4 Soil water - surface water interaction

J.C. van Dam, P. Groenendijk, J.G. Kroes, P.E.V. van Walsum

The interaction between soil and surface water system may be described by:
- Surface flow (runoff, runon and inundation); which is an overland water flow;
- Subsurface flow, or drainage and infiltration; which is a shallow or deep water flow through the soil system.

Different options for this interaction are described in this paragraph.

4.1 Surface flow

Surface flow is regarded as the overland water flow that results in interaction between soil and surface water system. Several water fluxes play a role in this interaction where the so-called ponding reservoir playes a crucial role (Figure 11). This ponding reservoir may be regarded as a thin layer of water on top of the soil surface, which can store water to a certain maximum.

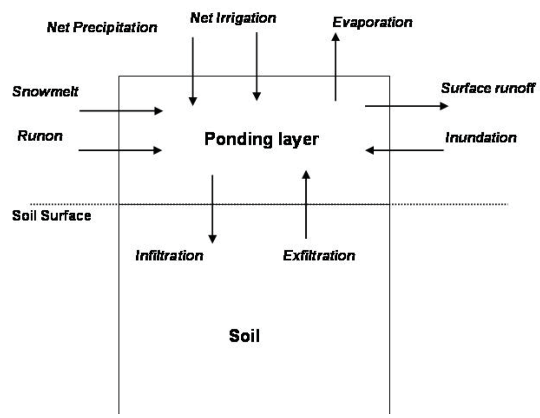


Figure 11 The water fluxes on the soil surface

The water balance of this ponding reservoir is:

$$\Delta pond = P_{net} + I_{net} + q_1 + M + q_{runon} - q_{runoff} - E_{pond} \tag{4.1}$$

where: $\Delta pond$ is the storage change of the ponding reservoir (cm d^{-1}), P_{net} is the net precipitation flux (cm d^{-1}), I_{net} is the net irrigation flux (cm d^{-1}), q_1 is the flux between the ponding layer and the 1st model compartment (cm d^{-1}, exfiltration is upward and has a positive value, infiltration is downward and has a negative value), M is snowmelt (cm d^{-1}), q_{runon} is an external runon flux, e.g. from a neighbouring field (cm d^{-1}), q_{runoff} is discharge to/from the surface water system (cm d^{-1}, as runoff with a positive value, as inundation with a negative value).

4.1.1 Surface runoff and inundation

Surface runoff is simulated when the groundwater level rises above the soil surface or when the infiltration capacity of the soil is not sufficient to infiltrate all the water. In either case the groundwater level will fill the ponding reservoir until a certain threshold ponding level (h_{pond}) is exceeded. When this exceedance occurs, surface runoff as:

$$q_{runoff} = \frac{1}{\gamma_{sill}} \left(h_{pond} - z_{sill} \right)^{\beta_{sill}} \tag{4.2}$$

where h_{pond} is the ponding depth of water (cm) on the soil surface, z_{sill} the height (cm) of the sill which is equal to the maximum ponding height ($h_{pond,max}$) or to the surface water level, γ_{sill} the runoff/inundation resistance (d) and β_{sill} an exponent (-).

Surface runoff occurs when $h_{pond} > z_{sill}$; inundation occurs when $h_{pond} < z_{sill}$.
The maximum ponding height without surface runoff is determined by the irregularities of the soil surface. As surface runoff is a rapid process, the sill resistance γ_{sill} will typically have values of less than 1 d. For most SWAP applications, realistic dynamic simulation of surface runoff is not required, but only the effect of surface runoff on the soil water balance is relevant. Then a rough estimate of γ_{sill} is sufficient, e.g. $\gamma_{sill} \approx 0.1$ d. When the dynamics of surface runoff are relevant, the values of γ_{sill} and β_{sill} might be derived from experimental data or from a hydraulic model of soil surface flow.

Model input

Variable	*Code*	*Description*
$h_{pond,max}$	PONDMX	Ponding height (cm)
γ_{sill}	RSRO	Runoff/inundation resistance (d)
β_{sill}	RSROEXP	Exponent in runoff/inundation relation (-)

4.1.2 Surface runon

Surface runon is supplied to the model as an external source. It originates from an external source (runoff from a neighbouring field) which supplies excess water.

Model input

Variable	*Code*	*Description*
q_{runon}	RUFIL	File with external runon flux, e.g. from a neighbouring field (cm d^{-1})

4.2 Drainage and infiltration

Lateral field drainage fluxes, q_{drain} (cm d^{-1}) to the drainage system may be defined in different forms. Four methods can be used to calculate q_{drain}:

- Linear or tabulas $q_{drain}(\phi_{gwl})$ relation (Par. 4.2.1)

- drainage equations of Hooghoudt and Ernst (Par. 4.2.2)
- drainage/infiltration to/from surface water systems (basic drainage, Par. 4.2.3)
- interaction with a simplified surface water system (extended drainage, Par. 4.2.5)

4.2.1 Linear or tabular relation

A linear or tabular relation between groundwater level and drainage flux q_{drain} (cm d^{-1}) may be applied:

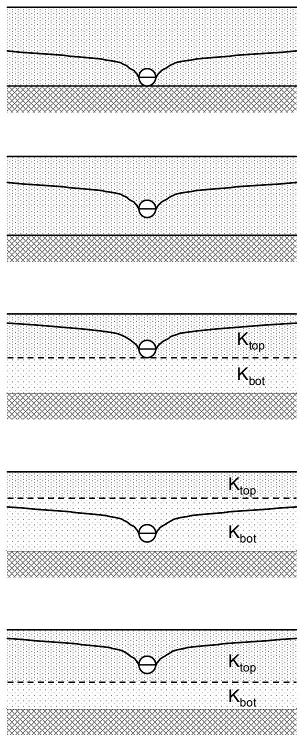
$$q_{drain} = \frac{\phi_{gwl} - \phi_{drain}}{\gamma_{drain}} \tag{4.3}$$

where ϕ_{gwl} is the phreatic groundwater level midway between the drains or ditches (cm), ϕ_{drain} the drain hydraulic head (cm) γ_{drain} the drainage resistance (d). In case of non-linear relations between q_{drain} and ϕ_{gwl}, tabular values of q_{drain} as function of ϕ_{gwl} are input.

Model input		
Variable	*Code*	*Description*
ϕ_{gwl}	GWL	Groundwater level (cm, negative below soil surface)
q_{drain}	Qdrain	Drainage flux (cm d^{-1}) as a function of groundwater level

4.2.2 Drainage equations of Hooghoudt and Ernst

The drainage equations of Hooghoudt and Ernst allow the evaluation of drainage design. The theory behind these equations is clearly described in Ritzema (1994). Five typical drainage situations are distinguished (Figure 12). For each of which the drainage resistance γ_{drain} can be defined.



Schematization	Soil profile	Drain position	Theory
	homogeneous	on top of impervious layer	Hooghoudt Donnan
	homogeneous	above impervious layer	Hooghoudt with equi-valent depth
	two layers	at interface of the two soil layers	Hooghoudt
	two layers ($K_{top} < K_{bot}$)	in bottom layer	Ernst
	two layers ($K_{top} < K_{bot}$)	in top layer	Ernst

Figure 12 Five field drainage situations considered in SWAP (aftere Ritzema, 1994)

Homogeneous profile, drain on top of impervious layer

The drainage resistance is calculated as:

$$\gamma_{\text{drain}} = \frac{L_{\text{drain}}^2}{4K_{\text{hprof}}\left(\phi_{\text{gwl}} - \phi_{\text{drain}}\right)} + \gamma_{\text{entr}} \tag{4.4}$$

with K_{hprof} the horizontal saturated hydraulic conductivity above the drainage basis (cm d^{-1}), L_{drain} the drain spacing (cm) and γ_{entr} the entrance resistance into the drains and/or ditches (d). The value for γ_{entr} can be obtained, analogous to the resistance value of an aquitard, by dividing the 'thickness' of the channel walls with the permeability. If this permeability does not differ substantially from the conductivity in the surrounding subsoil, the numerical value of the entry resistance will become relatively minor.

Homogeneous profile, drain above impervious layer

This drainage situation has been originally described by Hooghoudt (1940). The drainage resistance follows from:

$$\gamma_{\text{drain}} = \frac{L_{\text{drain}}^2}{8K_{\text{hprof}}D_{\text{eq}} + 4K_{\text{hprof}}\left(\phi_{\text{gwl}} - \phi_{\text{drain}}\right)} + \gamma_{\text{entr}} \tag{4.5}$$

where D_{eq} is the equivalent depth (cm).

The equivalent depth was introduced by Hooghoudt to incorporate the extra head loss near the drains caused by converging flow lines. We employ in SWAP a numerical solution of Van der Molen and Wesseling (1991) to calculate D_{eq} (Ritzema, 1994). A typical length variable x is used:

$$x = \frac{2\pi\left(\phi_{drain} - z_{imp}\right)}{L_{drain}} \tag{4.6}$$

If $x < 10^{-6}$, then:

$$D_{eq} = \phi_{drain} - z_{imp} \tag{4.7}$$

with z_{imp} the level of the impervious layer. If $10^{-6} < x < 0.5$, then:

$$F(x) = \frac{\pi^2}{4x} + \ln\left(\frac{x}{2\pi}\right) \tag{4.8}$$

and the equivalent depth equals:

$$D_{eq} = \frac{\pi\, L_{drain}}{8\left(\ln\left(\frac{L_{drain}}{\pi\, r_{drain}}\right) + F(x)\right)} \tag{4.9}$$

with r_{drain} the radius of the drain or ditch. If $0.5 < x$, then:

$$F(x) = \sum_{j=1,3,5,..}^{\infty} \frac{4\,e^{-2jx}}{j\left(1 - e^{-2jx}\right)} \tag{4.10}$$

and equivalent depth again follows from Eq. (4.9).

Heterogeneous soil profile, drain at interface between both soil layers

The equivalent depth D_{eq} is calculated with the procedure of Eq. (4.6) to (4.10). The drainage resistance follows from:

$$\gamma_{drain} = \frac{L_{drain}^2}{8\,K_{hbot} D_{eq} + 4 K_{htop}\left(\phi_{gwl} - \phi_{drain}\right)} + \gamma_{entr} \tag{4.11)}$$

with K_{htop} and K_{hbot} the horizontal saturated hydraulic conductivity (cm d^{-1}) of upper and lower soil layer, respectively.

Heterogeneous soil profile, drain in bottom layer

The drainage resistance is calculated according to Ernst (1956) as:

$$\gamma_{drain} = \gamma_{ver} + \gamma_{hor} + \gamma_{rad} + \gamma_{entr} \tag{4.12}$$

where γ_{ver}, γ_{hor}, and γ_{rad} are the vertical, horizontal and radial resistance (d^{-1}), respectively. The vertical resistance is calculated by:

$$\gamma_{ver} = \frac{\phi_{gwl} - z_{int}}{K_{vtop}} + \frac{z_{int} - \phi_{drain}}{K_{vbot}} \tag{4.13}$$

with z_{int} the level of the transition (cm) between the upper and lower soil layer, and K_{vtop} and K_{vbot} the vertical saturated hydraulic conductivity (cm d^{-1}) of the upper and lower soil layer, respectively. The horizontal resistance is calculated as:

$$\gamma_{hor} = \frac{L_{drain}^2}{8\, K_{hbot} D_{bot}} \tag{4.14}$$

with D_{bot} the contributing layer below the drain level (cm), which is calculated as the minimum of (ϕ_{drain} - z_{imp}) and ¼ L_{drain}. The radial resistance is calculated by:

$$\gamma_{rad} = \frac{L_{drain}}{\pi \sqrt{K_{hbot} K_{vbot}}} \ln\left(\frac{D_{bot}}{u_{drain}}\right) \tag{4.15}$$

with u_{drain} the wet perimeter (cm) of the drain.

