The Chapter 8 Measuring- and Regulating Structures

To be able to distribute the water in an irrigation system, control of water levels and regulating and measuring of flows are required. In channels flow rates can be determined by measuring the water velocity and the wet area which is related to the water level. Further are there dedicated structures to measure and regulate flow rates, mainly based on the principle of the Bernouilli equation. Shortly the hydraulic losses occurring with hydraulic structures are discussed.

The main types of measurement and/or regulating structures are discussed, namely the short- and broad crested weir, and the orifice. Finally various regulating structures commonly used in irrigation practice are dealt with.

What you should know after studying this chapter

1 Flow measurements in channels, flow –velocity-area-water level relation. Velocity measurements with floats, propellers and acoustic equipment. Measurement of water depth: needle, staff gauge, float, pressure.
2 Hydraulic principles of structures, Bernouilli equation, energy depth, maximum possible flow, critical depth, sub- and super critical flow, straight and curved flow lines, hydro-static pressure, energy losses in an open- and closed conduit, entry losses, exit losses, friction losses, losses with a sudden enlargement
3 Broad- and short crested weir, orifice, free flow, submerged or drowned flow, perfect and imperfect flow
4 Measuring structures: normal weir, Thomson weir, Cipoletti weir
5 Off-take structures (regulating and measurement structures): Romijn movable weir, flat gate with measurement structure, Metergate, Crump-de-Gruyter gate, Constant Head Orifice, Neyrtec Module distributor
6 Level regulators: Fixed weir, movable weir, sliding gate, siphon, automatic level controllers (up- and downstream controlled.

8 Measuring- and Regulating Structures

8.1 Introduction

8.1.1 General

To guarantee a good water distribution within an irrigation system is measuring and controlling the required flow and water levels, for which measuring and regulating structures are used.
Also with drainage control is required, for instance to delay low discharges (measures to prevent drying out) or to temporary store or divert high discharges.

Various measuring and regulating structures exist, from very simple weir distribution structures in which everything is fixed and the water proportionally is distributed according to the width of the weirs, to advanced electric-mechanical mechanisms remotely regulated by computers. In principle for every structure the passing flow should be determined. The flow rate is defined as:

$$Q = v \cdot A \tag{8.1}$$

With:

Q	=	flow rate	[m^3/s]
v	=	velocity	[m/s]
A	=	wet area	[m^2]

The most manifest method to measure a flow rate is to measure the velocity (v) and the wet area (A) separately.

8.1.2 Measuring flow rates in conduits

The velocity can be measured using the following methods:

- Floats: Although this method appears to be old fashioned and impractical the method can be useful to get a quick indication of the velocities in the channels.
- Propellers: Propeller measurements are accurate. Mostly they are used for river flows. For permanent measurement they are not suitable because of their vulnerability.
- Acoustic: Acoustic measurements are used as standard for measuring flows in pipelines and open channels (see figure 8.1).

In open channels the wet area is found by measuring the water level, when the relation between water level and wet area is known (h – A graph). For this the section of the channel has to be measured. There are various ways to measure the water level.

- The most accurate way is to measure the level with a measuring rod although this is not practical for irrigation.
- Mostly the water level is measured with a staff gauge. This can be done electronically as well as visually. The advantage of visual measurement at location is that also the users observe how much water is let in.

- A variant is the use of a float mostly placed in a separate stilling well to filter out disturbing high frequencies. In the Netherlands often floats are used to measure levels.
- Other possibilities are the use of a pressure sensor, placed at a fixed elevation in the channel and measures the water pressure, which simply can be converted to a water level. Alternatively air-bubbles are released at a fixed level in the channel and the required release pressure is measured.

The advantage of flow rate measurement by measuring the velocity and water level in a channel section is that no energy loss occurs. This type of flow measurements is not yet common in water management but is evolving. Nearly all flow measurements are carried out with gauging structures.

Measuring principle

The measuring principle of FLOW-2000 is based on the absolute difference in run-time method; a method with which it is possible to separately check each individual measurement. Herewith sound pulses are sent by transducers from A to B and vice versa, under an angle through the river (see fig. 1).

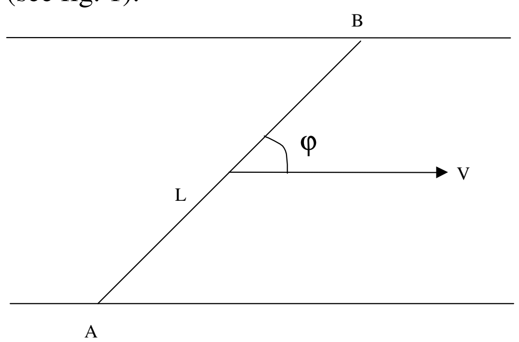


Figure 1

In the case of no current the running time from A to B (t_{AB}) is equal to that of B to A (t_{BA}):

$$t_{AB} = t_{BA} = \tfrac{L}{C}$$

in which:
L is the length of the acoustic measurement line
C is the celerity of sound in water (= 1400 – 1550 m/s)

In case of a flow as sketched in figure 1 the sound pulse from A to B is accelerated and that from B to A is delayed according to:

$$t_{AB} = \frac{L}{C + V\cos\varphi} \quad \text{and} \quad t_{BA} = \frac{L}{C - V\cos\varphi}$$

in which:
V is the velocity in the direction of the axis of the river
ν is the angle between this axis and the measuring line.

V can be deduced from the difference of the running times.

To eliminate the influence of the cross flow on the result of the measurement, perpendicular on the first measuring line a second measuring line is placed together forming a measuring cross.

Because the sound pulses of the FLOW-2000 are sent through the same water contemporary the celerity of sound is for both the same. Therefore with the FLOW-2000 it has become possible to measure at these locations where large variations in speed of sound occur; for example nearby large power plants or in the wake of ships.

Figure 8.1 Principle of measurement of the acoustical measuring instrument Flow-2000

8.2 Hydraulic principles of structures

To understand flow phenomena at the location of structures some knowledge of hydraulics is required. The much used principles are summarised below (see lectures Fluid Mechanics).

8.2.1 Bernoulli's law

Bernoulli's law is the most important law for the computation of measuring and regulating structures. In fact Bernoulli states that the amount of energy (sum of the kinetic- and potential energy) in flowing water is constant when no energy losses occur. With hydro-static pressure distribution the amount of energy of flowing water can be written as:

$$E = \rho \,.\, g \,.H = \rho \,.\, g \,.\, h + ½ \,.\, \rho \,.\, v^2 + \rho \,.\, g \,.\, z \qquad (8.2)$$

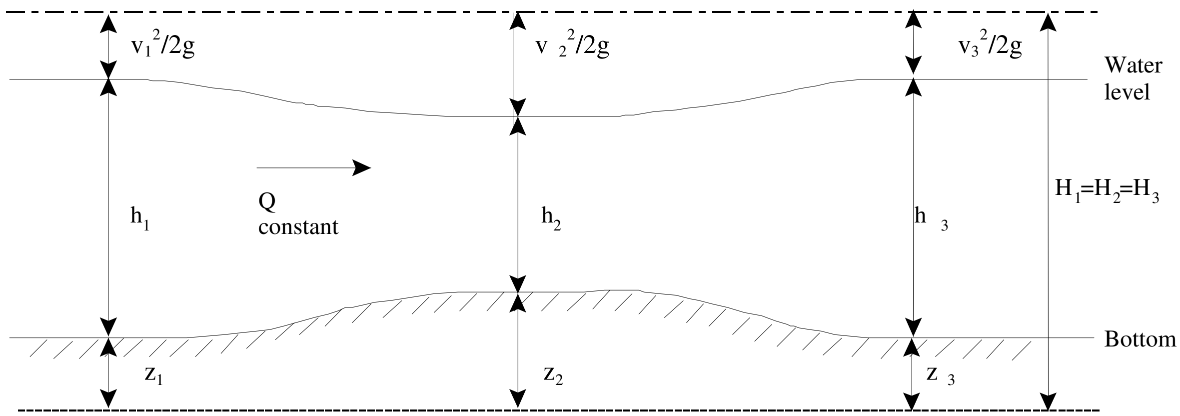
Or:

$$H = h + z + \frac{v^2}{2.g} \qquad (8.3)$$

With:

H	=	energy depth	[m]
h	=	water depth	[m]
z	=	bed level	[m]
v	=	velocity	[m/s]
g	=	acceleration of gravity	[m/s^2]
Δ	=	specific mass	[kg/m^3]

The last notation often is used, with H representing the energy depth. When written in the former manner it clearly can be noted that the amount of energy of a flow consists of a potential fraction and a kinetic fraction.



N.B. - friction is ignored.
- in the sections the flow lines are straight, so there is hydro-static pressure

Figure 8.2 Potential and kinetic energy

8.2.2 Maximum flow with a given energy depth

With a certain ratio between potential and kinetic energy the flow is maximal. This can be recognised as follows. When there is potential energy only, the flow is zero, the water is stagnant. When there is kinetic energy only, then the flow rate is zero as well, because the water depth has to be nil; and consequently the wet section is zero. The maximum flow rate lies between these extremes.

The maximum flow rate with a given energy depth H can be found by writing the flow rate per unit width (=q [m^3/m.s]) as function of the water depth h. The maximum flow rate is found to set the differential of q to h equal to zero. With the relation q = v.h:

$$H = h + \frac{v^2}{2.g} = h + \frac{q^2}{2.g.h^2} \tag{8.4}$$

This can be written to q^2:

$$q^2 = 2 \,.\, g \,.\, h^2 \,.\, H - 2 \,.\, g \,.\, h^3 \tag{8.5}$$

So q^2 = f(h). For which h is q^2 maximal?

$$\frac{\delta q^2}{\delta h} = 4 \,.\, g \,.\, h \,.\, H - 6 \,.\, g \,.\, h^2 = 0 \;\Rightarrow h_c = \frac{2}{3}H \tag{8.6}$$

The remaining part (=H – 2/3 H) is kinetic energy:

