to too low a groundwater level, navigation may be hampered, and/or side slopes may become unstable.
- Maximum Allowable Storage Level/MASL: As the water level cannot be kept at DDL constantly (this would lead to economically unfeasible drainage systems), there is a need to define a highest boundary: the MASL. This boundary is equal to the DDL, plus the maximum tolerable rise of the water level in the system. The MASL is determined by the agricultural drainage criteria on which the design of the field drainage system is based (Chapter 17) and by the design criteria which apply to the main drainage system (Chapter 19). The determination of MASL is also based on economic considerations: it is the level at which the investments needed in the drainage system (enlarging storage capacity) outweigh the risk of economic losses (chance of exceeding MASL, multiplied by losses incurred by yield reduction or damage to canals, structures, housing, etc.).

Depending on the envisaged land use and on the operation and maintenance requirements of the drainage system, the DDL and the MASL may vary throughout the year. For the design of the outlet structure, the most unfavourable levels should be selected in combination with the highest outer water levels: usually this will be the lowest DDL and MASL and/or the smallest difference between MASL and DDL.

24.3 Design of Gravity Outlet Structures

This section reviews the various types of gravity outlet structures, presents the relationships between storage and the hydraulic design of gravity outlets, and formulates guidelines for selecting hydraulic dimensions.

24.3.1 Types of Gravity Outlet Structures

Gravity outlet structures can be a drainage sluice with doors, a gated culvert, or a siphon.

Drainage Sluice

A drainage sluice consists of a weir and a set of doors. Each of the two doors hinges around a vertical axis, and is positioned in such a way that inner water can flow freely to the outer water, whereas they prevent a flow in the opposite direction (Figure 24.18).

The doors will remain closed as long as the pressure from the outer water is greater or equal to the pressure from the inner water. In case of fresh water on both sides, an equilibrium situation occurs when the water levels on both sides are equal. When the outer waters are salty, or when they contain a considerably larger sediment load than the inner waters, the densities differ, so that an equilibrium situation will occur when the inner water level is higher and compensates for the (denser) outer water. In case of salt outer water, the inner water level must be around 1.012 times the outer water level to have equilibrium (Section 24.3.3).

A drainage sluice can be self-operating, manually operated, or automatically operated.

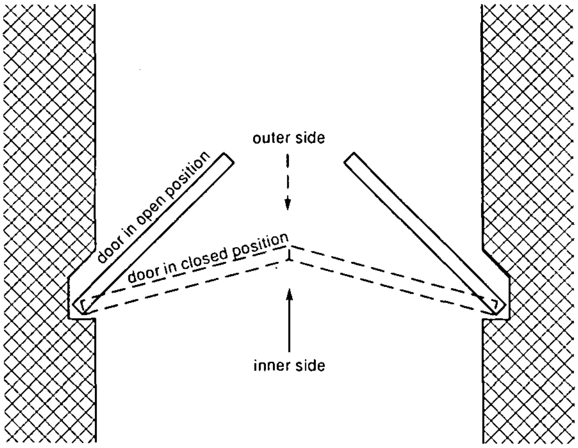

Figure 24.18 Plan view of an outlet sluice with vertical doors (Smedema and Rycroft 1983)

The principle of self-operation is based on differences in water pressure. As soon as the pressure exerted by the inner water against the doors exceeds the pressure of the outer water, the doors will open and water will flow out of the drained area. The doors close when the pressure from the outer water is higher than the inner. For this kind of operation, the sluice should be fitted with vertical doors, which should be constructed as indicated in Figure 24.18. In closed position, the doors should form a 'V' to counteract the outer pressure. In open position, the doors should not be allowed to open entirely to enable the outer water to exert the pressure that is needed to close the doors again. To facilitate and quicken the closing process, the doors must be balanced in such a way that only little overpressure is needed to close them in a very short time (seconds).

It should be noted that the overpressure needed to operate the doors is also needed to overcome friction forces.

Manual operation requires a watchman, who monitors the levels and operates the doors in accordance with a given strategy. In the case of automatic operation this process is automated.

Drainage sluices enable self-operation and offer possibilities for navigation during the drainage period, provided that the water velocities in the sluice are not too high. Other advantages are that, by applying two doors, larger single openings can be realized, that no energy supply is required, and that no personnel is needed to operate them. Therefore, drainage sluices can even be applied in remote areas. Nevertheless, there are certain circumstances where the operation of the drainage sluice should be either manual or automated: if there is an infrequent need to use the drainage sluice (e.g. where the outer water levels fluctuate with the season), when large waves play a role (so that the doors would be continuously opening and closing because of fluctuating outer water levels), and/or when quality control of inner or outer water is required. In these circumstances, the water levels on either side of the doors should be monitored, either by man or by measuring equipment, and the doors should be opened when the outer water levels are sufficiently lower than the inner levels.

From a hydraulic point of view, the drainage sluice can be considered a broad-crested weir, which means that the streamlines over the weir are practically straight and parallel. To obtain this situation, the length (L) of the weir crest should be related to the total upstream energy head (H) as $0.07 \leq H/L \leq 0.50$. A smooth transition between the crest and the downstream slope reduces the head loss over the structure. The recommended slope of the downstream face of the weir equals 1:6 (Figure 24.19).

Assuming that the flow upstream of the weir is sub-critical, two different flows may occur over it, namely sub-critical and critical flow. These flow conditions have their own flow pattern and corresponding equations for calculating the discharge. Note that these equations can be applied for steady-state conditions only.

According to Chapter 7, Section 2.4, the total upstream energy level is defined as

$$H = h_u + \frac{v_u^2}{2g} \tag{24.7}$$

where

H = upstream energy level (m)
h_u = upstream water level (m)
v_u = upstream flow velocity (m/s)
g = acceleration due to gravity (m/s²)

The energy levels are defined relative to the weir crest.