Heterogeneous soil profile, drain in top layer

Again the approach of Ernst (1956) is applied (Eq. (4.12)). The resistances are calculated as:

$$\gamma_{ver} = \frac{\phi_{gwl} - \phi_{drain}}{K_{vtop}} \tag{4.16}$$

$$\gamma_{hor} = \frac{L_{drain}^2}{8\, K_{htop} D_{top} + 8\, K_{hbot} D_{bot}} \tag{4.17}$$

$$\gamma_{rad} = \frac{L_{drain}}{\pi \sqrt{K_{htop} K_{vtop}}} \ln\left(g_{drain} \frac{\phi_{drain} - z_{int}}{u_{drain}}\right) \tag{4.18}$$

with D_{top} equal to *(ϕ_{drain} - z_{int})* and g_{drain} is the drain geometry factor, which should be specified in the input. The value of g_{drain} depends on the ratio of the hydraulic conductivity of the bottom (K_{hbot}) and the top (K_{htot}) layer. Using the relaxation method, Ernst (1962) distinguished the following situations:

- $K_{hbot}/K_{htot} < 0.1$: the bottom layer can be considered impervious and the case is reduced to a homogeneous soil profile and $g_{drain} = 1$;
- $0.1 < K_{hbot}/K_{htot} < 50$: g_{drain} depends on the ratios K_{hbot}/K_{htop} and D_{bot}/D_{top}, as given in Table 1.
- $50 < K_{hbot}/K_{htot}$: $g_{drain} = 4$.

Table 1 The geometry factor g_{drain} (-), as obtained by the relaxation method (after Ernst, 1962).

K_{hbot}/K_{htop}	D_{bot}/D_{top}					
	1	2	4	8	16	32
1	2.0	3.0	5.0	9.0	15.0	30.0
2	2.4	3.2	4.6	6.2	8.0	10.0
3	2.6	3.3	4.5	5.5	6.8	8.0
5	2.8	3.5	4.4	4.8	5.6	6.2
10	3.2	3.6	4.2	4.5	4.8	5.0
20	3.6	3.7	4.0	4.2	4.4	4.6
50	3.8	4.0	4.0	4.0	4.2	4.6

Model input

Variable	*Code*	*Description*
L_{drain}	LM2	Drain spacing (m)
u_{drain}	WETPER	Wet perimeter of the drain (cm)
ϕ_{drain}	ZBOTDR	Level of drain bottom (cm)
γ_{entr}	ENTRES	Drain entry resistance (d)
z_{imp}	BASEGW	Level of impervious layer (cm)
K_{htop}	KHTOP	Horizontal hydraulic conductivity top layer (cm d^{-1})
For a non-homogeneneous soil profile:		
K_{hbot}	KHBOT	Horizontal hydraulic conducyivity bottom layer (cm d^{-1})
z_{int}	ZINTF	Level of interface of fine and coarse soil layer (cm)
K_{vtop}	KVTOP	Vertical hydraulic conductivity top layer (cm d^{-1})
K_{vbot}	KVBOT	Vertical hydraulic conductivity bottom layer (cm d^{-1})
g_{drain}	GEOFAC	Geometry factor of Ernst (-)

4.2.3 Basic drainage

A simple, basic interaction between groundwater and a maximum of 5 surface water systems may be simulated.

The drainage/infiltration (q_{drain}) to/from each surface water system *i* is calculated as:

$$q_{drain,i} = \frac{\phi_{gwl} - \phi_{drain,i}}{\gamma_{drain,i}} \qquad (4.19)$$

where $q_{drain,i}$ is the drainage/infiltration (cm d^{-1}) to/from surface water system *i*, the drainage base $\phi_{drain,i}$ is equal to the surface water level of system *i* (cm below the soil surface), ϕ_{gwl} is the groundwater level (cm below the soil surface), $\gamma_{drain,i}$ is the drainage or infiltration resistance from system i (d).

Model input		
Variable	*Code*	*Description*
	NRLEVS	Number of drainage levels (-)
Specify for each drainage level:		
γ_{drain}	DRARES	Drainage resistance (d)
γ_{inf}	INFRES	Infiltration resistance (d)
L_{drain}	L	Drain spacing (m)
ϕ_{drain}	ZBOTDR	Level of drainage medium bottom (cm)

4.2.4 Interflow

In some applications one may wish to define one of the systems as an interflow system, which has a rapid discharge with short residence times of the water in the soil system.
Interflow should always be assigned to the highest order or level of distinguished drainage systems. This may be applied for either basic or extended drainage options. (paragraphs 4.2.3 and 4.2.5).

The interflow towards surface water systems *n* is calculated as:

$$q_{drain,n} = A_{\text{int}erflow}\,(\phi_{gwl} - \phi_{drain,n})^{B_{\text{int}erflow}} \tag{4.20}$$

where: $q_{drain,n}$ is the interflow towards surface water system n, $A_{\text{int}erflow}$ and $B_{\text{int}erflow}$ are respectively coefficient (d^{-1}) and exponent (-) in the relation.

Model input		
Variable	*Code*	*Description*
$A_{\text{interflow}}$	COFINTFLB	Coefficient for interflow relations (d)
$B_{\text{interflow}}$	EXPINTFLB	Exponent for interflow relation (-)

4.2.5 Extendend drainage

This paragraph describes an extended drainage option, which may be applied when the interaction between groundwater and surface water system can limited to a single representative groundwater level and a single representative surface water level. The interaction between these two levels is described with extensive options and documented hereafter.

The groundwater-surface water system is described at the scale of a horizontal subregion. Only a single representative groundwater level is simulated, which is 'stretched' over a scale that in reality involves a variety of groundwater levels. In the following, due consideration will be given to the schematization of the surface water system, the simulation of drainage/sub-irrigation fluxes (including surface runoff), and the handling of an open surface water level.
The surface water system is divided into a maximum of five channel orders:
- primary water course (1^{st} order);
- secondary water course(s) (2^{nd} order);
- tertiary water courses (3^{rd} order);
- pipe drains (4^{th} order);

- trenches (5th order).

An example of a surface water system with three channel orders is shown in Figure 13.

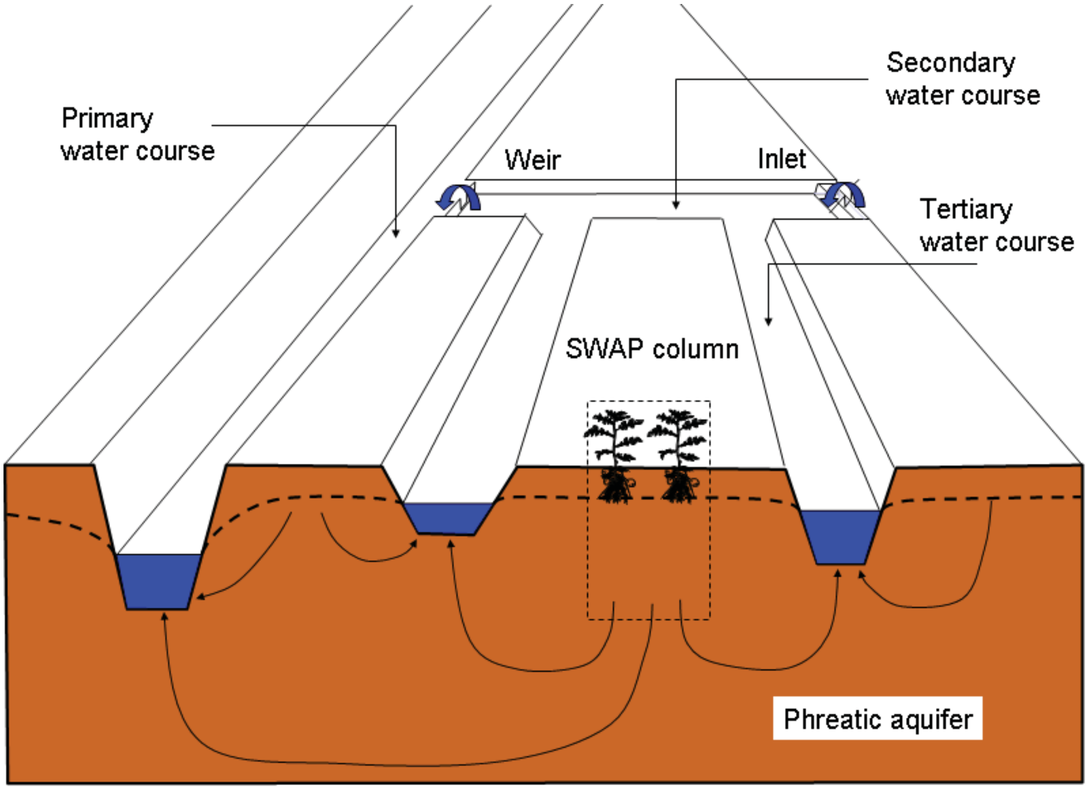


Figure 13 Schematized surface water system. The primary water course functions separately from the others, but it does interact with the SWAP soil column by the drainage or infiltration flux

Each order of channels is defined by its channel bed level, bed width, side-slope, and spacing. For practical cases, the representative spacing L_i (m) is derived by dividing the area of the subregion A_{reg} (m^2) by the total length of the i^{th} order channels, l_i (m):

$$L_i = \frac{A_{reg}}{l_i} \tag{4.21}$$

In the surface water model, we assume that the different channels orders are connected in a dendritic manner. Together they form a surface water 'control unit' with a single outlet and, if present, a single inlet. The surface water level at the outlet is assumed to be omnipresent in the subregion. Friction losses are neglected and thus the slope of the surface water level is assumed to be zero. This means that in all parts of the subregion the surface water level has the same depth below soil surface. Its presence, however, is only locally felt in a water course if it is higher than the channel bed level. If it is lower, the water course is free draining, or remains dry if the groundwater level is below the channel bed.

In most applications, the control unit will include the primary watercourse. It is, however, possible to specify that the primary watercourse, e.g. a large river, functions separately from the rest of the subregional surface water system. In that case it has its own surface water level. This level has to be specified in the input, because it is determined by water balances and flows on a much larger scale than that of the modelled subregion. In the real situation there may be some interaction between the primary water course and the control unit: for instance a pumping station for removal of drainage water, and/or an inlet for letting in external surface water supply (Figure 13). The hydraulics of such structures are not included in the model.

The channels do not only act as waterways for surface water transport. Depending on the groundwater level and the open surface water level, the channels will also act as either drainage or sub-irrigation media. In the system modelled by SWAP, it is possible that more than one type of surface water channel becomes active simultaneously. For these situations one can best speak of 'multi-level' drainage or sub-irrigation. In the following, we will refer to channels in terms of their 'order' if their role as part of the surface water system is being considered. When considering their drainage characteristics we will refer to them in terms of their 'level'.