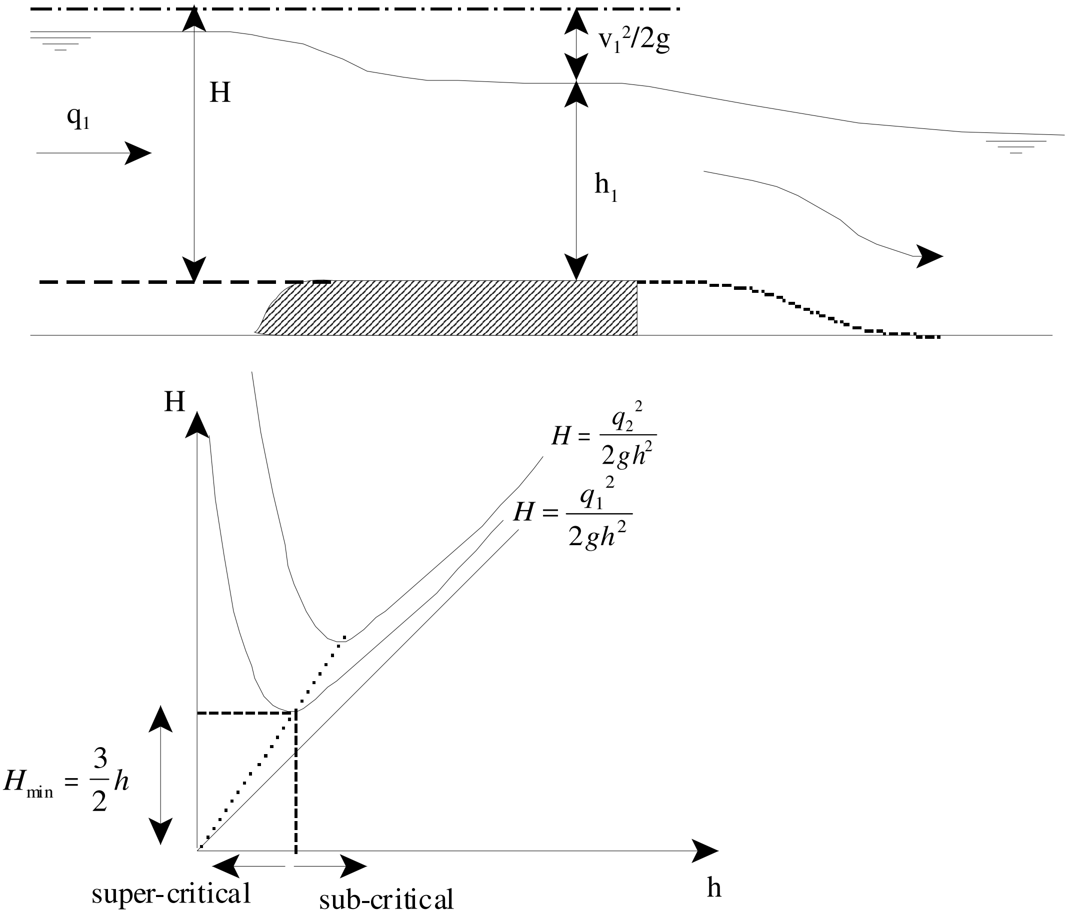
$$\frac{v^2}{2.g} = \frac{1}{3}H \Rightarrow v_c = \sqrt{\frac{2}{3}.g.H} = \sqrt{g.h_c} \tag{8.7}$$

It can be concluded that for a certain energy depth H_1 one maximal flow rate q_{max} exists. This maximum flow occurs when the velocity depth is equal to 1/3 H and the water depth is 2/3 H, with H being the energy depth above the bed (or crest). Similarly can be deduced that with a given q_{max} a flow condition exists with a minimum energy depth H_1, and that that occurs with a water depth h_c equal to 2/3 H_1 (see figure 8.3).

The term critical velocity proceeds from the fact that with this velocity the water flows as fast as the celerity of a disturbance (Fr = 1). This includes that with water velocities equal to or larger than the critical velocity a downstream disturbance (damming up or waves) has no influence on the flow at the location- or upstream of where critical flow appears.

8.2.3 Straight and curved stream lines

Bernoulli's law applies for a hydrostatic pressure only. With curved flow lines the water pressure is not hydro-static and Bernoulli's law can not be used. In the following the relation between flow lines and pressure distribution will be indicated shortly. In fact a hydrostatic pressure means that each water particle pushes on the underlying particle with the same weight, resulting in a linear pressure distribution, because the lowest particle carries the accumulated weight of the water particles above. When the flow lines are straight the pressure distribution perpendicular the direction of the flow does not change, so that a hydrostatic pressure distribution is maintained. When the lowest particle suddenly "sinks" then at that moment the pressure on the sinking particle of the particles lying above is less, because of the inertia of the particles lying above. Consequently the pressure on the particles below has become less than hydrostatic, depending on the acceleration with which the particle sinks.



H = h + v2/2g

q=v*h

$$H = h + \frac{q^2}{2gh^2}$$

q is constant = q_1

For which h is H minimal (with which minimal energy-depth can q_1 been discharged)?

$$\frac{dH}{dh} = 0 \rightarrow 1 - \frac{2q_1^2}{2gh^3} = 0 \rightarrow q_1^2 = gh^3$$

$$H_{min} = h + \frac{gh^3}{2gh^2} = \frac{3}{2}h \quad or \quad h = \frac{2}{3}H = h_c \quad (critical - depth)$$

$$\frac{v^2}{2g} = \frac{1}{3}H_{min} = \frac{1}{2}h_c \quad or \quad v_c = \sqrt{gh_c}$$

Figure 8.3 Theory critical-, sub-critical- and super-critical flow

In the flow lines such "sinking" is displayed in a curvature of the flow lines. The stronger the curvature the larger the deviation from the hydrostatic pressure. An upward curvature against the acceleration of gravity, gives an increase of the hydrostatic pressure and consequently a reduction of the velocity term; and a downward curvature a reduction of the hydrostatic pressure and therewith an increase of the velocity term (see figure 8.4). Note that a curvature in the horizontal plain has no influence on the vertical hydrostatic pressure distribution.

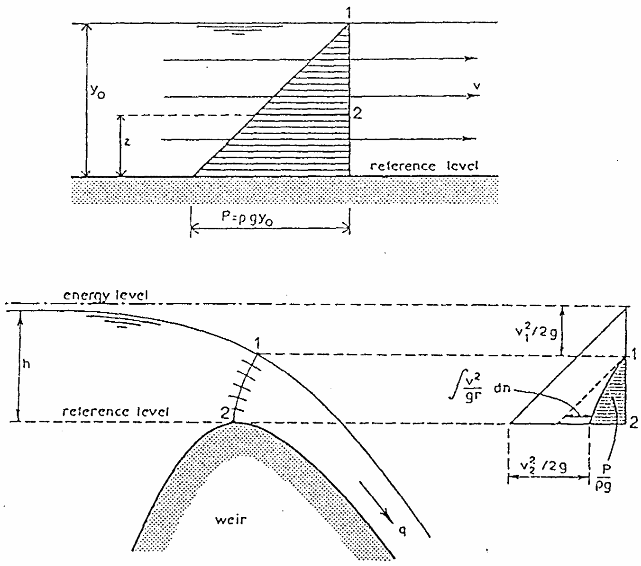


Figure 8.4 Curvature in stream lines causes a change in pressure

8.2.4 Local energy losses in closed conduits

<u>Straight closed conduit</u>
When connecting two reservoirs A and B with a straight closed conduit, energy losses occur at the entry and the end of it and further friction losses in the closed conduit itself. The total loss of energy, expressed in loss of pressure depth is:

$$dH = C_e \frac{v^2}{2g} + C_f \frac{v^2}{2g} + C_o \frac{v^2}{2g} \tag{8.8}$$

With:

dH	=	total energy loss	[m]
C_e	=	entry loss coefficient	[-]
C_f	=	friction loss coefficient	[-]
C_o	=	exit loss coefficient	[-]

Entry loss:
Is calculated as the kinetic energy loss between the throat with contraction : and the closed conduit itself.

$$C_e = \left(\frac{1}{\mu} - 1\right)^2 \tag{8.9}$$

In which : is the contraction coefficient. The larger : (e.g. with a gradual transition), the smaller is C_e. : varies from 0.5 (tube of Borda) to 1.0 (rounded transition).

Friction loss:
Is calculated with Chezy or Strickler; e.g.:

$$C_f = \frac{2.g.L}{k_s^{\,2}.R^{4/3}} \tag{8.10}$$

Exit loss:
C_o indicates the coefficient of the exit energy loss. When all kinetic energy of the water is lost here and the velocity in the closed conduit would be constant over the complete area, C_o would be equal 1. Because the average velocity is used, is $C_o = \forall$. Herewith $\forall$ is the energy coefficient and can be assumed 1.1 as an average.

The total energy loss is thus:

$$\boxed{dH = \left(\left(\frac{1}{\mu} - 1\right)^2 + \frac{2.g.L}{k_s^{\,2}.R^{4/3}} + \alpha\right)\frac{v^2}{2g}} \tag{8.11}$$

With long closed conduits the middle term is by far the most important, so that the entry and exit losses mostly can be neglected. However with short closed conduits the first and last term prevail and especially with short wide closed conduits the friction loss can be neglected.

<u>Sudden enlargements</u>
With an enlargement in a closed conduit energy losses will occur, which will be larger as the section of transition is shorter. The appropriate value for the transition angle ∗ is ± 8° with which the energy loss is about:

$$\pm 0{,}15\frac{v_1^{\,2} - v_2^{\,2}}{2g} \tag{8.12}$$

In this case the flow still follows the wall. With larger ∗ the flow detaches so that the losses increase. Then is spoken of a sudden enlargement. In case ∗ = 90° the energy loss is:

$$\frac{(v_1 - v_2)^2}{2g} \tag{8.13}$$

Is $A_2 = nA_1$, then is $v_2 = \frac{v_1}{n}$ and the expression modifies into:

$$\frac{(v_1 - v_1/n)^2}{2g} = \frac{v_1^{\,2}}{2g}\left(1 - \frac{1}{n}\right)^2 \tag{8.14}$$

By making the transition gradual the energy losses can be limited. E.g. if n = 2, thus $v_2 = ½v_1$, then with a sudden enlargement the energy loss is $1/4.\frac{v_1^{\,2}}{2g} = 0.25\frac{v_1^{\,2}}{2g}$.

With a gradual transition (∗ = 8°) the energy losses become:

$$0.15\times\frac{v_1^{\,2} - v_2^{\,2}}{2g} = 0.15\times\frac{v_1^{\,2} - \frac{1}{4}v_1^{\,2}}{2g} = 0.11\frac{v_1^{\,2}}{2g} \tag{8.15}$$

Contractions
With a gradual contraction of a closed conduit hardly any energy loss occurs, all energy is conserved.

8.3 Types of measuring and regulating structures

8.3.1 General

There are many types of measuring and regulating structures. When flow characteristics are considered two types can be distinguished; the weir and the orifice. In this paragraph both types will be dealt with, followed by free and submerged flows.

8.3.2 Weirs

The weir exists in two designs. The broad crested weir and the short crested weir. With the same upstream water level a short crested weir has a larger discharge than the broad crested weir, because at the crest of the short crested weir the flow lines are curved downwards, causing the resultant pressure to be lower than with straight flow lines.

Broad crested weir
When at first looking at a broad crested weir (see fig 8.5) the discharge formula can be deduced from an energy evaluation. When assumed that the accelerating flow has no (extra) energy loss and that the flow at top of the crest is critical (which means that the water level sets itself to the maximal discharge) than the formula applies:

$$Q = c_v \, . \, c_d \, . \, 1.7 \, . \, b \, . \, h^{3/2} \tag{8.16}$$

With:

Q	=	discharge	[m^3/s]
c_v	=	approach velocity correction coefficient	[-]
c_d	=	crest coefficient	[-]
1.7	=	2/3 √(2/3 g)	[$m^{1/2}/s$]
b	=	width of weir	[m]
h	=	water depth upstream of the weir	[m]

With low approach velocity c_v = 1 and with a smooth and rounded off and sufficiently long broad crest is c_d = 1.