Subcritical Flow
Subcritical flow occurs when the weir is submerged, i.e. when $h_d \geq 2/3$ H (Figure

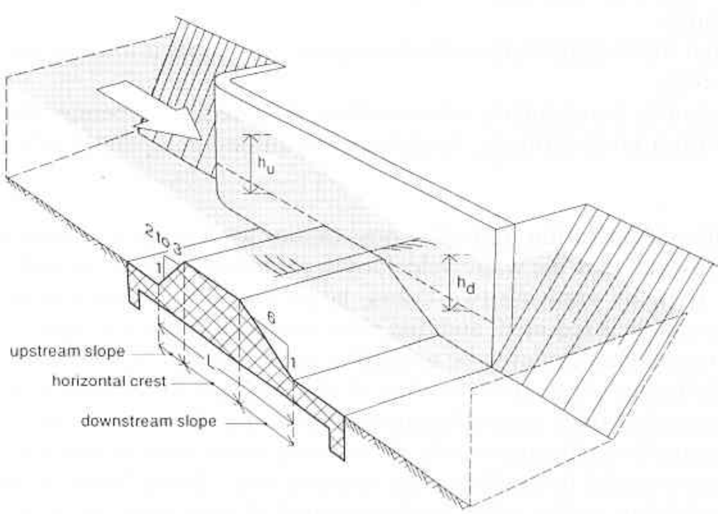

Figure 24.19 The broad-crested weir

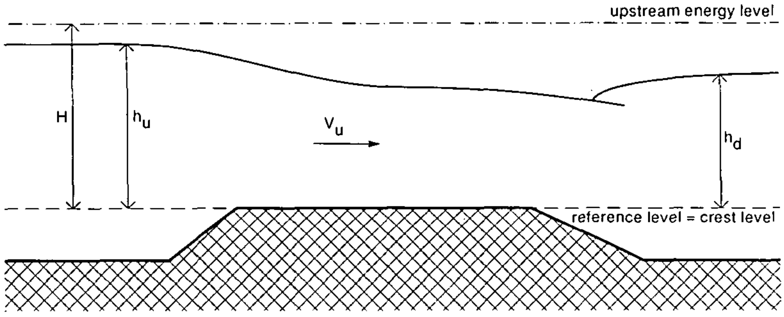

Figure 24.20 Subcritical flow over a weir

24.20). This means that the discharge depends on the upstream energy level and the downstream water level.

$$\text{for } h_d \geq \frac{2}{3} H: Q = \mu\, b\, h_d \sqrt{2\, g(H-h_d)}$$

When the upstream flow velocity remains low, the velocity head $v_u^2/2g$ is small and can be neglected. The upstream energy level H in the above equation can be replaced by the upstream water level h_u.

$$\text{for } h_d \geq \frac{2}{3} h_u: Q = \mu\, b\, h_d \sqrt{2\, g(h_u-h_d)} \tag{24.8}$$

where, in addition to the symbols already defined

- Q = discharge (m^3/s)
- b = width of the outlet (m)
- h_d = downstream water level (m)
- μ = discharge coefficient, which includes losses due to friction and contraction over the weir (–)

Thijsse (see De Vries et al. 1947-1951) gives some indicative values of μ, valid for subcritical flow conditions.

- μ = 1.3 for smooth surfaces, rounded crests, gentle downstream slope, small difference in head
- μ = 1.1 for average conditions
- μ = 0.9 for rough surfaces, sharp crests, steep downstream slope, large difference in head

Critical Flow

Flow is in a critical state when the inertial and gravitational forces are in equilibrium, which occurs when the Froude number equals 1. The Froude number can be determined by

$$Fr = \frac{v}{\sqrt{g\, R \cos s}} \tag{24.9}$$

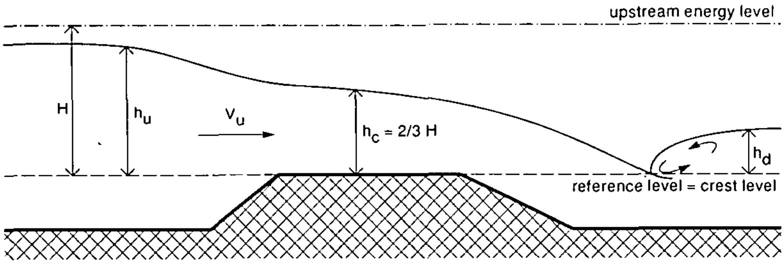

Figure 24.21 Critical flow over a weir

where

Fr = Froude number (–)
v = average flow velocity (m/s)
R = hydraulic radius, being the wetted area divided by the wet perimeter (m)
s = slope of the energy line (–)

For a constant upstream water level, maximum discharge occurs during critical flow, i.e. when the discharge is independent of the downstream water level (h_d < 2/3 H) (Figure 24.21)

$$\text{for } h_d < \frac{2}{3}H: \quad Q = C_d\, b\, \frac{2}{3}H \sqrt{2g\frac{1}{3}H}$$

or by neglecting the velocity head v_u

$$\text{for } h_d < \frac{2}{3}h_u: \quad Q = C_d\, b\, \frac{2}{3}h_u \sqrt{2g\frac{1}{3}h_u} \qquad (24.10)$$

where

C_d = discharge coefficient (–)

Note that under critical flow conditions, the water level above the crest (critical depth h_c) depends on the upstream energy level as

$$h_c = \frac{2}{3}H \approx \frac{2}{3}h_u \,(\mathrm{m})$$

The discharge coefficient C_d includes losses due to friction and contraction and depends on the shape of the weir and the upstream water level. The value of C_d can be determined as $C_d = 0.93 + 0.10\, H/L$ for $0.10 \leq H/L \leq 0.70$ (Bos 1989).

Gated Culverts

Gated culverts are applied when the outlet structure does not have a navigation function. The cross-section of a culvert can be circular, square, or rectangular. An advantage of a culvert is that the top of the embankment (inspection road) will remain undisturbed.

To prevent the outer water from entering the drained area, the structure is fitted with a gate, which can be operated either by hydraulic forces or manually. The operation principle is almost the same as for sluice doors, the main difference being that the gate usually rotates around a horizontal axis. In closed position (during high outer water levels), the door slants slightly outwards, which is preferred to the vertical position because it closes better (its own weight component helps keep it closed). In times of discharge, the gate will not open completely, thereby ensuring that it will close when the outer water level rises again. These gates are called 'flap gates' (Figure 24.22).

Compared with the drainage sluice, self-operating gated culverts will have extra head loss during discharge, because extra head is not only needed for the flow through the culvert and to open the door, but also to compensate for the weight of the gate. Nevertheless, by applying relatively light material, counterweights, and minimal friction in the hinges, flap gates with head losses of practically nil have been developed.

As manually and automatically operated gates can be fully removed from the flow, they will not disturb the flow, so that no extra head loss will occur with such gates.

To calculate the discharge through a culvert, basic formulae for culvert flows can be used (e.g. French 1986; Chow 1959; USBR 1983). For practical purposes, six types of culvert flow can be identified (Figure 24.23).