When the groundwater level rises above the soil surface, the soil surface also starts to function as a 'drainage medium' generating surface runoff. The storage of water on the soil surface itself, however, is simulated by SWAP as 'ponding' (Par. 4.1).

Model input

Variable	*Code*	*Description*
n	NRSRF	Number of subsurface drainage levels (-)
Specify for each level:		
i	LEVEL	Drainage level number (-)
-	SWDTYP	Type of drainage medium (open = 0, closed = 1)
L	L	Spacing between channels/drains (m)
z_{bed}	ZBOTDRE	Altitude of bottom of channel or drain (cm)
ϕ_{avg}^{min}	GWLINF	Groundwater level for maximum infiltration (cm)
$\gamma_{drain,inp}$	RDRAIN	Drainage resistance (d)
$\gamma_{inf,inp}$	RINFI	Infiltration resistance (d)
γ_{entry}	RENRTY	Entry resistance (d)
γ_{exit}	REXIT	Exit resistance (d)
-	WIDTHR	Bottom width of channel (cm)
-	TALUDR	Side-slope of channel (-)

4.2.5.1 Surface water balance

For the water balance of the subregion as a whole, we assume that the soil profile 'occupies' the whole surface area, even though part of the area is covered by surface water. In other words, the water balance terms of the soil profile that are computed per unit area ($cm^3\ cm^{-2}$) have the *same numerical value for the subregion as a whole.* This implies that the evapotranspiration of surface water is set equal to the actual evapotranspiration of land surface. For reasons of simplicity evapotranspiration and precipitation are not included in the water balance of surface water. We do, however, compute storage characteristics of the surface water based on the lengths of the water courses and the wetted cross sections. There

is thus a 'duplicate use' of part of the area, introducing some extra storage in the system, which in reality does not exist. The approach followed here is only valid for subregions with a limited area of surface water, certainly not more than 10%.
The surface water balance equation for the control unit is formulated as:

$$V_{\mathrm{sur}}^{j+1} - V_{\mathrm{sur}}^{j} = \left(q_{\mathrm{sup}} - q_{\mathrm{dis}} + q_{\mathrm{drain}} + q_{\mathrm{c,drain}} + q_{\mathrm{run}} \right) \Delta t^{j} \tag{4.22}$$

where V_{sur} is the regional surface water storage (cm^3 cm^{-2}), q_{sup} is the external supply to the control unit (cm^3 cm^{-2} d^{-1}), q_{dis} is the discharge that leaves the control unit (cm^3 cm^{-2} d^{-1}), $q_{\mathrm{c,drain}}$ is bypass flow (cm^3 cm^{-2} d^{-1}) through cracks of a dry clay soil to drains or ditches, q_{run} is the surface runoff/runon (cm^3 cm^{-2} d^{-1}), Δt is the time increment (d), and superscript j is the time level.

The regional surface water storage V_{sur} (cm^3 cm^{-2}) is the sum of the surface water storage in each order of the surface water system:

$$V_{\mathrm{sur}} = \frac{1}{A_{\mathrm{reg}}} \sum_{i=1}^{\mathrm{n}} l_i A_{\mathrm{d},i} \tag{4.23}$$

in which A_{reg} is the total area of the subregion (cm^2), l_i the total length of channels/drains of order i in the subregion (cm), and $A_{\mathrm{d},i}$ is the wetted area of a channel *vertical* cross-section (cm^2). The program calculates $A_{\mathrm{d},i}$ using the surface water level ϕ_{sur}, the channel bed level, the bottom width, and the side-slope. Substitution of Eq. (4.21) in Eq. (4.23) yields the expression:

$$V_{\mathrm{sur}} = \sum_{i=1}^{\mathrm{n}} \frac{A_{\mathrm{d},i}}{L_i} \tag{4.24}$$

Channels of order i only contribute to the storage if $\phi_{\mathrm{sur}} > z_{\mathrm{bed,i}}$. The storage in pipe drains is assumed to be zero. Eq. (4.24) is used by the model for computing the storage from the surface water level and vice versa, per time step. Prior to making any dynamic simulations, a table of channel storage as a function of discrete surface water levels is derived.

4.2.5.2 Drainage resistance (subregional approach)

Prior to any calculation of the drainage/sub-irrigation rate, we determine whether the flow situation involves drainage, sub-irrigation, or neither. No drainage or sub-irrigation will occur if both the groundwater level and surface water level are below the drainage base.
Drainage will only occur if the following two conditions are met:
- the groundwater level is higher than the channel bed level;
- the groundwater level is higher than the surface water level.

Sub-irrigation can only occur if the following two conditions are met:
- the surface water level is higher than the channel bed level;
- the surface water level is higher than the groundwater level.

In both cases we take for the drainage base, ϕ_{drain} (cm), either the surface water level, ϕ_{sur} (cm), or the channel bed level, z_{bed} (cm), whichever is higher:

$$\phi_{\mathrm{drain}} = \max\left(\phi_{\mathrm{sur}}, z_{\mathrm{bed}} \right) \tag{4.25}$$

The variable ϕ is defined positive upward, with zero at the soil surface.

An example of a single-level drainage case is given in Figure 13. In this example we assume that:

- the considered channel is part of a system involving equidistant and parallel channels, all of the same order;
- the recharge R is evenly distributed and steady-state.

For such situations several drainage formula exist, as described in Par.4.2.2.
The drainage resistance for the subregional approach is defined as:

$$\gamma_{\text{drain}} = \frac{\phi_{\text{avg}} - \phi_{\text{drain}}}{R} \qquad (4.26)$$

where ϕ_{avg} is the mean groundwater level of the whole subregion, and ϕ_{drain} the hydraulic head of the drain or ditch (cm), the so-called drainage base.
Note that instead of the maximum groundwater level ϕ_{gwl} midway between the drains or ditches (eq. (4.3)), the mean groundwater level ϕ_{avg} is used. The two definitions of γ_{drain} in eq. (4.3) and (4.26) differ by the so-called shape factor: the shape factor is the ratio between the mean and the maximum groundwater level elevation above the drainage base. The shape factor depends on the vertical, horizontal, radial and entrance resistances of the drainage system (Ernst, 1978). For regional situations, where the 'horizontal' resistance to flow plays an important role, the shape factor is relatively small ($\approx$ 0.7). The smaller the horizontal resistance becomes, the more 'rectangular' the water table: in the most extreme case with all the resistance concentrated in the direct vicinity of the channel, the water table is level, except for the abrupt drop towards the drainage base. In that case the shape factor becomes equal to unity (see Par.4.2.2).

The model calcutes drainage using a total drainage resistance:

$$\gamma_{\text{drain}} = \gamma_{\text{drain,inp}} + \frac{L_{\text{drain}}}{u_{\text{drain}}} \gamma_{\text{entry}} \qquad (4.27)$$

where: $\gamma_{\text{drain,inp}}$ is input to the model, u_{drain} is the wetted perimeter (cm), γ_{entry} is the entrance resistance (d)

In case of sub-irrigation, the entrance resistance (then denoted as γ_{inf}) can differ from that for drainage (γ_{drain}): it can either be higher or lower, depending on local conditions. A substantial raising of the surface water level can for instance result in infiltration through a 'bio-active' zone (e.g. involving pores of rain worms) which will reduce the entrance resistance. In most situations with sub-irrigation the radial resistance will be higher than with drainage, because the wetted section of the subsoil is less than in the situation with drainage (the groundwater table becomes concave instead of convex). Especially if the conductivity of the subsoil above the drainage base is larger than in the deeper subsoil, the sub-irrigation resistance γ_{inf} will be substantially higher than the drainage resistance γ_{drain}. In view of these various possible practical situations, the model has the option for using sub-irrigation resistances that differ from the ones for drainage (e.g. $\gamma_{inf} \approx 3/2\ \gamma_{drain}$ in Figure 14).

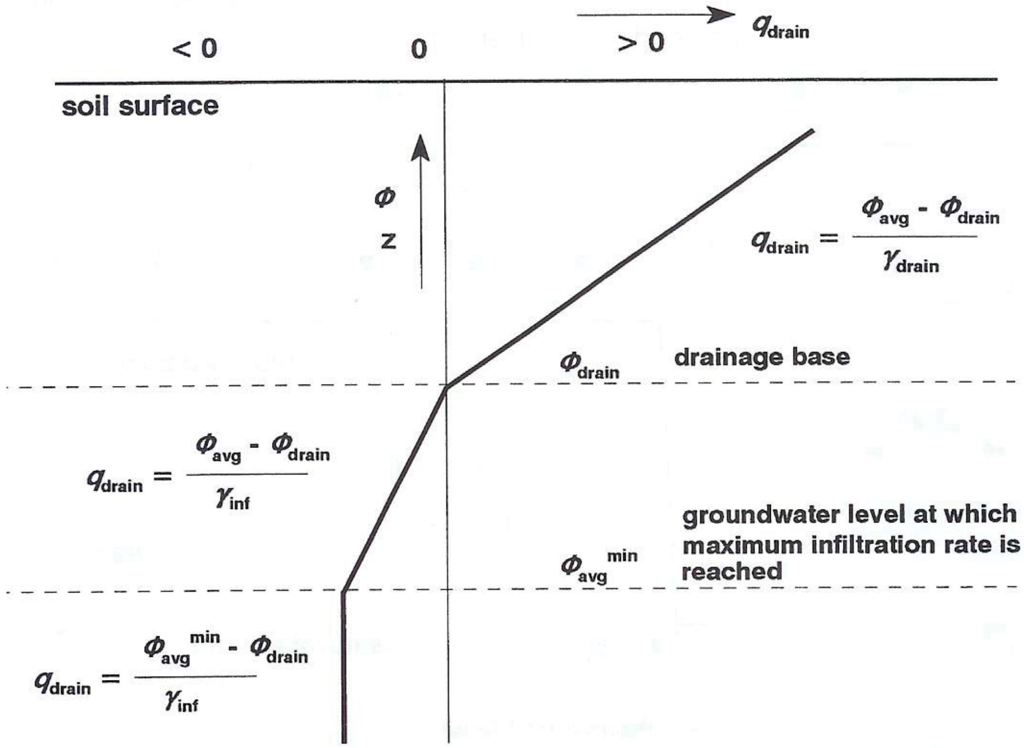


Figure 14 Lineair relationships between drainage ($q_{drain} > 0$) and infiltration ($q_{drain} < 0$) flux and mean groundwater level ϕ_{avg}

An additional model option is to limit the simulated sub-irrigation rate. Such a limitation is needed because the sub-irrigation rate does not increase forever when the groundwater level drops: asymptotically a maximum rate is reached. This maximum rate is determined by the surface water level, the geometry of the wetted channel cross-section and the permeability of the subsoil. For practical reasons we have not set a limit to the sub-irrigation rate itself (Figure 14). Instead, we have limited the simulated sub-irrigation rate by defining the groundwater level ϕ_{avg}^{min} at which the maximum sub-irrigation rate is reached. The linearised relationship, given by Eq. (4.26), is not valid at lower groundwater levels.

Because the non-steady groundwater flow is simulated as a sequence of steady-state conditions, we use the linearised relation between q_{drain} and ϕ_{avg}. This approach is only valid if the drainage resistance is concentrated in the direct vicinity of the channel cross-section, i.e. that the radial resistance is far more important than the horizontal resistance. In such cases the shape factor approaches unity. This contrasts with the case of 'perfect' drains where the shape factor varies with time, depending on the sequence of preceding recharges. After a 'storm recharge' the drainage flow to 'perfect' drains is much higher than the flow predicted by the steady-state relationship. In most situations however, the radial resistance is much higher than the horizontal one, and the use of a steady-state relationship for non-steady simulations will not lead to major errors.