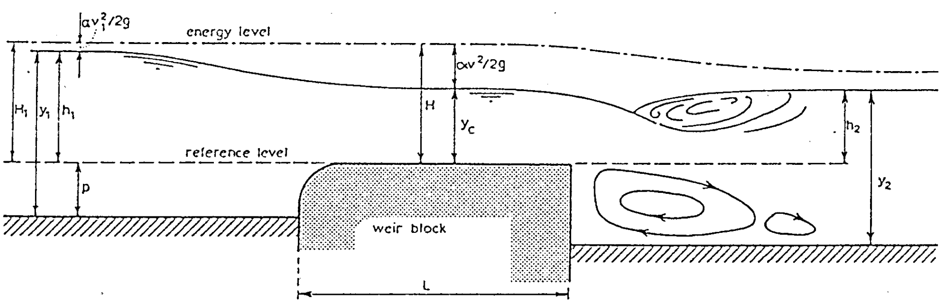


Figure 8.5 Flow over a broad crested weir

<u>Short crested weir</u>
In principles the short crested weir functions similarly as the broad crested weir, however with the difference that the flow lines at the top of the crest are curved. Therefore it is more difficult to deduce a discharge formula. Therefore mostly the same formula is used as for the broad crested weir with a correction coefficient for the curved flow lines, varying between 1.1 and 1.3. The equation for a short crested weir with free flow also reads:

$$Q = c_v \, . \, c_d \, . \, 1.7 \, . \, b \, . \, h^{3/2}$$

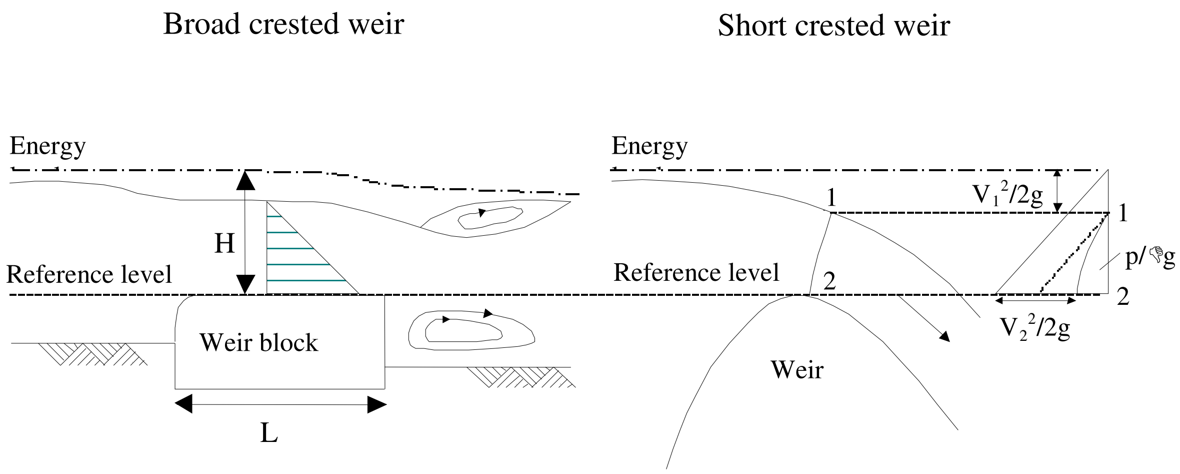


Figure 8.6 Pressure distribution on top of a broad- and short crested weir.

Herewith the same deduction as for the broad crested weir can be used, however with the difference that the coefficient c_d is not 1, but can go up to 1.3.

8.3.3 Orifice (Underflow gate)

The following type of structure dealt with is the orifice. The flow characteristic of an orifice is different from the weir (see figure 8.7).

Two basic flow conditions exist, with downstream free discharge of critical flow or submerged outflow water level.

Contrary to the weir, variation in the upstream water level has little effect on the flow rate.

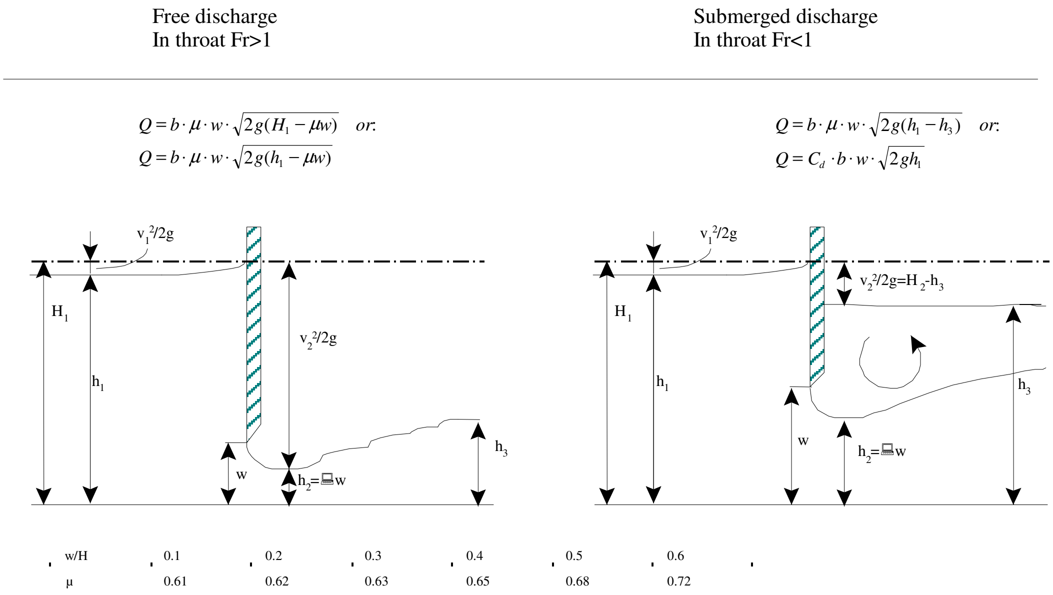


Figure 8.7 Flow through an orifice

Free flow

Like with the weir flow through the orifice can be free or submerged. Also here applies that free flow (which means independent of the downstream level) is preferred. The discharge formula for an orifice with free flow can be found by applying Bernouilli:

$$Q = b.\mu..w\sqrt{2.g.(H_1 - \mu.w)} \tag{8.17}$$

: is determined in practice, and appears to be dependent of w/H. In practice H_1 will be assumed equal to h_1, except when the velocity is high and a correction term has to be introduced:

$$Q = c_e.b.w.\sqrt{2.g.(h_1 - h_2)} \tag{8.18}$$

With:

Q	=	discharge	[m^3/s]
c_e	=	effective discharge coefficient free flow	[-]
w	=	orifice height	[m]
g	=	acceleration of gravity	[m/s^2]
b	=	width of orifice opening	[m]
h_1	=	water depth upstream of the orifice	[m]
h_2	=	water depth immediately downstream of the orifice	[m]
:	=	vertical contraction of the underflow	[-]
H_1	=	energy height	[m]

w/H	0.1	0.2	0.3	0.4	0.5	0.6
Φ	0.61	0.62	0.63	0.65	0.68	0.72

Submerged flow

Also for this flow condition Bernoulli can be applied when is assumed that the flow lines in the throat section (at h_2) are straight and that between h_1 and h_2 no energy losses because of friction or contraction occur. Now the high water level h_3 is relevant to establish the potential energy in the section of h_2; giving the equation

$$Q = C_d.b.w.\sqrt{2.g.(h_1 - h_3)} \tag{8.19}$$

In which:

C_d = submerged (drowned) discharge coefficient [-]

C_d depends on h_3/w and h_1/w and h_1/h_3

The Bernouilli equation can be rewritten as:

$$Q = C_d.b.w.\sqrt{2.g.h_1} \tag{8.20}$$

All variations between h1, w_1 and h_3 have been incorporated in C_d, which was determined by H.R. Henry with laboratory tests, see figure 8.8.

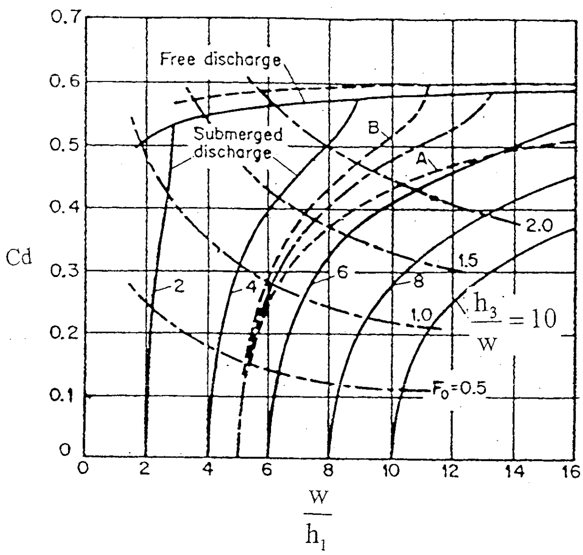


Figure 8.8 Discharge coefficient for vertical sluice gate

Other flow conditions

With orifices still different flow conditions like transfer from free critical flow to submerged, and from wholly free flow when the water level is lower than the gate without touching it. These flow conditions are treated in the lecture “Polders and Flood Protection”.

8.3.4 Perfect and imperfect flow

A weir is called perfect in case the flow over the weir is free, i.e. the downstream water level does not affect the flow, which is when the critical water velocity is reached or exceeded. In the discharge formulas for weirs and orifices discussed in the following is departed from a perfect flow. In case the downstream water level has influence, is spoken of imperfect or submerged flow and different formulas apply. In practice the discharge formulas for perfect flow are used, with a correction for imperfect flow.

With the use of imperfect weirs the operational water management of the channel system becomes much more complex compared with perfect weirs, because with the setting of the discharges also the water levels in the downstream section have to be taken into account.