A culvert will have full flow when the downstream end is submerged (Type 1). If the downstream end is not submerged, the culvert will have full flow when the upstream water level is high (i.e. when $h_u > 1.5d$) and the culvert can be regarded as hydraulically long (Type 2). Whether a culvert is hydraulically long or short depends on factors such as the bottom slope, the ratio between the culvert length and height (L/d), the ratio between the entrance radius and the culvert height (r/d), and the water levels at both ends. When $h_u > 1.5d$ and the culvert is hydraulically short, the flow is of Type 3. When the upstream water level is less than 1.5d, the downstream water level may be higher than the critical depth of the flow (Type 4). For lower downstream water levels, flow will be of Type 5 when the slope of the culvert bottom is subcritical, and of Type 6 when the slope is supercritical.

For Flow Types 4, 5, and 6, the entrance of the culvert acts like a weir and the discharge coefficient varies approximately between 0.75 and 0.95 (Chow 1959).

Note that, in the profiles shown in Figure 24.23, contraction due to the valve at the downstream side of the culvert is ignored. This is allowed only for manually operated gates.

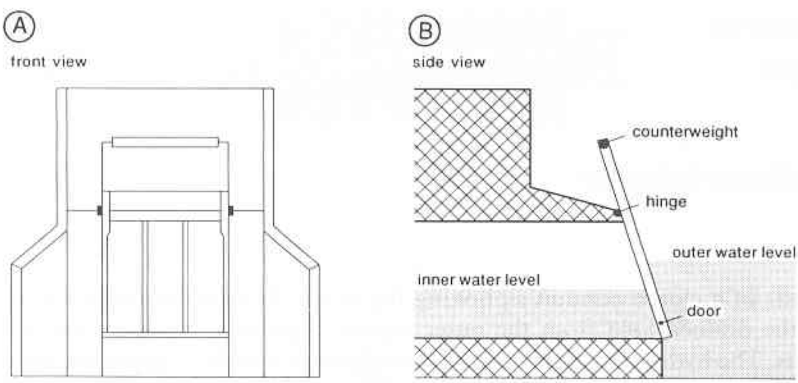

Figure 24.22 Flap gate: A: Front view; B: Side view

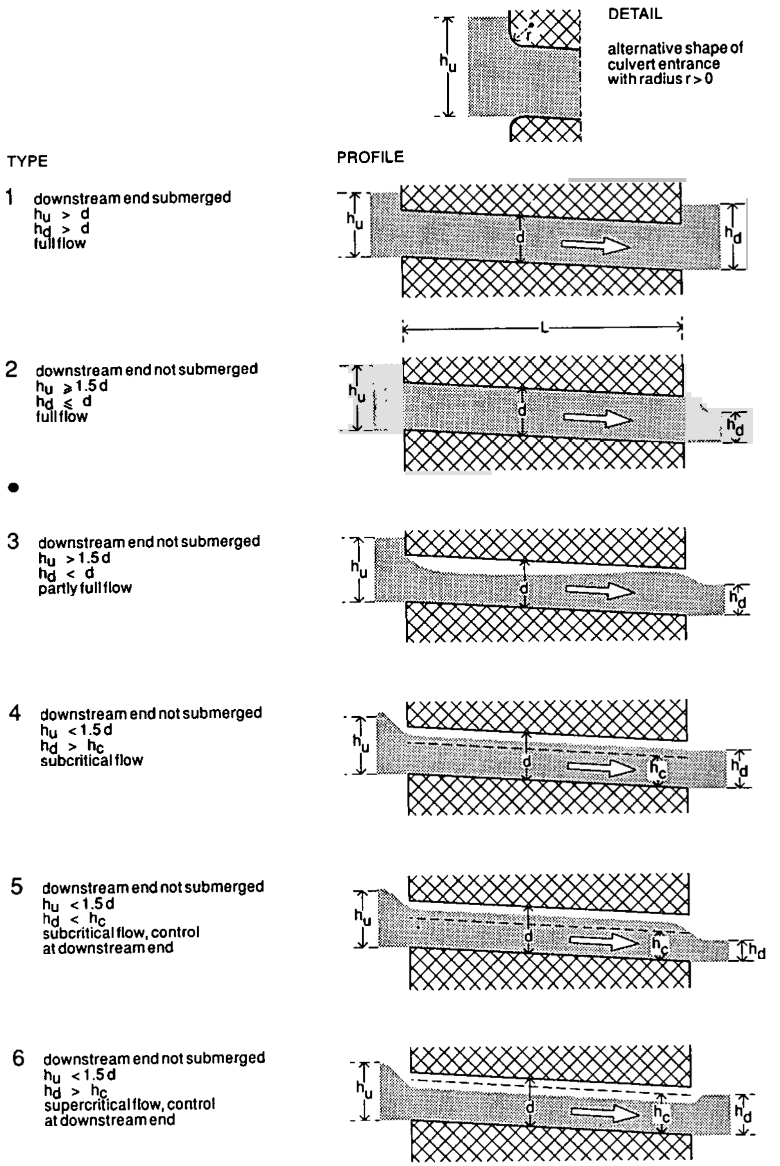

Figure 24.23 Classification of culvert flow

Siphons

A siphon consists of a closed conduit siphoning the water through an embankment that separates the drained area from the outer waters. Siphons are seldom applied as gravity outlets. The hydraulic calculation of flow through siphons is based on pipe-flow principles. Siphons will not be further discussed here. Reference is made to French (1986) and Chow (1964).

24.3.2 Location of Outlet Structures

The selection of the outlet location depends on hydrological considerations (tidal or non-tidal area), topographical considerations (lowest spot of the area to be drained), soil mechanical considerations (foundation possibilities), effects of wave attack, and sedimentation and scouring at the outer side of the outlet.

From a drainage point of view, the location of outlets in tidal areas is generally most favourable in those areas with the lowest low water levels. The site should preferably be selected near a natural gully, so that optimum use can be made of the natural drainage characteristics (Chapter 19). The outlet itself should not be constructed in the gully because this might pose foundation problems; instead, it should be placed just beside the gully and be connected to it by a short canal.

In a non-tidal river area, the outlet should be located at the lowest site in the drained area. Also in this case, use should be made of the existing natural drainage channels in the area. For drained areas with relatively long stretches along the river, it is advisable to apply two or even more outlets, because otherwise all the drainage water would have to be conveyed to the lowest part of the area before draining to the river, which would result in a relatively large storage area.

If the outer water levels cause a prolonged period of impeded drainage, additional measures (e.g. pumps) will be needed. Another alternative could be to construct a channel, parallel to the river, to a location further downstream. The slope of this channel should be less than the slope of the river bed, so that sufficient head can be obtained to allow gravity discharge.

The outlet structure should be protected from waves by an indent in the dike in a direction that depends on the predominant wave direction. Problems of sedimentation may then occur, however, for which additional measures are required (e.g. dredging and flushing; see Section 24.3.3).