4.2.5.3 Multi level drainage

For illustration purposes we consider a multi-level drainage involving third and fourth order systems (Figure 15):

- the third-order drainage system consists of ditches;
- the fourth-order system consists of subsurface drains;
- the ditches and drains are assumed to be equidistant and parallel.

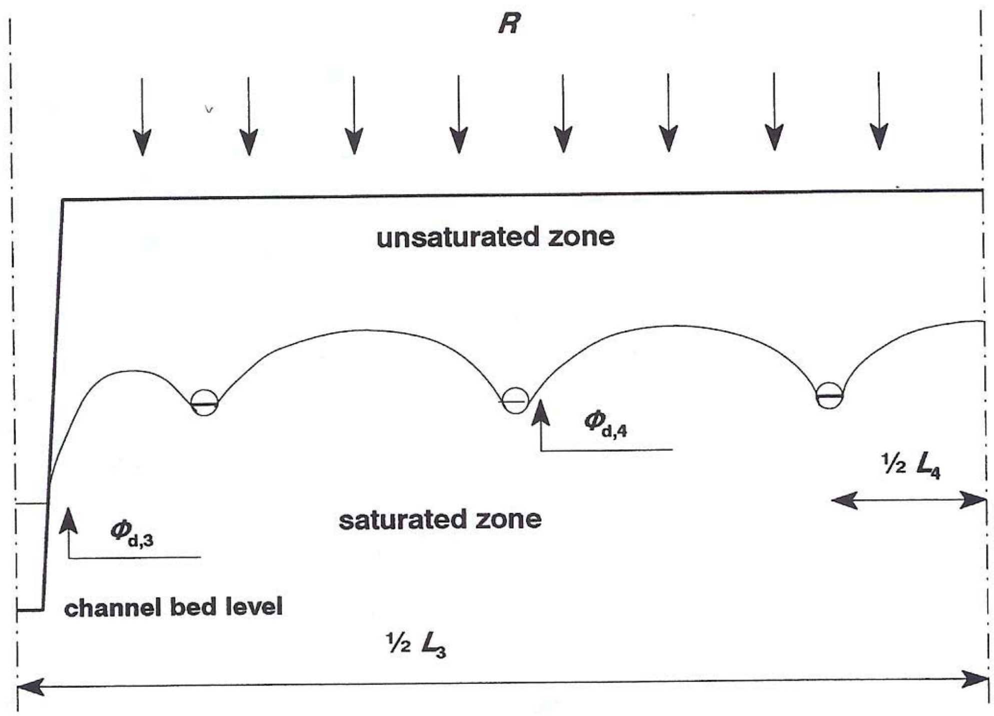


Figure 15 Cross-section of multi-level drainage, involving a third-order system of ditches and a fourth-order system of pipe drains

In this case of two-level drainage we need to quantify the drainage fluxes to both levels of drainage media. We implicitly assume that nearly all of the flow resistance is concentrated in the vicinity of the drainage media (channels and drains). In the most extreme case with only entrance resistance, the water level is horizontal, as shown in Figure 16. In such a case groundwater behaves as a linear reservoir, with outlets at different levels ('tank with holes', see Figure 18). This approach is valid if the main part of the drainage resistance is concentrated near the drains or ditches. For most soils in the Netherlands this seems a reasonable assumption.

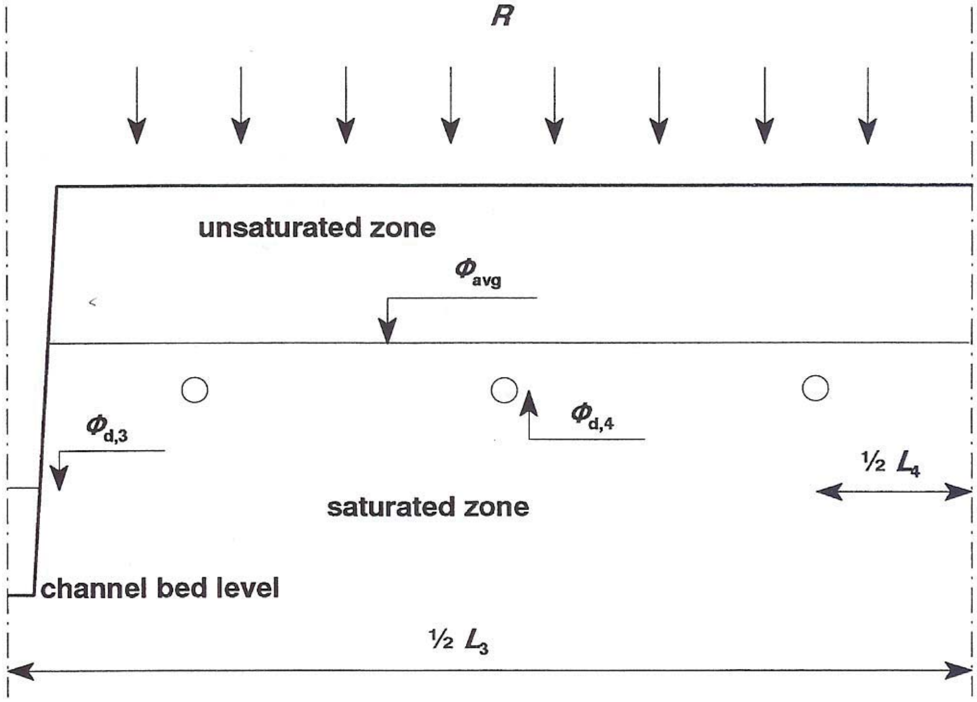


Figure 16 Cross section of multi-level drainage. The main part of the flow resistance is assumed to be located near the drains and ditches, which results in a horizontal groundwater table

Similar to the case of single-level drainage, a drainage level is only 'active' if either the groundwater level or the surface water level is higher than the channel bed level. The drainage base is determined separately for each of the drainage levels, using Eq. (4.25). In computing the total flux to/from surface water, the contributions of the different channel orders are simply added. For the situation with the groundwater level above the highest bed level and with the surface water level below the lowest one, for instance, the total drainage flux is computed with:

$$q_{\mathrm{drain}} = \sum_{i=1}^{\mathrm{n}} \frac{\phi_{\mathrm{avg}} - \phi_{\mathrm{d},i}}{\gamma_{\mathrm{d},i}} \tag{4.28}$$

where the drainage base $\phi_{\mathrm{d,i}}$ is in this case equal to the channel bed level, $z_{\mathrm{bed},i}$. If the surface water level becomes higher than the channel bed level $z_{\mathrm{bed},i}$, the latter is replaced by the surface water level.

Model input

Variable	*Code*	*Description*
ϕ_{sur}^{j}	WLP	Water level in primary water course as a function of date (cm)
$\phi_{\mathrm{sur}}^{j+1}$	WLS	Water level in secondary water course as a function of date (cm)

4.2.5.4 Procedure for surface water level as input

SWAP calculates the net discharge q_{dis}- q_{sup} between t^j and t^{j+1} for the given surface water levels ϕ_{sur}^j and ϕ_{sur}^{j+1} at the beginning an end of a time step, using Eq. (4.22) in a rearranged form:

$$q_{\text{dis}} - q_{\text{sup}} = \frac{V_{\text{sur}}^j - V_{\text{sur}}^{j+1}}{\Delta t^j} + q_{\text{drain}} + q_{\text{c,drain}} + q_{\text{run}} \qquad (4.29)$$

The terms on the right hand side are known or can be calculated (V_{sur} is a function of the known ϕ_{sur}). If the sum is positive, discharge has taken place and the supply is equal to zero. If the sum is negative, supply has taken place and the discharge is equal to zero.

4.2.5.5 Procedure for surface water level as output

This procedure calculates the surface water level from the surface water balance of a control unit. For each water management period a fixed or an automatic weir can be simulated. The settings of the weirs can be different for each management period, as can be the other input parameters of water management. One of the most important input parameters is the maximum rate at which water can be supplied from an external source (for sub-irrigation). During each time step, SWAP determines:

- the target level;
- whether the target level is reached, and the amount of external supply that is needed (if any);
- the discharge that takes place (if any) and the surface water level at the end of the time step.

In the case of a fixed weir, the target level coincides with the level of the crest (which is fixed during a certain management period, but can be changed from one period to the next). In the case of an automatic weir, the target level is determined by a water management scheme. This scheme gives the desired setting of the target water level $\phi_{\text{sur,tar}}$ in relation to a number of state variables of the system. At present it is possible to relate the target level to:

- the average groundwater level ϕ_{avg};
- the soil water pressure head h (cm) at a certain depth in the soil profile;
- total water storage of the unsaturated soil profile V_{uns} (cm).

A high groundwater level will lead to a lower target level, in order to minimize reduction of crop growth due to waterlogging. In nature reserves this criterium does not apply. A soil water pressure head gives a better indication of a threat of waterlogging, than the groundwater level only. The water amount that still can be stored in the soil profile, indicates the buffer capacity in case of heavy rainfall. Maintaining a certain minimum amount of storage, reduces the risk of flooding and subsequent discharge peaks.

Table 2 Example of a water management scheme, with $\phi_{sur,tar}$ the target level for surface water, the criterium $\phi_{avg,max}$ for the mean groundwater level (maximum), the criterium h_{max} for the pressure head (maximum) and $V_{uns,min}$ for the unsaturated volume (minimum). The program selects the highest target level for which all three criteria are met.

$\phi_{sur,tar}$ (cm)	$\phi_{avg,max}$ (cm)	h_{max} (cm)	$V_{uns,min}$ (cm)
-180	0	0	0
-160	-80	-100	1.5
-140	-90	-150	2.0
-120	-100	-200	2.5
-100	-120	-250	3.0
-80	-130	-300	4.0

An example of the water management scheme with target levels and criteria, is shown in *Table 2*. On the first line the minimum target level is specified. The criteria for this level (zeros) are dummies: the minimum target level is chosen whatever the prevailing conditions. The water management scheme selects the highest level for which all three criteria are met.

The water management scheme also has a *maximum drop rate* parameter, which specifies the maximum rate with which the target level of an automatic weir is allowed to drop (cm d^{-1}). This is needed to avoid situations in which the target level reacts abruptly to the prevailing groundwater level. An abrupt drop can cause instability of channel walls or wastage of water that could have been infiltrated. Such a situation can occur during a period with surface water supply and a rising groundwater level due to infiltrating water: the rising groundwater level can cause a different target level to be chosen for the surface water system.

After having determined the target level, the next step in the procedure is to determine whether it can be reached within the considered time step. If necessary, surface water supply is used to attain the target level. This supply is not allowed to exceed the maximum supply rate $q_{sup,max}$, which is an input parameter. For situations with supply, it is possible to specify a tolerance for the surface water level in relation to the target level. This tolerance, the *allowed dip* of the surface water level, can for instance be 10 cm. Then the model does not activate the water supply as long as the water level remains within this tolerance limit of the target level. An appropriate setting of this parameter can save a substantial amount of water, because quick switches between supply and discharge are avoided.