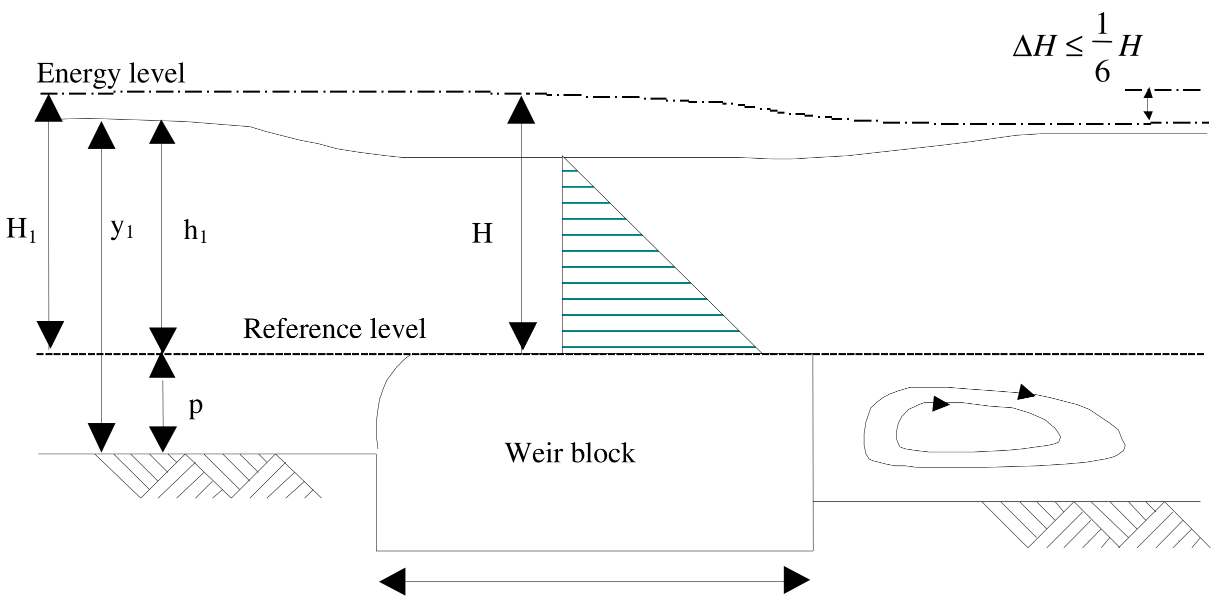


Figure 8.9 Submerged broad crested weir

In figure 8.9 the flow over an imperfect broad crested weir is shown. The discharge over the weir can not be determined explicitly because the flow above the crest is not critical. The flow will be less than with a perfect weir depending on the downstream water level. In practice a reduction coefficient is introduced in the discharge formula for the perfect weir to take the reduced discharge into account.

Often the problem is when the flow is perfect and when it becomes imperfect. A safe limit is that the downstream water level can not exceed the critical water depth above the crest, in which case the flow always will be perfect also with maximum energy dissipation (= $v^2/2g$). When not all kinetic energy is dissipated the downstream water level can become slightly higher than the critical water level above the crest. This is the case when a part of the kinetic energy is recovered as potential energy either by a gradual transmission or by a hydraulic jump. For a perfect weir the downstream water level can rise to a level of 1/6 H below the upstream water level, with which H is the energy level with respect to the crest.

For orifices it is more difficult to establish the limit between free and submerged flow, namely with free flow a hydraulic jump occurs behind the orifice changing the super-critical to sub-critical flow. The downstream water level rises depending on the energy dissipation, which in a hydraulic jump is always substantial. With a limited drop, the flow mostly is submerged and sub-critical.

8.4 Types of structures

8.4.1 Classification of structures

Looking at the functions of the structures the following types can be distinguished:

- Measuring structures
- Off-take structures (combined measuring and regulating structures)
- Level regulators

Measuring structures are needed to measure the flow. In an irrigation system notably measuring structures are used at the off-take points.
Examples of measuring structures are:

- Normal weir
- Thomson weir
- Cipoletti weir

Off-take- structures are needed to take off a certain flow from a main canal into a branch or a distribution canal. Common names are off-take regulator, intake structure, turnout, or head of off-take. An off-take structure has to regulate and measure the flow. In case it is not known how much water is let in, no further control is needed.
Examples of off-take structures are:

- The Romijn movable weir
- Flat gate with measuring structure
- Metergate
- Crump-de-Gruyter gate
- Constant Head Orifice
- Neyrtec Module distributor

Level controllers are needed to fix the water level at the location of a off-take structure, making the management of the off-take structures simpler.
Examples of level controllers are:

- Fixed weir
- Movable weir
- Gate with slide
- Siphon
- Automatic hydraulic level controllers

8.4.2 Measuring structures

Thomson weir (see figure 8.10)
A well known sharp crested weir is the V-shaped weir (Thomson weir). In case the V-shape has an angle of 90^o the discharge formula becomes:

$$Q = m \cdot h^{5/2} \qquad (8.21)$$

The variation of the flow with $h^{5/2}$ is among others caused by the contraction because of the V-shape. An other well-known and much used design of the sharp crested weir is the *Cipoletti weir*.

<u>Cipolletti weir (see figure 8.10)</u>
The Cipoletti weir is a modification of a normal sharp crested weir, and has a trapezoid shaped control section. The crest is horizontal and the sides slope out under an angle of 1 (hor.) to 4 (vert.), to compensate for the effect of the larger side contraction and curvature of the streamlines with high discharges. The discharge formula is given by:

$$Q = 1.9 \, . \, b \, . \, H^{3/2} \qquad (8.22)$$

The downstream water level can not be too high to allow for aeration underneath the nappe. Therefore a downstream water level of at least 0.05 m below the weir crest is a condition. The height of the crest above the bed of the upstream channel has to be at least twice the drop in water level over the weir, with a minimum of 0.30 meter. These conditions also apply for the distance between the sides of the control section and the side of the channel. The ratio between the upstream level (h) and the width (b) of the crest has to be smaller than or equal to 0.50.

The advantages of the Cipoletti weir are:

- Simple structure
- Floating material can pass without problems
- Flow measurements are simple.

The disadvantages are:

- The drop in water level is equal to the energy loss
- No possibility to regulate the flow.

8.4.3 Combined Measuring & Regulating Structures

<u>Romijn movable weir (see figure 8.11)</u>
The Romijn movable weir is developed in Indonesia for use in relative flat areas, where the water flow is variable over the year because of difference in crop water requirement during the growth season, and conditions of water shortage.

The Romijn movable weir is a combination of a broad crested weir and an orifice. The broad crested weir is connected to a steel guidance frame and can be moved up- and down to increase or reduce the flow. Often an extra opening is installed near the bottom in the guidance frame to flush through sediment deposited upstream the weir. Standards widths of Romijn movable weirs are 0.50, 0.75, 1.00 and 1.25 meter, for maximum discharges of 0.30, 0.45, 0.60 and 0.75 m3/s with a maximum drop over the weir of 0.50 m.