If the outlet has to discharge to a meandering river or to rapidly changing tidal forelands, locations that might be subject to meandering and/or scouring should be avoided, because both processes may affect the proper functioning of the outlet.

24.3.3 Discharge Capacities of Tidal Drainage Outlets

To calculate the discharge capacity of tidal drainage outlets, one needs data on inner water levels (DDL and MASL), the volume of water to be drained (represented by the drainage coefficient), a representative tidal curve of the outer water, hydraulic characteristics of the planned structure and of the foreshore channel, and the characteristics of the storage area.

The actual inner water levels depend on the incoming water, the volume that can be stored, and the volume that can be discharged during one tidal cycle. The incoming water is represented by the drainage coefficient, being a desired depth of excess water during a certain maximum period of time. Its background has already been discussed in Chapter 17. The volume of water that can be stored in the drainage area is the product of the total wetted surface area (area of canals and of storage basins) and the maximum allowable rise (MASL-DDL). To determine the storage capacity of a drained area, sloping sides of canals and/or storage reservoirs should be averaged between DDL and MASL, and the average area should be multiplied by the maximum allowable rise (MASL-DDL).

The stored water will have to be evacuated within the drainage period. The length of this period depends on the outer water levels, which are governed by the tidal fluctuation, in combination with river discharges.

From Figure 24.2, it could be observed that discharge starts when the water levels on both sides of the outlet are (more or less) equal. As was mentioned earlier, the design should take into account the head loss over the outlet structure and the higher density of outside waters. Discharge stops when the outside water level becomes higher than the level inside.

The volume to be evacuated through the outlet can be obtained by balancing the drainage volume with the available storage and keeping in mind that storage can be used only temporarily.

Computation of Outlet Width and Storage Capacity

To compute the outlet width and the corresponding required storage capacity in the drained area, the following procedure can be followed:

A. Outlet Width

Step 1: Select a design tidal range, which should reflect the most unfavourable outer water conditions for drainage. (In most cases, it equals the minimum tidal range.) The data should preferably cover a period of at least one lunar month, so that spring and neap tides are included.

Step 2: Determine the length of drainage periods T_D during the selected tidal cycle on the basis of the design drainage level and the maximum allowable storage level.

Step 3: Subdivide each drainage period into small periods Δt during which the conditions can be considered constant, so that the steady-state formulae for subcritical and critical flow conditions can be applied. The value of Δt can best be chosen in the range of 1500 to 3000 seconds.

Step 4: Choose a crest elevation and take it as reference level (see Section 24.3.4 for remarks on the best choice).

Step 5: For each time interval Δt, determine the flow situation and the corresponding values of h_u and h_d (Figure 24.24).

Water starts flowing through the drainage outlet when the water level inside exceeds the water level outside (provided that inside and outside waters have equal density). There will be subcritical flow as long as $h_d \geqslant 2/3\ h_u$. By expressing Equation 24.8 per unit sluice width, we obtain

$$v_{\Delta t} = q_s \Delta t = \mu\, h_d \Delta t \sqrt{2\,g\,(h_u - h_d)} \qquad (24.11)$$

where

$v_{\Delta t}$ = drained volume per metre sluice width during time step Δt (m^2)

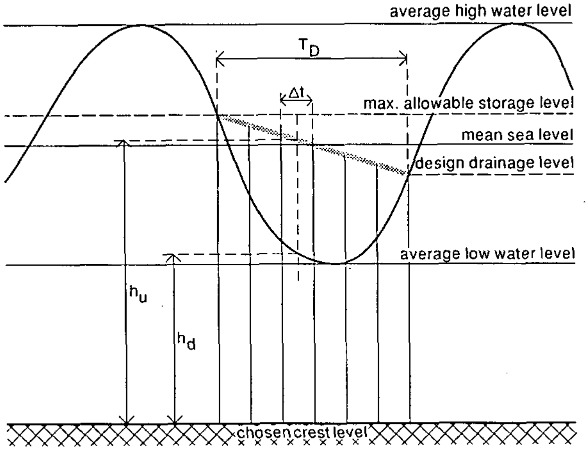

Figure 24.24 Determination of the water levels inside ($_u$) and outside ($_d$) at the middle of each time step during subcritical flow conditions

q_s = average drainage discharge per metre sluice width during time step Δt under subcritical flow conditions (m^2/s)
Δt = time step (s)
h_u = the average upstream water level, measured at the middle of time step Δt (m)
h_d = the average downstream water level, measured at the middle of time step Δt (m)

Select a value for the discharge coefficient μ.

Critical flow starts as soon as $h_d < 2/3\ h_u$ (Point A in Figure 24.25) and continues until the downstream water level starts to rise and rèaches the value $h_d = 2/3\ h_u$ again (Point B in Figure 24.25). During the period AB, the discharge over the weir is controlled by the critical depth of flow above the weir; $h_c = 2/3\ h_u$. Note that the values of h_u for these time steps can be determined by linear interpolation between A and B.

As long as critical flow conditions occur, $v_{\Delta t}$ can be calculated with Equation 24.10

$$v_{\Delta t} = q_c \Delta t = C_d \frac{2}{3} h_u \Delta t \sqrt{2g\frac{1}{3} h_u} \tag{24.12}$$

where

q_c = average drainage discharge per metre sluice width during time step Δt under critical flow conditions ($m^3/m.s$)

Select a value for the discharge coefficient C_d.

Step 6: Calculate the volume of drainage water that collects during a tidal day using

$$V = q\, T\, A_D \tag{24.13}$$

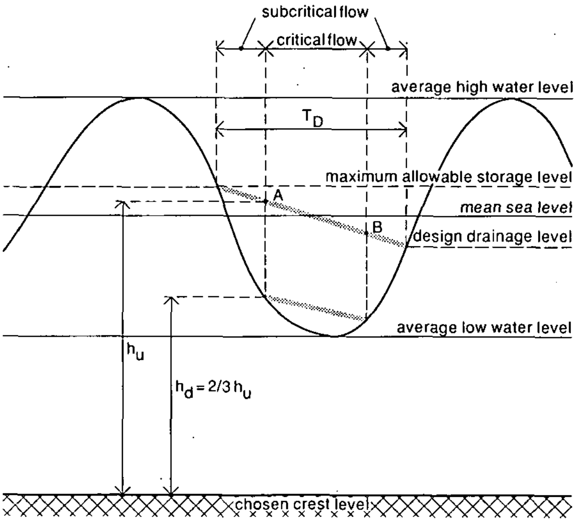

Figure 24.25 Determination of the duration of critical flow conditions

where

V = the total volume of water to be discharged during the drainage period (m^3)
q = the drainage coefficient (m/s)
T = length of tidal period (s)
A_D = the drainage area (m^2)

Note: Normally the drainage coefficient is expressed in mm/d. To convert it to m/s, multiply the coefficient by $10^{-3}/(24 \times 3600)$.