The final step in the procedure is to determine the discharge that takes place (if any) and the surface water level at the end of the time step. Discharge takes place if no supply is needed for reaching the target level. In that case the supply rate is set to zero. In the case of an automatic weir, the discharge follows simply from the water balance equation in the form given by Eq. (4.29), with q_{sup} set to zero and the storage V_{sur}^{j+1} set equal to the storage for the target level. The discharge q_{dis} is then the only unknown left, and can be solved directly.

In the case of a fixed weir, the discharge can not be determined so easily. For the 'stage-discharge' relationship $q_{dis}(\phi_{sur})$ of a fixed weir, we use:

$$q_{dis} = \alpha \left(\phi_{sur} - z_{weir} \right)^{\beta} \qquad (4.30)$$

in which z_{weir} is the weir crest level (cm), α is the discharge coefficient ($cm^{1-\beta}\ d^{-1}$), and β is the discharge exponent (-).

In hydraulic literature head-discharge relationships are given in SI-units, i.e. m for length and s for time and the discharge is computed as a volume rate ($m^3\ s^{-1}$). To facilitate the input for the user we conformed to hydraulic literature. This implies that the user has to specify the weir characteristics that define a relationship of the following form:

$$Q = \alpha_{weir} H^{\beta_{weir}} \tag{4.31}$$

where Q is the discharge ($m^3\ s^{-1}$), $H = \phi_{sur} - z_{weir}$ is the head above the crest (m) and α_{weir} is a weir coefficient ($m^{3-\beta}\ s^{-1}$), β_{weir} is a weir exponent (-).
The user has to compute the value of α_{weir} from the various coefficients preceding the upstream head above the crest. For instance, for a broad-crested rectangular weir, α_{weir} is (approximately) given by:

$$\alpha_{weir} = 1.7b \tag{4.32}$$

where 1,7 is the discharge coefficient of the weir (based on SI-units), b is the width of the weir (m).

To correct for units, the model carries out the following conversion:

$$\alpha = \frac{8.64 * 100^{(1-\beta_{weir})}}{A_{cu}} \alpha_{weir} \tag{4.33}$$

where A_{cu} is the size of the control unit (ha).
The model requires input of the size of the control unit (A_{cu}), which in simple cases will be identical to the size of the simulation unit.

Also a table can be used to specify this relationship. The relationship should be specified for all the management periods, *including* those with management using an automatic weir. In situations with increasing discharge, at a certain moment the capacity of the automatic weir will be reached. In such situations the crest is lowered to its lowest possible position, and the water level starts to rise above the target level. This type of situation can only be simulated correctly if the lowest possible crest level has been specified, and the discharge relationship has been defined accordingly.

To determine the discharge of a fixed weir, the stage-discharge relationship has to be substituted in the water balance equation of Eq. (4.22). The (unknown) surface water level ϕ_{sur}^{j+1} influences both V_{sur}^{j+1} and q_{dis}. This equation can not be solved directly because there can be a transition from a no-flow situation at the beginning of the time step to a flow situation at the end of the time step. For this reason an iterative numerical method is used to determine the new surface water level ϕ_{sur}^{j+1} and the discharge (see Par.4.2.5.6).

Model input

Variable	*Code*	*Description*
ϕ_{sur}	WLACT	Initial surface water level (cm)
-	OSSWLM	Criterium for warning about osscilation (cm)
For each management period specify:		
-	IMPEND	Date that management ends
-	SWMAN	Type of water management (1 = fixed weir crest, 2 = automatic weir)
q_{sup}	WSCAP	Surface water supply capacity (cm d^{-1})
-	WLDIP	Allowed dip of surface water level, before starting supply (cm)
-	INTWL	Length of water-level adjustment period (d)
Exponential discharge relation:		
A_u	SOFCU	Size of control unit (ha)
Specify for all periods:		
z_{weir}	HBWEIR	Weir crest (cm)
α_{weir}	ALPHAW	Alpha-coefficient of discharge formula
β_{weir}	BETAW	Beta-coefficient of discharge formula
Table discharge relation:		
Specify for all periods:		
-	ITAB	Index per management period (-)
ϕ_{sur}	HTAB	Surface water level (cm)
q_{dis}	QTAB	Discharge (cm d^{-1})
Automatic weir control:		
Specify for all periods:		
-	DROPR	Maximum drop rate of surface water level (cm d^{-1})
-	HDEPTH	Depth in soil profile for comparing with HCRIT (cm)
-		
-	IPHASE	Index per management period (-)
$\phi_{sur,tar}$	WLSMAN	Surface water level (cm)
$\phi_{avg,max}$	GWLCRIT	Groundwater level (cm)
h_{max}	HCRIT	Critical pressure head, max. value (cm)
$V_{uns,min}$	VCRIT	Critical unsaturated volume for all surface water levels (cm)

4.2.5.6 Implementation aspects

Schematization into subregions

A simulation at subregional scale will often not stand on its own. A relatively large study area will be divided into several subregions. The boundaries of the subregion(s) should be chosen in a judicious manner. Ideally a subregion is horizontal, has the same type of soil throughout, has a regularly structured dendritic surface water system, and has a groundwater level that does not vary much in depth (a few decimeters). In practice this will hardly ever be the case. By making the subregions very small, the variation of the groundwater depth will be limited, but the number of defined subregions will increase. Another disadvantage can be that the surface water system becomes divided into units that are smaller than the basic control unit which functions in the field. This makes it hard to translate practical water management strategies into model parameters and vice versa. It may also become difficult to compare measured and simulated water balances with each other, which hampers model calibration. The schematization into subregions is a compromise, affected by these aspects.

Schematisation of the surface water system

SWAP uses at most five distinct 'orders' of channels/drains, with exactly defined channel characteristics per order. In reality, the channel characteristics will not be exactly defined. Variations of channel depths by a few decimeters are quite normal. The classification should not involve more classes than necessary, as more classes require more input data and produce more output data. If this extra data load can not be justified by a significantly better simulation result, the extra data will simply be an extra burden and hamper result interpretation.

Obtaining model input data for the smaller channels is relatively straightforward. Each order of channels can be treated as a separate single-level drainage medium, for which data can be derived using formulae given in Par. 4.2.2. Getting data for the large primary water courses can be more involved, especially if the spacing is at a larger scale than the subregion itself. It will then become less realistic to (for these channels) use the mean groundwater level ϕ_{avg}. Instead, the position of the subregion with respect to two channels of the primary order should be taken into account. If, for instance, the subregion is roughly midway between two such channels, the drainage resistance for the maximum groundwater level ϕ_{gwl} should be used, but only for these large channels, not for the rest of the surface water system. In such a case it is obvious that the surface water level in the primary channel is determined by the water balance on a scale that is much larger than that of the subregion. It is then also appropriate to model the primary channel as being separate from the rest of the surface water system.

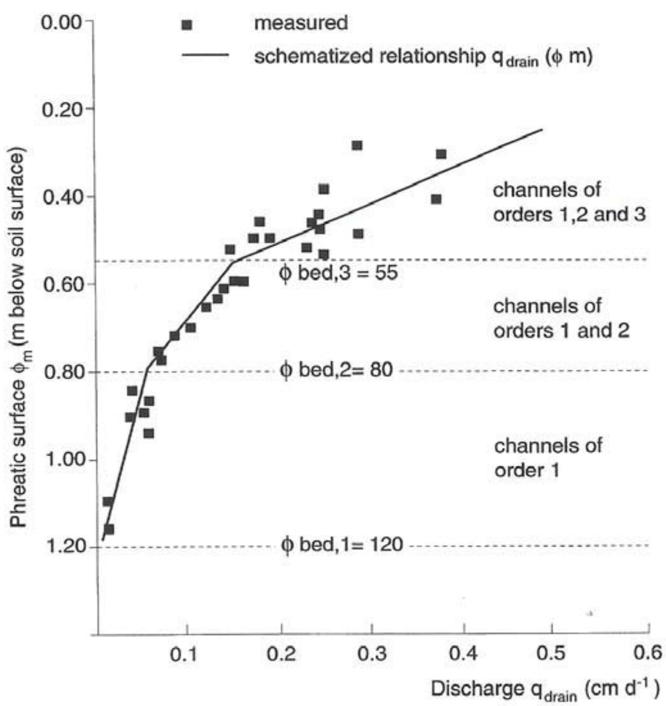


Figure 17 Discharge qdrain as function of mean phreatic surface ϕ_{avg} in the Beltrum area (Massop and de Wit, 1994)

An alternative way of making a schematization of the surface water system is by analysis of experimental data. In Figure 17 the results are shown of field measurements by Massop and De Wit (1994) for the Beltrum area. A discharge unit was identified and measurements were made of:

- total surface area;
- discharge at the outlet;
- mean groundwater level.

From Figure 17 one can see that the drainage base of the larger channels is roughly at z = -120 cm, as no discharges were measured below that level. The schematized $q_{\mathrm{drain}}(\phi_{\mathrm{avg}})$-relationship is a piece-wise linear function, with transition points at mean groundwater levels of 80 and 55 cm below soil surface. These transition points correspond to the 'representative' bed levels of the second and third order channels. The drainage resistance of the first order channels can be derived from the transition point at z = -80 cm in the following manner:

$$q(-80) = 0.05 = \frac{\phi_{\mathrm{avg}} - \phi_{\mathrm{d},1}}{\gamma_{\mathrm{d},1}} = \frac{-80+120}{\gamma_{\mathrm{d},1}} \tag{4.34}$$

which gives $\gamma_{\mathrm{d},1}$ = 800 d. The drainage resistance of the second-order channels follows subsequently from:

$$q(-55) = 0.15 = \frac{\phi_{\mathrm{avg}} - \phi_{\mathrm{d},1}}{\gamma_{\mathrm{d},1}} + \frac{\phi_{\mathrm{avg}} - \phi_{\mathrm{d},2}}{\gamma_{\mathrm{d},2}} = \frac{-55+120}{800} + \frac{-55+80}{\gamma_{\mathrm{d},2}} \tag{4.35}$$

which results in $\gamma_{\mathrm{d},2}$ = 365 d. Analogously, the drainage resistance of the third-order channels can be derived: $\gamma_{\mathrm{d},3}$ = 135 d.

Numerical schemes

The land surface model, in which the Richards' equation is solved, and the surface water model are coupled by means of an *explicit* numerical scheme. In other words, the surface water level update and the calculation of the drainage fluxes do *not* interact with the calculation of the soil water content and the groundwater level *within a time step*. Thus the drainage fluxes are computed using the groundwater level and the surface water level at the *beginning* of a time step.

The surface runoff (or runon), however, is computed with Eq. (4.2) using more up-to-date information: the ponding height h_{pond} at the *end* of a time step is used. This is made possible by the sequence of calculations in SWAP for situations with total saturation and ponding at the soil surface:

- first the Richard's equation is solved for the soil profile, with prescribed head $h = h_{\mathrm{pond}}$ at the soil surface;
- next the ponding depth hpond is updated from the water balance of the total soil profile, including surface runoff.

Explicit numerical schemes have the disadvantage that the computed levels can become unstable. To reduce the chance of oscillations in the simulated levels, the program reduces the time step automatically as soon as the ponding starts. If the specified 'ponding sill' has been set to zero, however, the first time step with surface runoff may lead to instability, because the time step is reduced from the second time step after ponding onwards. The user can avoid this instability by specifying a non-zero value for the maximum ponding depth, e.g. of 1 cm.