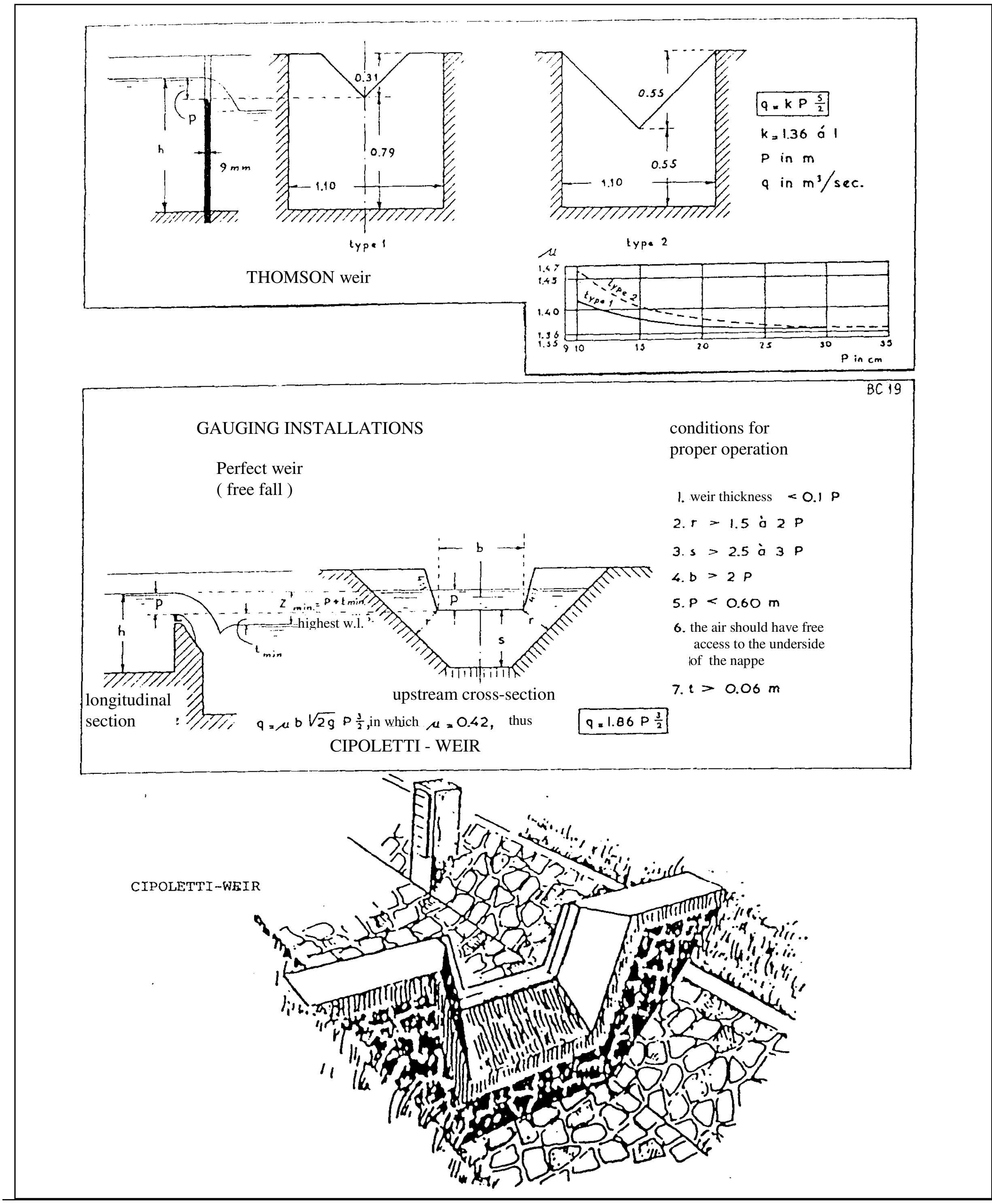


Figure 8.10 Thomson weir and Cipoletti weir

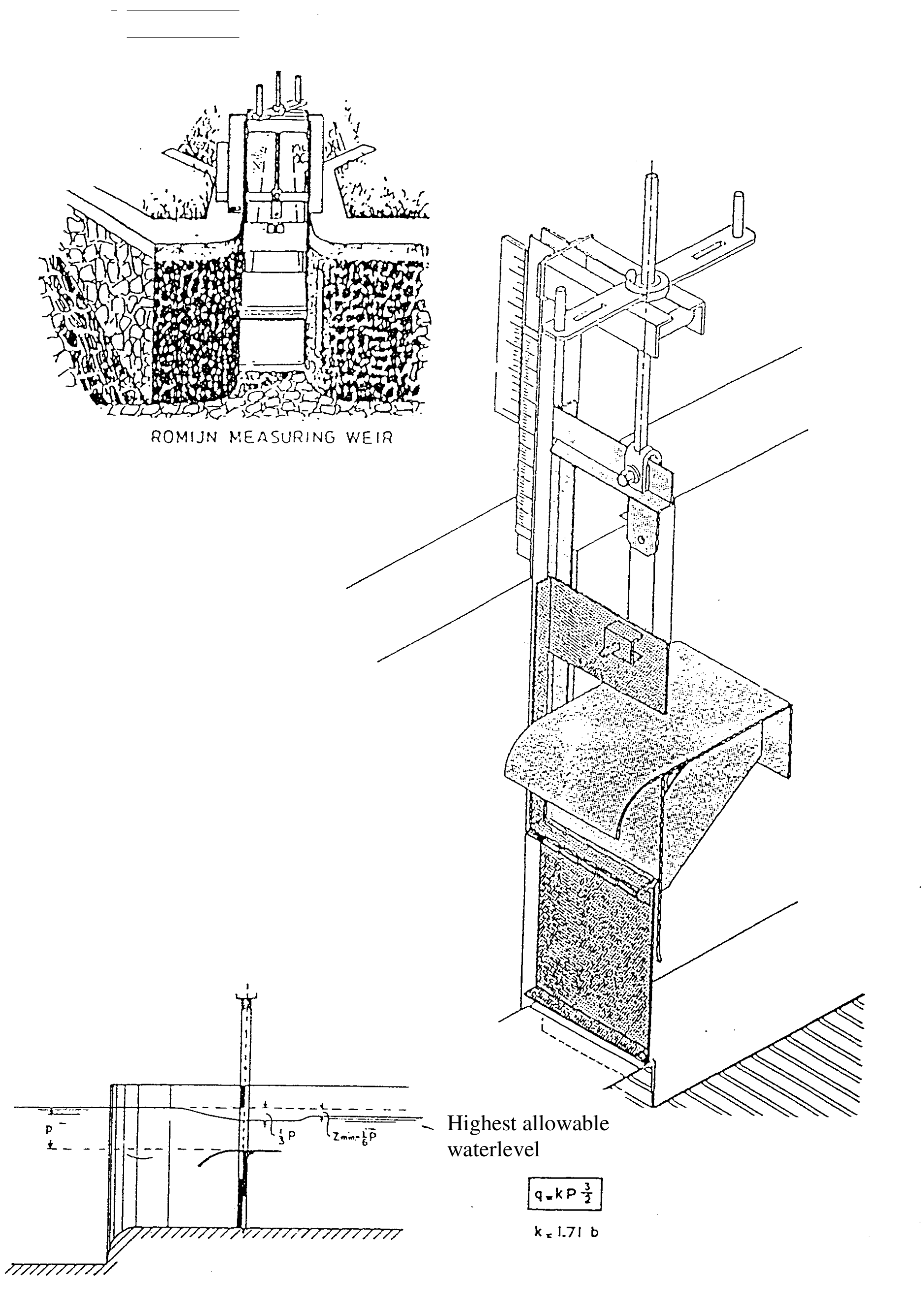


Figure 8.11 The Romijn-sliding gate

Measuring the discharge and setting the level is simple, although three staff gauges are required:

- One for the water level in the channel (in centimetres), the ‘counter-gauge’;

- One fixed to the frame (in centimetres), the ‘centimetre-gauge’;
- One logarithmic gauge moving together with the weir, the ‘litre gauge’.

Example

A water level in a secondary channel-is read to be 63 cm on the ‘counter-gauge’ and 124 l/s are let in in the tertiary canal. The movable gate is set in such a way that the 124 l/s of the litre-gauge is in equal position with the 63 cm on the centimetre gauge.

The advantages of the movable Romijn-weir are:

- The structure can measure and regulate contemporary
- The head-loss is relatively small
- Operation and setting is simple
- High accuracy

The disadvantages of the Romijn-weir are:

- Complicated and expensive structure
- The flow is sensitive for upstream water level variations ($H^{1\frac{1}{2}}$)
- The structure needs a high water level in the supply channel

Flat sliding gate (see figure 8.12)
The flat sliding gate is the simplest design of the orifice, with which the mostly metal sliding gate can be moved up and down as required, from completely closed to even above the water surface. As long the gate is partly under water, the simple formula applies:

$$Q = c.A.\sqrt{2.g.z} \qquad (8.23)$$

However when the gate has been raised out of the water the discharge formula of the weir applies.

Metergate (see figure 8.12)
The Metergate consists of a pipe of a standard diameter (often between 0.15 and 0.60 m.) which can be closed by means of a flat sliding gate. The flow through the structure can not be measured, although the maximum discharge can be determined from the diameter of the pipe and the gradient (head-loss divided by the length of the pipe) at free flow.

With submerged flow the conditions are much more complicated. The Metergate often is used in semi-technical irrigation systems as (tertiary) off-take installation, where finally the water is transferred to the water-users organisation. The discharge formula of the Metergate with free outflow (modular) is:

$$\mathrm{Q} = \mathrm{m.w.b.}\sqrt{2.\mathrm{g.z}} \qquad (8.24)$$

With:

m	=	discharge coefficient, about 0.55 to 0.60	[-]
w	=	height of the opening of the culvert	[m]
b	=	width of the culvert	[m]
z	=	water level drop over the gate,)H	[m]

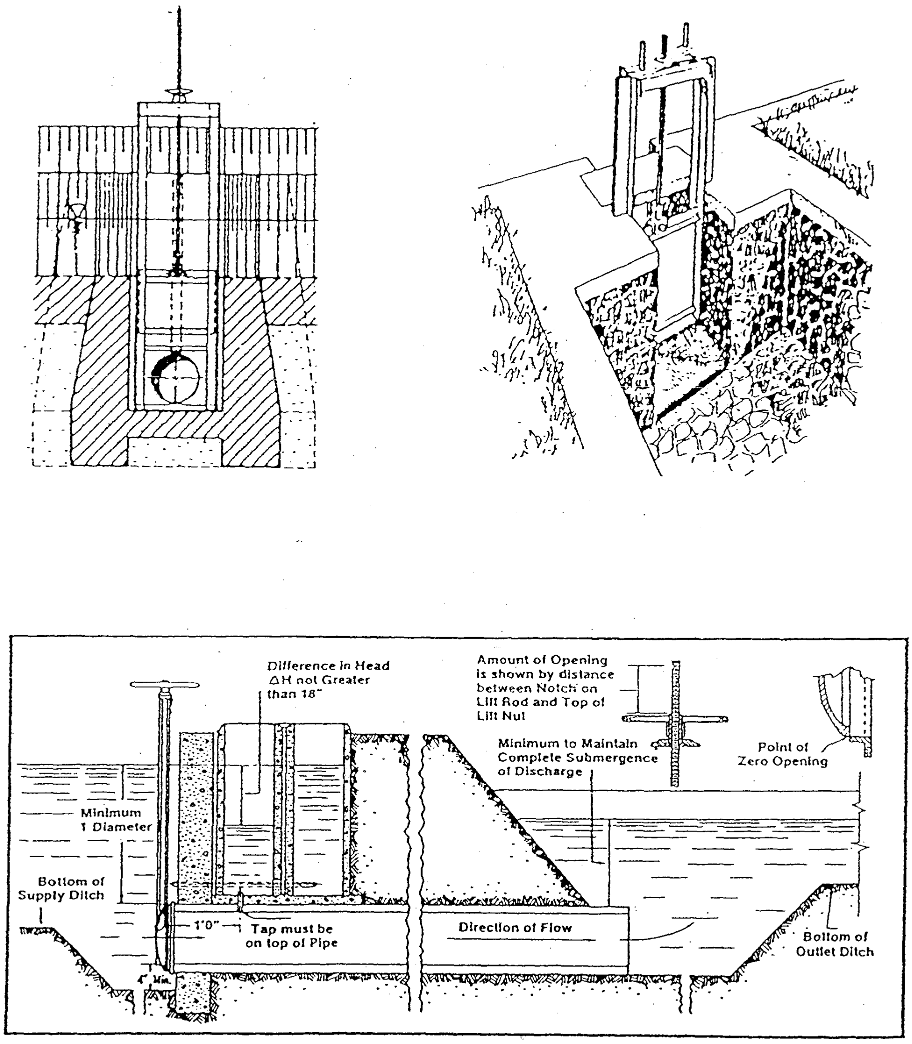


Figure 8.12 The flat sliding gate and Metergate

Tables are available (published by the U.S.B.R.) showing the discharges for the various standard designs of the Metergate. The drop z is measured by lowering a hook with two scales in the two gauging wells and reading the difference in water level.

Advantages of the Metergate are:

- The upstream water level can be regulated accurately (N.B. not relevant for off-take installation)
- The structure is sturdy and simple
- Sediment can be passed

Disadvantages of the Metergate are:

- The culvert catches suspended sediment
- The discharge can not be measured very accurately so that often an additional measuring structure is required

- For the discharge measurement two water levels and a sliding gate position have to be measured, which often is too complicated in practice
- The structure is expensive

<u>Crump-de-Gruyter sliding gate (figure 8.13)</u>
The Crump-de-Gruyter sliding gate is a streamlined flat sliding gate in a short flume. At the upstream side of the gate half a cylinder is attached to prevent strong curvature of the flow lines and thus contraction below the gate. By attaching this curvature at the upstream face at the bottom of the gate as used by Crump, the contraction coefficient can be assumed : = 1.0. In case of no curvature : has to be determined; and depends on the ratio of the upstream water level H and the gate opening y (Table 8.1).

Table 8.1 Crump de Gruyter: minimal level loss for free flow

k = : . y/H (For de Crump-de-Gruyter gate applies : = 1)	**Measurable reach** $\forall = z_{min}/H$	$\vartheta = Q_{max}/Q_{min}$
0,630	0,167	1
0,218	0,386	2
0,140	0,496	3
0,101	0,575	4
0,080	0,620	5
0,065	0,665	6
0,055	0,690	7
0,049	0,715	8
0,044	0,735	9
0,040	0,750	10

The discharge formula of the Crump-de-Gruyter gate is given as:

$$Q = C.b.\mu.y.\sqrt{2.g.(H - \mu.y)}\text{, with } C = 0.9 \text{ and } : = 1.0 \qquad (8.25)$$

Conditions for proper functioning are:
(i) the slide gate opening y is less than 2/3 H (to prevent the structure to function as a weir)
(ii) the condition for free flow has been fulfilled.

In case the downstream water level is too high the lower part of the slide gate will be submerged at the downstream side (and is spoken of a “drowned” gate). Then the installation is not any more modular and the above discharge formula is not longer valid. Thus it can be clearly observed if the installation functions properly. Concerning the modality the behaviour of this type measuring structure is governed by z_{min}, with which the rising of the water just does not occur and the underside of the gate remains visible.