Step 7: Calculate the design outlet width using

$$b = \frac{V}{\Sigma v_{\Delta t}} \qquad (24.14)$$

where $\Sigma v_{\Delta t}$ is the total potential discharge volume per unit outlet width during the tidal period considered.

B. Storage Capacity
During the storage periods T_S, the drainage water should be stored inside the drained area. The following procedure can be applied to determine the required storage capacity:

Step 8: Select from the design tidal range the longest period T_S during which no drainage is possible.

Step 9: Determine the drainage discharge Q by using

$$Q = q\,A_D \qquad (24.15)$$

Step 10: Calculate the volume of water V_S that needs to be stored during the period T_S by multiplying the discharge with the period selected in Step 8 using

$$V_S = Q\,T_S \tag{24.16}$$

Step 11: Calculate the average storage area A_S, i.e. the average of the storage areas at maximum allowable storage level (MASL) and design drainage level (DDL), using

$$A_s = \frac{V_s}{\text{MASL-DDL}} \tag{24.17}$$

Step 12: If the calculation has been made for *average* tidal conditions, neglecting spring and neap tides, it is advisable to create a storage area of at least 1.5 times the dimensions calculated under Step 11.

Example 24.3

Given (Figure 24.26):

Average highest water level	AHWL	=	4.38 m
Average lowest water level	ALWL	=	2.62 m
Maximum allowable storage level	MASL	=	3.90 m
Desired drainage level	DDL	=	3.60 m
Drainage coefficient	q	=	50 mm/day
Drainage area	A_D	=	5000 ha
Discharge coefficients	μ	=	1.2
	C_d	=	0.9
Acceleration of gravity	g	=	9.81 m/s^2

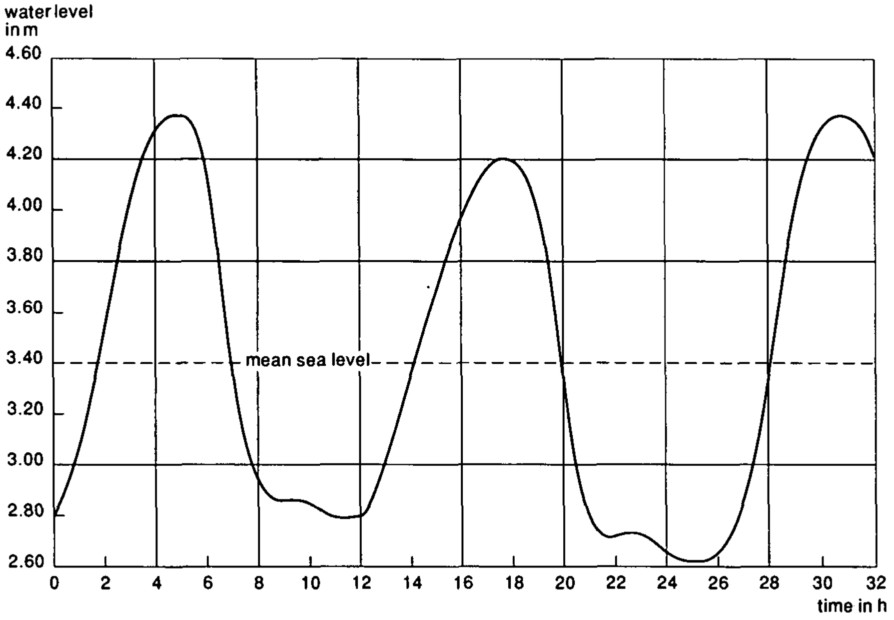

Figure 24.26 Average tidal levels used in Example 24.3

Asked:

A. Design width of the outlet structure;

B. Required storage area;

C. A check on the computations by a simple reservoir calculation.

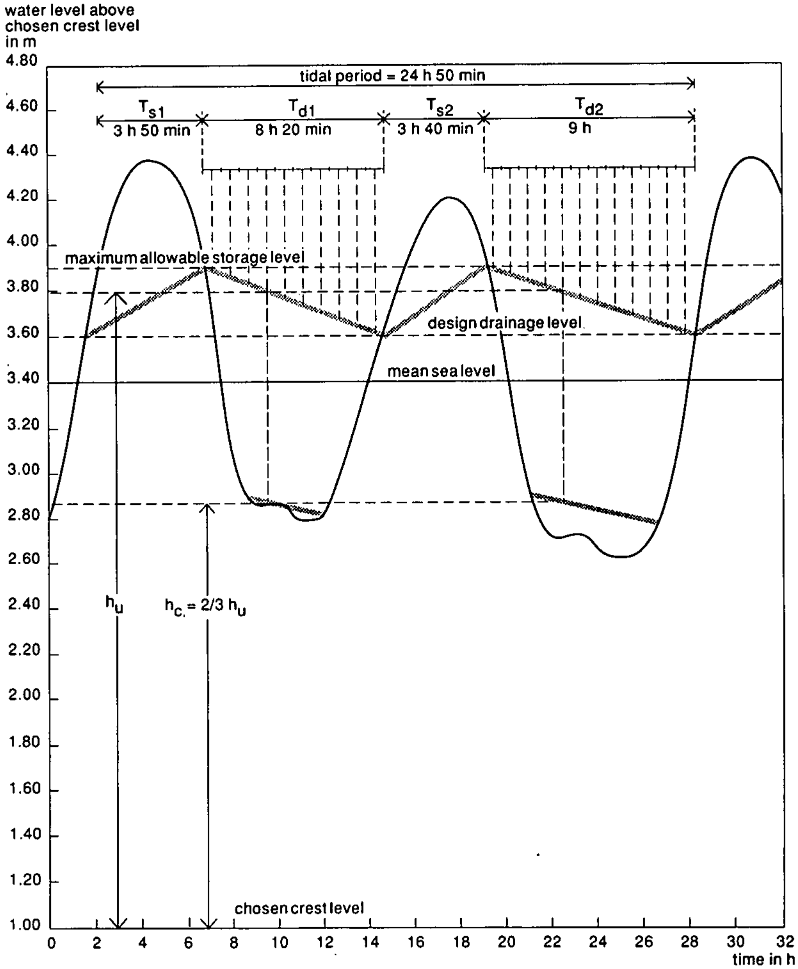

Figure 24.27 Determination of drainage and storage periods in Example 24.3

A. Design Width of the Drainage Outlet

Step 1: Representative tidal range: see Figure 24.26.