For computing the surface water level in situations with a fixed weir, an equation has to be solved involving a look-up table (storage as a function of surface water level) and an exponential discharge relationship (discharge of weir as a function of the surface water level). We use an implicit iterative procedure for this, involving the surface water level at the *end* of the time step. This scheme has the advantage of being very stable. The disadvantage is that the computed discharge might deviate from the 'average' discharge during the time step. But since the used time steps are relatively small (<0.2 d), the loss of accuracy is not significant.

It can nevertheless be possible, even without surface runoff, that the simulated surface water and groundwater levels become unstable. SWAP warns the user if large oscillations of surface or groundwater levels occur. In such a case the user should reduce the maximum time step. In general, a time step of 1/50 of the smallest drainage resistance should lead to a stable simulation. If, however, the surface water system is highly reactive to drainage flows, an even smaller time step may be required.

4.3 Residence time approach

4.3.1 Introduction

Following the discussion in Par. 4.2, the drain densities of a three level drainage system are defined as:

$$M_1 = \frac{\sum l_1}{A_{reg}}; \qquad M_2 = \frac{\sum l_2}{A_{reg}}; \qquad M_3 = \frac{\sum l_3}{A_{reg}} \tag{4.36}$$

where A_{reg} (cm^2) is the area of the subregion, Σl_1, Σl_2 and Σl_3 are the total lengths (cm) of respectively the first, second and third order drains and M_1, M_2, M_3 are the drainage densities (cm^{-1}) of respectively the first order, the second order and the third order drainage system. The drainage fluxes $q_{d,1}$, $q_{d,2}$ and $q_{d,3}$ (cm d^{-1}) are calculated by linearized flux-head relationships (see Eq. 4.26):

$$q_{d,1} = \frac{\varphi_{avg} - \varphi_{d,1}}{\gamma_1}; \qquad q_{d,2} = \frac{\varphi_{avg} - \varphi_{d,2}}{\gamma_2}; \qquad q_{d,3} = \frac{\varphi_{avg} - \varphi_{d,3}}{\gamma_3} \tag{4.37}$$

where ϕ_{avg} is the regional averaged groundwater level (cm), $\phi_{d,i}$ the drainage hydraulic head (cm) of drainage system order i, and γ_i the drainage resistance (d) of drainage system order i. This drainage concept is schematically illustrated in Figure 18, depicting a linear reservoir model with outlets at different heights.

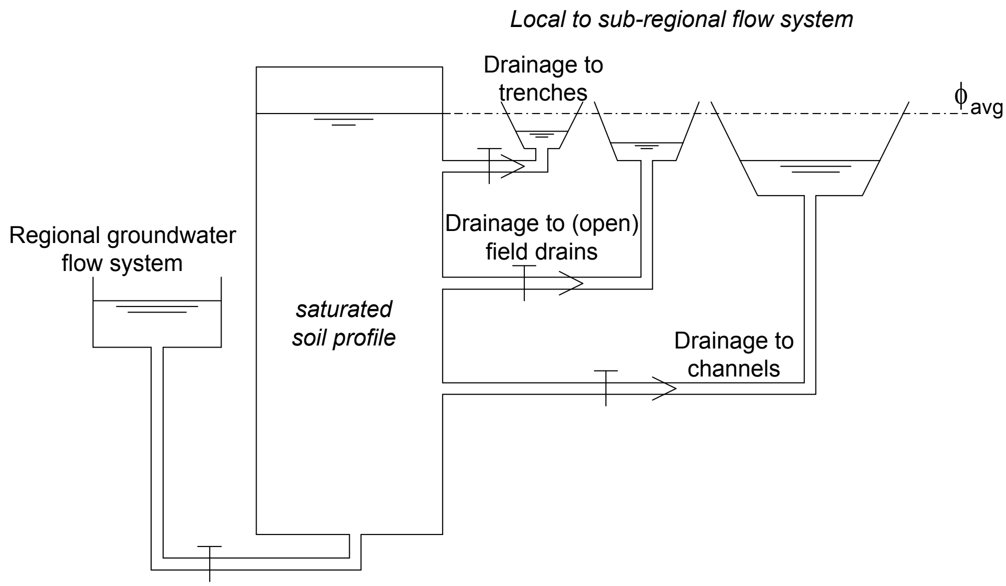


Figure 18 Illustration of regional drainage concept. The resistance mainly consists of radial and entrance resistance near the drainage devices

4.3.2 The horizontal groundwater flux

One-dimensional leaching models generally represent a vertical soil column. Within the unsaturated zone, chemical substances are transported by vertical water flows, whereas in the saturated zone the drainage discharge leaves the vertical column side-ways. For example in the ANIMO model (Rijtema et al., 1997), the distribution of lateral drainage fluxes with depth has been used to simulate the response of the load of chemicals on the surface water system to the inputs in the groundwater system. In this section, the concept for a distribution of lateral drainage fluxes with depth in an one-dimensional hydrological simulation model will be described. The following assumptions are made:

- steady groundwater flow and homogeneous distribution of recharge rates by rainfall;
- the aquifer has a constant thickness.

For convenience, only three levels of drains are considered, although the concept discussed here is valid for a system having any number of drainage levels.

Van Ommen (1986) has shown that for simple single level drainage systems, the travel time distribution is independent from the size and the shape of the recharge area. Under these assumptions, the average concentration of an inert solute in drainage water to a well or a watercourse, can mathematically be described by the linear behaviour of a single reservoir. This behaviour depends only on the groundwater recharge rate, the aquifer thickness and its porosity.

The non-homogeneous distribution of exfiltration points as well as the influence of chemical reactions on the concentration behaviour necessitates to distinguish between the hydraulic and chemical properties of different soil layers. In the drainage model, which describes the drainage discharge to parallel equidistant water courses, the discharge flow of system i, $Q_{d,i}$ is calculated as:

$$Q_{d,i} = L_i q_{d,i} \tag{4.38}$$

where L_i is the spacing of drainage system i. According to the Dupuit-Forcheimer assumption, the head loss due to radial flow and vertical flow can be ignored in the largest part of the flow domain. Following this rule, the ratio between occupied flow volumes V_i can be derived from the proportionality between flow volumes and discharge rates:

$$\frac{V_i}{V_{i-1}} = \frac{Q_{d,i}}{Q_{d,i-1}} \tag{4.39}$$

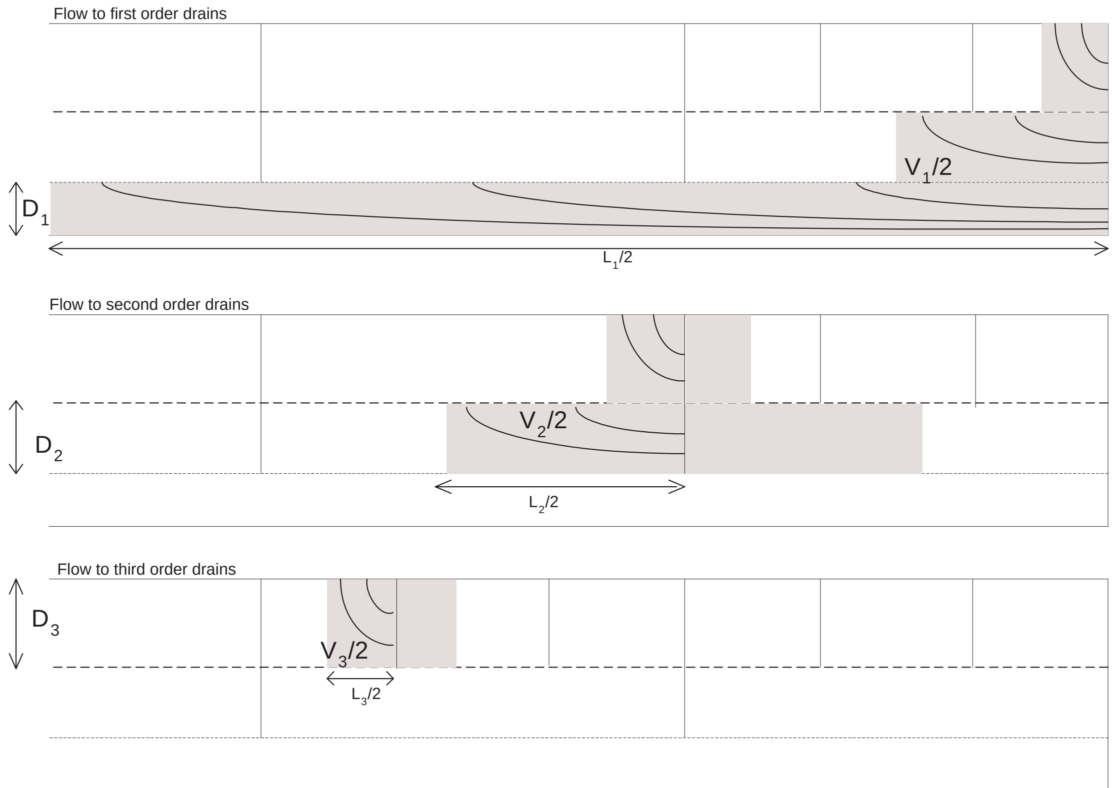


Figure 19 Schematization of regional groundwater flow to drains of three different orders

First order drains act also as field ditches and trenches and next higher drains act partly as third order drains. In the SWAP-model the lumped discharge flux per drainage system is computed from the relation between groundwater elevation and drainage resistance. Figure 19 shows the schematization of the regional groundwater flow, including the occupied flow volumes for the nested drain systems. The volume V_i consists of summed rectangles L_iD_i of superposed drains, where D_i is the thickness (cm) of discharge layer i.

The flow volume V_i assigned to drains of order 1, 2 and 3 is related to drain distances L_i and thickness D_i of discharge layers as follows:

$$V_1 = L_1D_1 + L_2D_2 + L_3D_3 \tag{4.40}$$

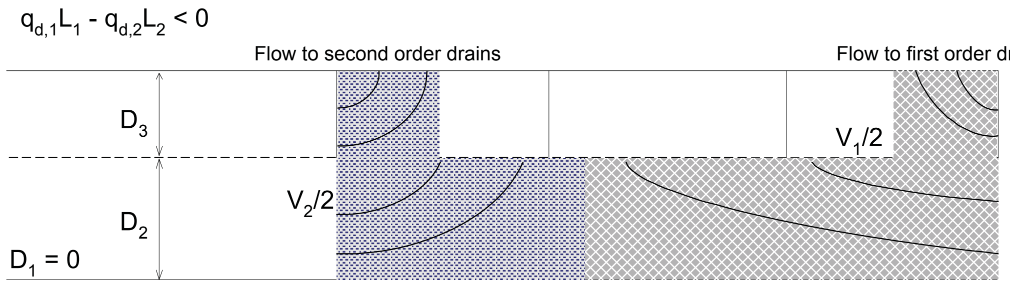
$$V_2 = L_2D_2 + L_3D_3 \tag{4.41}$$

$$V_3 = L_3D_3 \tag{4.42}$$

Rewriting Eq. (4.40) to (4.42) and substituting Eq. (4.38) and Eq. (4.39) yields an expression which relates the proportions of the discharge layer to the discharge flow rates:

$$L_1 D_1 : L_2 D_2 : L_3 D_3 = \left(q_{d,1} L_1 - q_{d,2} L_2\right) : \left(q_{d,2} L_2 - q_{d,3} L_3\right) : \left(q_{d,3} L_3\right) \qquad (4.43)$$

In theory, the terms $q_{d,1}\ L_1 - q_{d,2}\ L_2$ and $q_{d,2}\ L_2 - q_{d,3}\ L_3$ can take negative values for specific combinations of $q_{d,1}\ L_1$, $q_{d,2}\ L_2$ and $q_{d,3}\ L_3$. When $q_{d,1}\ L_1 - q_{d,2}\ L_2 < 0$ it is assumed that D_1 will be zero and the nesting of superposed flows systems on top of the flow region assigned to drainage class 1 will not occur. Likewise, a separate nested flow region related to a drainage class will not show up when $q_{d,2}\ L_2 - q_{d,3}\ L_3 < 0$. These cases are depicted schematically in Figure 20.