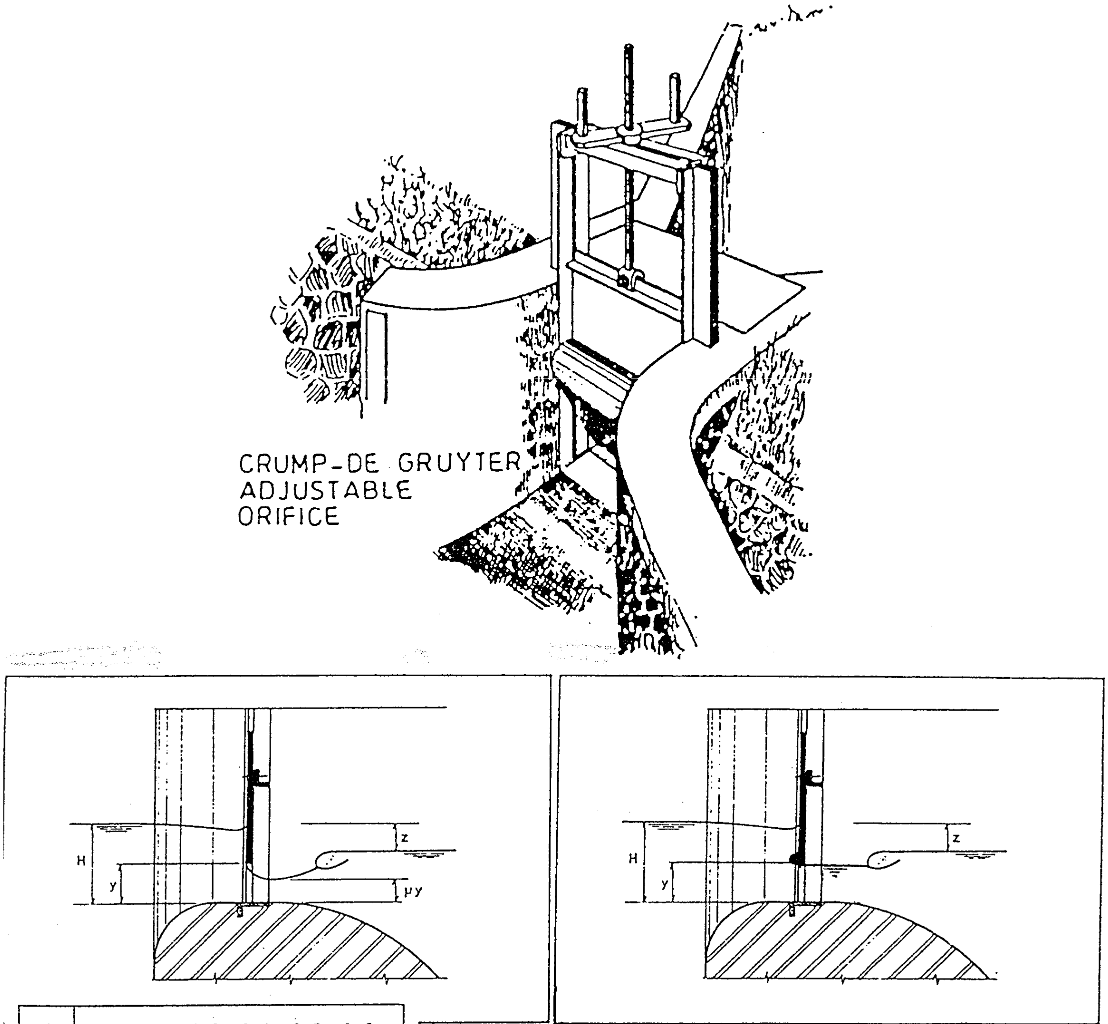
From tests a relation between k (= y / H_0 and $\forall$ (= z_{min} / H) appears. The minimum necessary drop increases relatively with rising upstream water level H. For the lowest possible upstream water level is the needed drop (z_{min}) only 0.17 H. For the “highest possible” upstream water level the minimum drop has increased to 0.75 H (see table 8.1).

Advantages of the Crump-de-Gruyter gate are:

- The structure can measure and regulate at the same time
- There are no problems with deposition of sediment in front of the structure
- The structure can be used in supply channels with large water level fluctuations.

Disadvantages of the Crump-de-Gruyter gate are:

- The structure is complicated and expensive
- The water level drops are relatively high
- Measuring the discharge requires two readings: the upstream water level and the gate opening
- Floating debris can be a problem.



y/H	0.1	0.2	0.3	0.4	0.5	0.6
μ	0.61	0.62	0.63	0.65	0.68	0.72

Figure 8.13 The Crump-de-Gruyter sliding gate

Constant Head Orifice (see figure 8.14)

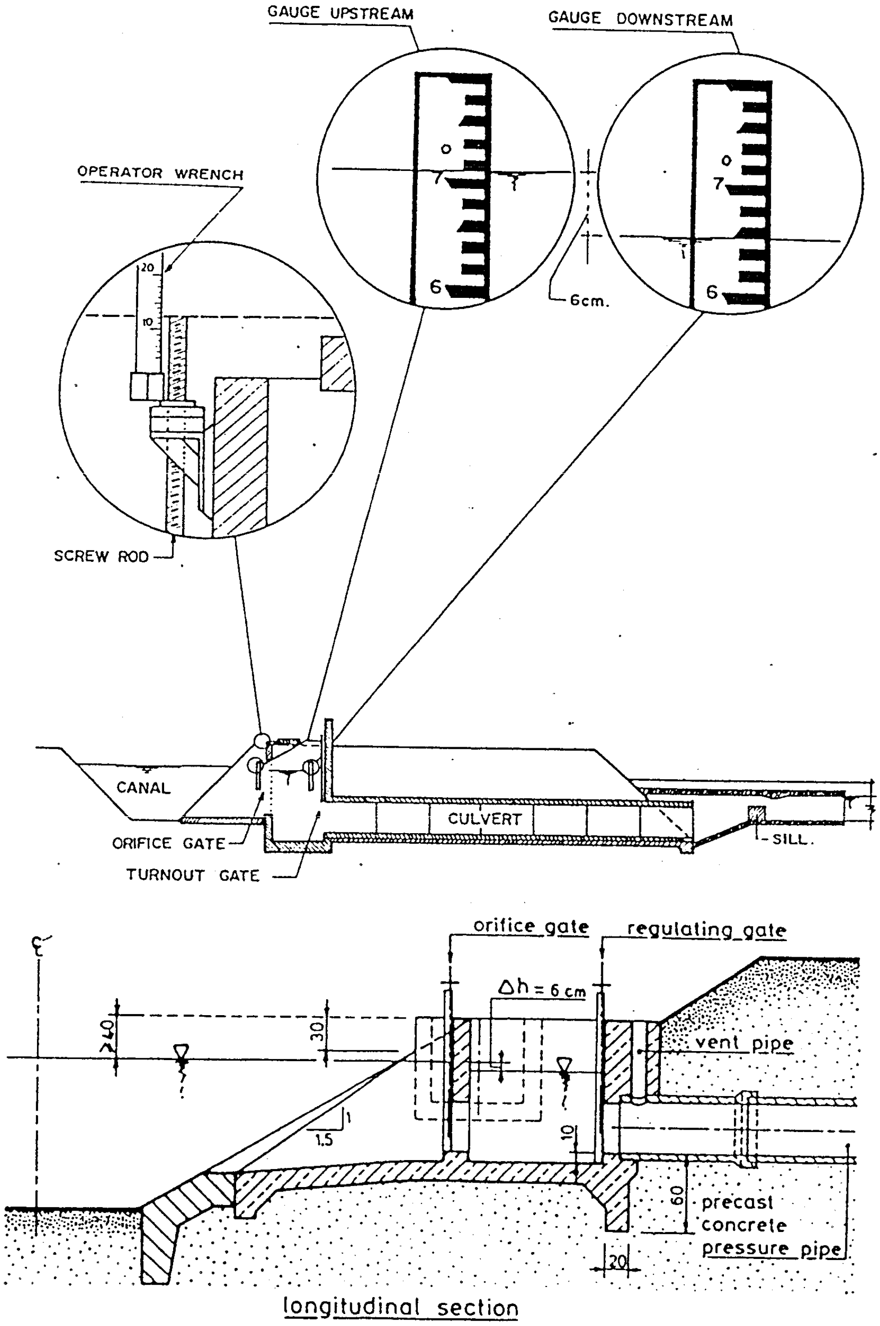


Figure 8.14 The Constant Head Orifice

The Constant Head Orifice" is a combination of a measuring and regulating structure which uses:

- An (upstream) adjustable submerged orifice to measure the flow
- A (downstream) adjustable outlet for the control.

The operation of the 'Constant Head Orifice' is based on the setting and maintaining a constant head over the upstream opening, mostly 0.06 m. The discharges can be set by changing the discharge area of the upstream orifice, and maintaining the head loss of 0.06 m over the orifice by adjusting the downstream "regulator'. The rather low value of 0.06 m is one of the factors contributing to the low accuracy of measuring the discharge. The introduction of a larger head loss will decrease this error, but introduce larger flow disturbances in the dampening basin between the two sections. The combined head loss over the upstream opening (0.06 m,) and the downstream opening is substantial.

The discharge formula for the Constant Head Orifice is synonymous with the discharge formula for the drowned orifice, with a head loss of 0.06 m, and reads:

$$Q = 0.716 \cdot b \cdot w \tag{8.26}$$

With:

b	=	opening width	[m]
w	=	gate opening	[m]

Advantages of the Constant Head Orifice gate are:

- The structure can measure and regulate at the same time
- There are no problems with deposition of sediment in front of the structure
- The structure can be used in supply channels with large water level fluctuations.

Disadvantages of the Constant Head Orifice are:

- The discharge measurements are inaccurate
- The head losses are high, often more than 0.25 m.
- Complex operation: Measuring the discharge requires two readings.
- Floating debris can not pass the structure.

<u>Neyrtec Distributor (see figure 8.15)</u>
Neyrtec distributors, formerly called Neyrpic, consist of separate modules and are (tertiary) off-take structures designed to simply pass a controllable and constant discharge, more or less independent of the upstream water level. The flow can be set on the desired discharge by opening and closing a combination of different modules. Each module is provided with a sliding gate which has to be either in the completely closed or completely open position.

After the modules have been set the discharge remains constant, even with variable upstream water levels within a certain range.
With low water levels the structure functions as a broad crested weir. When the water level rises and it touches the fixed plate it starts functioning as an orifice, which is accompanied by a reduction of the discharge. With the further rising of the water level the flow contraction caused by the upstream directed fixed plate becomes larger still, so that the discharge increases only slightly. Because the discharge of a certain module can not be changed, the only manner to change the discharge is by combining various gates of different widths.

In addition to the various widths there are four different types of modules manufactured, distinguished in terms of nominal discharge per unit width. These types are:

- Series X: 10 l/s per 0.10 m width
- Series XX: 20 l/s per 0.10 m width
- Series L: 50 l/s per 0.10 m width
- Series C: 100 l/s per 0.10 m width

The modules can have an index *1* or *2*, depending if the type has one or two fixed plates. The modules with two fixed plates allow a larger upstream water level variation than these with one.