Step 2: Determine graphically the length of the drainage periods T_{D1} and T_{D2}. Using Figure 24.27, we find T_{D1} = 30 000 and T_{D2} = 32 400 s.

Step 3: Select appropriate time steps Δt for each drainage period, for example: Δt_1 = 3000 s, and Δt_2 = 2700 s, so that 10 and 12 discrete steps, respectively, can be obtained.

Step 4: In this calculation, the crest elevation is chosen at 1.00 m above datum level.

Step 5: For each time step Δt, read the values of h_u and h_d from the graph of Figure 24.27. Check which flow conditions exist during the drainage periods T_{D1} and T_{D2} (Tables 24.2, 24.3: Column 4). Calculate for each time step Δt the corresponding $v_{\Delta t}$, using the equation for subcritical or for critical flow (Equation 24.11 or 24.12).

Step 6: Using Equation 24.13, determine the total volume of water V that has to be drained during one tidal day

$$V = q\, T_D\, A_D = \frac{24 \times 60 + 50}{24 \times 60} \times 50 \times 10^{-3} \times 5000 \times 10^4 = 2.59 \times 10^6\, m^3$$

Table 24.2 Discharges during drainage period T_{D1}

Time step	h_u	h_d	2/3 h_u	C_d or μ	Δt	$v_{\Delta t}$	Critical flow
1	2.89	2.70	1.93	1.20	3000	18767	No
2	2.86	2.18	1.91	1.20	3000	28666	No
3	2.83	1.92	1.89	1.20	3000	29206	No
4	2.80	1.88	1.87	1.20	3000	28754	No
5	2.77	1.88	1.85	1.20	3000	28282	No
6	2.74	1.80	1.83	0.90	3000	20878	YES! h_d < 2/3 h_u See Figure 24.27
7	2.71	1.81	1.81	1.20	3000	27381	No
8	2.68	1.95	1.79	1.20	3000	26567	No
9	2.65	2.20	1.77	1.20	3000	23533	No
10	2.62	2.48	1.75	1.20	3000	14797	No
					$\Sigma v_{\Delta t}$ =	246831	

Table 24.3 Discharges during drainage period T_{D2}

Time step	h_u	h_d	2/3 h_u	C_d or μ	Δt	$v_{\Delta t}$	Critical flow
1	2.89	2.81	1.93	1.20	2700	11406	No
2	2.87	2.37	1.91	1.20	2700	23970	No
3	2.84	1.95	1.90	1.20	2700	26451	No
4	2.82	1.75	1.88	0.90	2700	19619	Yes
5	2.80	1.72	1.86	0.90	2700	19376	Yes
6	2.77	1.74	1.85	0.90	2700	19134	Yes
7	2.75	1.67	1.83	0.90	2700	18893	Yes
8	2.73	1.63	1.82	0.90	2700	18653	Yes
9	2.70	1.64	1.80	0.90	2700	18414	Yes
10	2.68	1.71	1.79	0.90	2700	18176	Yes
11	2.65	1.91	1.77	1.20	2700	23580	No
12	2.61	2.30	1.74	1.20	2700	18378	No
					$\Sigma v_{\Delta t}$ =	236051	

Step 7: Calculate the width of the outlet using Equation 24:14

$$b = \frac{V}{\Sigma v_{\Delta t}} = \frac{2.59 \times 10^6}{(2.46 + 2.36) \times 10^5} = 5.36\,\mathrm{m}$$

Thus an outlet with two gates of 3 m each could be sufficient.
Note: It is always preferred to apply an outlet with more than one opening for considerations of maintenance and repair.

B. Required Storage Capacity
Step 8: During periods T_{S1} and T_{S2}, water has to be stored because during those periods no drainage is possible. T_{S1} has the longest duration: 3h50min.

Step 9: Determine the drainage discharge Q, by using Equation 24.15

$$Q = q \times A_D = \frac{50 \times 10^{-3}}{24 \times 3600} \times 5000 \times 10^4 = 28.9\,\mathrm{m^3/s}$$

Step 10: Using Equation 24.16, calculate the volume of water V_S that needs to be stored during the period T_S (being the longest period)

$$V_s = Q\,T_{s1} = 28.9 \times (3 \times 3600 + 50 \times 60) = 0.40 \times 10^6\,\mathrm{m^3}$$

Table 24.4 Reservoir analysis for the storage area. The required storage (Column 7) equals the accumulated inflow (Column 4) minus the accumulated outflow (Column 6)

1	2	3	4	5	6	7
Period	Duration (s)	Inflow (Step 9) ($m^3 \times 10^6$)	Cumulative inflow ($m^3 \times 10^6$)	Outflow (Step 5, 7) ($m^3 \times 10^6$)	Cumulative outflow ($m^3 \times 10^6$)	Required storage ($m^3 \times 10^6$)
T_{S1}	13800	0.40	0.40	0.00	0.00	0.40
T_{D1}	30000	0.87	1.27	1.32	1.32	-0.05*
T_{S2}	13200	0.38	1.65	0.00	1.32	0.34
T_{D2}	32400	0.94	2.59	1.27	2.59	0.00

* A negative value means that no storage is required

Step 11: Using Equation 24.17, calculate the average storage area A_S

$$A_s = \frac{V_s}{\text{MASL - DDL}} = \frac{0.40 \times 10^6}{3.90 - 3.60} = 1.33 \times 10^6\,\text{m}^2 = 133\,\text{ha}$$

Step 12: Because this calculation has been made for average tidal conditions, the advisable storage area should be 133 × 1.5 = 200 ha.

C. Checking the Calculations

We can check the calculations by making a simple reservoir analysis for the drained area as is presented in Table 24.4.

It appears from Table 24.4 that, at the end of a tidal day, the volume of inflow is the same as that of the outflow, and that therefore, as was mentioned, the minimum storage volume should be $0.40 \times 10^6\,\text{m}^3$.

Remarks on the Hydraulic Computation

The hydraulic computation method that has been presented can be used as a *first approximation* only, because the real situation has been simplified.

The first simplification was the value of the discharge coefficient μ. There are no proper formulae from which μ can be derived satisfactorily. Therefore, for large outlet structures, the discharge coefficient needs to be investigated by scale models or by simulation; for smaller structures reference is made to Bos (1989).

Generally, the more open the gates, the less the contraction will be. If the gates could open completely and no part of them were to protrude significantly, the discharge coefficient could have values of 1 (which means no contraction at the gates) or, under favourable conditions, of even more than 1.