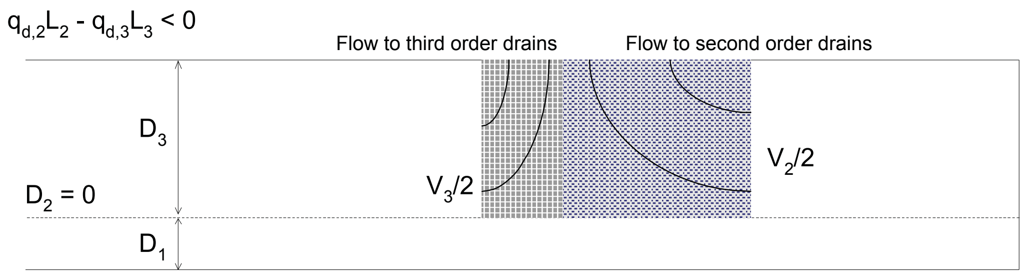


Figure 20 Schematization of regional groundwater flow to drains of three orders when either $q_{d,1}L_1 - q_{d,2}L_2 < 0$ or $q_{d,2}L_2 - q_{d,3}L_3 < 0$

If the soil profile is heterogeneous with respect to horizontal permeabilities, the heterogeneity can be taken into account by substituting transmissivities kD for layer thicknesses in Eq.(4.43):

$$(kD)_1 : (kD)_2 : (kD)_3 = \left(\frac{q_1 L_1 - q_2 L_2}{L_1}\right) : \left(\frac{q_2 L_2 - q_3 L_3}{L_2}\right) : \left(\frac{q_3 L_3}{L_3}\right) \qquad (4.44)$$

The thickness of a certain layer can be derived by considering the vertical cumulative transmissivity relation with depth as shown in Figure 21.

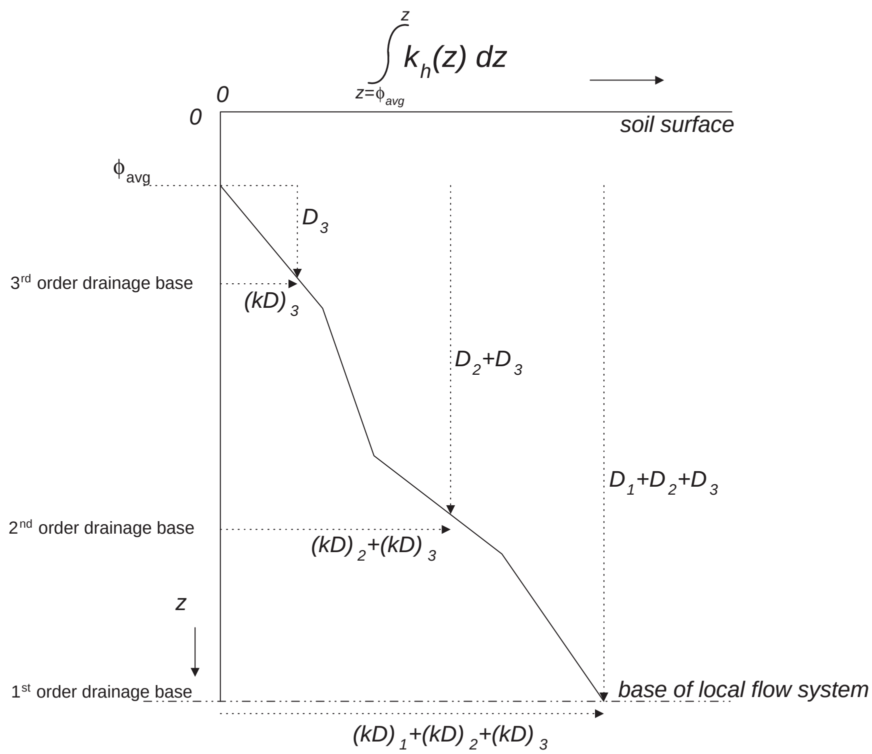


Figure 21 Discharge layer thickness D_i as function of cumulative transmissivity kD_i in a heterogeneous soil profile

The lateral flux relation per unit soil depth shows a uniform distribution. Lateral drainage fluxes $q_{d,k,i}$ to drainage system k for each nodal compartment i of the simulation model are calculated by:

$$q_{d,1,i} = q_{d,1} \frac{k_{h,i} \Delta z_i}{\sum_{i_{z=\varphi_{avg}}}^{i_{z=-D_1-D_2-D_3}} k_{h,i} \Delta z_i} \qquad \text{for} \qquad -D_1 - D_2 - D_3 < z < \varphi_{avg} \tag{4.45}$$

$$q_{d,2,i} = q_{d,2} \frac{k_{h,i} \Delta z_i}{\sum_{i_{z=\varphi_{avg}}}^{i_{z=-D_2-D_3}} k_{h,i} \Delta z_i} \qquad \text{for} \qquad -D_2 - D_3 < z < \varphi_{avg} \tag{4.46}$$

$$q_{d,3,i} = q_{d,3} \frac{k_{h,i} \Delta z_i}{\sum_{i_{z=\varphi_{avg}}}^{i_{z=-D_3}} k_{h,i} \Delta z_i} \qquad \text{for} \qquad -D_3 < z < \varphi_{avg} \tag{4.47}$$

where $k_{h,i}$ is the horizontal conductivity (cm d^{-1}) of compartment i, Δz_i is the thickness (cm) of compartment i, and $i_{Z=-D1-D2-D3}$ and $i_{z=\phi avg}$ are resp. the numbers of the bottom compartment and the compartment in which the regional groundwater level is situated. Water quality models such as ANIMO (Rijtema et al., 1997) compute the average concentration of discharge water which flows to a certain order drainage system on the basis of these lateral fluxes. The avering rules are:

$$\overline{c_1} = \frac{\sum_{i_z=\varphi_{avg}}^{i_{z=-D_1-D_2-D_3}} q_{d,1,i} c_i}{q_{d,1}} \qquad (4.48)$$

$$\overline{c_2} = \frac{\sum_{i_z=\varphi_{avg}}^{i_{z=-D_2-D_3}} q_{d,2,i} c_i}{q_{d,2}} \qquad (4.49)$$

$$\overline{c_3} = \frac{\sum_{i_z=\varphi_{avg}}^{i_{z=-D_3}} q_{d,3,i} c_i}{q_{d,3}} \qquad (4.50)$$

Using these average concentrations computed by a leaching model, the average concentration c_R at the scale of a sub-region is calculated as:

$$\overline{c_R} = \frac{q_{d,1}\overline{c_1} + q_{d,2}\overline{c_2} + q_{d,3}\overline{c_3}}{q_{d,1} + q_{d,2} + q_{d,3}} \qquad (4.51)$$

4.3.3 Maximum depth of a discharge layer

For the purpose of water quality simulations, the thickness of a model discharge layer has to be limited to a certain depth. In the water quality model, the maximum thickness D of a discharge layer has been set at:

$$D \leq \frac{L}{4} \qquad (4.52)$$

This rule of thumb is based on the assumption of a half-circular shape of streamlines in a flow field (Figure 22). The deepest streamline which arrives in the drain, originates from a point at distance L/2. It can be seen that following to the circular shape, the horizontal distance L/2 corresponds to the length 2D.

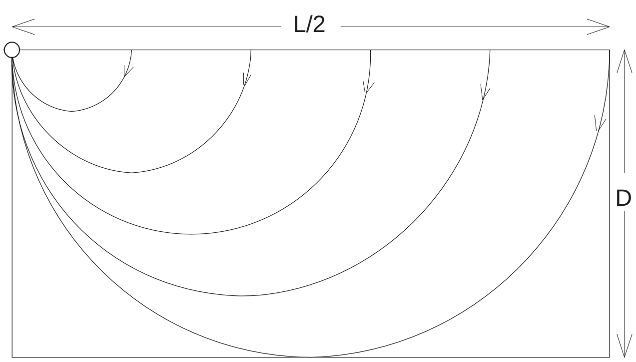


Figure 22 Flow field to a drain with half circular shaped sream lines

Homogeneous anisotropic soil profile

In the saturated zone, the horizontal permeability is often larger than the vertical permeability. General assumptions to deal with the transformation of the anisotropic conditions of a two-dimensional flow field are:

- hydraulic heads and flow rates are the same as in an isotropic situation
- x-coordinate: $x' = x\,\sqrt{(kv/kh)}$
- z-coordinate: $z' = z$
- permeability: $k' = \sqrt{(kv\ kh)}$

where the primes denote the transformed values of an anisotropic condition. Applying these assumptions to the relation between thickness of the discharge layer D and the horizontal drain distance L yields:

$$D' \leq \frac{L'}{4} \qquad \Rightarrow \qquad D \leq \frac{L}{4}\sqrt{\frac{k_v}{k_h}} \tag{4.53}$$

At first sight, this condition does not agree with the 'penetration depth' derived by Zijl and Nawalany (1993) for the estimation of the order of magnitude of the characteristic depth of the flow problem in case of a single layer model. However, these authors consider the wave length of an assumed sinusoidal shaped phreatic head. This assumption does not hold for most of the flow systems where only 1 or 2% of the area shows an upward discharge flux at the phreatic level. Transforming the wave length variable given by Zijl and Nawalany (1993) to the characteristic distance relevant for drainage systems (L/2) and taking into account the sinusoidal function can fully explain the difference between Eq. (4.53) and the 'penetration depth'.

