch bottom that is sufficient to catch the intercepted flow. A separate calculation is needed for the size of the drain needed, this can be found by the program DRMULTI.

INCEP, homogeneous profile

After the three general questions (notation of decimal, project name, and location), the programs moves on to specifics regarding the upstream conditions:

- The source: hillslope, canal or higher fields.

In the case of hillslopes:

- The upstream slope, as the ratio 1: *n* (vertical: horizontal) of which *n* is required.
- The upstream permeability, in metres per day.
- The upstream depth of the impermeable base, in metres below surface.
- The upstream depth of the groundwater, in metres below surface.
- Depth of drain, below the upstream soil surface, in metres.

In the case of a leaky canal at higher level:

- The water losses from the canal, flowing to both sides in the present situation in square metres per day.
- The water level in the canal above the nearby soil surface.
- The original groundwater level below surface.
- The required future groundwater level below surface.

In the case of flow from higher ground:

- The flow from higher lands.
- The required future groundwater level below surface.

These data appear on screen. If correct, ENTER 1, else 2 to restart the questions. If correct, the downstream conditions must be specified:

- Flat or sloping surface?
- If there is a further downward slope 1:*n*, the downstream *n* is required, which must be more than upstream.
- The downstream permeability, in metres per day.
- The downstream depth of the impermeable base, in metres below surface.
- Depth of drain, below the downstream soil surface, in metres. For hill slopes, the difference with the upstream value determines the difference in surface elevation near the drain.
- The required downstream depth of the groundwater, in metres below surface.

These data appear on screen. If correct, ENTER 1, else 2 to restart the questions. If correct, the necessary calculations are performed and the results shown on screen, the main one being the width of the drain trench or ditch bottom needed to catch the intercepted flow. In most cases, a normal trench width is sufficient, the main exception being permeable soils of considerable depth.

Calculating the lowering of the groundwater upstream of the drain is an option for hill slopes.

INCEP2, less permeable topsoil

The program treats a two-layered soil with an upper layer at least ten times less permeable that the second one. Only a change in slope is considered.

After the three general questions (notation of decimal, project name, and location), the program moves on to specifics. These are similar to those for INCEP, plus:

FIGURE A23.13
Printout of program INCEP

```
***** interceptor drain, homogeneous soil *****
==============================================
project: a; location: a1; case: a---01.txt

Upstream values
 tangent of slope                 .05 m/m  1:20.0
 diff. surface level at x=0        .00 m
 permeability                     3.00 m/d
 depth to impermeable layer       8.00 m
 depth of drain, upstream end     2.00 m
 drain above impermeable base     6.00 m
 radial resistance near drain      .48 d/m
 incoming flow                    1.05 m2/d
 thickness of incoming flow       7.00 m
 depth groundwater upstream       1.00 m

Downstream values
 zero slope, flat terrain
 diff. surface level at x=0        .00 m
 permeability                     3.00 m/d
 depth to impermeable layer       8.00 m
 depth of drain, downstream       2.00 m
 drain above impermeable base     6.00 m
 radial resistance near drain      .48 d/m
 head from radial resistance       .50 m
 incoming flow                    1.05 m2/d
 intercepted flow                 1.05 m2/d
 downstream flow                   .00 m2/d
 thickness of outgoing flow       6.50  m
 depth groundwater downstream     1.50  m

Required width of trench needed for groundwater control
 width 0.10 m sufficient
 WARNING: May not be sufficient for drain discharge!

 Use DRMULTI for drain sizes.
 Inflow into drain is 1.050 m2/d
 ============================================
 Upstream lowering by drain
     100%= .50 m
  lowering %   lowering m    distance x, m
-----------------------------------------------------
     100.           .50             .0
      90.           .45           13.8
      80.           .40           29.2
      70.           .35           46.9
      60.           .30           67.5
      50.           .25           92.0
      40.           .20          122.3
      30.           .15          161.6
      20.           .10          217.3
      10.           .05          313.4
============================================
```

FIGURE A23.14
Printout of program INCEP2

```
***** interceptor drain, two-layered soil *****
 ================================================
project: b; location: b1; case: b---01.txt
  tangent of slope upstream          .05 m/m  1: 20.0
  downstream slope zero,             flat terrain

  no difference in surface level at x=0
  permeability top layer             .30 m/d
  permeability second layer         3.00 m/d
  thickness top layer               4.00 m
  thickness second layer            4.00 m
  depth to impermeable layer        8.00 m

  depth of trench or ditch          2.00 m
  drain above soil transition       2.00 m
  radial resistance near drain       .78 d/m
  resulting head above drain         .50  m

  incoming groundwater flow          .65  m2/d
  outgoing groundwater flow          .00  m2/d
  intercepted by drain               .65  m2/d
  depth groundwater upstream        1.00  m
  depth groundwater downstream      1.50  m
  thickness of incoming flow        7.00  m
  thickness of outgoing flow        6.50  m

  Result: required bottom width     6.83 m
   corrected linear approximation   6.84 m

Use DRMULTI for drain sizes.
  Inflow into drain is .645 m2/d
============================================
```

- Permeability of top layer, metres per day.
- Permeability of second layer, metres per day.
- Thickness of top layer, metres.
- Thickness of second layer, metres.

All entry data appear on screen. If correct, ENTER 1, else 2 to restart the questions. If correct, the necessary calculations are performed and the results shown on screen, the main one being the width of the trench or ditch bottom needed to catch the intercepted flow. In contrast to the homogeneous case, where a small width is usually sufficient, a drain in less permeable topsoil requires a much wider trench. As this is often not feasible, several drains are needed. Their mutual distance can be estimated for the program ARTES for artesian conditions, their number from the total flow to be eliminated.

Continuation, output and examples

The process can be repeated in a new case belonging to the same project. With another project or END, the files are closed and the results written to file ID****.txt, where ID stands for "Interceptor Drain" and **** is the abbreviated project name. These filenames are mentioned in LISTID.TXT.

Figure A23.13 gives the output of INCEP for a hillslope in project 'a', at location 'a1'. It can be seen that the effect of the radial resistance is negligible in this case, as is usual for homogeneous permeable soils of rather shallow depth.

Figure A23.14 gives results for a case similar to Figure A23.13, but now with the upper 4 m of low permeability and for a leaky canal. The increase in necessary bottom width is dramatic. Although the flow is similar, the required width changes from less than 0.10 m to more than 6 m. As this is impractical, several drains will be needed.

The hydrological conditions are usually more complicated at such locations and often poorly known. Therefore, the programs can give rough guidelines only, and solutions must often be found in the field by trial and error, adding more drains if needed until the result is satisfactory.

The inflow per m' drain can be used as input in the program DRMULTI to find the necessary dimensions of the drain itself.

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FAO IRRIGATION AND DRAINAGE PAPER

Guidelines and computer programs for the planning and design of land drainage systems

The aim of this paper is to facilitate the planning and design of land drainage systems for sound land and water management for engineers and other professionals. The text of this publication provides guidelines for the appropriate identification of drainage problems, for the planning and design of field drainage systems (surface and subsurface) and the main drainage and disposal systems. The annexes provide more detailed information with technical background, appropriate equations, some cross-references for finding appropriate methodologies, and computer programs for calculation of extreme values, of permeability and some land drainage system parameters. The paper considers the integration of technical, socio-economic and environmental factors and the need for system users' participation in the planning, design, operation and maintenance processes.

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