Other aspects to be mentioned are hydraulic losses at the transitions between channel and structure and losses due to friction along the sides of the structure. Table 24.5 gives a first indication of the values for the coefficient which takes these losses into account.

Table 24.5 Head loss coefficients for hydraulic losses at the upstream and downstream transitions with the channel and for friction losses (Van der Kley and Zuidweg 1969)

Shape of side walls and crest	c_d
Crest elevation at channel bottom:	
- Sharp-cornered side walls	0.80
- Rounded cornered side walls	0.90
Crest elevation above channel bottom:	
- Sharp-cornered crest and side walls	0.72
- Rounded side walls:	
= Sharp-cornered crest	0.76
= Rounded crest	0.85

Other simplifications used were:
- A constant inflow rate equal to the drainage coefficient of the drained area;
- An average tidal curve for one tidal day.

Reality is of course different and more complex. In principle, it is possible to simulate the real situation rather satisfactorily by using hydraulic computer models in which the system 'inner water levels, storage, outlet structure, and outer water levels' can be schematized in one network as follows:
- Fluctuating inner water levels can be simulated on the basis of a design rainfall or a series of measured rainfall data;
- Fluctuating outer water levels can be simulated on the basis of the most important constituents that influence daily water levels (tidal area), on seasonal river water levels (non-tidal area), or on a combination of the two (tidal rivers);
- Various areas of the storage reservoir can be included in the network;
- The flow through the outlet as a result of the above-mentioned fluctuating water levels. Several outlet characteristics (including varying contraction coefficients) could be used as input.

In this way, we can simulate the functioning of an outlet for a relatively longer period (e.g. a month to include spring and neap tides, or a season to include extreme river flows), and to test the sensitivity of some parameters to obtain an insight into the design conditions. An example of such a simulation model is 'Drainage: Tidal Sluice Simulation' (Standa Vanacek 1990), which demonstrates the hydraulic functioning of the system 'drainage sluice, storage area, tidal outer water levels'. The model shows the sensitivity of design parameters (like storage area, crest width, and crest level) and the effect of numerical parameters (implicit versus explicit numerical solution methods, different time steps) on the design of a tidal drainage sluice.

A third element to be discussed is that the inner and outer waters have different densities (Figure 24.28). The doors of an outlet will remain closed as long as the forces acting upon them are in equilibrium.

Thus for equilibrium

$$\frac{1}{2}\rho_f g h_f^2 = \frac{1}{2}\rho_s g h_s^2 \qquad (24.18)$$

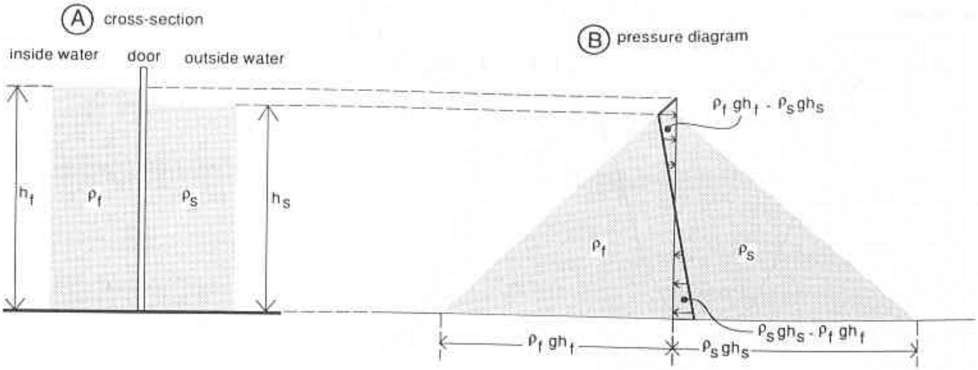

Figure 24.28 Hydrostatic pressure on a vertical interface between water bodies of different mass density; A: Cross section; B: Pressure diagram, showing the resultant pressure

where the subscripts f and s denote, respectively, fresh and salt water.

From Equation 24.18, it follows that

$$\frac{h_f}{h_s} = \sqrt{\frac{\rho_s}{\rho_f}} \tag{24.19}$$

For sea water ($\rho_s = 1025$ kg/m^3), the ratio $h_f : h_s$ becomes 1.012:1.
So, generally, if the outer water has a higher density (e.g. salt water, river water containing high sediment rates), a higher inner water level is needed to open the doors. Consequently, the doors will close earlier than when the densities on both sides are equal. Figure 24.29 shows the inner and outer water levels and the corresponding discharge curve of a drainage outlet in a typical tidal environment. Figure 24.29 shows that, because of the density differences, the actual drainage period will be shorter than that used in Example 24.3.

24.3.4 Design, Construction, Operation, and Maintenance

Crest Level

The crest level of a tidal outlet can best be chosen at a depth varying between 0.5 and 2 m below the outer low water level. As a low crest level reduces the period in which critical flow takes place and thus increases the capacity of the outlet, it is preferred to lower the crest level rather than to increase the outlet width (keeping the wetted area the same). However, the construction costs may increase significantly with lower crest levels. The stability of the side walls might be another restricting factor if the height of the side walls becomes large in relation to the outlet width.

The bottom level of the drainage canal that leads to the outlet, and that of the canal from the outlet to the receiving waters, govern the crest elevation as well. In case of highly elevated foreshores, for instance, or outer areas subject to sedimentation and siltation, it is not possible to maintain a relatively deep outer drainage canal.

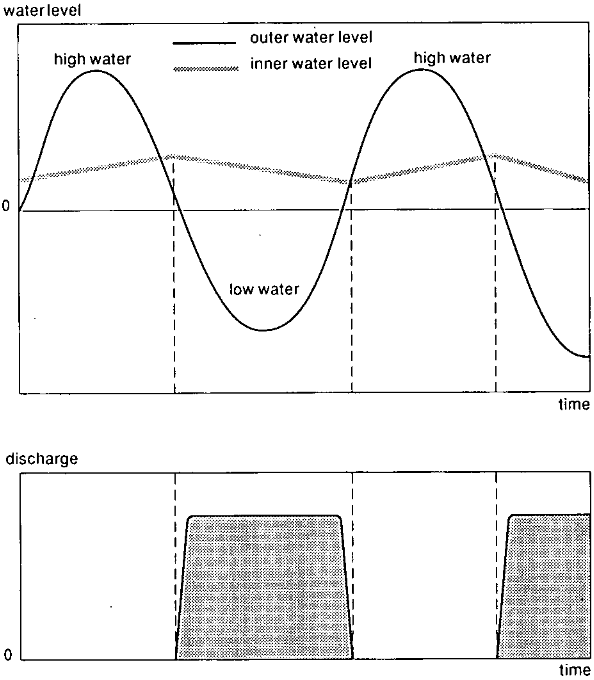

Figure 24.29 Water level fluctuations and discharge of an outlet structure under tidal conditions

The best solution then is to choose the crest level at the elevation of the foreshore. Relatively high crest elevations also occur in drained areas with a relatively high ground level and a corresponding high DDL. In such cases, the crest elevation might be chosen at, or even above, low water level.