Heterogeneous anisotropic soil profile

For heterogeneous soil profiles, an average value for the anisotropic factor √(kv/kh) has to be considered. The average horizontal and vertical conductivity is calculated as:

$$\overline{k_h} = \frac{\sum\limits_{i_{z=\varphi_{avg}}}^{i_{z=-D_1-D_2-D_3}} k_{h,i}\Delta z_i}{\sum\limits_{i_{z=\varphi_{avg}}}^{i_{z=-D_1-D_2-D_3}} \Delta z_i} \tag{4.54}$$

$$\overline{k_v} = \frac{\sum\limits_{i_{z=\varphi_{avg}}}^{i_{z=-D_1-D_2-D_3}} \Delta z_i}{\sum\limits_{i_{z=\varphi_{avg}}}^{i_{z=-D_1-D_2-D_3}} \dfrac{\Delta z_i}{k_{v,i}}} \tag{4.55}$$

and the maximum depth of the discharge layer bottom:

$$D \leq \frac{L}{4}\sqrt{\frac{\overline{k_v}}{\overline{k_h}}} \tag{4.56}$$

The assumption of cylindrical shaped streamlines is an abstraction of the actual streamline pattern. The condition (D ≤ L/4) based on this model assumption is most relevant at large D/L ratios. Ernst (1973) provides a mathematical formulation of a streamline pattern in a saturated soil profile of infinite thickness. Such a hydrological situation can be seen as the most extreme situation for evaluating the influence of the D/L-ratio. In reality, the drainage flow will occupy less space in the saturated groundwater body and the flow paths will be less deep. The streamlines can be described as:

$$\psi(x,z) = \frac{q_0}{\pi}\arctan\left(\frac{e^{\frac{-2\pi L}{L}}\sin\left(\frac{2\pi x}{L}\right)}{e^{\frac{-2\pi L}{L}}\cos\left(\frac{2\pi x}{L}\right)-1}\right) \tag{4.57}$$

where ψ (x,z) is the stream function and q0 is the discharge flow rate which originates from the area between x=0 en x=L/2. The streamline pattern is shown graphically in Figure 23, where the water enters the groundwater body along the line z=0 and the water is discharged by a drain at (0,0).

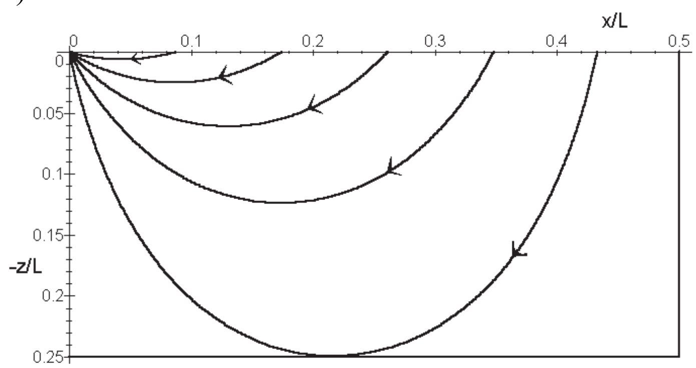


Figure 23 Stream line pattern in a groundwater system of infinite thickness

The majority of the precipitation surplus does not reach the line at depth -z/D=0.25. In this soil column, imaginary horizontal planes at z=-D can be considered. The streamline with its deepest point at -z/D=1, but not intersecting the line z=-D, bounds the stream zone which will never be found below z=-D. The following condition holds for the streamline with its tangent-line at z=-D:

$$\frac{\partial \psi(x,D)}{\partial x} = 0 \tag{4.58}$$

Evaluation of this expression yields a value for the horizontal coordinate of the point of contact between the streamline and the line z=-D. Together with the value z=-D, the horizontal distance can be substituted into the general stream function equation. This action yields a flow fraction ψ/q0 of the total drainage discharge which will never be found below the line z=-D. The depth has been transformed to a fraction of the drain distance to summarize all possible relations into one graph.

In a soil profile with infinite thickness, about 87% of the total drain discharge is conveyed above the plane at z=-L/4. In a deep soil profile with finite thickness, more than 87% of the total drain discharge will be transported above this plane.

4.3.4 Concentrations of solute in drainage water

The discharge layer approach assumes a uniform function of the lateral flux intensity with depth. Therefore, the vertical flux as a function of depth for a single drainage system can be described by a linear relation:

$$q(z) = \varepsilon \frac{dz}{dt} = \left(1 + \frac{z}{D}\right) q_{drain} + q_{bot} \tag{4.59}$$

where ε is the soil porosity (-), q the vertical flux (cm d^{-1}) and qbot the vertical flux across the lower boundary of the soil profile. The relations hold between the phreatic level at z = ϕavg and the lower boundary at z=-D (m). This equation can be used to derive the residence time T (d) as a function of depth, provided t = T_0 at z = ϕ_{avg}:

$$T = T_0 + \frac{\varepsilon D}{q_{drain}} \ln\left(\frac{q(\varphi_{avg})}{q(z)}\right) \tag{4.60}$$

Streamlines can be described mathematically by a stream function. For a two-dimensional transect between parallel drains, assuming a zero flux at the bottom of the aquifer and a negligible radial flow in the vicinity of the drains, the stream function ψ(x,z) can be given as a function of depth z and distance x relative to the origin at the bottom of the aquifer, as depicted in Figure 24:

$$\psi(x, z) = -\frac{R}{D} x (D + z) \tag{4.61}$$

where R is the net recharge and D is the thickness of the homogeneous layer.

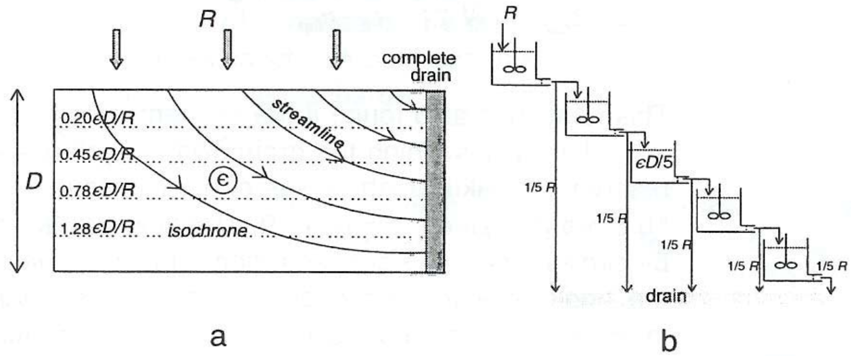


Figure 24 (a) Streamlines and isochrones of a siol profile with complete drains and (b) schematization of the flow pattern by a cascade of perfectly mixed reservoirs

Construction of isochrones for solute displacement after uniform infiltration at the phreatic level yields horizontal lines, because the vertical fluxes do not depend on the horizontal distance relative to the origin. In the model, the isochrones are regarded as imaginary boundaries between soil layers.

Each of the soil layers may be regarded as a perfectly mixed reservoir. Part of the inflow is conveyed to underlying soil layers, the remainder flows horizontally to the water course or drainage tube. Assuming a steady state situation and equal distances between the soil layers, the displacement of a non-reactive solute through this system may be described by a set of linear differential equations. For the first reservoir, the following equation applies:

$$\frac{\varepsilon D}{N}\frac{dc_1}{dt} = Rc_{inp} - Rc_1 \qquad (4.62)$$

where N is the number of soil layers and cinp is the input concentration. For an arbitrary reservoir i, the change in concentration is described by:

$$\frac{\varepsilon D}{N}\frac{dc_i}{dt} = \frac{N-i+1}{N}Rc_{i-1} - \frac{N-i+1}{N}Rc_i \qquad (4.63)$$

Assuming an initial concentration c_0 uniform over the entire depth, the solution to the differential equations yields the concentration course over time in reservoir j:

$$\frac{c_j(t) - c_{inp}}{c_0 - c_{inp}} = \sum_{i=1}^{j}\binom{N}{i-1}\binom{N-i}{j-i}(-1)^{i+1}e^{\frac{-(N-i+1)Rt}{\varepsilon D}} \qquad (4.64)$$

Since the outflows of all reservoirs are assumed to be equal, the resulting concentration in drainage discharge can be found as the average of all reservoirs. Lengthy, but straight forward algebraic summation of the binomial series in Eq. (4.63) yields a simple relation for the concentration in drainage water:

$$\frac{c_d(t) - c_{inp}}{c_0 - c_{inp}} = \frac{1}{N}\sum_{j=1}^{N}\frac{c_j(t) - c_{inp}}{c_0 - c_{inp}} = e^{\frac{-Rt}{\varepsilon D}} \qquad (4.65)$$

This relation is also found if the concentration in the drainage water is modelled by describing the groundwater system as one perfectly stirred reservoir. Breakthrough curves of the individual reservoirs as denoted in Figure 24 are presented in Figure 25. The flow averaged concentration (indicated by circles) fits to the concentration relation for the single reservoir approach. Overall effects of vertical dispersion which are introduced by defining distinct soils layers can thus be described by using one single reservoir. For the single drainage system, the simulation of solute migration by describing a vertical column with uniform lateral outflow agrees with the solutions found by Gelhar and Wilson (1974), Raats (1978) and Van Ommen (1986).

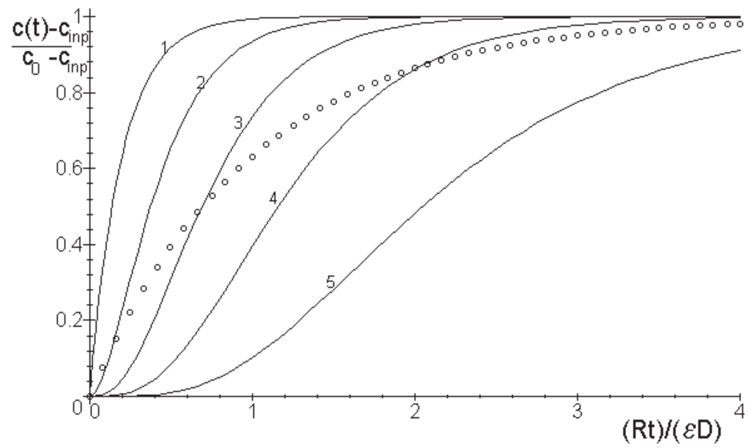


Figure 25 Step response of outflow concetrations per soil layer (numbered lines) and step response of the averaged concentration which enters the drains (circles)

4.3.5 Discussion

As a consequence of a number assumptions and schematization of the flow pattern, the model user should be aware of the following limitations:

- assumption of steady state during the time increment;
- constant depth of the drainage base;
- assumption of perfect drains;
- uniform thickness of the hydrological profile.

In most of the applications of the regional water quality model, the time step is set at 1 day up to 10 days. During an interval of 10 days, the drainage flux may vary as a result of variation of the meteorological conditions. For chemical substances which are bounded in the upper soil layers, the assessment of the solute discharge to the surface water may lead to considerable inaccuracies.

The boundary between the groundwater flow affected by the ‘local’ drainage system and the regional flow can be defined as the depth in the soil profile below which no direct discharge to surface water occurs (Figure 19). Above this depth, the larger part of the precipitation surplus flows to water courses and other drainage systems. This boundary depends on the deepest streamline discharging water to the drainage systems. It can be expected that the size of the subregion influences the depth of the boundary surface. With larger schematized areas, discharge water can originate from greater distances, having deeper streamlines. The influence of the seasonal variation of trans-boundary fluxes at the lower boundary of the modelled soil profile is not considered.

The uniform distribution of the lateral flux pattern is based on the assumption of perfect drains. In reality, the flow pattern converges in the surrounding area of the drain. The soil profile has a uniform depth. When the height difference between maximum groundwater level and drainage level is larger than a certain fraction of the depth of the saturated profile, this assumption may not be valid. In theory, these effects can be simulated by defining a correction function for the lateral flux relation with depth. From the point of view of data acquisition and validation of hydro-geological parameters, refinement of this relationship is questionable.

The Dupuit-assumption has been applied implicitly by assuming horizontal discharge layers. The discharge layer which corresponds to the channel system has been defined as a horizontal layer at the bottom of the local flow system. In reality, the water discharging to canals at larger distances infiltrates into the saturated zone. This water takes up some space in the upper zone of the groundwater system. A way to validate the 'discharge layer' approach presented above is by comparing a set of simulation results with the outcome of three dimensional streamline models at regional scale.