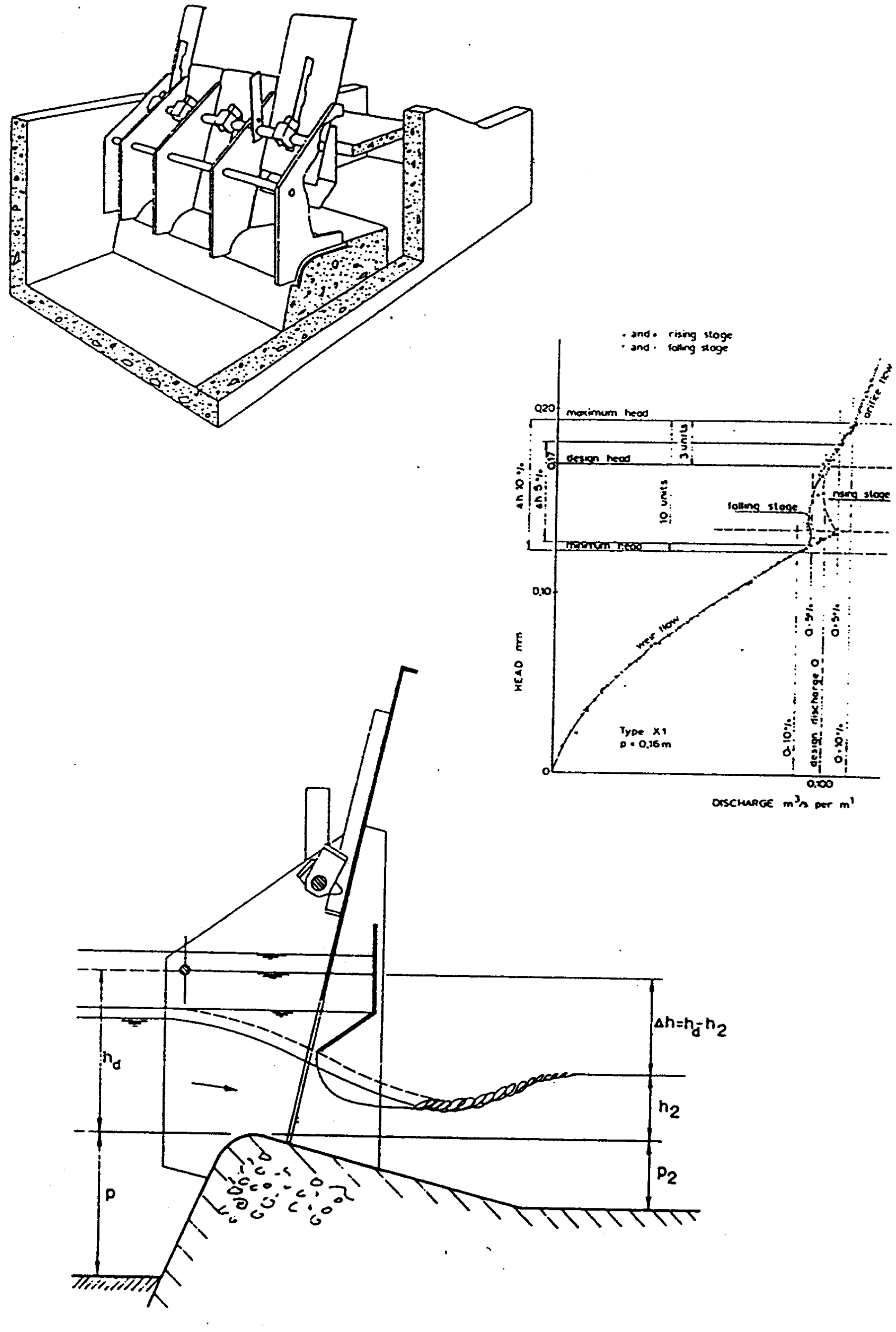


Figure 8.15 Neyrpic Distributor

8.4.4 Level controllers

To be sure that the desired discharge is passed through an off-take structure as set, it is necessary to keep the water level upstream the structure reasonable constant. To maintain this water level special water level controllers are constructed in the channels. Besides maintaining the level these controllers should be able to pass also surges while limiting the level variations as much as possible. To limit costs often the level for several off-takes is maintained by one level controller.

This requires high standards for the design of level regulating structures. Important selection criteria are:

- What is the maximum acceptable level variation
- Do supply variations occur at long intervals only so that a manual adjustment will be effective, or are the supply variations frequent and/or unexpected so that automatic adjustment is desired;
- Which level has to be regulated: upstream or downstream of the structure (upstream- or downstream control).

Level controllers mainly are used in primary and secondary channels. The following structures can be distinguished:

- Fixed weir
- Movable weir
- Gate with sliding door
- Siphon
- Automatic (hydraulic) level controllers
  - Begemann gate
  - Vlugter gate
  - Neyrtec AMIL
  - Neyrtec AVIS
  - Neyrtec AVIO

<u>Fixed weir</u>
With a fixed weir is, after completion of the structure, regulation impossible. The requirements to maintain the water level have to be in the design of the structure. Departing from the maximum passing flow and the maximum increase of the water level above the crest, the required width of the weir can be determined. The minimum water level is equal to the crest level. The level above the crest is proportional with $Q^{2/3}$. Fixed weirs with width regulation are not sensible as they reduce the width of the weir only to increase the water level variation.

Orifices with fixed openings are hardly used as large level variations will occur because the water level varies with Q^2.

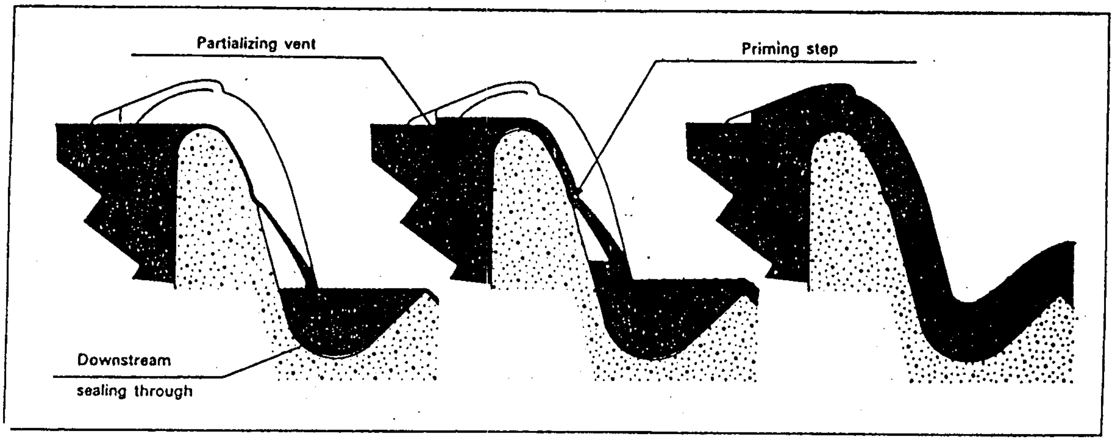
<u>Movable weir</u>
A movable weir functions according to the same principle as a fixed weir. They have the advantage that a water level rise with large flows can be limited by lowering the weir. Moreover with movable weirs it is also possible to vary the minimum required water level. The main disadvantage is that the structure is expensive to construct.

A compromise of a fixed and movable weir is the stop-log weir, an often used and inexpensive to construct. The installation and removal of the stop-logs is an unpleasant job however, and the stop-logs not fixed in the weir may be pilfered!

<u>Flat gate</u>
The flat gate passage is a simple and effective means to maintain the upstream water level in an irrigation channel. For the regulation of the water level it is sufficient to raise or lower the gate until the required value is reached. The design of the flat sliding gate mostly is based on the maximum (design) flow with which the head loss has to be as small as possible. This

means that the gate has to be out of the water during the maximum flow, so that the entry and exit losses has to be taken into account only. These are of the order)h = 0.5 $v^2/2g$, with v being the velocity in the opening.

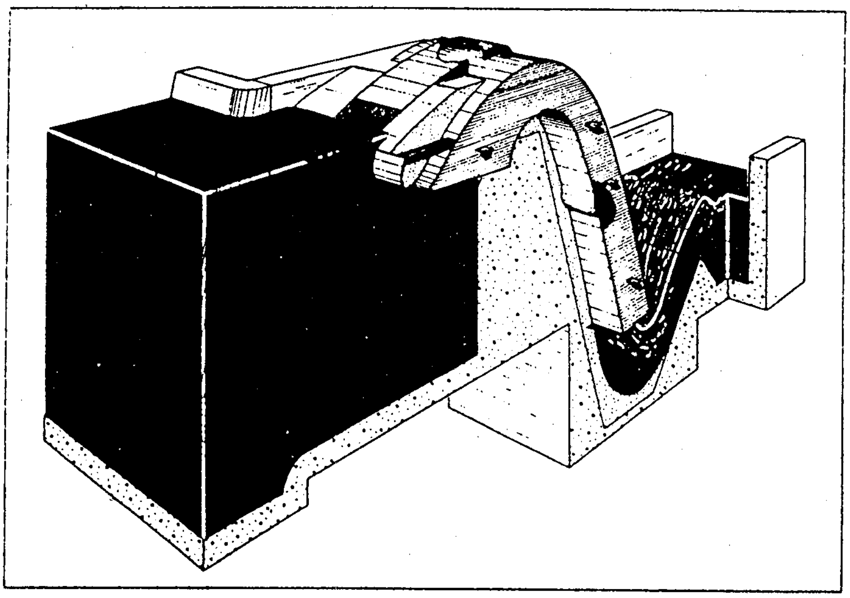
<u>Siphons (see figure 8.16)</u>



1) LOW DISCHARGE
Siphon operating as a weir

2) MEDIUM DISCHARGE
Partialized operation,
with varying air entrainment

3) MAXIMUM DISCHARGE
Siphon is fully primed



10 emergency siphons Si.1400 l/s for Euphrates Basin Reclamation Project (Syria)

Figure 8.16 Siphons

Siphons are used to prevent the rising of the upstream water level above a certain limit. They are able to discharge a large flow with limited width of the structure, which saves costs. This large discharge per unit width is caused by the sucking operation of the water in the siphon.

<u>Begemann gate (see figure 8.17)</u>
Originally intended for level management in rivers with an automatic safety against sudden floods, they have appeared to be very well suitable for level control for primary and secondary irrigation conduits.

With this gate the momentum of the upstream water pressure is in balance with the momentum of its own weight of the gate around a horizontal axis. The manner the gate reacts on level variations depends on:

- The elevation of the turning axis above the water level;
- The horizontal distance from the axis to the gate;
- The location of the centre of gravity.