When outlets are located upstream of the tidal reach of a river, the crest level must be chosen at 1.50 to 1.75 m below DDL (provided, of course, that the river water levels allow free drainage).

Doors and Gates

In tidal areas, either doors (with a vertical or horizontal axis) or sliding gates can be applied. In tidal areas, doors have the advantage of opening and closing by hydraulic forces only. Waves, however, may cause a repeated opening and closing, which not only allows outer water to intrude, but may also damage the doors and hinges. The gates should also be able to withstand the wave forces and should be able to pass these forces to their hinges.

The prevention of salt water intrusion via doors and gates can be realized by applying rubber profiles near the hinges and at the ends where they touch each other (or touch the side walls in case of only one door).

In non-tidal areas, sliding gates can better be applied, as they can still be opened when large differences between inner and outer water levels occur. Furthermore, it is possible to maintain higher inner water levels (when desired), which is not possible with outlet doors that open towards the outer water levels.

To prevent a hampered operation of the outlet and to protect the doors and gates from damage by floating debris, trash-racks should be applied at the inner side to collect the debris.

Doors and gates can be maintained and repaired by closing the outlet temporarily with stoplogs, for which slots in the sidewalls are required in which the logs can slide. These slots should be provided at both sides of the gate to cope with varying inner and outer water levels.

In case of a tidal outlet, a second set of doors might be constructed, in order to ensure extra safety of the drained area against high outer waters.

The height of the doors should, of course, be at the same elevation as the dike, to prevent flooding during extremely high outer water levels.

For gravity outlets that consist of more than one opening, it is advisable to have the same dimensions for all openings. This will allow a standard design for the doors/gates and for other mechanical items, and will make them exchangeable.

24.3.5 Other Aspects

As the tidal outlet is part of the protection system of an area, it should be constructed in such a way that it does not weaken this defence. This implies that seepage under the structure should be prevented, which can be realized by applying sheet piles. In case of relatively high outer water levels, it may even be necessary to apply sheet piles not only underneath the structure, but also next to both side walls.

High velocities through outlets should be avoided to prevent scouring and damage to banks and the structure itself (Chapter 19). This can be achieved by applying larger cross-sections for outlets and channels (resulting in lower velocities) and/or by lining the channel banks and protecting the outlet channel. On the other hand, sedimentation in the canals should be prevented by flushing the canals, for which certain minimum velocities are required.

Besides the problem of the intrusion of poor quality outer water, the quality of the drainage water is of increasing importance. As long as the quantity of polluted drainage water is small compared to the outer water, and the characteristics of the pollution allow for natural breakdown, no special measures need be taken. In areas where water of good quality is of importance, however, special measures might be needed, such as:

- Removing/diminishing the source of contamination;
- Restricting drainage from pollutive sources like industry, intensive agriculture;
- Purifying polluted drainage water before discharging it to outer waters, which is hardly feasible for drainage water that is polluted by agricultural practices;
- Discharging the drainage water in smaller quantities. This measure will not only require more storage, but is also related to the acceptability of storing polluted water in the drainage system;
- Discharging further downstream or at various locations.

(Chapter 25 elaborates on water quality in further detail.)

When the outer channel is subject to sedimentation and siltation (tidal foreshores, rivers with high sediment loads), regular flushing and/or dredging might be required. In tidal areas, flushing can be realized by constructing the required storage reservoir

Figure 24.30 Plan view of a storage reservoir, which is also used to flush the outer channel

next to the outlet structure (the outer doors) on the inland side, followed by a second set of doors (inner doors), which separate the reservoir from the drained area (Figure 24.30).

During normal operation, the outer doors open and close to allow for drainage, while the inner doors are kept open constantly. When flushing of the outer channel is required, the outer doors are kept open and the inner doors will close when the outer water levels become higher than the inner water levels. This will cause the water level in the storage area to rise to high water level. During the following drainage period, much more hydraulic head will be available, so that the outer channel can be flushed successfully. This, however, requires embankments encircling the storage area to offer protection against high outer water levels (Van der Kley and Zuidweg 1969; Smedema and Rycroft 1983).

References

Bos M.G. (ed.) 1989. Discharge measurement structures. ILRI Publication 20, Wageningen, 401 p.

Chow V.T. 1959. Open-channel hydraulics. McGraw-Hill, New York, 680 p.

Chow V.T. 1964. Handbook of applied hydrology. McGraw-Hill, London, 1418 p.

De Vries, J.E., H. Lammers and A.P. Potma 1947-1951. De Technische Vraagbaak : Handboek in vijf delen voor de gehele techniek. 5th ed. Volume W, Weg- en waterbouwkunde, Deventer.

French, R.H. 1986. Open channel hydraulics. McGraw-Hill, New York, 705 p.

Kalkwijk, J.P.Th. 1984. Tides. International Institute for Hydraulic and Environmental Engineering, Delft.

Pugh, D.T. 1987. Tides, surges, and mean sea level. Wiley, New York, 472 p.

Savenije, H. 1992. Rapid assessment technique for salinity intrusion in alluvial estuaries. International Institute for Hydraulic and Environmental Engineering, Delft.

Schureman, P. 1958. Manual of harmonic analysis and prediction of tides. Coast and Geodetic Survey, U.S. Department of Commerce, Special Publication No. 98, Washington, 317 p.

Smedema, L.K. and D.W. Rycroft 1983. Land drainage : planning and design of agricultural drainage systems. Batsford, London, 376 p.

Standa Vanecek 1990. Drainage : tidal sluice simulation. International Institute for Hydraulic and Environmental Engineering, Delft.

U.S. Bureau of Reclamation 1983. Design of small dams. U.S. Bureau of Reclamation, Department of the Interior, Washington, 816 p.

Van der Kley, J. and H.J. Zuidweg 1969. Polders en dijken. Agon Elsevier, Amsterdam, 342 p.

Van Os, A.G. and G. Abraham 1990. Density currents and salt intrusion. International Institute for Hydraulic and Environmental Engineering, Delft.