Prof. Vlugter carried out test and published recommendations. The Begemann gate can be very simply manufactured and maintained and completely automatically maintains the upstream level within a certain range. The downstream water level has to be below the underside of the gate because otherwise the moment balance would be affected.

<u>Vlugter gate (see figure 8.17)</u>
This gate functions similarly as the Begemann gate to keep the upstream level constant. By attaching a cylindrical body to the downstream face of the gate, with the axis of the cylinder and hinge of the gate coinciding, the affect of the downstream water level is eliminated. Therefore the gate also can be used in locations where the drop is limited.

<u>Neyrtec AMIL: upstream automatic level controller (see figure 8.17)</u>
The AMIL is an automatic hydraulic structure, which can maintain a constant upstream water level independent of the passing flow. This is managed by a special design and construction of the structure (developed by Neyrtec). The AMIL consists of:

- An in vertical sense rounded gate radially attached to a hinge. The gate fits in a trapezoidal shaped section
- A floater is attached to the upstream face of the gate
- Two counterweights, both adjustable.

The AMIL gates are described by the dimension ‘D’, which is about equal to the width of the water surface in cm. They are available in widths of 0.80 to 8.00 meter, for flows from 0.10 to 55 m^3/s.

The upstream water depth is about equal to 0.45 D, and the gate rises to a maximum of 0.225 D. The wet flow profile at the gate is about 0.2 D^2 and 0.35 D^2 directly upstream of the gate. The head loss can vary from some centimetres with the minimum flow rate to 1 meter with a flow rate of 50 m^3/s.

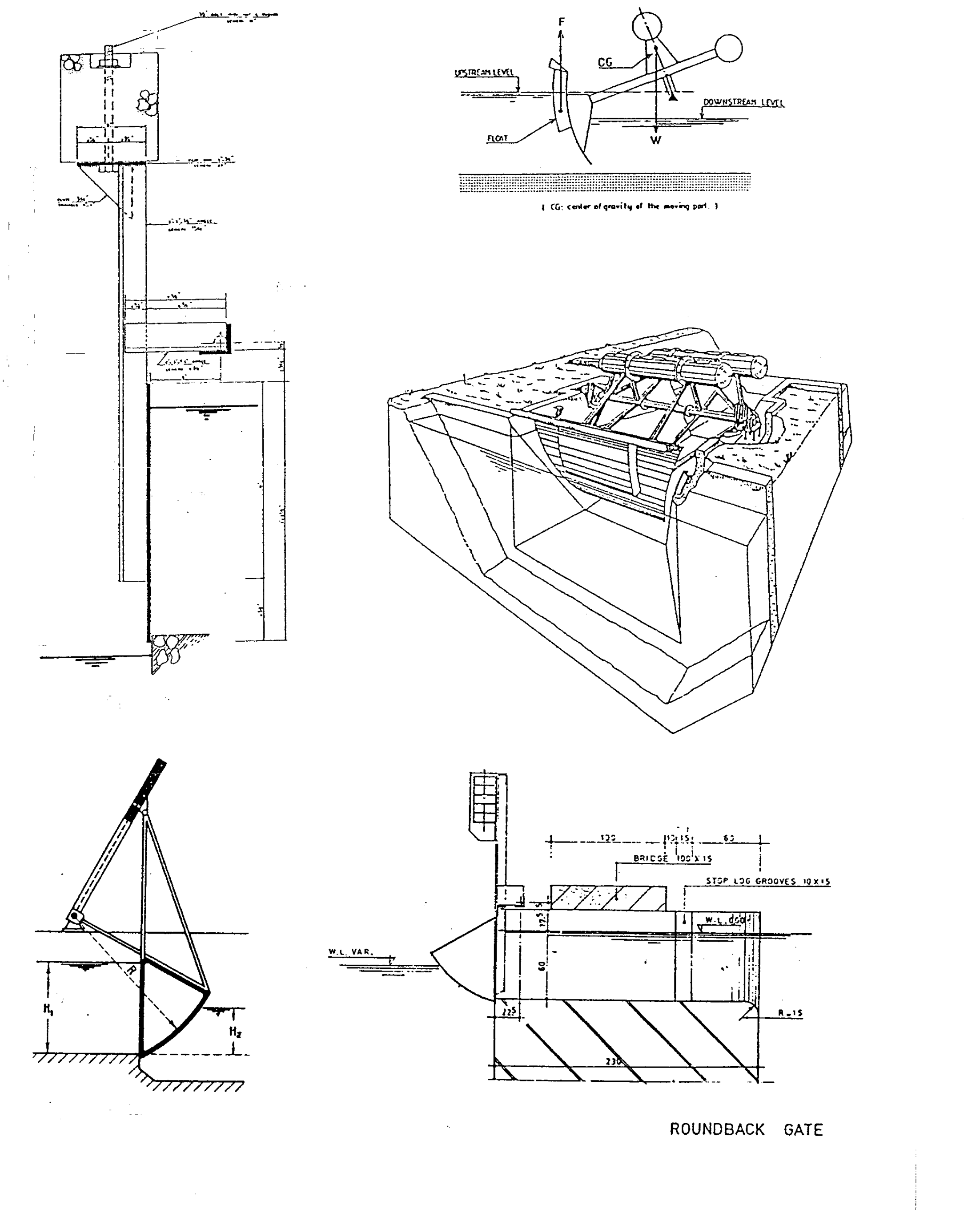


Figure 8.17 The Begemann-gate, Neyrpic-AMIL gate and Vlugter-gate

<u>Neyrtec AVIS: downstream automatic level controller (see figure 8.18)</u>
The AVIS is a hydraulic automatic structure that reacts on the downstream water level. The structure consists of:

- A cylindrical shaped gate with a trapezoidal cross section;
- A steel framework;
- A floater;
- An adjustable counterweight.

When the downstream water level drops below the desired level, it lowers the floater and rises the gate so that water flows into the downstream section and the water level rises again.

Subsequently the gate lowers a little bit. Thus a larger or smaller flow is passed depending on the deviation from the desired downstream water level.

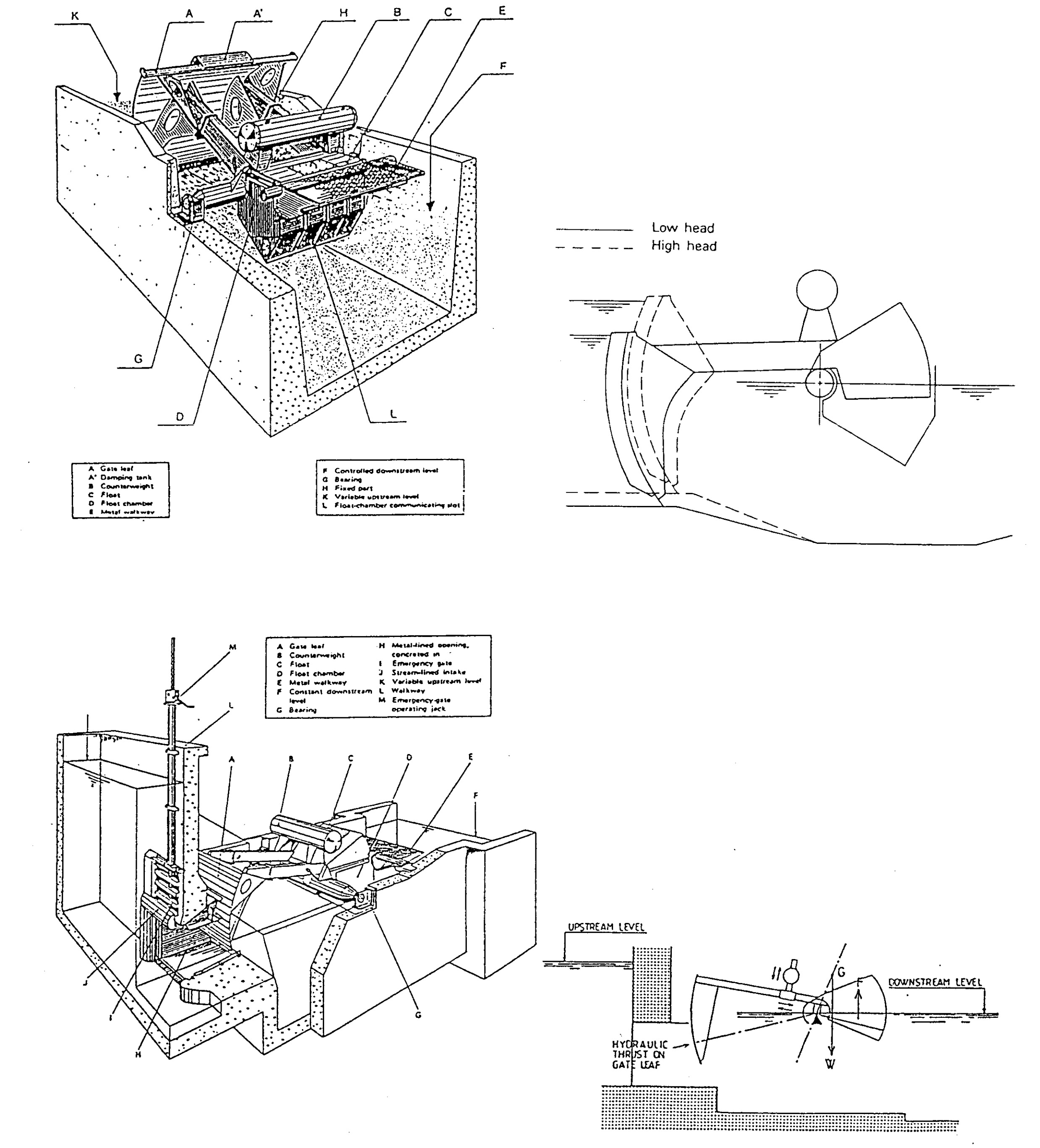


Figure 8.18 The Neyrpic-AVIS and Neyrpic-AVIO gates

The AVIS structures are noted in terms of the floater's radius 'r' in cm, and the bed-width in cm at the location of the gate. Two groups of the AVIS are available, one for large drops and one for small drops. The AVIS with small drops differs from the other one with the same floatation by a wider gate of lesser height. To prevent vibration there remains a small opening between the gate and the sides in the completely closed position. When operation requires complete closure of the flow, the AVIS should be replaced by the AVIO which is provided with an upstream (emergency) passage.

Neyrtec AVIO: downstream level regulator (see figure 8.18)
The AVIO is a variant of the AVIS. The AVIO is used when the flow in the supply canal is large, and the flow to be taken-off is small, such as from primary canals and storage ponds. The choice between a structure provided with the open (AVIS) and gated passage is determined by the maximum drop occurring between the upstream and the downstream to be regulated water level.

The AVIO is notified in terms of the radius of the floater ‘r’ in cm, and the passage section of the gate opening in dm^2, e.g. AVIO 56/25. The necessary dimensions of the gate can be determined with the figures in the brochure of Neyrtec concerned.